GEO-TECHNOLOGY ASSOCIATES, INC.

GEOTECHNICAL AND
ENVIRONMENTAL CONSULTANTS

A Practicing Geoprofessional Business Association Member Firm

August 11, 2016

Toll Brothers, Inc.
516 North Newtown Street Road
Newtown Square, PA 19073

Attn: Mr. Gary Chase

Re: Preliminary Geotechnical Exploration
Crebilly Farm
Westtown Township, Chester County, Pennsylvania

Gentlemen:

In accordance with our agreement dated June 6, 2016, Geo-Technology Associates, Inc. (GTA) has performed a preliminary geotechnical exploration for the proposed residential subdivision, located on the Crebilly Farm property in Westtown Township, Chester County, Pennsylvania. The exploration consisted of performing 40 Standard Penetration Test (SPT) borings and excavating 23 test pits within the proposed roadway and stormwater management areas, performing preliminary infiltration testing within the proposed stormwater management (SWM) facility locations, examining the encountered materials for engineering classifications, and performing limited laboratory testing. The results of field and laboratory testing and preliminary recommendations regarding design and construction of the proposed subdivision and associated SWM facilities are included in this report.

We appreciate the opportunity to have been of assistance to you on this project. Should you have questions, please contact our office at (302) 326-2100.

Sincerely,
GEO-TECHNOLOGY ASSOCIATES, INC.

Edwin F. Williams
Associate

Christopher M. Reith, P.E.
Principal

COMMONWEALTH OF PENNSYLVANIA
REGISTERED PROFESSIONAL
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TAH/EW/CMR/amd
161348
Attachments

REPORT OF PRELIMINARY GEOTECHNICAL EXPLORATION

CREBILLY FARM
Westtown Township, Chester County, Pennsylvania

August 2016

Prepared For:

TOLL BROTHERS, INC.
516 North Newtown Street Road
Newtown Square, PA 19073

Attn: Mr. Gary Chase

Prepared By:

GEO-TECHNOLOGY ASSOCIATES, INC.
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WESTTOWN TOWNSHIP, CHESTER COUNTY, PENNSYLVANIA
AUGUST 2016

INTRODUCTION

This report presents the results of our preliminary geotechnical exploration performed on the property located in the northwest corner of West Street Road and Wilmington Pike in Westtown Township, Chester County, Pennsylvania. We understand that Toll Brothers is considering the purchase of the subject site for construction of a residential community. The gross site area is 322.4 acres. GTA was proved two concept plans designated Plan B and C, prepared by Eastern States Engineering. The concept plans indicate the two alternate layouts of the subdivision and stormwater management areas. The proposed subdivision will be a mix of estate/executive lots, executive/courtyard lots, and carriage homes. According to the concept plans, the house totals range from 300 to 347 units. The plans included boundary information, potential lot and roadway configuration, and the locations of the proposed stormwater management facilities. Proposed and existing grades and utility locations were not provided for our review at the time this report was prepared.

In conjunction with the proposed development, Geo-Technology Associates, Inc. (GTA) was retained to perform a preliminary geotechnical exploration of the project site. The scope of this study included a field exploration, laboratory testing, and engineering analysis. Included in our field exploration were SPT borings performed at 40 locations to scheduled depths of 15 feet below the existing ground surface, test pits excavated at 22 locations to depths of approximately 7½ to 11½ feet below the existing ground surface, and field infiltration testing at 13 locations. Limited laboratory testing was performed to confirm the visual classifications and characterize general subsurface conditions. Conclusions and recommendations regarding the site development were derived from engineering analysis of field and laboratory data, and review of the previously referenced concept plans. It should be noted that structural details and final site grading or utility plans were not available at the time our exploration was performed. As such, GTA recommends that a design phase geotechnical review of the site be performed upon finalization of the site layout to verify that geotechnical considerations are addressed.

SITE DESCRIPTION

The subject site is located northwest of the intersection of Wilmington-West Chester Pike (Route 202) and West Street Road (Route 926), in Westtown Township, Chester County, Pennsylvania, as shown on the *Site Location Map, Figure 1*, included in Appendix A. Specifically, the subject site is comprised of eleven lots, identified as Tax Parcels 67-4-029, 67-4-029.1 through 67-4-029.4, 67-4-030 through 67-4-033, 67-4-033.1, and 67-4-134, totaling approximately 322.4 acres. The site is bound by West Pleasant Grove Road followed by residential properties to the north, by West Street Road followed by residential and agricultural properties to the south, by Wilmington-West Chester Pike followed by residential and commercial properties to the east, and South New Street followed by agricultural property and wooded land to the west. At the time the field exploration was performed, the subject site was primarily an undeveloped property containing a few single-story and two-story residential structures, barns, stables, and horse training facilities in the central and western portions of the site. The site also contained asphalt paved and gravel driveways and various utilities associated with the existing structures. GTA understands that there were residential structures on the eastern portion of the site that were demolished. Remnant slabs and demolition debris were present on this portion of the site at the time our field exploration was performed.

The eastern and northwestern portions of the site were utilized for agricultural crop production. The northeast, eastern-central, and southwestern portions of the site is comprised of wooded property containing small to large trees and light to moderate underbrush. A small pond was located in the south-central portion of site. A stream, identified as Radley Run, is located in the southwest portion of the site near the farm entrance. Additionally, unnamed tributaries and associated wetlands were observed in the southern-central and northwestern portions of the site bisecting the property. The site topography is generally gently to steeply sloping, with surface drainage generally directed toward Radley Run and the unnamed tributaries to the south and west. Ground surface elevations range from approximately elevation (EL) 380 in the southeast corner of the site, near the residential structure located adjacent to the intersection of West Street Road and Wilmington-West Chester Pike, to approximately EL 250 in the southwest portion of the site, near Radley Run. Ground surface elevations were based on Google Earth Imagery from 2011 and limited survey data and should be considered approximate.

