GEOTECHNICAL SERVICES
AMSTERDAM ROAD RECONSTRUCTION, PHASE 1
FORT WRIGHT, KENTUCKY

Prepared for:

CT CONSULTANTS, INC.
FORT MITCHELL, KENTUCKY

Prepared by:

GEOTECHNOLOGY, INC.
ERLANGER, KENTUCKY

Date:

AUGUST 21, 2017

Geotechnology Project No.:

J029038.01

SAFETY
QUALITY
INTEGRITY
PARTNERSHIP
OPPORTUNITY
RESPONSIVENESS

GEOTECHNOLOGY INC
FROM THE GROUND UP

August 21, 2017

Mr. Martin Hellmann, PE
CT Consultants, Inc.
2161 Chamber Center Drive
Fort Mitchell, Kentucky 41017

Re: Geotechnical Services
Amsterdam Road Reconstruction, Phase 1
Fort Wright, Kentucky
Geotechnology Project No. J029038.01

Ladies and Gentlemen:

Presented in this report are the results of our geotechnical exploration completed for the Amsterdam Road Reconstruction, Phase 1 project to be located between General Drive and Redwood Drive in Fort Wright, Kentucky. Our services were performed in general accordance with our Proposal P029038.01, which was dated February 23, 2017, and was authorized by the April 6, 2017 Subconsultant Agreement between Geotechnology, Inc. and CT Consultants, Inc.

We appreciate the opportunity to provide the geotechnical services for this project. If you have any questions regarding this report, or if we may be of any additional service to you, please do not hesitate to contact us.

Respectfully submitted,
GEOTECHNOLOGY, INC.

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1.0 INTRODUCTION

Geotechnology, Inc. (Geotechnology) prepared this geotechnical exploration report for CT Consultants, Inc. (CT) for the Amsterdam Road Reconstruction, Phase 1 project to be located between General Drive and Redwood Drive in Fort Wright, Kentucky. Our services were performed in general accordance with our Proposal P029038.01, which was dated February 23, 2017, and was authorized by the April 6, 2017 Subconsultant Agreement between Geotechnology and CT.

The purposes of the geotechnical exploration were: to evaluate the general subsurface profile at the site and to relate the engineering properties of the soils and bedrock; that is their classification, strength and compressibility characteristics, to subgrade preparation for the proposed roadway reconstruction and to the design of the project retaining walls. Our scope of services included a site reconnaissance, geotechnical borings, laboratory testing, engineering analyses, preparation of this report, and preparation of design drawings for the project retaining walls. The retaining wall design drawings will be issued separately at a later date.

A copy of "Important Information about This Geotechnical-Engineering Report" that is published by the Geotechnical Business Council of the Geoprofessional Business Association is included in Appendix A for your review.

2.0 PROJECT INFORMATION & SITE CONDITIONS

2.1 General

For the purposes of this report, we have assumed that Amsterdam Road runs east-west in the project area. The reconstruction project begins approximately 265 feet west of the intersection with General Drive and ends at approximately 43 feet east of the intersection with Redwood Drive for a total length of about 2,261 feet. We understand that the reconstruction project will include widening the roadway approximately 7 to 10 feet to accommodate a sidewalk and new curb and gutter. The project will also include the construction of a new storm sewer beneath the roadway and the construction of a 1,200 linear foot retaining wall. The existing roadway follows two primary terrain types; steeply sloping side-hill terrain along the west half or so of the project and gently sloping ridgetop terrain along the east half of the project.

2.2 Project Plans

The preliminary plan and profile drawings for the proposed road reconstruction and new storm sewer are shown on Sheets PNP-1 through PNP-5 of the undated CT Project Plans, which we received electronically from CT on August 1, 2017. CT also provided us with the undated preliminary plan and profile drawings for the proposed retaining wall, as shown on Sheets PNP-W1 through PNP-W3. Roadway cross sections are shown on Sheets XS-1 through XS-14. These drawings will be referred to as the CT Project Plans throughout this report.

2.3 Steeply Sloping Side-Hill Terrain - West End of Project to Station 21+90

The existing terrain in this portion of the project consists of a hillside that slopes down to the south at gradients that are primarily on the order of 3 horizontal to 1 vertical (3H:1V) to 2H:1V. Amsterdam Road climbs uphill through this area along the approximate middle of the slope, with the ground sloping down from left to right across the roadway. The slope on the downhill (south) side of the roadway is primarily wooded and continues down to the south toward a valley bottom that drains to the west into a tributary of Pleasant Run Creek. The existing Amsterdam Road was constructed with typical hillside cutting on the upslope side of the road and filling on the downslope side. Residential lots with homes, retaining walls, and yard space was similarly constructed on portions of the north side of the roadway.

The roadway will be widened in this area by extending the section to the south beyond the existing shoulder and over the steeply sloping terrain. New fills on the order of 2 to 6 feet deep will be required to establish the widened roadway over the sloping ground. Locally, the proposed fills will be as deep as 8 feet or more. The fills are planned to be retained with a new 1,200 linear foot wall, the location of which is shown on the CT Project Plans. The north side of the improved section will include minor filling on the order of about a foot or less to provide drainage. Cutting is not shown on the CT Project Plans on the uphill side of the roadway.

2.4 Gently Sloping Ridgetop Terrain – Station 21+90 to East End of Project

The existing roadway leaves the steeply sloping side-hill terrain at approximate Station 21+90 and continues to the east up and along a relatively gently sloping ridgetop to the end of the project. The gently sloping ridgetop extends well beyond each side of the roadway. The widened roadway will be established by minor cuts and fills on the order of 2 feet or less on both sides of the roadway through this area. Residential lots with homes and yard space line the majority of both sides of the roadway in this area.

2.5 Proposed Storm Sewer

Approximately 2,253 linear feet of 12-inch diameter storm sewer will be installed beneath the roadway and/or curb and gutter locations throughout the project alignment. The proposed storm sewer will be installed by traditional cut and cover methods with invert levels approximately 5 to 8 feet below the existing ground surface, which is approximately 5 to 9 feet below the proposed grades.

2.6 Existing Utilities

There are several existing utilities located within and/or paralleling the existing roadway, which include, but are not limited to, overhead wires, water mains, sanitary sewers and gas mains. It is our understanding that the sanitary sewer and water main will remain in place; a portion of the gas main from about Stations 12+10 to 13+40 will be relocated; and the power poles located at Stations 10+30, 13+98, and 18+05 will either be temporarily relocated or moved to allow for the construction of the retaining wall.

2.7 Site Reconnaissance and Existing Hillside Stability

Based on an engineering site reconnaissance performed in March 2017, the existing Amsterdam Road pavement is in fair to poor condition. Observed pavement distresses include alligator cracking, longitudinal cracking, edge cracking, diagonal cracking, transverse cracking, creep and potholes. Pavement distresses indicative of creep and possibly landslide movements were observed along the downslope edge of the road between approximate Stations 14+30 and 15+30 and approximate Stations 15+70 to 16+40, which are located in the vicinity of addresses 1731 to 1739 Amsterdam Road. The noted instability was observed to be at locations where additional fill appears to have been placed along the downslope edge of the road in order to create parking areas. Several relatively short, stacked stone retaining walls are located near the north edge of the road in front of addresses 1731 through 1739. The stone walls were observed to have bowed sections, cracked and patched areas, and areas where stone has toppled from the top of the wall.

Creep movement was observed along both the upslope and downslope sides of the road as evidenced by leaning, swept and bowed trees from the intersection with General Drive to the intersection with Ridgewood Road. Creep movement is the slow, almost imperceptible lateral movement of the overburden soils due to gravity. Over years of time, the lateral movements can manifest into multiple inches of movement.

In addition, possible landslide ground movement was observed on the slope between Amsterdam and Ridgewood Roads. Observations included: longitudinal ground cracking to the center of Ridgewood Road; alligator cracking and patching; and leaning, swept or bowed trees on the slope below Ridgewood Road.

3.0 SUBSURFACE EXPLORATION

The subsurface exploration consisted of 19 new borings (numbered 201 through 222, excluding 212, 215, and 217, which were not drilled) and 3 historic borings (numbered 104, 9 and 10). The new boring locations were selected by us and staked in the field by us. The borings in the western half of the project were selected to allow the development of a series of upslope-downslope cross sections along the hillside. The boring locations, ground surface elevations, and cross sections were then surveyed by CT relative to their survey control and benchmark elevation. A few of the new borings were moved during drilling due to access issues or existing utility conflicts. The distance, direction, and elevation change of each boring that was moved was measured using a tape measure and hand level. The as-drilled locations of the borings and the locations of the cross sections are shown on our Boring Plan, which is included in Appendix B.

The new borings were drilled between April 4, 2017 and April 12, 2017 with a track-mounted drill rig advancing hollow-stem augers. Sampling of the overburden soils and bedrock was accomplished ahead of the augers, with either 2-inch-outside-diameter (O.D.) split-spoons or 3-inch-O.D. thin-walled Shelby tubes in general accordance with the procedures outlined by ASTM D1586 and ASTM D1587, respectively. Standard Penetration Tests (SPTs) were performed while obtaining the split-spoon samples to determine the N-values[1] of the sampled material.

