Draft Preliminary Geotechnical Engineering Report
Piscataway Drive Slope Failure

Fort Washington, Prince George's County, Maryland

Prepared For

Prince George's County Government, Maryland

THIS DRAFT REPORT WAS PREPARED FOR THE EXPRESS PURPOSES OF PROVIDING ADVICE AND TECHNICAL EXPERTISE TO THE COUNTY EXECUTIVE STAFF TO ASSIST IN THEIR DECISIONAL PROCESSES

Prepared By

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May 19, 2014

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May 19, 2014

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Subject: Draft Preliminary Geotechnical Engineering Report
Piscataway Drive Slope Failure
Fort Washington, MD, Prince George's County, Maryland
KCI Job. No.: 07100627.W

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Dear Mr. Majett:

KCI Technologies, Inc. (KCI) has completed the preliminary geotechnical exploration for the Piscataway Drive slope failure.

The attached report presents a description of the existing site, subsurface conditions encountered, and recommendations for stabilizing the failed slope.

We appreciate the opportunity to provide these services and look forward to serving as your geotechnical consultant throughout this project. Please contact us if you have any questions regarding the information presented.

Sincerely,

KCI TECHNOLOGIES, INC.

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"PROFESSIONAL CERTIFICATION: I HEREBY CERTIFY THAT THESE DOCUMENTS WERE PREPARED OR APPROVED BY ME, AND THAT I AM A DULY LICENSED PROFESSIONAL ENGINEER UNDER THE LAWS OF THE STATE OF MARYLAND, LICENSE NO.:_25981__ EXPIRATION DATE:__3/22/15___"

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EXECUTIVE SUMMARY

This report contains the results of our subsurface explorations and preliminary geotechnical evaluations for repairing the approximately 1,500-foot section of failing slopes and ground movements along Piscataway Drive, Fort Washington, Prince George's County, Maryland. We performed 15 soil test borings, 10 cone penetration tests; and installed one piezometer to explore the subsurface conditions at the site. Additionally, we installed six inclinometers to monitor ground movements.

The test borings and CPT data revealed a soil profile consisting of three distinct strata within their termination depths, consistent with published geology. Stratum I (Ta, Nangemoy Formation) generally consisted of moist, brown, light brown, dark gray, very loose to medium dense Silty Sand, Clayey Sand, Sand with Gravels, and interbedded with soft to stiff Sandy Silt and Sandy Clay layers. A 20 to 30-ft thick layer of Stratum II (Marlboro Clay, Tm) underlies Stratum Ta. It consisted of moist to wet, reddish brown, brown, light gray to gray, Lean Clay with occasional thin lenses of micaceous Silt. Locally, we encountered Fat Clays within this stratum. Beneath the Tm, we encountered Stratum III (Aquia Formation, Ta) which consisted of moist to wet, olive gray, greenish gray to dark gray, Silty Sand and Sandy Silt with mica and calcareous shell fragments scattered throughout the stratum.

Based on preliminary site evaluations, analyses and review of historic information, the existing Marlboro Clay stratum made the site susceptible to slope failures. The intense and rapid infiltration of rainfall that occurred prior to the slope failure created saturated soil conditions resulting in significant loss of shear strength. The exploration data provided evidence of a failure plane within the Marlboro Clay stratum.

KCI proposes three options to stabilize the slopes with each soil-structure system extending beyond the anticipated failure planes. They are: 1) Drilled Shaft foundation along the east and west slopes abutting the roadway and Micropile Anchors at the head scarp upslope; 2) Drilled Shaft Foundation for east slope and Micropiles for west slope; 3) Micropiles for both east and west slopes. We anticipate that the resulting ground movements indicated by the inclinometer readings will have significant implications for the slope rehabilitation options. We therefore recommend that additional detailed analyses and design, constructability evaluation and cost analyses be performed for each option as part of the design purposes.

Design and construction considerations should not be based solely on the executive summary without reading the entire report.

1.0 INTRODUCTION

1.1 PROJECT INFORMATION

The project is located in Prince George's County in the proximity of 13700 to 13816 Piscataway Drive, Fort Washington, Prince George's County, Maryland. The site is bordered by Piscataway Creek on the east and Pine Road to the west. The Piscataway Drive, which traverses the site, is bordered by steep slopes on both sides with homes perched above and below the roadway. Figure 1 illustrates the site.

Historically, the slopes above and below Piscataway Drive have been experiencing surficial movement over a long period of time, but on May 4, 2014, significant failure.

Cracks began appearing in the pavements on Piscataway Drive on May 2, 2014 and escalated into major slope failures and pavement distress on May 4, 2014. Prior to May 2, there were no visible cracks or fractures on the slopes. Cracks, however, appeared on the slopes and widened on May 4 resulting in continuous fracture and downward movement of the western slope for a distance of approximately 450 feet long. The depth of failure along the slopes ranged from about 4 feet to about 20 feet. The deeper failure depths were results of root bulbs from several toppled trees during the slope failure.

The slope failure has directly threatened six homes, disrupted power, water supply, communications and other services to an additional 22 homes along the Piscataway Drive. It has also jeopardized the use of most of roadway from 13700 Piscataway Drive to the southernmost part of the drive. The affected portion of Piscataway Drive remains closed and the County has determined and declared numerous homes in the vicinity of the slide unfit and/or unsafe for occupancy.