PROPOSED CONSTRUCTION

Based on the concept plans provided by Eastern States Engineering, GTA understands that the subject property is planned to be developed as a residential community, which will include up to 347 homes. The subdivision will have access points from Wilmington-West Chester Pike, West Pleasant Grove Road and West Street Road; and a network of interior roadways and cul-de-sacs will provide access to the residential units. The proposed site grading and utility plans were not available at the time our exploration was performed. GTA anticipates that significant cuts and fills will be required for general site grading, given the moderately to steeply sloping topography and significant relief changes on portions of the property. Also, it is anticipated that deep excavations may be required for basement construction and utility installation. Specific building construction types and structural loading information were not available at the time this report was prepared. For the purposes of this report, the structures are assumed to be cast-in-place concrete and timber frame construction. Based on previous projects of similar scope, maximum wall loads of four kips per linear foot and maximum column loads of 20 kips were assumed in our analyses.

The preliminary concept plans for stormwater management (SWM) within the proposed subdivision includes construction of 13 basins located at various points across the planned community. At the time our investigation was performed, specific details regarding the types of stormwater management practices were not available. It is anticipated that best management practices for water quality and quantity management will be utilized to comply with Pennsylvania and Chester County specifications and regulations regarding stormwater design.

RELEVANT GEOLOGY

According to *The Preliminary Bedrock Geologic Map of a Portion of The Wilmington 30- by 60- Minute Quadrangle, Southeastern Pennsylvania*, published by Pennsylvania Department of Conservation and Natural Resources (2005), the subject site is primarily located within the Glenarm Wissahickon formation of the Piedmont Physiographic Province. Specifically, the map indicates that the majority if the site is underlain by the Doe Run schist which is identified as garnet-staurolite-kyanite pelitic schist with abundant biotite and muscovite. The residual soils resulting from the weathering of the parent bedrock of the Doe Run schist can result in low plasticity silts and clays transitioning to non-plastic sands with lesser percentages of silt and clay. These materials generally become increasingly stiff or dense with depth; although, differential weathering can often result in softer zones within otherwise very dense weathered rock material.

The above-referenced bedrock geology map also indicates that a small potion site along the western property boundary may underlain by Ultramafic rock, which is described as primarily serpentinite containing magnesium-rich rocks derived from pyroxenite and peridotite. The residual soils resulting from the weathering of the parent bedrock of the Ultramafic rock can result in high plasticity soils with low unit weights.

According to the U.S. Department of Agriculture (USDA) web soil survey, the soils underlying the site are mapped as the Glenville silt loam (GlB, GlC), Glenelg silt loam (GgB, GgC), Chester silt loam (CdB), Baile silt loam (Ba), Codorus silt loam (Co), Gaila silt loam (GaD) and Hatboro silt loam (Ha) series soils. The Glenelg, Chester and Gaila series soils are described as being well-drained, with depths to the water of more than 6 feet and a depth to bedrock generally ranging from 60 to 120 inches. The Glenville and Codorus series soils are described as being moderately well-drained, with depths to the water of approximately 6 to 36 inches and a depth to bedrock generally ranging from 15 to 99 inches. The Baile and Hatboro series soils are described as being poorly-drained, with depths to the water of approximately 0 to 6 inches and a depth to bedrock generally ranging from 60 to 99 inches. These soils were typically mapped in the low lying areas. Refer to the publications for additional information.

SUBSURFACE EXPLORATION

The subsurface conditions at the subject site were explored by performing SPT borings at 40 locations and excavating test pits at 22 locations. The test borings, identified as Borings B-1 through B-40, were drilled from July 27 through August 2, 2016 at various points along the proposed roadway alignments to scheduled depths of 15 feet below the existing ground surface. The test pits, identified as TP-1 through TP-13, were excavated on July 25 through 26, 2016 within the proposed SWM areas and proposed roadway alignments to depths of approximately 7½ to 11½ feet below existing ground surface. The test pits were excavated by R. Keating and

Sons, Inc. using a Case 580 Super M backhoe. The test pit and boring locations were field surveyed by representatives of Northeast Surveyors, LLC with the approximate locations depicted on *Figure 2: Exploration Location Plan,* included in Appendix A.

The test borings were drilled on July 27 with an ATV-mounted Diedrich D50 drill rig equipped with 3¼-inch hollow-stem augers and an automatic hammer. Standard Penetration Testing was performed in the boreholes with sampling performed at approximate 2-foot intervals in the upper 10 feet of drilling and at 5-foot intervals thereafter. Standard Penetration Testing involves driving a 2-inch outside diameter (O.D.), 1⅜-inch inside diameter (I.D.) split-spoon sampler with a 140-pound hammer free-falling 30 inches. The number of blows required to drive the sampler was recorded in intervals of 6 inches. The total number of hammer blows required to drive the sampler from the 6 to 18-inch interval is the SPT N-value.

The soil samples retrieved from the test pits and borings were delivered GTA's laboratory for visual classification by engineering personnel and limited laboratory testing. The soil descriptions indicated on the logs are based on visual observations using the Unified Soil Classification System (USCS) of the individual soil samples as summarized in the *Notes for Exploration Logs* included in Appendix B, supplemented by the laboratory test results.

SUBSURFACE CONDITIONS

In agreement with the published geology, the test borings and test pits typically encountered surficial topsoil underlain by residual soils consistent with the Glenarm Wissahickon Formation throughout the maximum depths explored. Topsoil/cultivated soil was encountered at the ground surface of the exploration holes and measured about 2 to 15 inches thick. Below the surficial topsoil, Boring B-28 encountered existing fill materials comprised of silt and sand mixtures with lesser amounts of rock fragments. The existing fill was encountered to a depth of approximately 2 feet below the ground surface and is likely native material that was placed for construction of the local farm road where Boring B-28 was performed. It was also located near the residential dwellings that have been razed.