As each boring was advanced, the Drilling Foreman kept a field log of the subsurface profile noting the soil and bedrock types and stratifications, groundwater, SPT results, and other pertinent data. Observations for groundwater were made in the borings during drilling and at the completion of drilling. The bore holes were backfilled immediately, such that long-term water readings could not be taken.

Representative portions of the split-spoon samples were placed in glass jars with lids to preserve the in-situ moisture contents of the samples. The Shelby tubes were capped and taped at their ends to preserve the in-situ moisture contents and densities of the samples, and the tubes were transported and stored in an upright position. The glass jars and Shelby tubes were marked and labeled in the field for identification when returned to our laboratory.

4.0 LABORATORY REVIEW AND TESTING

Upon completion of the fieldwork, the samples recovered from the borings were transported to our Soil Mechanics Laboratory, where they were visually reviewed and classified by the Project Geotechnical Engineer.

Laboratory testing was performed on selected soil and rock samples to estimate engineering and index properties. Laboratory testing of the selected soil samples included various combinations of the following tests: moisture content, Atterberg limits, gradation (particle-size) analyses, and unconfined compressive strength. The results of these tests are summarized in the Tabulation of Laboratory Tests in Appendix D, along with the corresponding laboratory test forms. Additionally, the results of laboratory index tests are presented on the boring logs and are summarized on the cross sections.

The final boring logs were prepared by the Project Geotechnical Engineer on the basis of the field logs, the visual classification of the soil and bedrock samples in the laboratory, and the

[1] The Standard Penetration Test Value, or N-value, is defined as the number of blows required to drive the split-spoon sampler 12 inches with a 140-pound hammer falling 30 inches. Since the split spoon sampler is driven 18 inches or until refusal, the blows for the first 6 inches are for seating the sampler, and the number of blows for the final 12 inches is the N-value. Additionally, "refusal" of the split-spoon sampler occurs when the sampler is driven less than 6 inches with 50 blows of the hammer.

laboratory test results. The final boring logs are included in Appendix C. Soil and Rock Classification Sheets are also included in Appendix C, which describe the terms and symbols used on the boring logs. The dashed lines on the boring logs indicate an approximate change in strata as estimated between samples, whereas a solid line indicates that the change in strata occurred within a sample where a more precise measurement could be made. Furthermore, the transition between strata can be abrupt or gradual.

5.0 SUBSURFACE CONDITIONS

5.1 Stratification

In general, two different subsurface profiles were encountered in the borings.

West of approximate Station 17+50, the subsurface profile consists primarily of artificial fill, underlain by native glacial and residual lean clay and fat clay soils underlain by relatively shallow interbedded shale and limestone bedrock. An exception is that native colluvial lean clay soils were encountered in three of the borings above the residual soil and/or bedrock. Bedrock was encountered on the order of 10 feet deep or less in this section.

East of approximate Station 17+50, the borings encountered deeper glacial outwash deposits consisting of intermixed layers of cohesive and granular soils, underlain by the residual soils and interbedded shale and limestone bedrock. The depth to bedrock was encountered as deep as 58.3 feet below the ground surface where deeper borings were performed.

The different subsurface profiles are similarly depicted on the referenced USGS bedrock geology mapping as discussed in Section 5.1.6 of this report. More specific descriptions of the subsurface strata are provided below, and the boring logs containing detailed material descriptions are located in Appendix C.

5.1.1 Pavement & Topsoil

The asphalt pavement was encountered in each boring performed in the roadway. Six of the 22 borings were performed outside of the roadway. The asphalt pavement was measured to be approximately 2.5 to 18 inches thick in the roadway borings. Granular base was encountered beneath the asphalt pavement in eight of the 16 roadway borings and was measured to be approximately 2 to 12 inches thick.

Topsoil was only encountered in Boring 202 and was noted to be 3 inches thick.

5.1.2 Artificial Fill

Artificial fill was encountered beneath the ground surface or the asphalt pavement in 12 of the 22 borings. The thickness of the artificial fill, where encountered, ranged from 0.3 to 4.5 feet. The artificial fill was described as varying degrees of brown, black, gray and green in color and typically included intermixed layers of lean clay, fat clay and sandy lean and fat clays with and without asphalt fragments, limestone fragments, gravel, brick fragments, shale fragments, topsoil, limestone floaters, roots, oxide concretions and oxide stains. The consistencies of the fill widely ranged from very soft to stiff with N-values ranging from 2 to 22 blows per foot (bpf). The majority of the N-values were 12 bpf or less. One N-value was greater than 50 bpf, which was due to encountering limestone floaters larger than the size of the sampler opening. It is noted that 2 layers of granular artificial fill were encountered in Borings 205 and 222 at depths less than 3 feet below the ground surface. Their descriptions primarily included asphalt fragments and limestone floaters. Two moisture contents of the artificial fill were 23.3 and 25.2 percent.

5.1.3 Glacial Outwash Deposits

Glacial outwash deposits are soils that have been deposited, transported, or reworked in place by the advancement or retreat of a glacier across the area. The glacial soils consisted of intermixed layers of lean clay, fat clay, sand and silt, which were either cohesive or granular in nature. The glacial soils were encountered beneath asphalt pavement and/or artificial fill in 15 of the 22 borings. The thickness of the glacial soils ranged from 1.3 feet to as much as 55.8 feet. The glacial soil was described as varying degrees of brown, orange, red and gray in color and was noted to be with and without gravel, oxide stains and concretions, shale fragments, limestone fragments and floaters, and roots.

The granular soils were encountered in seven of the 22 borings. Consistencies widely varied from very loose to dense with N-values ranging from 2 to 50 bpf, with the majority of the N-values below 14 bpf. Three moisture contents of the granular soil were 20.6, 23.0 and 30.2 percent. Atterberg limits tests were performed on three samples of the glacial soil in accordance with AASHTO T89 & T90 methods. The samples were classified as the following soil types according to the Unified Soil Classification System and AASHTO methods: SC (A-2-6), SC-SM (A-2-4), and CL (A-6). Particle-size analyses were performed on these three samples as well, and the results of those tests are shown on the Tabulation of Laboratory Tests and the test forms, which are both included in Appendix D of this report. One sample of the clayey sand had an unconfined compressive strength of 1,730 pounds per square foot (psf) and a natural dry density of 95.0 pounds per cubic foot (pcf).

The cohesive soils were encountered in 14 of the 22 borings. Consistencies widely varied from soft to very stiff with N-values typically below 16 bpf. Several moisture contents of the cohesive soil ranged from 14.8 to 31.4 percent, with the majority of the values being in the low to mid-twenties. Five Atterberg limits tests were performed on samples of the cohesive soil, which resulted in classifications of either CL or CH (A-7-6, A-6, or A-4) soil types. Particle-size analyses were performed on these five samples as well, and the results of those tests are shown on the Tabulation of Laboratory Tests and the test forms, which are included in Appendix D of this report. Unconfined compressive strengths of three cohesive soil samples ranged from 1,630 to 4,420 psf with natural dry densities ranging from 97.9 to 121.7 pcf.

5.1.4 Colluvium

Colluvial soils (or colluvium) are formed by the downslope movement of soil and rock material under the influence of gravity. Colluvium is often characterized by a dense clay matrix with randomly oriented shale fragments, limestone fragments and limestone floaters. Colluvial soils are a sign of current and/or past instability. The native colluvium was encountered beneath the asphalt pavement and/or artificial fill, in Borings 203, 104 and 9, below depths of 1.0, 4.5 and 3.9 feet, respectively. The thickness of the colluvium ranged from 1.5 to 3.4 feet. The colluvium was described as varying degrees of brown, medium stiff or stiff lean clay, with and without roots, shale fragments, limestone fragments and limestone floaters. The N-values of the colluvium ranged from 6 to 20 bpf. The moisture content of one sample of colluvium was 19.5 percent.

5.1.5 Residuum

Residual soils (or residuum) are formed by the in situ weathering of the underlying parent bedrock into a soil and can be identified by traces of horizontal bedding planes within the soil. Residual soils were encountered beneath the topsoil, the artificial fill, the glacial soils and/or the colluvium in 12 of the 22 borings. The thickness of the residual soils ranged from 1.5 to 10.0 feet and were described as brown, or olive brown in color, with red and gray descriptions in the deeper layers. The residual soil consisted primarily of lean clay and fat clay with and without roots, oxide stains and concretions, limestone fragments and floaters, and traces of bedding planes. The consistency of the residual soil was medium stiff, stiff, or very stiff with typical N-values ranging from 7 to 25 bpf. A few N-values were higher than 50 bpf, which was due to encountering limestone floaters larger than the size of the sampler opening.

Four moisture contents of the residual soil were 18.4, 19.6, 20.5 and 25.8 percent. Two samples of the residuum classified as A-6/CL soils. Particle-size analyses were performed on these two samples as well, and the results of those tests are shown on the Tabulation of Laboratory Tests and the test forms, which are included in Appendix D of this report.