1.2 SCOPE OF SERVICES

The purpose of this study is to obtain specific subsurface data at the site, review existing site geologic data and assess the cause of the slope failure and develop recommendations for:

- Rehabilitating the slope failure;
- Reconstructing Piscataway Drive;
- Repairing the utilities; and
- Options for moving forward in the design and construction phases.

Assessments of site environmental conditions or the presence or absence of pollutants in the soil, rock, surface water, or groundwater of the site were beyond the proposed objectives of our studies.

The report for this study includes the following:

- A brief review of our field and laboratory test procedures and their results
- Evaluation of subsurface conditions to include:
  - Review of surface topographic features and site conditions
  - Review of site geologic conditions
  - Review of near surface soil conditions
  - Estimates of subsurface profiles, as necessary, to illustrate subsurface conditions
- A review of possible causes of slope failure
- Evaluation of various alternatives for stabilizing the slopes
- Recommendations for stabilizing the slopes, reconstructing the affected portions of the Piscataway Drive, repairing the damaged utilities, and
- Options for moving forward in the design and construction phases

2.0 EXISTING SITE AND SUBSURFACE CONDITIONS

2.1 EXISTING SITE CONDITIONS

KCI conducted a site reconnaissance on May 3rd and 4th, 2014. The purpose of the site reconnaissance was to observe and document existing surface conditions. Information gathered during the site visit and site GIS data provided to us by Messieurs Unmesh Patel and Dwight Joseph of Prince George's County were used to help us interpret the subsurface data and to detect conditions that could affect our evaluations and recommendations.

The site topography is generally hilly. Piscataway Drive traverses the site. The difference in elevations between the top of the hill and the Piscataway Drive is approximately 65 feet. The elevation difference between the highest and lowest point of the site is approximately 100 feet. There are several residential buildings east and west of the Piscataway Drive. The slopes west of the roadway are about 1.5 Horizontal to one Vertical (1H:1V) or steeper downwards towards the Piscataway Drive. The eastern slopes are generally 1.5H: 1V to 3H: 1V or gentler towards the Piscataway Creek. The slopes are generally covered with thick brush and large trees.

Soils when exposed appeared soft, moist and generally silty sands with organics. We did not observe any rock outcrops. Though it had rained the previous night, there was no evidence of surficial or ponded water except areas where underground water force main had cracked. Prior to the visit, the area had experienced high levels of precipitation over a short period of time.

We observed evidence of slope failure on both sides of the roadway. Additionally there were several cracks openings on the order of three to six inches in width in the pavement structure. The pavement edges had also settled several inches with the highest settlement of about four feet occurring around 13700 Piscataway Drive. The soil mass near the top of the hill had moved laterally downslope towards the roadway about two to three feet, and on the average, had settled approximately eight feet. Vertical cracks were visible due to this movement. Several trees had toppled, as a result of the slope failure, and had snapped the overhead utility lines. We observed evidence of past slope movement which appears to be surficial movement of the soil mass. At a nearby previously condemned two-story structure located at 13710 Piscataway Drive we observed evidence of lateral movement and settlement cracks. There were several fissures in the driveway and around the house. We observed several distresses in the foundation wall. We further observed that the driveway leading to the garages is no longer accessible due to ground movement.

Residents recall minor sloughing of the slopes which are consistent with our observations during the site reconnaissance. We, however, did not observe any evidence of past slope repairs nor were we provided any records indicating that. We did not observe any storm drainage system in the vicinity of the failed slopes.

Underground utilities consist of water and sewer mains with service power and other lines to the various premises within the site. There are overhead utility which consist of power, communications and cables lines.

2.2 GEOLOGIC SETTING

Based on a review of the Geologic Map of Prince George's County (2003), the site is underlain by unconsolidated sediments ranging in geologic times from Holocene to Lower Paleocene. It is dominated by relatively thick, tripartite Paleocene-Eocene section- the Aquia (Ta) and Nanjemoy (Tn) Formations separated by the 20 to 30 feet of Marlboro Clay (Tm) (Figures 3A & 3B, Appendix A). These three units have an aggregate thickness of about 300 feet in outcrop. Both the Aquia and Nanjemoy are variably muddy, fossiliferous greensands in contrast to the Marlboro which is a thin but persistent pinkish to gray plastic clay. The Paleocene-Eocene section includes about 500 ft. of sediment.

The Aquia is composed of sand, fine-to medium-grained, poorly sorted well sorted, containing as much as 40 percent glauconite. Thin layers of calcareous shelly sandstone are scattered through the unit giving it the "salt and pepper" speckled. It is generally greenish gray to medium gray in color.

The Nanjemoy consists of mostly quartz sand, fine-to coarse-grained, with a variable amount of interstitial silt-clay and as much as 50 percent of green glauconite, also imparting a "salt and pepper" aspect to the sediments. Poorer outcrops are found along the Piscataway Creek. The glauconite sand in this formation is medium-gray to dark greenish gray, where unweathered. The silty-clay is dark-gray to chocolate-brown in color.

The Marlboro Clay is a continuous stratum throughout Southern Maryland. It is poorly exposed, mostly because it is thin and covered by slumping of the overlying sediments. In the valleys of the Piscataway and Mattawoman Creeks, the clay is effectively buried Holocene alluvium. Scattered patches of typically brownish red Marlboro clays are exposed along MD 210 just north of Piscataway Creek in Prince Georges County. The Marlboro Clay is a thin but highly distinctive unit composed of dense, brittle clay, ranging from thickly-bedded to finely laminated, lenticular or hummocky in part, containing partings and thin lenses of micaceous and lignitic laminated silt. It is usually pale-red to silvery-gray, and contains minor interbedded silt which is yellowish gray to pale-gray in color.