Below the topsoil and/or existing fill, the borings and test pits encountered fine-grained residual soils visually classified as silts and clays with lesser percentages of sand and rock fragments to depths of approximately 2 to 9½ feet blow existing grades. Underlying the fine-grained soils, the borings and test pits typically encountered granular residual soils visually classified as silty sand with varying amounts of rock fragments, generally transitioning into highly weathered rock. Highly weathered rock was encountered at boring locations B-5, B-9, B-11 through B-16, B-18 through B-20, B-23 through B-27, B-34, B-35, and B-37 through B-40 at depths of approximately 8½ to 14 feet below ground surface. At several locations the drill rig was able to auger through the weathered rock. Auger refusal was not encountered to the explored depths. Highly weathered rock was also identified at Test Pit locations TP-2, TP-5, TP-8, and TP-12 through TP-17 at depths of approximately 7 to 10½ feet below ground surface. Refusal of the excavation equipment was encountered in TP-2, TP-8, and TP-12 through TP-17 at depths of about 7½ to 10½ feet below the existing grades.

Uncorrected SPT N-values for the encountered surficial fine-grained soils ranged from 2 to 14 blows per foot, averaging 6 bpf, indicating these soils are generally medium stiff. The uncorrected N-Values for the granular materials ranged from 4 to 50, averaging 16, which indicates the soils were generally medium dense. The silty sands generally transformed into highly weathered rock materials with uncorrected N-values of 50 or more blows per increment. Hard augering and excavation difficulties were also experienced in the highly weathered rock at depths ranging from about 5 to 13 feet below existing grades and as indicated on the logs.

Groundwater was observed at 11 of the exploration locations at depths ranging from 4.4 to 12.9 feet below the ground surface, corresponding to elevations ranging from approximately EL 302 to 337. The remaining test locations were dry to their cave depths or termination depths. Groundwater levels were recorded during the exploration and again prior to backfilling the exploration holes. Most of the test borings were left open to collect 24-hour groundwater measurements; however, test borings conducted within the horse pastures and the test pits were backfilled upon completion for safety considerations. The observed water levels in the higher areas of the site are likely perched water trapped in sandy lenses over dense weathered rock. Water levels encountered in the exploration locations in the low lying areas are considered to be the seasonal water table. It should be noted that fluctuations of ground water levels of several feet typically occur seasonally with variations in precipitation and runoff. During the wet season of the year (late winter/early spring) groundwater or "perched" water conditions can develop locally within existing granular soils above the less permeable layers such as the very dense weathered rock and/or bedrock surface. Refer to the boring and test pit logs included in Appendix B for detailed information.

INFILTRATION TESTING

Thirteen proposed stormwater management (SWM) facility locations were evaluated for infiltration potential of the underlying soils. In order to estimate the unsaturated hydraulic conductivity or infiltration rate of the soils at these locations, a single-ring infiltrometer test (ASTM D5126) was performed within holes offset from Test Pits TP-1 through TP-13. The test depths were established to maintain a minimum of 3 feet of separation between the test elevations and hydraulically limiting zones.

The testing consisted of seating an open-bottom 12-inch diameter casing approximately 4 inches into the hand-trimmed subgrade soils. The holes were then pre-soaked, and water level measurements were taken with time until a steady state condition was observed. The tests were conducted for approximately 2 hours, and the unfactored steady-state values recorded over the last 1-hour time period are listed below. It should be noted that infiltration rates can vary widely with variations in soil texture and gradation.

Test Pit	Test Depth (ft)	Soil Description	Uncorrected Field Infiltration Rate
TP-1	4	Silty SAND, contains rock fragments	2 inches per hour
TP-2	2 ½	Silty GRAVEL with sand	2 inches per hour
TP-3	4 ½	Silty SAND, contains rock fragments	1 inches per hour
TP-4	5 ½	SILT, contains rock fragments	No discernible movement
TP-5	3 ½	Silty Clayey SAND with gravel	½ inches per hour
TP-6	4 ½	Silty SAND, contains rock fragments	2 inches per hour

Test Pit	Test Depth (ft)	Soil Description	Uncorrected Field Infiltration Rate
TP-7	3	Silty SAND, contains rock fragments	2 inches per hour
TP-8	3	Silty SAND, contains rock fragments	1 inches per hour
TP-9	5	Silty Clayey SAND	½ inches per hour
TP-10	4	Sandy SILT, contains rock fragments	No discernible movement
TP-11	4	Silty SAND with gravel	1 inch per hour
TP-12	4 ½	Silty SAND with gravel	2 inches per hour
TP-13	3 ½	Well-graded GRAVEL with silt and sand	4 inches per hour

LABORATORY ANALYSIS

Selected samples obtained from the test pits were tested for grain-size analysis, Atterberg Limits, and natural moisture contents. The grain-size analysis and Atterberg Limit testing were performed to determine the Unified Soil Classification System (USCS) designation and the USDA soil classification for the soil. USCS classifications provide information regarding soil behavior beneath pavement and foundation systems and the USDA soil classification can provide information regarding hydraulic conductivity of the soils. The results of testing are as summarized in the table below:

SUMMARY OF LABORATORY TESTING

Test Pit	Depth (ft)	USCS Classification	USDA Classification	LL%	PI%	NMC %
TP-1	4	Silty SAND (SM)	Sandy Loam	33	8	16.7
TP-2	2 ½	Silty GRAVEL with sand (GM)	Sandy Loam	32	6	12.4
TP-3	4 ½	Silty SAND (SM)	Sandy Loam	26	2	15.2
TP-4	2-5	SILT (ML)	---	33	8	24.8
TP-5	3 ½	Silty Clayey SAND with gravel (SM-SC)	Sandy Loam	28	6	13.1
TP-6	4 ½	Silty SAND (SM)	Sandy Loam	38	8	19.8

TP-7	3	Silty SAND (SM)	Sandy Loam	36	7	20.2
TP-8	3	Silty SAND (SM)	Loam	28	NP	10.2
TP-9	2-6	Silty SAND with gravel (SM)	---	33	5	14.0
TP-9	5	Silty Clayey SAND (SC-SM)	Sandy Loam	29	7	13.8
TP-10	4	Sandy SILT (ML)	Loam	31	7	15.1
TP-11	4	Silty SAND (SM)	Loam	43	7	33.2
TP-12	4 ½	Silty Sand with gravel (SM)	Sandy Loam	29	4	16.0
TP-13	3 ½	Well-graded GRAVEL with silt and sand	Sandy Loam	32	7	9.2

NP= Non-Plastic, LL = Liquid Limit, PI = Plastic Index, NMC=Natural Moisture Content

Two bulk samples obtained from Test Pits TP-4 and TP-9 were tested for moisture-density relationships in accordance with the Standard Proctor (ASTM D698) testing for use in evaluating the suitability of these soils for reuse as fill. The bulk samples were also subjected to California Bearing Ratio (ASTM D1583) (CBR) testing for use in evaluation of pavement subgrade support quality. Results of these tests are summarized in the following table.

SUMMARY OF COMPACTION AND CBR TESTING
(ASTM D698, Standard Proctor; ASTM D1883, CBR)

Test Pit No.	Depth (ft)	Maximum Dry Density (PCF)	Optimum Moisture (%)	NMC (%)	CBR at 0.1 at 95% Compaction
TP-4	2 to 5	105.4	19.1	24.8	6.1
TP-9	2 to 6	111.6	16.3	14.0	7.4

Natural soil moisture contents for the samples tested ranged from 7.8 to 59.0 percent, averaging approximately 17 percent. The higher moisture contents were generally associated with the more fine-grained samples near the ground surface and moderately plastic soils. Grain-size distribution test reports, moisture-density relationship curves, CBR test reports and natural moisture test results are included in Appendix C.

ANALYSIS AND RECOMMENDATIONS

Based upon the results of this study, it is our opinion that construction of the proposed subdivision is feasible, given that the following recommendations are followed, and that the standard level of care is maintained during construction. It should be noted that problems associated with perched groundwater, shallow weathered rock, and wet, sensitive soils could be encountered during construction. Discussions of these issues, as well as general site development procedures are included in the following paragraphs.

Earthwork

As previously discussed, the subject site contains a few residential structures, barns, stables, and horse training facilities in the central and western portions. Additionally, the site contains asphalt and gravel driveways and various utilities associated with existing structures, as well as remnant slabs and demolition debris on the eastern portion. The general sequence of construction should consist of demolition and removal of existing and abandoned structures not to remain; including their below grade components such as underground storage tanks, foundations, concrete floor slabs, and utilities. Any excavations made for the removal of below grade tanks, foundations, utilities or drain tiles in structural areas should be backfilled with compacted structural fill meeting the requirements outlined below.

Prior to the placement of any structural fill, the area should be stripped to remove any vegetation, cultivated soil/topsoil, organic material, surface debris, existing fill materials or other unsuitable materials. Topsoil/cultivated soil was encountered at depths ranging approximately 2 to 15 inches and root balls from the larger trees may extend 2 to 3 feet. The actual stripping thickness will be dependent on localized topsoil development, root mat thickness, precipitation, soil moisture, construction traffic disturbance and contractor care. Topsoil should be stripped from within a minimum of 5 feet beyond the proposed building and pavement limits. The topsoil may be stockpiled onsite for future use in landscaped areas but would not be suitable for reuse in structural areas. Based on our on-site observations, localized areas of existing fill associated with the on-site farming roads and previous development will likely be encountered. These fill materials are not considered suitable for foundation support and should be evaluated before leaving in place for any slabs or roadway support. Additional subsurface explorations may be necessary in areas that had been previously developed if structures or infrastructure are planned.

Following stripping, the building and pavement areas to receive fill should be proof-rolled to locate any soft or loose areas on the subgrade. Any surficial materials identified as unstable or unsuitable should be undercut to a stable stratum and backfilled with structural fill or stabilized as recommended in the field by the Geotechnical Engineer. It should be noted that the stripping of organics, subgrade evaluations, undercutting of any unsuitable/unstable material, and placement of controlled, compacted fill should be observed by a geotechnical engineer or their qualified representative. Near surface fine-grained soils will generally be wet of their optimum moisture and will be sensitive to heavy construction traffic. Care should be taken during mass grading to not disturb the subgrade soils in structure areas. Drying of the subgrade may be necessary before placing compacted structural fill. New structural fill should be placed in lifts and compacted in accordance with the specifications included in this report.

We recommend that positive drainage be maintained across the site during construction to prevent ponding of water, since the exposed subgrades could destabilize in combination with construction traffic and precipitation. Furthermore, heavy construction traffic should generally be run on designated haul roads during periods of wet weather to reduce the potential for destabilization of more subgrade areas than necessary. If the subgrade is disturbed by construction traffic and becomes unstable, undercutting and replacement of these surficial materials will be required.