5.1.6 Bedrock

The artificial fill and native overburden glacial, colluvial and residual soils at the site are underlain by bedrock consisting of interbedded shale and limestone layers. Bedrock was encountered in all but five of the borings.

According to the 1971 USGS Geologic Map of Part of the Covington Quadrangle, Northern Kentucky, the bedrock immediately underlying the overburden soils along the western third of the project belongs to the Fairview Formation. The referenced map indicates that the ground surface along the eastern two-thirds of the project site is underlain by deep glacial outwash deposits as described earlier in this section of the report.

The referenced USGS Map indicates that the Fairview Formation is comprised of interbedded shale and limestone, with the shale being 45 to 60 percent of the formation and limestone comprising the remaining 40 to 55 percent of the formation. The shale is described as greenish-gray to medium-gray, weathers dusky yellow and light olive gray, and laminated to thinly bedded. The limestone is described as medium-gray or medium-light gray, fine or coarse-grained, in even to irregular and sometimes lenticular beds, generally less than 4 inches thick, but locally up to 10 to 15 inches thick.

Bedrock in the Northern Kentucky Area is typically categorized as highly weathered, weathered, or unweathered, based on the degree of weathering of the shale component. The highly weathered zone is typically the uppermost zone, wherein the shale is brown in color and has almost weathered to a clay. In the intermediate weathered zone, the shale is typically olive brown with occasional gray and is stronger than the shale in the highly weathered zone. In the unweathered parent zone, the shale is gray and is stronger than the shale in the weathered zones. The shale in each of the three (3) zones is interbedded with limestone. It is not uncommon for one or both of the weathered shale bedrock zones to be absent due to differential weathering, erosion, or prior excavation. The Rock Classification Sheet, which is included in Appendix C, describes the varying degrees of weathering along with the rock strength descriptions that are used on the boring logs. The depth to the surface of the bedrock ranged from 4.5 to 9.5 feet in the western third of the project, and then transitions to between 14.5 and 58.3 feet along the eastern two-thirds of the project.

Interbedded highly weathered shale and limestone bedrock was encountered in 14 of the 22 borings. The thickness of the highly weathered zone, where encountered and penetrated, ranged from 2.5 to 5.0 feet. Moisture contents of selected samples of the shale from the highly weathered zone ranged from 11.9 to 19.5 percent.

The interbedded weathered shale and limestone zone of bedrock was encountered in 11 of the 22 borings. The thickness of the weathered zone, where encountered and penetrated, ranged from 2.5 to more than 5.8 feet. Moisture contents of selected samples of the shale from the weathered zone ranged from 5.7 to 14.9 percent.

The surface of the interbedded unweathered shale and limestone bedrock was encountered in 12 of the 22 borings below depths ranging from 12.0 to 17.0 feet in the western third, transitioning from 17.0 to 53.3 feet in the eastern two-thirds. Moisture contents on selected samples of the shale from the unweathered zone ranged from 3.5 to 3.8 percent.

Regarding the limestone, these layers are predominantly unweathered, and their strengths are estimated to range from medium strong to very strong (i.e., uniaxial compressive strengths ranging from 4,000 psi to upwards of 30,000 psi). Occasionally, layers are encountered within the bedrock profile where groundwater seepage is concentrated and weathering of the limestone layers is more advanced.

5.2 Groundwater Conditions

All but five of the borings were noted to be dry during and upon completion of drilling. In the remaining five borings, groundwater was encountered within the bedrock, or within lenticular deposits of the coarser-grained deeper glacial soils. The borings were backfilled immediately, therefore, no static water readings could be taken.

Based on our local experience, periodic groundwater seepage can occur as perched water within artificial fill, at the fill/native soil interface, in lenticular deposits of coarser-grained soils, at the soil/bedrock interface, and along limestone layers within the bedrock. Locally concentrated flow may occur along fractures within the bedrock or within saturated zones of overburden soils that were not encountered by the borings. Groundwater levels and seepage/flow rates are expected to vary with time, location, and amount of precipitation.

6.0 CONCLUSIONS AND RECOMMENDATIONS

Based on our engineering reconnaissance of the site, the borings, a visual examination of the recovered samples, the laboratory test results, our understanding of the proposed project, our engineering analyses, and our experience as Consulting Soil and Foundation Engineers in the Northern Kentucky Area, we have reached the following conclusions and make the following recommendations of this report.

6.1 Site and Subsurface Condition Summary

For the approximate western half of the project, Amsterdam Road climbs uphill from west to east along the approximate mid-slope of a hillside that slopes down to the south toward a valley bottom that drains into a tributary of Pleasant Run Creek. The downslope side of the road ranges in gradient from approximately 3 to 2 horizontal to 1 vertical (3 to 2H:1V) from west of General Drive to just west of Morris Road. Two potential landslide areas were observed along the downslope side of the road between addresses 1731 and 1739, where fill was placed along the south edge of the road. The eastern half or so of the project, from just west of Morris Road to just east of Redwood Drive, is located on gently sloping ridgetop terrain. In addition, possible landslide ground movement was observed on the slope between Amsterdam and Ridgewood Roads.

In general, two different subsurface profiles were encountered in the borings. West of approximate Station 17+50, the subsurface profile consists primarily of artificial fill, then glacial and residual lean clay and fat clay soils, underlain by relatively shallow interbedded shale and limestone bedrock. Native colluvial lean clay soils were encountered in three of the borings above the residual soil and/or bedrock as well. Bedrock was encountered on the order of 10 feet deep or less in this section. East of approximate Station 17+50, the borings encountered deeper glacial outwash deposits consisting of intermixed layers of cohesive and granular soils, underlain by deeper residual soils and interbedded shale and limestone bedrock. The depth to bedrock was encountered as deep as 58.3 feet below the ground surface in this section.

6.2 Storm Sewer Installation

Approximately 2,253 linear feet of 12-inch diameter storm sewer will be installed beneath the roadway and/or curb and gutter locations throughout the project alignment. The proposed storm sewer will be installed by traditional cut and cover methods with invert levels approximately 5 to 8 feet below the existing ground surface, which is approximately 5 to 9 feet below the proposed grades. Based on the boring results, the storm sewer trench excavations west of about Station 17+50 will encounter primarily artificial fill and native soils consisting of lean and fat clays, with some of the excavations potentially encountering the interbedded shale and limestone bedrock. East of about Station 17+50, the trench excavations will transition into the deeper glacial profile, which consists of intermixed layers of lean clay, fat clay, sand and silt. It should be noted that the artificial fill soils, as well as the native overburden soils may contain shale fragments, limestone fragments and limestone floaters throughout the project area. In addition, the Contractor should be aware that glacial soils that are granular in nature will tend to cave during trench excavation, and may require sheeting, shoring or other means of support by the Contractor to maintain safe working conditions in the trenches, and to protect pavement, structures and infrastructure near the trenches.

The Contractor should recognize that the majority of the storm sewer installation is located within close proximity to or crossing above or below other existing utilities. All of the pressurized pipelines (e.g., water mains, gas mains, etc.) in these areas should be evaluated by the Contractor to determine the impact of the potential excavation and backfilling operations on the current thrust restraint of these pipelines. These evaluations should highlight critical areas where thrust restraint must be maintained and/or supplemented during the storm sewer installations. This was not included in our scope of services and was not performed by Geotechnolgy.

In our opinion, trench excavations into the artificial fill, the native soils, and into the interbedded highly weathered shale and limestone bedrock can be made with large track-mounted hoes. If the excavations proceed deeper into the zone of weathered bedrock, the difficulty of making trench excavations will increase in relation to the frequency, thickness, and nature of the intact interbedded limestone layers encountered and the strength of the shale layers. Excavation difficulties will increase substantially if the excavations proceed down into the unweathered zone of interbedded shale and limestone bedrock. In the weathered zone of the bedrock, large track-mounted hoes with ripping teeth, and possibly hoe rams, may be required. Hoe rams, track hoes with ripping teeth, and/or rock saws would be required to penetrate into the stronger unweathered zone of bedrock, if encountered. Blasting of the bedrock is typically not permitted in urban settings. The Contractor should be aware of the possibility of encountering the bedrock in some locations and should be prepared for the difficulty that bedrock excavation may present. As previously mentioned, limestone layers are predominantly unweathered, and their strengths are estimated to range from medium strong to very strong (i.e., uniaxial compressive strengths ranging from 4,000 psi to upwards of 30,000 psi).

In the excavations that encounter bedrock, the Contractor should select a method of excavating the bedrock that will allow adequate care to be taken to protect nearby structures and infrastructure from vibration or other potential damage that might be caused during bedrock excavation. In addition, the Contractor should note that it is difficult to shear limestone layers neatly in the sides of trench excavations. Frequently, when limestone layers are encountered in trench excavations, the tendency is for the layers not to break even with the sides of the excavations, but rather to be pulled up in larger chunks, which tend to heave and ravel the bedrock and soils outside the limits of the intended trench. Where trench excavations will be made immediately adjacent to existing pavements, structures, or utilities, it should be anticipated that there will be areas where this heave and raveling due to removal of limestone layers could cause collateral damage to the features adjacent to the trench, and the damaged items would have to be restored. In areas where such collateral damage cannot be tolerated, the bedrock should be pre-cut or sheared to reduce the risk of heaving and raveling outside the intended limits of the trench excavation. We recommend that the Contractor complete preconstruction surveys of the property located near the proposed alignment to document existing conditions prior to any excavations.