2.3 SUBSURFACE EXPLORATIONS AND IN-SITU TESTING

KCI's sub-contractors, CenKen Group, LLC (CenKen) and Hillis-Carnes Engineering Associates, Inc. (HCEA) performed emergency subsurface explorations in the areas of the failing slope. The exploration program consisted of 15 standard penetration test (SPT) borings and 10 cone penetrometer test (CPT) soundings. Additionally, we installed six inclinometers and one groundwater monitoring well (piezometer). We conducted the subsurface explorations from May 6 to May 15, 2014 in accordance to the procedures presented in Appendix B. The depth of the explorations ranged from 40 feet to 100 feet into natural soils. The approximate exploration boring and tests locations are shown on Figure 2 in Appendix A. The boring logs and CPT are included in Appendix B.

2.3.1 Standard Penetration Test

We drilled test borings in general accordance with ASTM D420 procedures presented in Appendix B. The borings were advanced using ATV drill rigs equipped with hollow stem augers (HSA) and mud-rotary drilling in cased holes in general accordance with ASTM D1452. We conducted continuous standard penetration tests (SPTs) in the borings in general accordance with ASTM D 1586.

We performed standard penetration tests (SPT) borings in accordance with ASTM D1586. The SPT method consisted of advancing a two-inch diameter sampling spoon to a depth of 18 inches by driving it with a 140-pound hammer falling 30 inches. The values reported on the boring logs are the blows required to advance three successive six-inch increments. The first six-inch increment is considered as seating. The sum of the number of blows for the second and third increments is the "N" value. The "N" value is used to infer the general indications relative density and compressibility of the soils. KCI obtained soil samples using the SPT method and sampling was performed at two and half-foot intervals to a depth of ten feet below existing ground surface (bgs) and every five feet thereafter to boring terminations depth. We obtained representative disturbed soil samples during these tests and used them to classify the soils encountered. We placed the recovered representative soil samples in six-inch glass jars and transported to the laboratory for testing.

KCI geotechnical engineers visually classified the recovered soil samples in general accordance with ASTM D 2488 Standard Practice for Description and Identification of Soils. We classified soil samples with respect to texture in accordance with the Unified Soil Classification System (USCS). Boring logs describing the subsurface soils and groundwater conditions encountered at each of the boring locations are presented in Appendix B. The existing ground surface elevations indicated on the logs are based on field survey information provided by KCI-Survey.

2.3.2 Cone Penetration Test

We performed cone penetration tests (CPT) soundings in general accordance with ASTM D5778 at ten locations within the general project area between May 9 and 13, 2014. We use the results of the soundings to characterize the existing subsurface conditions within the unstable ground and slope areas. In addition, we performed localized pore pressure dissipation tests at test locations CPT-1 and CPT-5. The approximate test locations are shown on the attached Figure 2. We have provided summary tables soundings of the CPT results in Appendix B. We terminated the sounding depths at pushing refusals between 38 and 75 feet below existing ground surface, typically in excess of about 55 feet. We inferred soils in general accordance to Soil Behavior Types proposed by Robterson (1990).

We performed CPT tests in general accordance with ASTM D5778. CPT permits continuous explorations and profiling of the subsurface conditions while minimizing retrieval of subsurface materials. This exploration method employs sensors that are pushed into the ground to infer the properties of both soils and pore fluids. Known as direct-push technology, this method can map out the vertical and lateral extents of stratigraphic layers, as well as the distribution of groundwater conditions.

In combination with the test boring information, we will use the CPT results to identify loose/soft and disturbed soils strata and weak zones, and predict or confirm the existing failure planes at depth. Also, it will provide soil and groundwater data for characterizing the stress history and shear strength parameters of in-situ soil materials. By using standard engineering correlations, the geotechnical properties of stratigraphic layers can be inferred. Inferred properties include constrained modulus, undrained shear strength, residual shear strength, friction angle, overconsolidation ratio, and the coefficient of consolidation.

2.3.3 Undisturbed Soil Sampling

Split-barrel samples are suitable for visual examination and classification tests but are not sufficiently intact for quantitative laboratory tests requiring undisturbed samples. Therefore, we obtained relatively undisturbed samples in selected borings by drilling to the desired depth and hydraulically forcing a section of 3-inch O.D., 16 gauge steel tubing into the soil. The sampling procedure is described by ASTM D 1587. We carefully removed each tube, together with the encased soil, from the ground, made airtight and transported to the laboratory. The appropriate test boring records show depths of undisturbed samples.

2.3.4 Soil Conditions

Figures 4A, 4B, and 4C in Appendix A depict generalized subsurface profiles at the project site across the slope failures. The subsurface conditions encountered at the boring locations are shown on the test boring records in Appendix B. Also, the inferred subsurface conditions at the CPT sounding locations are shown on the CPT records in Appendix C. These test boring records and profiles represent our interpretation of the subsurface conditions based on visual examination of field samples and laboratory tests. The lines designating the interfaces between various strata on the test boring records represent the approximate interface locations. The actual transitions between strata may be gradual.