The on-site materials classified as ML (silt), SM (silty sand), SC-SM (silty, clayey sand) and with some limitations CL (lean clay), are considered suitable for use in structural fill construction. Any large rock fragments encountered during construction should be removed or processed to less than 6 inches in size and mixed with suitable residual soils. Any materials classifying as CL, CH, and MH, if encountered, should generally not be used for structural fill within the upper 1 to 2 feet of pavement subgrade or beneath foundations without chemical stabilization, but can be used for construction of stormwater management berms or in the nonstructural areas.

At the time this study was performed, some of the soils were wet of the optimum moisture content for compaction, with moisture contents in the range of 8 to 59 percent, compared to optimum moisture contents in the range of 16 to 19 percent. Moisture conditioning of the on-site, non-plastic granular soils should not be a significant problem during favorable weather conditions. However, the fine-grained or plastic, granular soils will require significantly more drying effort if they are wet of optimum at the time earthwork proceeds. The excavated materials will generally need to be within 2 to 3 percentage points of the optimum moisture for compaction before compactive effort is applied. Off-site borrow, if required, should meet Unified Soil Classification System (USCS) designation SC, SM, SP, GP, GM, or GW and be approved by the Geotechnical Engineer prior to use. All structural fill should be constructed in maximum 8-inch thick loose lifts and be compacted to the following specifications:

COMPACTION SPECIFICATIONS

Fills supporting foundations, retaining walls, floor slabs, and within walls or slopes steeper than 5H:1V	95% of ASTM D698 Moisture: within 3% of optimum
Fills within top 1 foot of pavement subgrade	98% of ASTM D698 Moisture: within 2% of optimum
Fills below 1 foot of pavement subgrade	95% of ASTM D698 Moisture: within 3% of optimum

Fill subgrades and each lift of fill should be observed and tested by a soils technician on a full-time basis, under the supervision of a registered engineer as required per the International Residential Code. All compactive effort should be verified by in-place density testing. New fills constructed on slopes steeper than 5H:1V (horizontal to vertical) should be keyed into existing slopes for stability considerations. All fill slopes steeper than 5H:1V should generally be placed as structural fill and be controlled and compacted to minimum densities as specified above. Slopes constructed steeper than 3H:1V should be evaluated for stability and may need to be designed with reinforcement.

Subsurface Utilities

The natural soils are considered suitable for support of below grade utilities. Granular bedding may be required to provide uniform support if soft/loose soils, groundwater, or rock are encountered as dictated by site conditions or as required by local code. Refusal was encountered within the highly weathered rock at Test Pits TP-2, TP-8, and TP-12 through TP-17, at depths of approximately 7½ to 10½ feet below existing grades. As such, deeper trench excavations may be difficult in these areas. It should be expected that excavations into the weathered rock or beyond the bucket refusal depth may not be possible without the use of large excavation equipment equipped with rock teeth or rippers, blasting, or other special rock removal techniques. We recommend that the rock excavation be completed prior to construction of foundations, subsurface utilities, or site retaining walls to avoid the potential problems that could result from vibrations caused by the rock removal operations.

We recommend that the construction documents identify all excavation as "unclassified" to avoid disputes that often arise as to the definition of rock. If excavation must be bid as "classified" then your agreement must include a definition of rock. An example definition of rock for contractual purposes is presented below:

> Rock is defined as massive bedrock that cannot be dislodged by a D-9 Caterpillar tractor, or equivalent, equipped with a hydraulically operated power ripper, or by a Caterpillar 245 excavator, or equivalent, equipped with rock teeth but without the use of hoe rams or other breaking techniques. Boulders or masses of rock exceeding 1 cubic yard in volume shall also be considered rock excavation. This classification does not include materials such as loose rock, concrete or other materials that can be removed by means other than breaking by hoe rams, etc., but which for reasons of economy in excavating the Contractor chooses to remove by other methods.

If excavation is bid as "classified" then a rock excavation allowance should be established and be included in the base bid with add/deduct unit prices per cubic yard (measured in-place) to adjust the base allowance. It should be noted that variations in the depth to partially weathered rock will exist between boring locations and rock may be encountered at shallower depths across the site during mass excavation.

Groundwater or perched water was encountered in some of the test borings and test pits at depths of approximately 4 ½ to 13 feet below the existing ground surface. Therefore, groundwater could impact utility construction, particularly in the low-lying areas of the site adjacent to the streams and wetlands, and perched water could be encountered at the soil/weathered rock interface during the wet season. Problems associated with groundwater include seepage into the excavation, loss of stability, sidewall collapse, and sloughing of soils. These problems can be reduced through the use of dewatering techniques, such as sumps, but will likely be marginally effective. Trench shields may also be required for support of vertical cut excavations where utilities are deeper than 4 feet to reduce sidewall collapse. Due to the potential for collapse of unsupported excavations in granular soils, the utility contractor should be prepared to provide adequate earth support and dewatering systems during utility construction.

Utility pipe systems below pavement and other structural areas should be backfilled using compacted structural fill. The backfill should be placed and compacted in accordance with our *Earthwork* recommendations.

Foundations

Assuming maximum wall loads of 4 klf and column loads of 20 kips; the proposed structures may be supported on shallow spread footings designed for a net allowable bearing pressure of up to 3,000 pounds per square foot (psf). Minimum widths for wall footings of 16 inches and column footings of 24 inches are recommended when design based on 3,000 psf results in a more narrow footing. Settlement on the order of 1-inch total and ½-inch differential can be anticipated, based on the assumed loads. Exterior footings should be founded a minimum of 36 inches below the final exterior grades to provide protection from frost action, unless otherwise required by local code.