The scope of this project included borings that were performed at widely spaced intervals. Therefore, we recommend that the specifications for this project be based on unclassified excavation, not on separate cost items for soil excavation and bedrock excavation. We recommend that the base bid for the project include the cost of excavating the materials encountered within the specified depths, regardless of soil or bedrock characteristics.

It is noted that some of the artificial fill and native overburden soils encountered within the borings were very soft to medium stiff in consistency. If soft or unstable soils are encountered at the bottom of the trench excavations, we recommend that the unstable materials be undercut to stiff native soils, bedrock, or to a maximum depth of 18 inches below the pipe invert level, for the full trench width and be replaced with compacted crushed stone to provide a stable trench bottom. We recommend the compacted crushed stone be wrapped with a non-woven geotextile to minimize the migration of fine-grained soils and fine granular bedding into the crushed stone. The depth of the undercut and crushed stone fill below the pipe invert will vary with the unstable soil conditions encountered, but can be limited to a maximum of 18 inches below pipe invert level. We recommend the crushed stone backfill be placed and compacted in accordance with the recommendations for backfilling presented in the second to last paragraph of this section, and that the specified pipe bedding over the geo-textile wrapped compacted crushed stone.

We recommend that the Contractor be responsible for the stability and safety of excavations and should exercise necessary precautions to shore, slope or otherwise maintain stable trench excavations to protect workers, surrounding ground, adjacent pavement, structures, and infrastructure, including utilities. It is noted that there are several utilities located along the proposed storm sewer alignment that will be crossed or will be paralleled. These trenches should be made and maintained in accordance with all Federal, State and Local regulations.

The Contractor should be required to dewater excavations for this project, and to maintain the excavations in a dewatered condition so that the pipe bedding, pipe materials, and backfill can be placed and compacted under dry conditions.

If granular soils, lean clay or shale backfill is allowed in the trenches, we expect that the excavated materials, exclusive of the thick limestone layers, can be used as backfill after the appropriate granular pipe bedding and backfill is installed. Fill materials should not include asphalt, concrete, trash, construction or demolition debris, topsoil or frozen material. Large pieces of limestone, which tend to nest or retard compaction, should be excluded from the backfill. Smaller pieces of limestone that can be broken up and dispersed so that they do not nest or retard compaction can be incorporated in the backfill provided that proper protection of the pipe from these pieces of limestone is provided.

Normal and recommended utility construction practice is to bed and backfill pipes with granular fill to a specified height above the crown of the pipe. Compaction of pipe bedding and trench backfill to a moist, firm, dense condition is important throughout the alignment as new pavements will be constructed above the backfilled excavations. Granular bedding and backfill should be compacted to at least 80 percent relative density per ASTM D4253 and D4254 for soils that do not exhibit a well-defined moisture-density relationship, or to at least 95 percent of the standard Proctor maximum dry density, ASTM D698, for soils that exhibit a well-defined moisture-density relationship. We recommend that granular soils, silty clay, clay and shale backfill for this project be placed in shallow level layers, 4 to 8 inches in maximum loose thickness, and be compacted to densities not less than 95 percent of the standard Proctor maximum dry density, ASTM D698. The backfill soils should be moisture-conditioned to within the range of 2 percent below to 3 percent above the optimum moisture content at the time of compaction. Shale should be pulverized to a soil-like consistency and moisture conditioned the same as a soil. If the trench backfill was poorly placed and/or compacted, it would likely settle under its own weight over time and in turn, cause settlement of the overlying new pavements after construction. As such, we recommend that the trench backfilling procedures be reviewed and the compaction of the backfill be tested for moisture and density on a regular basis by a Soils Technician under the direction of a Registered Professional Geotechnical Engineer. We recommend that we be retained to provide these services.

If flowable fill is used, we recommend that it have a design strength of at least 30 psi for stability and not greater than 100 psi for future excavatability.

6.3 Roadway Design & Construction

Proposed fill amounts up to about 8 feet and cuts up to about 2 feet will be required to reach the proposed grades. It is our understanding that the proposed pavement section will be selected by CT and will be installed in accordance with the Kenton County Subdivision Regulations.

6.3.1 Pavement Demolition

We recommend that the pavement demolition include the removal of the existing asphalt pavement in full. In addition, the existing granular base, which was primarily encountered in the eastern half of the boring locations, should be undercut as necessary to provide enough thickness for the new full-depth (new asphalt and new granular base) pavement section.

If vegetation, topsoil and/or heavy root systems are encountered within the proposed roadway limits, we recommend their removal from the proposed cut, fill, pavement and structure areas prior to the placement of new fill or new pavement section. The vegetation, including the heavy root systems, should be disposed of off site in accordance with applicable regulations. Topsoil should be stockpiled for landscape purposes if permitted by the specifications.

The spoils from the undercut excavations, including the existing pavement, vegetation, and/or heavy root systems, should be hauled off site. The undercut granular base materials are suitable for use in the new subgrade fills provided they are well-mixed with clayey soils prior to use. It is our opinion that placing additional fill on the slope above or below the road, except as shown on the CT Project Plans, could act as a driving force to the already creeping and/or landsliding hillside, resulting in a risk of future slope instability.

6.3.2 Poor Subgrade Undercut Recommendation

After the pavement demolition and undercut operations, and after making the required excavations in the cut areas, the exposed subgrade should be thoroughly proofrolled using a heavily loaded piece of rubber-tire equipment, such as a loaded single-axle dump truck, under the review of the Project Geotechnical Engineer or a representative thereof. Soft or yielding soils observed during the proofrolling should be undercut to stiff non-yielding cohesive soils or medium dense to dense well-graded cohesionless soils. We recommend that the depth of undercut below proposed subgrade be limited to 12 inches or less. If soft or yielding soils are encountered at the recommended maximum undercut depth of 12 inches, the subgrade should be stabilized at those depths using an approved biaxial or triaxial geogrid (e.g., Tensar BX-1200 or TriAx TX160) and two 6-inch thick lifts of compacted crushed stone. We recommend that the Contract Documents include a bid item for the recommended undercutting, as deemed necessary, and their replacement with new compacted and tested fill on a “per cubic yard of in-place compacted fill” basis.

6.3.3 Site Fill

Fill materials should consist of approved on-site clay soils, bedrock, or approved borrow materials that are relatively free of topsoil, vegetation, trash, construction or demolition debris, frozen materials, particles over 6 inches in maximum dimension, (such as limestone floaters), or other deleterious materials.

The existing artificial fill, the native overburden soils and bedrock from the proposed cuts and storm sewer trench excavations, and the poor subgrade undercut soils are, in our opinion, suitable for use in the new fills provided that these soils conform to the recommendations contained in this report regarding suitable fill materials.

The shale and limestone bedrock is, in our opinion, suitable to be incorporated into the fill provided that the shale is pulverized to a soil-like consistency and moisture-conditioned, and provided that the limestone is broken up and dispersed so as not to cause nesting or retard compaction. The maximum dimension of the broken-up limestone floaters in the fills should be limited to 6 inches. Larger pieces of limestone, if not capable of being broken up, should be wasted off site.

The new fill should be placed on generally level surfaces, in shallow level lifts (or layers), 6 to 8 inches in loose thickness. Each lift should be moisture-conditioned to within the recommended moisture content range provided in Table 1, and compacted with a sheepsfoot roller or self-propelled compactor to at least the minimum percent compaction indicated in the same table. Moisture-conditioning may include: aeration and drying of wetter soils; wetting drier soils and/or shales; and/or thoroughly mixing wetter and drier soils into a uniform mixture.

Table 1. Percent compaction and moisture-conditioning recommendations for fill and backfill.

Area	Minimum Percent Compaction[a,b]	Acceptable Moisture Content Range[c]
Pavement subgrade ≤ 12 inches below subgrade	100% of SPMDD	0% to +2% of OMC
Non-pavement subgrade	95% of SPMDD	-2% to +3% of OMC

[a] SPMDD = standard Proctor maximum dry density determined from ASTM D698.
MPMDD = modified Proctor maximum dry density determined from ASTM D1557.
[b] For granular soils that do not exhibit a well-defined moisture-density relationship, refer to Section 6.2 for minimum relative density requirements.
[c] OMC = optimum moisture content determined from ASTM D698 or ASTM D1557.

Groundwater is not expected to have a significant adverse effect on the proposed earthwork construction; however, the Contractor must be prepared to remove seepage that accumulates in excavations, on fill surfaces or at subgrade levels.

Maintaining the moisture content of pavement subgrade soils within the recommended range provided in Table 1 is important during and after construction. The subgrade soils should not be allowed to become excessively wet or dried during or after construction, and measures should be taken to prevent water from ponding on these soils and to prevent these soils from desiccating during dry weather.