Consistent with the published geologic mapping, the borings and CPT soundings encountered three major natural strata underlying existing 6-inch thick asphalt pavement structure and Fill materials. The natural soils include an upper sand stratum (Nanjemoy, Tn Stratum) overlying Marlboro Clay (Tm Stratum) and Aquaia Formation (Ta Stratum). These strata are briefly described in the following paragraphs.

Existing FILL (F):
This two to six feet thick stratum was encountered typically below the existing asphalt pavement (borings B-1 through B-10) and at borings B-14, B-15 and B-17 (within the vicinity of an abandoned building structure). Existing FILL materials consisted of a heterogeneous mixture of brown to reddish brown Silty Sand, Clayey Sand and Gravels with deleterious materials such as asphalt fragments, decomposed wood and organics. Soft silt and clay materials were locally encountered at boring B-15. The SPT N-values ranged from 3 to 19 blows per foot (bpf) indicating very loose to medium dense, typically loose relative density.

Stratum I: Natural Silty SAND, Clayey SAND, Sandy SILT (Tn Stratum)

This stratum was encountered below existing Fill or occurred as the top stratum in several test borings up to a depth of about 15 feet bgs in the elevated upslope areas. It appears to thin out towards the low lying and downslope areas towards the wetlands and stream (e.g., in the general area borings B-1 and B-4, and from B-13 towards B-16, etc.) It generally consisted of moist, brown, light brown, dark gray, very loose to medium dense Silty Sand (SM), Clayey Sand (SC), coarse Sand (SP) with Gravels, and interbedded with soft to stiff Sandy Silt (ML) and Sandy Clay (CL) layers. The SPT blow counts ranged from 3 to 12 bpf indicating very loose to medium dense, typically loose relative density. Soils appeared to be slightly plastic.

Stratum II – CLAYS (Tm Stratum)

Marlboro Clay stratum was encountered below the Tn Stratum at each of the exploration locations. It varied in thickness from 15 to 30 feet with the base typically at approximate El. 78 and El. 74; and locally at approximate El. 50 at the lower topographic areas, and up to El. 135 at the higher elevations. It generally consisted of moist to wet, reddish brown, brown, light gray to gray, Lean Clay (CL) with occasional thin lenses of micaceous Silt. Locally, we encountered occasional Fat Clays (CH) within this stratum. The SPT N-values ranged from 3 to 14 bpf indicating generally soft to stiff compactness, typically medium stiff. The moisture content of the tested samples ranged from 14 to 48 percent. The Liquid Limit ranged from 26 to 59 percent with Plasticity Index (PI) ranging between 10 and 30 percent, indicating typically high to very high plasticity soils. We noted, however, that the clay soils appeared to be brittle.

Stratum III – SAND AND SILT (Ta Stratum)

This stratum was encountered below the Marlboro Clay to the boring termination depths. It generally consisted of moist to wet, olive gray, greenish gray to dark gray, Silty Sand (SM) and Sandy Silt (ML) with mica and calcareous shell fragments scattered throughout the stratum. The SPT N-values ranged from 5 to over 100 bpf (characterized by spoon refusals in the cemented layers), indicating generally loose to very dense relative density. The loose zones appeared to occur at the interface with the Marlboro Clay. The relative density appears to be typically medium dense to dense compactness, and/or stiff to hard compactness. The moisture content of the tested samples ranged from 20 to 30 percent, with non-plastic to slight plasticity (PI less than 4 percent).

2.3.5 Groundwater and Cave-in Conditions

We observed and recorded groundwater and cave-in depth information in each boring during drilling (within the drill augers), and several hours after completion of drilling (and removal of the augers). In addition, we have installed a piezometer near Boring B-2 to record long term groundwater levels. Table 2-

1 below provides a summary of groundwater conditions and cave-in depths. Where encountered, groundwater and/or perched water conditions generally occurred at depths between 10 and 60 feet bgs.

Cave-in occurred at depth between 14 and 65 feet bgs following removal of the drill augers. Cave-in may be due to the collapse of soils after removing augers at the completion of drilling. However, in granular soils, cave-in depths may be due to the presence of saturated soil conditions arising from groundwater and/or perched-water conditions.

Because of the presence of clayey and silty nature (characterized by relatively impermeable conditions) within portions of the site soils, site soils have the potential of developing perched water conditions. In addition, seasonal and/or long-term fluctuations of the groundwater levels and/or perched water may occur due to variations in rainfall, evaporation, soil capillary, construction activity, ground conditions and surface runoff, and other site-specific factors, and should be anticipated.

Table 2.1: Summary of Groundwater Condition

Boring Nos.	**Groundwater Levels**				**Cave-in Depths (ft)**
	Depth (ft) (in augers)	**Depth (ft) (0 hr)**	**Depth (ft) (>24 hrs)**	**Elevation (ft)**	
B-1	43.8	57.8	-	11.8	14
B-2	31.2	11.1		63.3	21
Observation Well	-	36.7	37	37.6	-
B-3	48	42	10	69.7	18
B-4	-	59	34.3	63.3	36
B-6	Mud rotary drilling				
B-7	20.5	12	11	104.0	65
B-8	Mud rotary drilling				
B-9	18	17	15.5	105.2	
B-10	Mud rotary drilling				
B-11	Dry	-	-	-	92

Table 2.1: Summary of Groundwater Condition

Boring Nos.	Groundwater Levels				Cave-in Depths (ft)
	Depth (ft) (in augers)	Depth (ft) (0 hr)	Depth (ft) (>24 hrs)	Elevation (ft)	
B-13	Mud rotary drilling				
B-14	54.8	36	9.8	99.6	25
B-15	67.5	59	10.1	97.8	28
B-16	Mud rotary drilling				
B-17	Mud rotary drilling				

Piezometers

KCI subcontractor, CenKen, installed one piezometer near Boring B-2 on May 13, 2014 to monitor long term groundwater levels. The screen was installed at between 35 to 50 feet below the existing ground surface. A KCI engineer obtained the initial water-level reading on May 13, 2014 using a groundwater monitoring meter. We plan to perform daily readings to monitor the long-term fluctuations of the water table at that location.