Footings should be supported on the medium dense or stiff natural soils or on new properly compacted structural fill. In localized areas, it may be necessary to undercut foundations at saturated zones or where soft/loose soils are encountered. The decision to undercut footings should be made in the field during footing construction. Based on the test borings and test pits excavations of basements can generally be accomplished by conventional means provided the site grades are not lowered significantly. However, difficult excavation may be encountered in the vicinity of Test Pits TP-2, TP-8, and TP-12 through TP-17 where refusal was encountered within the dense weathered rock materials.

GTA recommends that concrete placement be performed the same day footings are excavated to prevent exposure of the soils at footings level and potential weakening of the soils.

Groundwater was encountered at 11 locations at depths ranging from 4.4 to 12.9 feet below the ground surface. It is believed that the encountered water was a result of perched conditions and/or water influenced by nearby wetlands or streams. Depending on the site grades and basement elevations, problems may be encountered during foundation construction during the wet season or after periods of heavy precipitation. If perched water or groundwater in encountered, a layer of open-graded aggregate can be placed across the basement subgrade to facilitate drainage and protect the subgrade soils. Additionally, the use of dewatering devices such as sumps or gravity flow trenches will likely be sufficient in aiding in dewatering. Construction of permanent exterior and interior drains with interior sump pumps are recommended to direct accumulated subsurface drainage away from the foundation.

Detailed foundation evaluations should be performed in each footing excavation prior to the placement of reinforcing steel or concrete. These evaluations should be performed by a representative of the Geotechnical Engineer to confirm that the allowable soil bearing capacity is available. The foundation bearing surface evaluations should be performed using a combination of visual observation, comparison with the test pits, hand-rod probing, and Dynamic Cone Penetrometer (DCP) testing.

Floor Design

Floor slabs can be designed as concrete slabs on grade. GTA recommends that the concrete floor slabs supported on grade be founded on a 4-inch (minimum) coarse granular layer covered with polyethylene vapor barrier to interrupt the rise of capillary moisture through the slab. Imported washed gravel or crushed stone materials meeting the gradation of AASHTO No.

57 aggregate are considered suitable for the granular layer. Natural and compacted fill subgrades for support of the floor slabs should be observed to evaluate stability prior to placement of concrete. The slabs may bear on wall or footing projections, but they should be isolated and jointed so that the foundation walls can settle slightly without affecting the slab.

Lateral Earth Pressure

Below grade walls and retaining walls will have to be designed to resist lateral earth pressures from the retained soils. The following properties may be used in the design of below grade foundation walls and retaining walls. These properties consider the use of either the on-site granular soils or on-site fine-grained soils as structural fill.

LATERAL EARTH PRESSURE SUMMARY

Soil Property	On-Site, Granular Soils
Unit Weight, γ	125 pcf
Angle of Internal Friction, Φ	30°
Coefficient of Active Earth Pressure (Ka)	0.33
Coefficient of Passive Earth Pressure (Kp)	3.00
Coefficient of Earth Pressure at Rest (Ko)	0.5
Base Friction, tan δ	0.5
Equivalent Fluid Pressure (Unrestrained Top of Wall)	42 psf/ft
Equivalent Fluid Pressure (Restrained Top of Wall)	63 psf/ft

Drainage panels and a perimeter drain should be provided behind below grade walls and retaining walls to carry away any infiltrating surface water so that hydrostatic pressures do not develop. The perimeter drain should consist of a minimum 4-inch diameter slotted or perforated pipe encased in a minimum of 6 inches of crushed stone that is wrapped by a geotextile filter. The crushed stone should meet the gradational requirements of AASHTO Size No. 57 aggregate. The perimeter drain should tie into a sump pit, adjacent storm sewer, or off-site drainage system. Where retaining walls are used, the collection system should discharge water to weepholes, which are at least two inches in diameter and spaced at maximum eight feet on center. All below grade foundation walls adjacent to occupied spaces should be waterproofed.

Pavements

GTA recommends that the upper 12 to 18 inches of pavement subgrade be constructed of on-site granular soils with characteristics tabulated below:

Liquid Limit (AASHTO T89)	35% or less
Plasticity Index (AASHTO T89, T90)	15% or less
Maximum Dry Density (AASHTO T99)	105 pcf or greater
California Bearing Ratio	5% or greater

Based on the results of our laboratory testing, soils with these characteristics should be readily available at the site. However, some of the surficial fine-grained soils are moisture sensitive and micaceous and generally have a low sheer strength without confinement. Undercutting, replacing with granular soils, crushed stone, or the use of geosynthetics may be necessary in some areas where destabilization of the subgrade occur. Prior to construction of pavement sections, the pavement subgrade should be reviewed to verify design parameters and proof-rolled with a loaded tri-axle dump truck under the direct supervision of the Geotechnical Engineer to evaluate stability. Unsuitable soils should be over-excavated to a stable layer.

The natural site soils may become disturbed and softened from excess moisture and construction equipment traffic. Contractors should anticipate that remedial work could be required to achieve a stable subgrade prior to placing stone and paving, even if the subgrade soils had previously been compacted to the required densities. Prudent planning and earthwork procedures will reduce the potential necessity for remedial work. Road fills should be placed and compacted in accordance with the recommendations outlined in the *Earthwork* section of this report.

Heavy construction traffic should not be allowed on partial pavement sections since such traffic can damage the pavement. The paving contractor should be advised that they must control construction traffic to limit disturbance of previously approved subgrade, stone base course, or completed asphalt. Some patching and repair may be necessary prior to placement of the final wearing surface layer of asphalt due to construction traffic.

SWM Facilities

Based on our observations made during the subsurface exploration, it is our opinion that managing stormwater quality through the use of infiltration will be feasible with some limitations in portions of the site. However, the surficial fine-grained soils could impact the design and construction of the proposed facilities. Where infiltration is desired, it is recommended that the proposed subgrades be extended through the fine-grained soils in to the sandy residual soils. If the subgrades need to be undercut below the design grade, the proposed subgrade elevations can be re-established with ASTM C33 sand (concrete sand) or AASHTO #57 stone.