Surface drainage should be directed away from the edges of proposed or existing pavements so that water does not pond next to pavements or flow onto pavements from unpaved areas. Such ponding or flow can cause deterioration of pavement subgrades and premature failure of pavements. If drainage ditches are used to intercept surface water before it reaches the pavements, the ditches should have an invert at least 6 inches below the pavement subgrade and have a sufficient longitudinal gradient to rapidly drain the ditches and prevent ponding of water. In those areas where exterior grades do not fully slope away from the edges of the proposed pavement, we recommend that edge drains be installed along the perimeter of the pavement. It is our understanding that edge drains will be installed along both edges of the proposed pavement for this project.

We recommend that the earthwork operations be carried out during the drier season of the year and that a sufficient gradient be maintained at the ground surface to prevent ponding of surface water. In our experience, the weather conditions are historically more favorable for earthwork during the months of May through October in the Northern Kentucky Area. Regardless of the time of year, asphalt, concrete, or fill should not be placed over frozen or saturated soils, and frozen or saturated soils should not be used as compacted fill or backfill.

Best management practices (BMPs) should be implemented to reduce the effects of erosion and the siltation of adjacent properties. Upon completion of earthwork, disturbed areas should be stabilized.

6.3.4 Subgrade Preparation and Aggregate Base

Prior to the placement of asphalt pavement or aggregate base, the proposed pavement subgrades should be proofrolled with a heavily loaded piece of rubber-tire equipment, such as a loaded single-axle dump truck, under the review of the Project Geotechnical Engineer or representative thereof. Soft or yielding soils observed during the proofroll should be undercut to stiff, non-yielding soils; however, we recommend the depth of undercut below subgrade be limited to 12 inches as previously discussed. The undercuts should be backfilled in accordance with 6.3.2 of this report. We recommend that the top 8 inches of clayey subgrade be scarified and recompacted per the requirements presented in Table 1.

We recommend that caution be exercised so that the proposed aggregate base does not become saturated during or after construction. Water trapped in the aggregate base is capable of freezing, causing it to expand within the voids it occupies. Consequently, ice lenses may form and potentially heave the pavement. Furthermore, the thawing process can soften underlying cohesive subgrades, which reduces the pavement support provided by the subgrade, giving rise to “pumping” of the pavements under loads. Preferably, the aggregate base should be a free-draining material with provisions for draining the base through a system of underdrains.

6.4 Proposed Retaining Wall

6.4.1 General Conditions

As previously mentioned, a retaining wall will be constructed along the downslope side of the road, between approximate Stations 9+85 and 21+90. The wall will have an exposed height ranging from about 2 to 8 feet above the existing ground surface in order to reach the proposed grades.

The boring results indicate that two different subsurface conditions exist along the roadway project. West of approximate Station 17+50, the subsurface profile consists primarily of relatively shallow sloping bedrock with a depth of 10 feet or less. East of approximate Station 17+50, the borings encountered deeper glacial soils with a depth to bedrock transitioning from about 17.0 to 58.3 feet below the ground surface. The gradient of the bedrock surface transitions from sloping at the west end of the project to relatively flat near approximate Station 17+50.

Prior to our site reconnaissance, borings, and analysis of the cross sections, it was discussed that bedrock embedment of the proposed retaining wall might be recommended throughout the entire roadway alignment due to the steep hillside conditions. However, based on our observations in the site reconnaissance, in conjunction with a review of the Cross Sections shown on Drawing Nos. 102 through 104, it is our opinion that bedrock embedment will be necessary from about Station 9+85 to Station 17+50 where bedrock is relatively shallow, sloping bedrock is a factor, and possible landslides were observed in the field. East of Station 17+50, where generally flat bedrock and deep glacial soils exist, it is our opinion that bedrock embedment of the proposed retaining wall will not be necessary.

6.4.2 Retaining Wall Construction Considerations

During the project planning stage, consideration was given to different types of retaining walls and methods of construction which included open cut benching for a conventional concrete cantilevered retaining wall and top down construction for drilled shaft or driven pile installations.

The open cut cantilevered concrete retaining wall has disadvantages including space limitations, interference with multiple existing utilities, potential deep excavation required to found the wall on bedrock, and seasonal weather construction issues, so this option was not recommended.

The driven pile option has disadvantages including high cost due to expensive mobilization fees and expensive steel members, difficulty with driving piles into the bedrock, and excessive noise and vibration during installation in a residential area, so this option was not recommended.

The drilled shaft option has advantages including accessibility due to space limitations, moderate drill rig mobilization cost, less noise and vibration than driven piles, and all weather installation.

Based on the disadvantages of the other options and the advantages of the drilled shaft option, it was preliminarily recommended that the entire length of the retaining wall consist of reinforced concrete drilled shaft construction.

6.4.3 Drilled Shaft Wall Discussion

The formal design of this wall will be subsequently submitted under separate cover. The drilled shafts west of approximate Station 17+50 will socket into and cantilever out of the bedrock. The drilled shafts east of approximate Station 17+50 will be designed to bear a sufficient depth into the glacial soils as necessary to resist the appropriate lateral earth pressures in that section.

In general, the retaining wall will have a total width of approximately 3 feet, which will include reinforced drilled shafts, most likely 24 to 30 inches in diameter with an anticipated center to center spacing of 3 to 4 feet, and unreinforced 24-inch diameter plug shafts. There are some locations where the retaining wall will have exposed heights up to 8 feet above the existing ground surface. Consideration will be given to using beams and precast concrete panels instead of, or in combination with, plug shafts in order to avoid placing sonotube forms an excessive height above the existing ground surface.

It should be noted that casing of the shafts to avoid cave-in may be required where the granular glacial soils are encountered. We recommend that the Contract Documents include an item for casing shafts on a cost per shaft basis as needed and/or as recommended by the Project Geotechnical Engineer or his/her representative. We recommend that the installation of all drilled shafts be reviewed by the Project Geotechnical Engineer or his/her representative in order to determine whether the design criteria are being met. Specific drilled shaft recommendations will be presented on the formal design drawings.

The drilled shaft spoils should, in general, be suitable for incorporation into the proposed fills. We recommend that the drilled shaft spoils not be placed on the slope below the retaining wall, or in other areas at the project site unless shown on the CT Project Plans. As previously explained, placing fill on the creeping/landsliding hillside could act as a driving force and result in future instability of the hillside. The backfill of the proposed retaining wall should be placed as a compacted and tested fill on level benches in accordance with Section 6.3.3 due to the fact that this fill will be supporting the pavement, curb, and/or roadway shoulder throughout the alignment.

7.0 RECOMMENDED ADDITIONAL SERVICES

The conclusions and recommendations given in this report are based on: Geotechnology's understanding of the proposed design and construction, as outlined in this report; site observations; interpretation of the exploration data; and our experience. Since the intent of the design recommendations is best understood by Geotechnology, we recommend that Geotechnology be included in the final design and construction process, and be retained to review the project plans and specifications to confirm that the recommendations given in this report have been correctly implemented. We recommend that Geotechnology be retained to participate in prebid and preconstruction conferences to reduce the risk of misinterpretation of the conclusions and recommendations in this report relative to the proposed construction of the subject project.

Since actual subsurface conditions between boring locations may vary from those encountered in the borings, our design recommendations are subject to adjustment in the field based on the subsurface conditions encountered during construction. Therefore, we recommend that Geotechnology be retained to provide construction observation services as a continuation of the design process to confirm the recommendations in this report and to revise them accordingly to accommodate differing subsurface conditions. Construction observation is intended to enhance compliance with project plans and specifications. It is not insurance, nor does it constitute a warranty or guarantee of any type. Regardless of construction observation, contractors, suppliers, and others are solely responsible for the quality of their work and for adhering to plans and specifications.

8.0 LIMITATIONS

This report has been prepared on behalf of, and for the exclusive use of, the client for specific application to the named project as described herein. If this report is provided to other parties, it should be provided in its entirety with all supplementary information. In addition, the client should make it clear that the information is provided for factual data only, and not as a warranty of subsurface conditions presented in this report.

Geotechnology has attempted to conduct the services reported herein in a manner consistent with that level of care and skill ordinarily exercised by members of the profession currently practicing in the same locality and under similar conditions. The recommendations and conclusions contained in this report are professional opinions. The report is not a bidding document and should not be used for that purpose.

Our scope for this phase of the project 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 boring logs regarding odors noted or unusual or suspicious items or conditions observed are strictly for the information of our client. Our scope did not include an assessment of the effects of flooding and erosion of creeks adjacent to the project site.

The analyses, conclusions, and recommendations contained in this report are based on the data obtained from the subsurface exploration. The field exploration methods used indicate subsurface conditions only at the specific locations where samples were obtained, only at the time they were obtained, and only to the depths penetrated. Consequently, subsurface conditions may vary gradually, abruptly, and/or nonlinearly between sample locations and/or intervals.

The conclusions or recommendations presented in this report should not be used without Geotechnology's review and assessment if the nature, design, or location of the facilities is changed, if there is a substantial lapse in time between the submittal of this report and the start of work at the site, or if there is a substantial interruption or delay during work at the site. If changes are contemplated or delays occur, Geotechnology must be allowed to review them to assess their impact on the findings, conclusions, and/or design recommendations given in this report. Geotechnology will not be responsible for any claims, damages, or liability associated with any other party's interpretations of the subsurface data or with reuse of the subsurface data or engineering analyses in this report.