We installed and have been monitoring the groundwater levels in general accordance with ASTM D5092. The details of general installation procedures are provided in Appendix B.

2.4 SLOPE MOVEMENT MONITORING

Our subcontractor, CenKen, installed six inclinometer casings from May 9 to May 13, 2014 to monitor further slope movements. We installed the casings at an average depth of 70 feet below the existing slope surface. A KCI engineer commenced obtaining the baseline inclinometer readings from May 12 and 13, 2014 using a probe-type inclinometer. We plan to perform daily inclinometer readings to determine potential progressive slope movements prior to the slope stabilization. We will provide the results of our slope monitoring along with final recommendations in a brief memorandum within two weeks from our last survey.

We are monitoring the slope movement in general accordance with ASTM D 6230. The details of general installation procedures and typical inclinometer survey procedures are provided in Appendix B.

2.5 LABORATORY TESTING

We performed laboratory testing on representative soil samples (from disturbed jar samples and undisturbed Shelby Tube samples) to confirm visual soils classifications and to determine physical properties of in-situ soils. The laboratory tests were conducted in general accordance with ASTM standards and included the following:

	No. of Tests
• Natural Moisture Content (ASTM D 2216)	42
• Classification Tests, including:	
- Atterberg Limits (ASTM D 4813)	29
- Sieve Analysis (ASTM D 422)	24
• Direct Shear Test (ASTM D 3080)	5
• CIU Triaxial Test (ASTM D 4767)	1
• One-Dimensional Consolidation Test (ASTM D 2435)	1

We have provided details of laboratory testing procedures and the laboratory test results in Appendix C. Due to the slope failure and unstable ground issues associated with the presence of the Marlboro Clay stratum at the project site, we performed laboratory testing to determine shear strength parameters (undrained direct shear, DS and consolidated undrained Triaxial, (CIU) and deformation characteristics (one dimensional consolidation) of the Tm stratum. Table 2-2 provides a summary of the shear strength test results.

Table 2-2: Summary of Shear Strength Results for Marlboro Clay (Tm)										
Boring No.	**Sample**	**Test Type**	**USCS**	**Cohesion c', (psf)**	**Friction Angle, ^{0}**	**Moisture Content, (%)**	**Unit Weight, (pcf)**	**LL %**	**PI %**	**Fines %**
B-13	ST-1 (22'-24')	DS	ML	997	29.3	36	115	48	18	78
B-13	ST-2 (28'-30')	DS	CL	473	22.4	32	118	39	14	100
B-14	ST-1 26.5'-28.5')	DS	CL	650	14.2	27	122	38	16	100
B-15	ST-1 (22'-24')	DS	CL	11.3	31	35	117	40	16	90
B-15	ST-1 (22'-24')	CIUC	CL	130	18.4	33	121	40	16	90
B-17	ST-1 (22'-24')	DSR	CL	759	29.7	44	116	47	28	74

*DS=Direct shear testing conducted at 0.01 in/minute shearing rate without residual cycles

**DSR = Direct shear testing conducted at 0.01 in/minute shearing with residual cycles

3.0 GEOTECHNICAL EVALUATIONS

3.1 SLOPE STABILITY ANALYSES

KCI performed preliminary global stability analyses for the pre-existing failure conditions of the slopes. This enabled us to back-calculate the critical shear strength parameters of the Marlboro Clay (Tm Stratum) under marginal stability conditions (defined by Factor of safety, FS = 1.0 or less). Based on the results of the subsurface explorations, we developed a typical subsurface profile for a critical slope section for our analyses as depicted in Appendix D. We have assumed that the phreatic water level was developed in the upper Tn (Stratum I) during slope failure.

We selected preliminary design soil parameters based on the field and laboratory test results, and our experiences with similar soil materials. We used the General Limit Equilibrium/Morgenstein-Price (GLE) method for the slope stability analyses to satisfy both force balances and moment balances of soil slices in order to find the most critical slip surface and the minimum factor of safety (FS) of the slope. We utilized both circular slip search and block slip search for the back analyses. We conducted our slope stability analyses using the software Slide Version 6.029 developed by RocScience Inc. We have analyzed several slope scenarios as part of the back-calculation evaluations using the following laboratory soil parameters and slope conditions as summarized in Table 3.1.

Table 3.1: Definition of Back Analyses Cases

	Soil Properties: Marlboro Clay (Tm Stratum)	
Assumed Slope Conditions	**C' (psf)**	**ϕ' (°)**
A. Groundwater depth at 10 feet and rear tension cracks	130	18
B. Groundwater depth at 10 feet and no tension cracks	130	18
C. Groundwater depth at 5 feet and rear tension cracks	130	18
D. Groundwater depth at 5 feet and no tension cracks	130	18
E. Groundwater depth at 10 feet and rear tension cracks	130	14
F. Groundwater depth at 10 feet and no tension cracks	130	14

Table 3.1: Definition of Back Analyses Cases		
	Soil Properties: Marlboro Clay (Tm Stratum)	
Assumed Slope Conditions	**C' (psf)**	**ϕ' (°)**
G. Groundwater depth at 5 feet and rear tension cracks	130	14
H. Groundwater depth at 5 feet and no tension cracks	130	14

We have provided detailed of our slope analyses in Appendix D. The results of our preliminary slope stability analyses are summarized in Table 3.2.