The guidelines established in the Pennsylvania Stormwater Best Management Practices Manual, Appendix C *Site Evaluation and Soil Testing* indicates that the minimum infiltration rate for all runoff reduction and infiltration practices is 0.1-inch per hour. Also, a vertical separation of two (2) feet from the seasonal high groundwater elevation is required. Infiltration is not considered practical in the areas near test pits TP-4 and TP-10 due to shallow limiting zones and/or lower infiltration rates.

Unfactored field measured infiltration rates ranged from no discernable rate to greater than 4 inches per hour at the tested locations and depths. However, we recommend that a design infiltration rate of no more than 25 to 50 percent of the field measured rate be used for the final design of the facility. We do not recommend averaging rates at various locations and applying the averaged rate to the site or per facility. This recommendation is based on the inherent problems associated with these systems as they become less permeable due to densification during construction and partial clogging or siltation occurring over time. Additionally, design phase infiltration testing should be performed to confirm the preliminary rates in this report.

Once the design of the proposed facilities has been completed, GTA should be provided the opportunity to review the plans to evaluate if the geotechnical issues have been addressed. Also, GTA should be provided the opportunity to review the facility subgrade during construction and perform additional field testing, if warranted. This is to observe compliance with the design concepts, specifications or recommendations, and to allow for field changes in the event that the soils conditions differ from that anticipated prior to that start of construction.

CONSTRUCTION OBSERVATION

We recommended that during construction of the subject project, a geotechnical engineer be retained to provide observation and testing services for the following items.

- Perform a supplemental subsurface investigation for the building, retaining wall, and deep utility excavations.
- Perform additional infiltration testing at alternate depths and/or locations.
- Review final civil and structural plans to evaluate if they conform with the intent of this report.
- Observe the proof-rolling of fill and pavement subgrades prior to placing fill or base course to evaluate stability.
- Provide observation and testing services during fill placement to evaluate if the work is being performed in accordance with the project specifications and intent of this report.
- Review excavated footings for compliance with the project drawings and the intent of this geotechnical report.
- Provide Special Inspections as required by the project specifications and Westtown Township requirements for the clubhouse.

LIMITATIONS

This report, including all supporting test boring, test pit logs, field data, field notes, laboratory test data, calculations, estimates, and other documents prepared by GTA in connection with this project, has been prepared for the exclusive use of Toll Brothers pursuant to the agreement between GTA and Toll Brothers, Inc., and in accordance with generally accepted engineering practice. All terms and conditions set forth in the Agreement and the General Provisions attached thereto are incorporated herein by reference. No warranty, express or implied, is given herein. Use and reproduction of this report by any other person without the expressed written permission of GTA and Toll Brothers, Inc. is unauthorized and such use is at the sole risk of the user.

The analysis and recommendations contained in this report are based on the data obtained from limited observation and testing of the encountered materials. Test borings and test pits indicate soil conditions only at specific locations and times and only to the depths penetrated. They do not necessarily reflect strata variations that may exist between the test pit locations. Consequently, the analysis and recommendations must be considered preliminary until the subsurface conditions can be verified by direct observation at the time of construction. If variations in subsurface conditions from those described are noted during construction, recommendations in this report may need to be re-evaluated.

In the event that any changes in the nature, design, or location of the facilities or lots are planned, the conclusions and recommendations contained in this report should not be considered valid unless the changes are reviewed and conclusions of this report are verified in writing. Geo-Technology Associates, Inc. is not responsible for any claims, damages, or liability associated with interpretation of subsurface data or re-use of the subsurface data or engineering analysis without the expressed written authorization of Geo-Technology Associates, Inc.

The scope of our services for this geotechnical exploration did not include any environmental assessment or investigation for the presence or absence of wetlands, or hazardous or toxic materials in the soil, surface water, groundwater or air, on or below or around this site. Any statements in this report or on the logs regarding odors or unusual or suspicious items or conditions observed are strictly for the information of our Client.

This report and the attached logs are instruments of service. The subject matter of this report is limited to the facts and matters stated herein. Absence of a reference to any other conditions or subject matter shall not be construed by the reader to imply approval by the writer.

161348 **GEO-TECHNOLOGY ASSOCIATES, INC.**

Important Information about This

Geotechnical-Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

The Geoprofessional Business Association (GBA) has prepared this advisory to help you – assumedly a client representative – interpret and apply this geotechnical-engineering report as effectively as possible. In that way, clients can benefit from a lowered exposure to the subsurface problems that, for decades, have been a principal cause of construction delays, cost overruns, claims, and disputes. If you have questions or want more information about any of the issues discussed below, contact your GBA-member geotechnical engineer. Active involvement in the Geoprofessional Business Association exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project.

Geotechnical-Engineering Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical-engineering study conducted for a given civil engineer will not likely meet the needs of a civil-works constructor or even a different civil engineer. Because each geotechnical-engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client. *Those who rely on a geotechnical-engineering report prepared for a different client can be seriously misled.* No one except authorized client representatives should rely on this geotechnical-engineering report without first conferring with the geotechnical engineer who prepared it. *And no one – not even you – should apply this report for any purpose or project except the one originally contemplated.*

Read this Report in Full

Costly problems have occurred because those relying on a geotechnical-engineering report did not read it *in its entirety*. Do not rely on an executive summary. Do not read selected elements only. *Read this report in full.*

You Need to Inform Your Geotechnical Engineer about Change

Your geotechnical engineer considered unique, project-specific factors when designing the study behind this report and developing the confirmation-dependent recommendations the report conveys. A few typical factors include:

- the client's goals, objectives, budget, schedule, and risk-management preferences;
- the general nature of the structure involved, its size, configuration, and performance criteria;
- the structure's location and orientation on the site; and
- other planned or existing site improvements, such as retaining walls, access roads, parking lots, and underground utilities.