The recommendations included in this report have been based in part on assumptions about variations in site stratigraphy that may be evaluated further during earthwork, utility, and wall construction. Geotechnology should be retained to perform construction observation and continue its geotechnical engineering service using observational methods. Geotechnology cannot assume liability for the adequacy of its recommendations when they are used in the field without Geotechnology being retained to observe construction.

A copy of "Important Information about This Geotechnical-Engineering Report" that is published by the Geotechnical Business Council (GBC) of the Geoprofessional Business Association (GBA) is included in Appendix A for your review. The publication discusses some other limitations, as well as ways to manage risk associated with subsurface conditions.

APPENDIX A – IMPORTANT INFORMATION ABOUT THIS GEOTECHNICAL-ENGINEERING REPORT

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.

Geotechnical 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 civil engineer may not fulfill the needs of a constructor — a construction contractor — or even another civil engineer. Because each geotechnical- engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client. No one except you 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 the Full Report

Serious problems have occurred because those relying on a geotechnical-engineering report did not read it all. Do not rely on an executive summary. Do not read selected elements only.

Geotechnical Engineers Base Each Report on a Unique Set of Project-Specific Factors

Geotechnical engineers consider many unique, project-specific factors when establishing the scope of a study. Typical factors include: the client's goals, objectives, and risk-management preferences; the general nature of the structure involved, its size, and configuration; the location of the structure on the site; and other planned or existing site improvements, such as access roads, parking lots, and underground utilities. Unless the geotechnical engineer who conducted the study specifically indicates otherwise, do not rely on a geotechnical-engineering report that was:

- not prepared for you;
- not prepared for your project;
- not prepared for the specific site explored; or
- completed before important project changes were made.

Typical changes that can erode the reliability of an existing geotechnical-engineering report include those that affect:

- 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. *Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.*

Subsurface Conditions Can Change

A geotechnical-engineering report is based on conditions that existed at the time the geotechnical engineer performed the study. *Do not rely on a geotechnical-engineering report whose adequacy may have been affected by*: the passage of time; man-made events, such as construction on or adjacent to the site; or natural events, such as floods, droughts, earthquakes, or groundwater fluctuations. *Contact the geotechnical engineer before applying this report to determine if it is still reliable.* A minor amount of additional testing or analysis could prevent major problems.

Most Geotechnical Findings Are Professional Opinions

Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. Geotechnical engineers review field and laboratory data and then apply their professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ — sometimes significantly — from those indicated in your report. Retaining the geotechnical engineer who developed your report to provide geotechnical-construction observation is the most effective method of managing the risks associated with unanticipated conditions.

A Report's Recommendations Are Not Final

Do not overrely on the confirmation-dependent recommendations included in your report. *Confirmation-dependent recommendations are not final,* because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations *only* by observing actual subsurface conditions revealed during construction. *The geotechnical engineer who developed your report cannot assume responsibility or liability for the report's confirmation-dependent recommendations if that engineer does not perform the geotechnical-construction observation required to confirm the recommendations' applicability.*

A Geotechnical-Engineering Report Is Subject to Misinterpretation

Other design-team members' misinterpretation of geotechnical-engineering reports has resulted in costly problems. Confront that risk by having your geotechnical engineer confer with appropriate members of the design team after submitting the report. Also retain your geotechnical engineer to review pertinent elements of the design team's plans and specifications. Constructors can also misinterpret a geotechnical-engineering report. Confront that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing geotechnical construction observation.

Do Not Redraw the Engineer's Logs

Geotechnical engineers prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical-engineering report should *never* be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, *but recognize that separating logs from the report can elevate risk.*

Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can make constructors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give constructors the complete geotechnical-engineering report, *but* preface it with a clearly written letter of transmittal. In that letter, advise constructors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with the geotechnical engineer who prepared the report (a modest fee may be required) and/or to conduct additional study to obtain the specific types of information they need or prefer. A prebid conference can also be valuable. *Be sure constructors have sufficient time* to perform additional study. Only then might you be in a position to give constructors the best information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions.

Read Responsibility Provisions Closely

Some clients, design professionals, and constructors fail to recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that have led to disappointments, claims, and disputes. To help reduce the risk of such outcomes, geotechnical engineers commonly include a variety of 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.

Environmental Concerns Are Not Covered

The equipment, techniques, and personnel used to perform an *environmental* study differ significantly from those used to perform a *geotechnical* 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 environmental problems have led to numerous project failures.* If you have not yet obtained your own environmental information, ask your geotechnical consultant for risk-management guidance. *Do not rely on an environmental report prepared for someone else.*

Obtain Professional Assistance To Deal with Mold

Diverse strategies can be applied during building design, construction, operation, and maintenance to prevent significant amounts of mold from growing on indoor surfaces. To be effective, all such strategies should be devised for the *express purpose* of mold prevention, integrated into a comprehensive plan, and executed with diligent oversight by a professional mold-prevention consultant. Because just a small amount of water or moisture can lead to the development of severe mold infestations, many mold- prevention strategies focus on keeping building surfaces dry. While groundwater, water infiltration, and similar issues may have been addressed as part of the geotechnical- engineering study whose findings are conveyed in this report, the geotechnical engineer in charge of this project is not a mold prevention consultant; *none of the services performed in connection with the geotechnical engineer's study were designed or conducted for the purpose of mold prevention. Proper implementation of the recommendations conveyed in this report will not of itself be sufficient to prevent mold from growing in or on the structure involved.*

Rely, on Your GBC-Member Geotechnical Engineer for Additional Assistance

Membership in the Geotechnical Business Council of 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. Confer with you GBC-Member geotechnical engineer for more information.

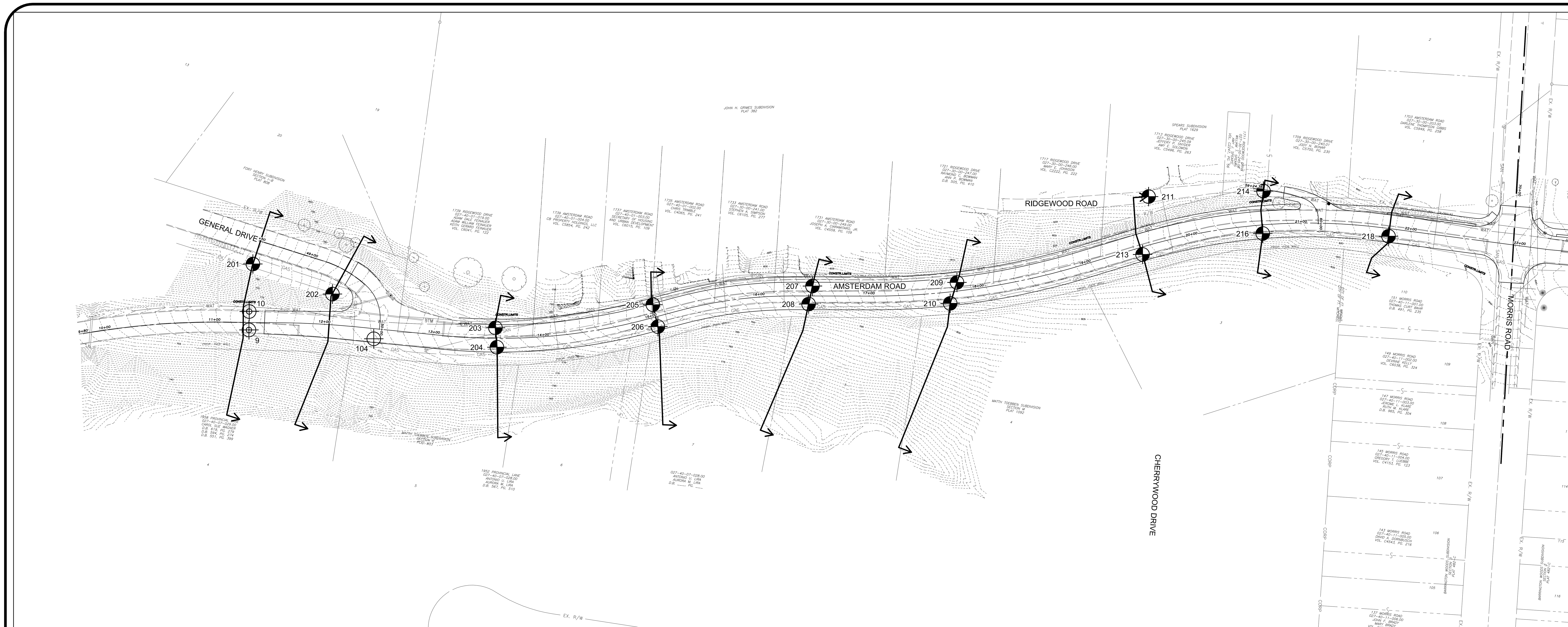
GBC GEOTECHNICAL BUSINESS COUNCIL *of the Geoprofessional Business Association*