Table 3.2: Summary Results of Pre-Failure Slope Analyses					
Case	**Hw**	**Tension Cracks Exist**	**ϕ' (°)**	**c' (psf)**	**FS**
A	10	Yes	18	130	1.02
B	10	No	18	130	1.13
C	5	Yes	18	130	0.91
D	5	No	18	130	0.98
E	10	Yes	14	130	0.84
F	10	No	14	130	0.97
G	5	Yes	14	130	0.77
H	5	No	14	130	0.78
H_w = Vertical height of water below the existing ground surface FS = Minimum Factor of Safety					

The results of our preliminary analyses confirmed that slope failure likely occurred under fully saturated slope conditions within the overburden Tn stratum and Marlboro Clay as indicated by the laboratory testing data. Pending additional testing, we recommend that residual soil shear strength from the CIUC test (cohesion, c'= 130 psf, friction angle = 18 degrees) be used for the Marlboro Clay in preliminary evaluations of slope stabilization options. Also the groundwater level should be set at 5 feet or less below grade for design stabilization efforts.

3.2 POTENTIAL CAUSES OF THE EXISTING LANDSLIDE

There are several causes such as, geological, morphological, physical and human activity that can render a site susceptible to landslide and ground movements. When such conditions exist, only one trigger is needed to cause the slope to fail/slide. Trigger is an external stimulus such as intense rainfall and storm water infiltration, earthquake shaking, volcanic eruption, storm waves, or rapid stream erosion that caused a near-immediate response in the form of a landslide by rapidly increasing the imposed stresses or by reducing the strength of slope materials due to significant pore pressure developments within saturated soils.

Based on our preliminary site evaluations and analysis and our review of historic information, the geology of the site, in particular the presence of the Marlboro Clay, made it susceptible to landslide and ground settlements. The trigger was intense and rapid infiltration of rainfall that occurred prior to the slope failure.

Our post-failure subsurface explorations confirmed that three geologic formations are present at the site. Of particular concern is the Marlboro Clay which is sandwiched between the upper Nanjemoy and the lower Aquia formations. Historic information (Pomeroy, 1988, *Map Showing Landslide Susceptibility in Maryland, USGS Miscellaneous Field Studies Map MF-2048*) suggests that Marlboro Clay is one of the Coastal Plain geologic formations highly susceptible to slope failures. Localized and mass ground movements associated with slumps and earthflows are known to be associated with Marlboro Clay with numerous slope failures having occurred in south-western and east-central Prince George's County.

During wet periods as rainfall percolates downward through the overlying permeable sandy and silty soils, it encounters the relatively impermeable Marlboro Clay layer. The microstructure of Marlboro clay makes it difficult for water to infiltrate. Thus, infiltrated water will move primarily along the surfaces of the clay layers. Over time, this water may gradually dissipate with little easing of the pore-pressures and causing little or no slope movements. However, during the recent intense and rapid rainfall recorded at the project site, the infiltrated water was not able to quickly dissipate in the Marlboro Clay and generated massive pore-pressure built up in the saturated sediments. These high pore pressures resulted in shear strength degradation and creating weak subsurface zones with significant reduction in the frictional resistance along the contacts between the saturated soil and the Marlboro Clay. This condition produced new slide surfaces and potentially regenerated existing failure planes leading to the on-going slope failures and landslide at Piscataway Drive.

As depicted on the Subsurface Profiles Figures 4A, 4B and 4C (Appendix A), we have estimated approximate depths of the landslide and slope failure planes based on the test borings and CPT soundings and the residual strengths from laboratory testing. Our visual examination of extracted undisturbed Shelby tube sample ST-1 from boring B-15 provided evidence of a near horizontal failure plane between depths of 23.2 and 23.6 feet bgs, corresponding to approximate El. 85 (See Figure No. 5 in Appendix A). In addition, during drilling at boring B-17, we encountered loss of drilling fluid mud between depth of 25 and 26 feet bgs (approximate El. 85). This may be indicative of a failure plane.

4.0 GEOTECHNICAL RECOMMENDATIONS

4.1 GENERAL

The material within the landslide area has been weakened by the movement of soil mass and has thus lost some amount of shear strength. Also, our test results indicate that pore-pressures have not dissipated, hence, the continual recorded movement. Furthermore, with lots of crack openings within the site, infiltration of water will generate more pore-pressure and further destabilizing the slopes and causing more movement. Thus, the failed slopes have to be repaired immediately.

4.2 SLOPE STABILIZATION OPTIONS

To stabilize the failed slopes, KCI examined several methods and have performed preliminary analyses on three. We are proposing three preliminary alternatives for stabilization of the failed slopes and landslide areas at the project site as presented in Table 4-1. The conceptual designs of the stabilization alternatives are also provided.

As discussed previously, the major geotechnical issue relates the presence of saturated overburden soils overlying the impermeable Marlboro Clay which is known to be susceptible to landslides and slope failures. The interface between the overburden soils and clay strata loses significant frictional resistance when subjected to undrained conditions due to water infiltration leading to pore pressure build-up. The resulting loss of shear strength indicates that there is insufficient resistance along the interface to resist driving forces thus leading to slope instability.