Typical changes that could erode the reliability of this report include those that affect:

- the site's size or shape;
- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light-industrial plant to a refrigerated warehouse;
- the elevation, configuration, location, orientation, or weight of the proposed structure;
- the composition of the design team; or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes – even minor ones – and request an assessment of their impact. *The geotechnical engineer who prepared this report cannot accept responsibility or liability for problems that arise because the geotechnical engineer was not informed about developments the engineer otherwise would have considered.*

This Report May Not Be Reliable

Do not rely on this report if your geotechnical engineer prepared it:

- for a different client;
- for a different project;
- for a different site (that may or may not include all or a portion of the original site); or
- before important events occurred at the site or adjacent to it; e.g., man-made events like construction or environmental remediation, or natural events like floods, droughts, earthquakes, or groundwater fluctuations.

Note, too, that it could be unwise to rely on a geotechnical-engineering report whose reliability may have been affected by the passage of time, because of factors like changed subsurface conditions; new or modified codes, standards, or regulations; or new techniques or tools. *If your geotechnical engineer has not indicated an "apply-by" date on the report, ask what it should be*, and, in general, *if you are the least bit uncertain* about the continued reliability of this report, contact your geotechnical engineer before applying it. A minor amount of additional testing or analysis – if any is required at all – could prevent major problems.

Most of the "Findings" Related in This Report Are Professional Opinions

Before construction begins, geotechnical engineers explore a site's subsurface through various sampling and testing procedures. *Geotechnical engineers can observe actual subsurface conditions only at those specific locations where sampling and testing were performed.* The data derived from that sampling and testing were reviewed by your geotechnical engineer, who then applied professional judgment to form opinions about subsurface conditions throughout the site. Actual sitewide-subsurface conditions may differ – maybe significantly – from those indicated in this report. Confront that risk by retaining your geotechnical engineer to serve on the design team from project start to project finish, so the individual can provide informed guidance quickly, whenever needed.

This Report's Recommendations Are Confirmation-Dependent

The recommendations included in this report – including any options or alternatives – are confirmation-dependent. In other words, *they are not final*, because the geotechnical engineer who developed them relied heavily on judgment and opinion to do so. Your geotechnical engineer can finalize the recommendations *only after observing actual subsurface conditions* revealed during construction. If through observation your geotechnical engineer confirms that the conditions assumed to exist actually do exist, the recommendations can be relied upon, assuming no other changes have occurred. *The geotechnical engineer who prepared this report cannot assume responsibility or liability for confirmation-dependent recommendations if you fail to retain that engineer to perform construction observation.*

This Report Could Be Misinterpreted

Other design professionals' misinterpretation of geotechnical-engineering reports has resulted in costly problems. Confront that risk by having your geotechnical engineer serve as a full-time member of the design team, to:

- confer with other design-team members,
- help develop specifications,
- review pertinent elements of other design professionals' plans and specifications, and
- be on hand quickly whenever geotechnical-engineering guidance is needed.

You should also confront the risk of constructors misinterpreting this report. Do so by retaining your geotechnical engineer to participate in prebid and preconstruction conferences and to perform construction observation.

Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can shift unanticipated-subsurface-conditions liability to constructors by limiting the information they provide for bid preparation. To help prevent the costly, contentious problems this practice has caused, include the complete geotechnical-engineering report, along with any attachments or appendices, with your contract documents, *but be certain to note conspicuously that you've included the material for informational purposes only.* To avoid misunderstanding, you may also want to note that "informational purposes" means constructors have no right to rely on the interpretations, opinions, conclusions, or recommendations in the report, but they may rely on the factual data relative to the specific times, locations, and depths/elevations referenced. Be certain that constructors know they may learn about specific project requirements, including options selected from the report, *only* from the design drawings and specifications. Remind constructors that they may perform their own studies if they want to, and *be sure to allow enough time* to permit them to do so. Only then might you be in a position to give constructors the information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions. Conducting prebid and preconstruction conferences can also be valuable in this respect.

Read Responsibility Provisions Closely

Some client representatives, design professionals, and constructors do not realize that geotechnical engineering is far less exact than other engineering disciplines. That lack of understanding has nurtured unrealistic expectations that have resulted in disappointments, delays, cost overruns, claims, and disputes. To confront that risk, geotechnical engineers commonly include explanatory provisions in their reports. Sometimes labeled "limitations," many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The personnel, equipment, and techniques used to perform an environmental study – e.g., a "phase-one" or "phase-two" environmental site assessment – differ significantly from those used to perform a geotechnical-engineering study. For that reason, a geotechnical-engineering report does not usually relate any environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated subsurface environmental problems have led to project failures.* If you have not yet obtained your own environmental information, ask your geotechnical consultant for risk-management guidance. As a general rule, *do not rely on an environmental report prepared for a different client, site, or project, or that is more than six months old.*

Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

While your geotechnical engineer may have addressed groundwater, water infiltration, or similar issues in this report, none of the engineer's services were designed, conducted, or intended to prevent uncontrolled migration of moisture – including water vapor – from the soil through building slabs and walls and into the building interior, where it can cause mold growth and material-performance deficiencies. Accordingly, ***proper implementation of the geotechnical engineer's recommendations will not of itself be sufficient to prevent moisture infiltration.*** Confront the risk of moisture infiltration by including building-envelope or mold specialists on the design team. ***Geotechnical engineers are not building-envelope or mold specialists.***

GBA GEOPROFESSIONAL BUSINESS ASSOCIATION

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