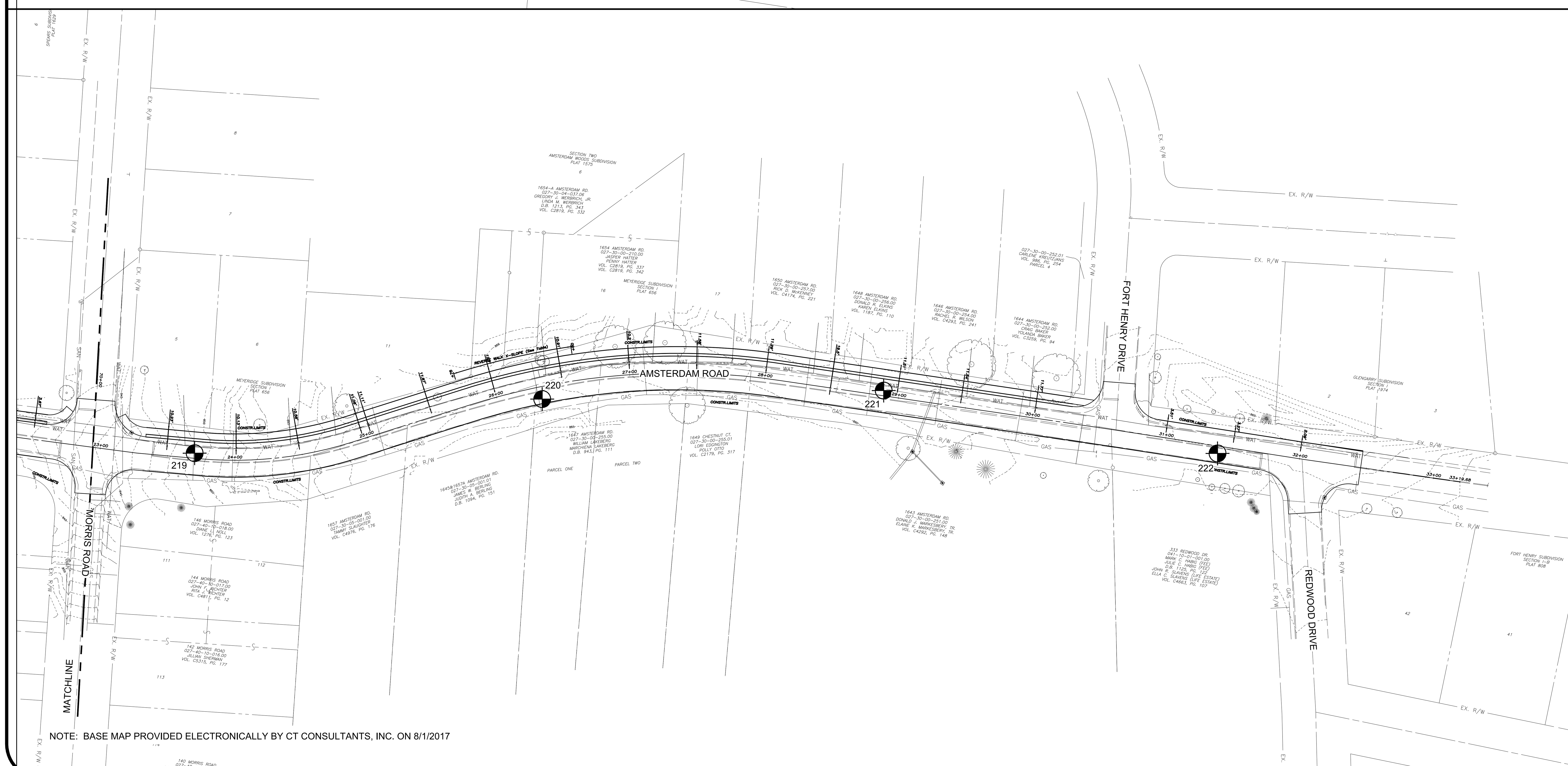
8811 Colesville Road/Suite G106, Silver Spring, MD 20910
Telephone: 301/565-2733 Facsimile: 301/589-2017
e-mail: info@geoprofessional.org www.geoprofessional.org

APPENDIX B – PLANS

Boring Plan, Drawing No. 101

Cross Sections, Drawing Nos. 102 through 104





NORTH

0 25 50 100
SCALE: 1" = 50'

INDICATES CROSS SECTION LOCATION

INDICATES TEST BORING LOCATION 2017 (J029038.01)

INDICATES TEST BORING LOCATION 2008 (070087E)

INDICATES TEST BORING LOCATION 1995 (91018E)

Drawing Revisions

Date:	Description:

GEOTECHNOLOGY INC
FROM THE GROUND UP

Project: Amsterdam Road Reconstruction, Phase 1

Location: Fort Wright, Kentucky

Title: BORING PLAN

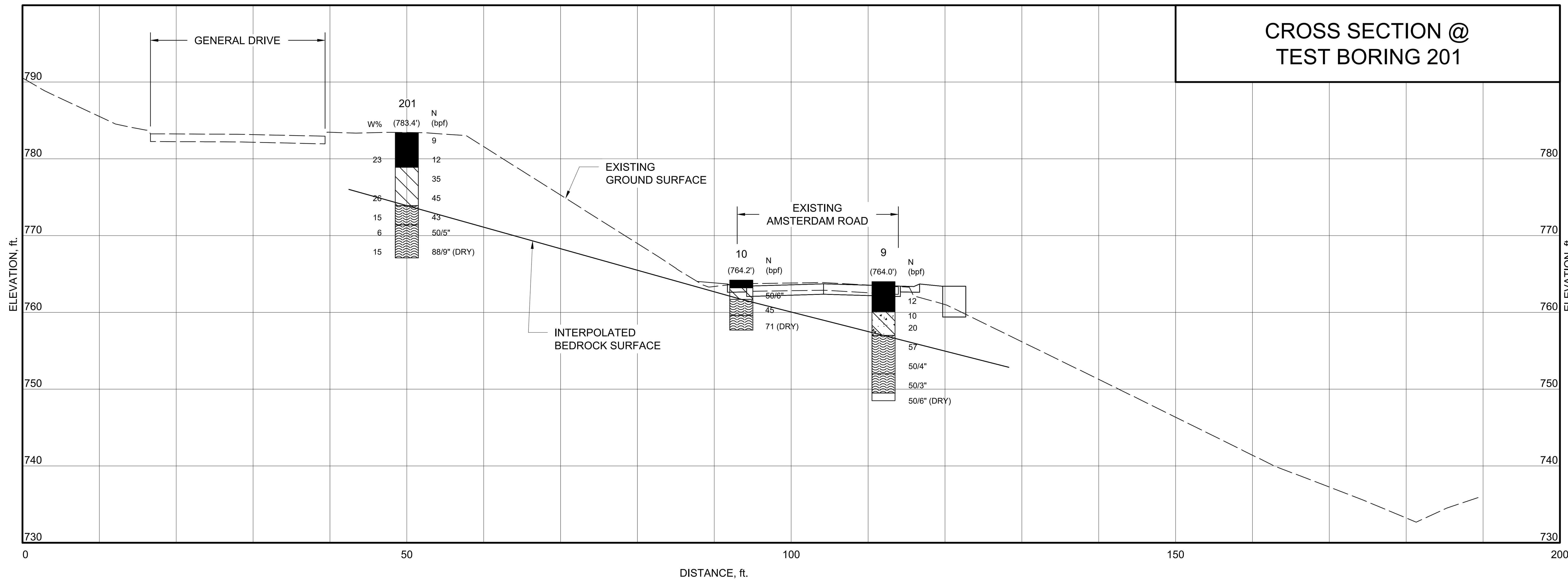
Client: CT Consultants, Inc.