In order to stabilize the slope and mitigate ground movements, measures should be taken to provide additional resistance and reduce slope driving forces risk to minimize the risk to public properties and life. Note that the proposed slope stabilization schemes are designed to stabilize the upper slope portions above Piscataway Drive roadway and protect the roadway. Note that we did not provide stabilization for the slope portion further downhill toward the river due to the anticipated lower risk to public properties and lives.

Table 4.1: Summary of Proposed Preliminary Slope Stabilization Options		
Option	**Grade & Backfill**	**Structural Element Support**
1	Backfill slopes (3H:1V) above roadway and support using an 8-foot high soldier-lagging wall	Drilled Shaft Foundation and Micropile (Mini-pile) Anchors
2	Limited Slope Regrading	Drilled Shaft and Micropile
3	Limited Slope Regrading	Micropile

Option 1: This alternative includes a combination of ground stabilization partial backfill and mid-slope stabilization and protection. This method involves the installation of two rows of drilled shaft foundations along both sides of the Piscataway Drive, a retaining wall with backfill, and two rows of micropiles (mini-piles) near the existing head scarp. These reinforcements will be extended beyond the failure surfaces. This stabilization is associated with the installation of structural elements with high strength, which introduce forces to oppose movement and support the sliding mass, resulting in stabilization of the landslide. Partial slope backfilling supported with a retaining wall along the roadway to stabilize the toe of slope. We have provided details of the conceptual design on Figure D-9 in Appendix D.

Option 2: This alternate involves ground stabilization using drilled shafts along the roadway and slope reinforcement using micropiles along the entire western side to reinforce the failed slopes with limited regrading. On the eastern slopes, we recommend one row drilled shafts installed beyond the failure surface and embedded in Ta Stratum. We have provided details of the conceptual design on Figure D-10 in Appendix D.

Option 3: This alternate is similar to Option 2; however, we use only micropiles for both ground stabilization and reinforcement with limited regrading. The method involves the installation of micropiles throughout the slopes on both sides of Piscataway Drive. The mini piles will be extended beyond the failure surface to a minimum depth of 50 feet into Ta Stratum. We have provided details of the conceptual design on Figure D-11 in Appendix D.

Our analyses indicate that each of the options will adequately stabilize the slopes and mitigate additional movements within the vicinity of improvement. However, the drilling and grouting equipment used for micropile installation is relatively small and can be mobilized in constrained and restrictive areas that would prohibit the entry of conventional heavy drilled shaft-installation equipment. In addition, micropile installation will not be impacted by overhead power lines or other obstructions as are conventional drilled shaft systems. The equipment can be mobilized up steep slopes and in remote locations. Also, drilling and grouting procedures associated with micropile installations will not cause significant site disturbance or damage to adjacent existing structures and buried utility mains when proper drilling and grouting procedures are used.

We anticipate that the resulting ground movements indicated by the inclinometer readings will have significant implications for the slope rehabilitation options. Therefore, we will revise the proposed stabilization options accordingly, and recommend that additional detail analyses and design, constructability evaluations and cost analyses be performed for each option as part of the final design purposes.

4.3 UTILITY COORDINATION AND RECONSTRUCTION RECOMMENDATIONS

The Utility Coordination efforts should continue and should include meeting and talking with each utility company to discuss the impacts to their facilities and potential mediation once the slope is stabilized.

WSSC Facilities: The existing eight-inch Ductile Iron Pipe (DIP) water main (12/20/02 Contract) and the eight-inch Concrete Sanitary Line (6/1/70 Built date) will need to be replaced within the proposed length of the roadway reconstruction (approximately 1,500 linear feet). KCI recommends that both lines be replaced within the existing footprint location in relation to the existing roadway. Prior to the soil failure the water and sanitary house connections ran under the failing slope; these connections collapsed during the failure event. KCI recommends that after the soil stabilization the replacement design should incorporate the use of a carrier pipe. A design will avoid the need to have the services running through the selected stabilized slope treatment.

KCI recommends the proposed water main and sanitary sewer replacement work be performed under the same construction contract.

Electric, Cable TV (CATV) and Telephone Facilities: PEPCO previously maintained a pole line along the southern edge of paving of Piscataway Drive which carried a single phase primary electric cables as well as third parties; COMCAST and Verizon cables. Temporarily the electric line has been de-energized and picked up from the broken poles and lifted to avoid danger to the crews working in the area. PEPCO is evaluating a temporary and permanent solution based upon the method and implementation of the slope stabilization.

Initially, it is anticipated that the impacted single phase pole line be reconstructed in a similar alignment and fashion as the system prior to the slope failure. The downstream and upstream poles should be evaluated in relation for vertical lift and tension impact sustained during the event and change pending line and grade. KCI recommends the collapsed service pole which was carrying the electric, CATV and telephone underground services be relocated along the common driveway of the impacted properties to avoid services running through the selected stabilized slope treatment.

4.4 ENVIRONMENTAL COORDINATION

On May 12, 2014, KCI performed wetland delineation within the vicinity of Piscataway Drive in Fort Washington, Maryland. KCI identified one palustrine forested wetland at the base of the slope below Piscataway Drive, as well as two associated stream channels, designated intermittent and perennial, respectively. KCI contacted the Maryland Department of the Environment (MDE) and the US Army Corps of Engineers on May 16, 2014 and the agencies concurred that the work constitutes an emergency.

MDE specified that if any access through regulated resources is needed in order to complete the repairs, a Joint Permit Application (JPA) must be submitted within 30 days. Impacts to wetlands and waterways should be minimized to the amount necessary to repair the slope. KCI contacted the Chesapeake Critical Areas Commission (CAC) on May 15, 2014 to make them aware of the ongoing emergency activities. A CAC letter will be prepared during final design.

4.5 ROADWAY RECONSTRUCTION AFTER THE SLOPE STABILIZATION PROCESS HAS BEEN COMPLETED

Utilizing the topographic survey, KCI will develop a baseline that will closely match the centerline of the existing roadway. This baseline will serve as the centerline for the reconstructed roadway. KCI will generate and evaluate the existing roadway profile since portions of the roadway have settled significantly. We will generate a revised roadway profile for the posted 25 mph per American Association of State Highway and Transportation Officials (AASHTO) – A policy on Geometric Design of Highways and Streets (Chapter 5: Local Roads and Streets). KCI will develop a typical Rural Secondary Residential roadway section for a 22-foot wide crowned roadway with 2% roadway cross-slopes. We will vary roadside grading to closely match the condition prior to the slope and roadway failure to reduce impacts to the existing residences. KCI will generate existing ground cross-sections with the proposed new roadway section superimposed to develop grading limits and earthwork requirements. We will place impermeable side ditches where necessary to divert the sheet flow of water away from the roadway into existing or proposed cross pipes.

KCI anticipates that during construction, once the slopes are stabilized (and all major construction equipment is no longer required to utilize the existing roadway), the existing pavement will be thoroughly broken up, scarified or removed. The embankment and subgrade will be placed along with any ditch and required cross pipes (existing cross pipes shall be cleaned). We will use the Prince George's County pavement section, or provide a recommended pavement design including a six-inch underdrain along both roadway edges. The underdrain will be outlet to the fill slopes. Guardrail will be required along the east side of the roadway for most if not the entire length of the reconstruction. Curbing may also be placed along the east side of the roadway to divert water away from the fill slopes to curb openings and stabilized slope channels.

5.0 STRATEGY FOR MOVING FORWARD

This report provides a preliminary concept design prepared after reviewing the feasibility of several options. We have developed the recommended concept to an approximate 20% design stage. KCI will now work with Prince George's County DPW&T to consider option for moving forward with the recommendation contained in this report.

6.0 BASIS OF RECOMMENDATIONS

General

1. This report has been prepared to aid in the evaluation for the proposed construction described in this report. Adequate recommendations have been provided to serve as a basis for design and preparation of plans and specifications. The opinions, conclusions, and recommendations contained in this report are based upon our professional judgment and generally accepted principles of geotechnical engineering. Inherent to these are the assumptions that the earthwork and foundation construction should be monitored and tested under the guidance of a geotechnical engineer licensed in the State of Maryland or his representative.

Explorations

2. The analyses and recommendations provided are, of necessity, based on project information available at the time of the actual writing of the report, including existing site, surface and subsurface conditions that existed at the time the exploratory borings were drilled. Further assumption has been made that the limited exploratory borings, in relation to both the lateral extent of the site and to depth, are representative of general conditions across the site. The nature and extent of variations between these explorations may not become evident until further explorations and construction. If variations from anticipated conditions then appear evident, it will be necessary to revise the recommendations in this report.

3. The soil strata described in the text and indicated on the subsurface profiles are intended to convey generalized trends in subsurface conditions. Boundaries between strata are approximate and idealized, and developed by interpretations of widely spaced explorations and sampling; actual soils transitions are probably more erratic. Refer to boring logs for specific information.

4. Groundwater level readings have been made in the drill holes at times and under conditions stated on the boring logs. These data have been reviewed and interpretations have been made in this report. Fluctuations in the level of the ground water may occur due to variations in rainfall, temperature, and other factors occurring since the time measurements were made.

Review

5. This report has been prepared based on plans and description of the proposed construction cited herein. In the event that any changes in the nature, design or location of the proposed construction, the conclusions and recommendations contained in this report shall not be considered valid unless the changes are reviewed and conclusions of this report modified or verified in writing by KCI. We recommend that KCI be provided the opportunity for a general review of design and specifications so that our recommendations may be properly interpreted and implemented in the design specifications.

Uses of Report

6. This final report has been prepared for the exclusive use of Prince Georges County Government and other members of the design team for specific application of the Engineering Design services for the **Piscataway Drive Slope Failure,** Fort Washington, Maryland. Our professional services were performed in accordance with generally accepted soil and foundation engineering principles and practices; no other warranty, expressed or implied, is made. KCI assumes no responsibility for interpretations made by others on the work performed by KCI.

7. In the event the County proceeds forward with construction, this report is for design purposes only and is not sufficient to prepare an accurate bid. Contractors wishing a copy of the report may secure it with the understanding that its scope is limited to design considerations only. We recommend that this report be made available in its entirety including attachments and appendices to contractors for informational purposes only. The project plans or specifications should include the following note:

A geotechnical report has been prepared for this project by KCI Technologies, Inc. This report is for informational purposes only and shall not be considered as part of the contract documents. The opinions and conclusions of KCI represent our interpretation of the subsurface conditions and the planned construction at the time of the report preparation.

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