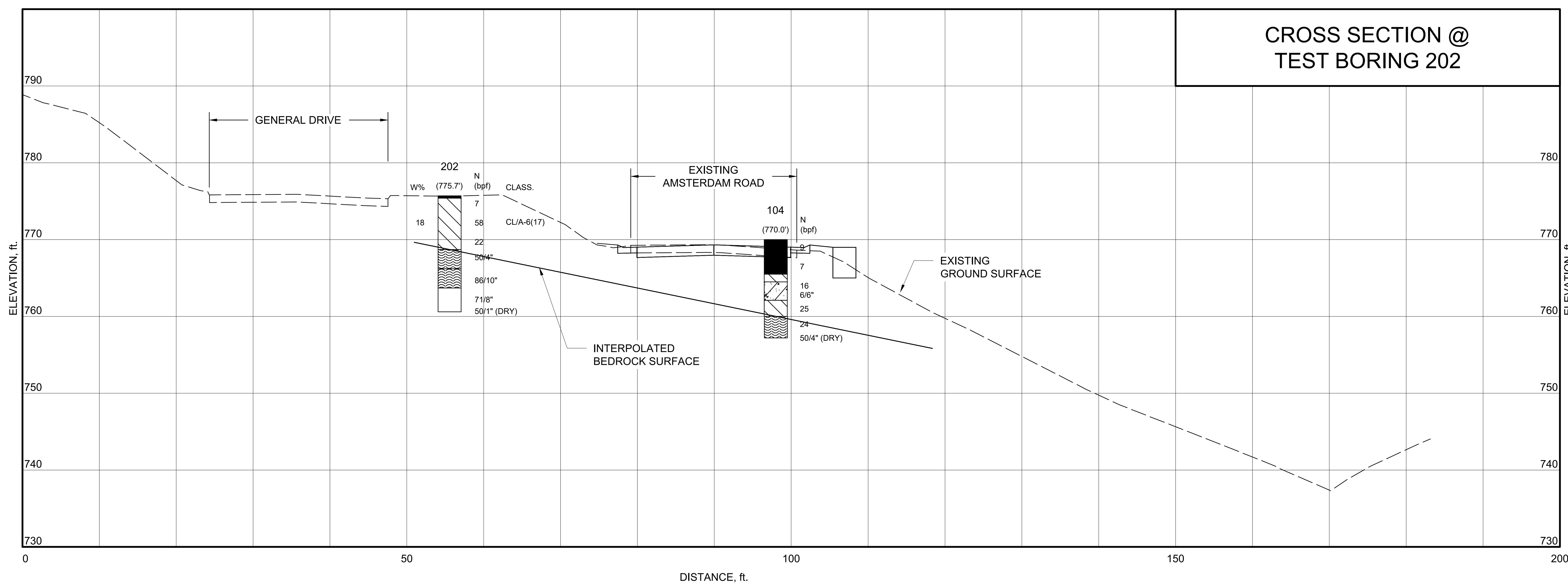
Date: 8/21/2017

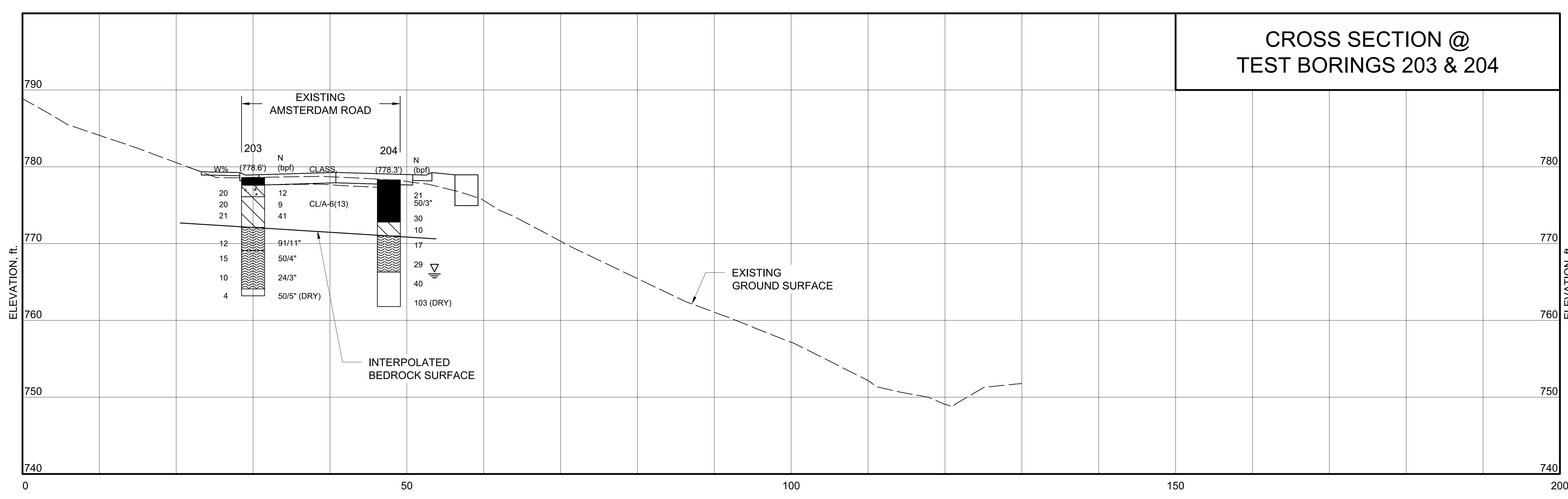
Project No.: J029038.01

Drawing No.: 101

Date Printed: 8/21/2017 5:45 PM Path: \\10.0.10.12\network storage\data\projects\J029038.01\Draw_Plans for Submittal\J029038.01-101-104.dwg







NOTE: PROPOSED GROUND SURFACES ESTIMATED BASED ON SECTIONS PROVIDED ELECTRONICALLY BY CT CONSULTANTS, INC. ON 8/1/2017

LEGEND

- FILL
- SAND
- SILT
- LEAN CLAY
- FAT CLAY
- LEAN CLAY (COLLUVIUM)
- FAT CLAY (COLLUVIUM)
- HIGHLY WEATHERED SHALE AND LIMESTONE BEDROCK
- WEATHERED SHALE AND LIMESTONE BEDROCK
- UNWEATHERED SHALE AND LIMESTONE BEDROCK
- WATER FIRST NOTED
- WATER AT COMPLETION
- (DRY) BOREHOLE DRY AT COMPLETION OF DRILLING

Drawing Revisions

Date: | Description:

GEOTECHNOLOGY INC
FROM THE GROUND UP

Project: Amsterdam Road Reconstruction, Phase 1

Location: Fort Wright, Kentucky

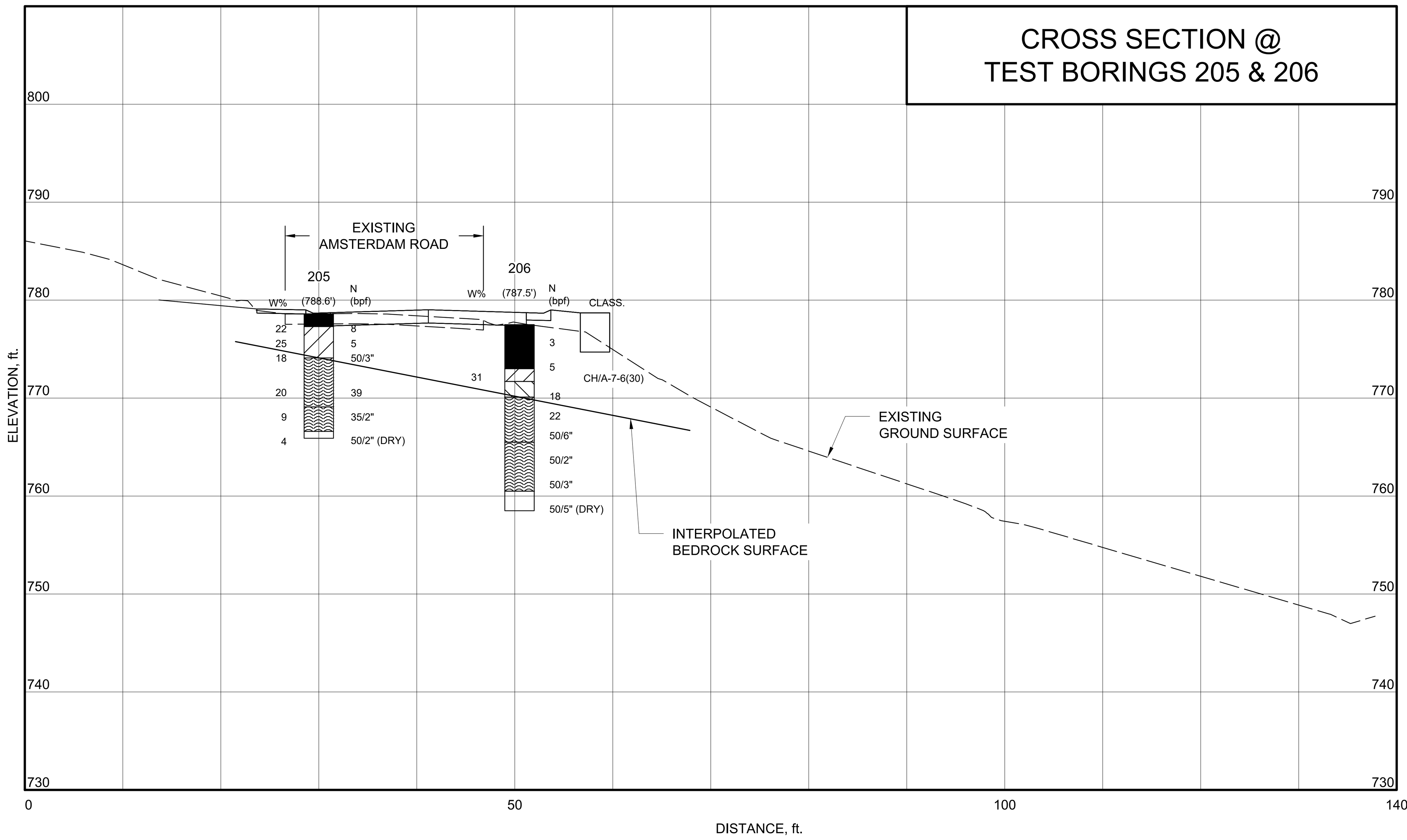
Title: CROSS SECTIONS

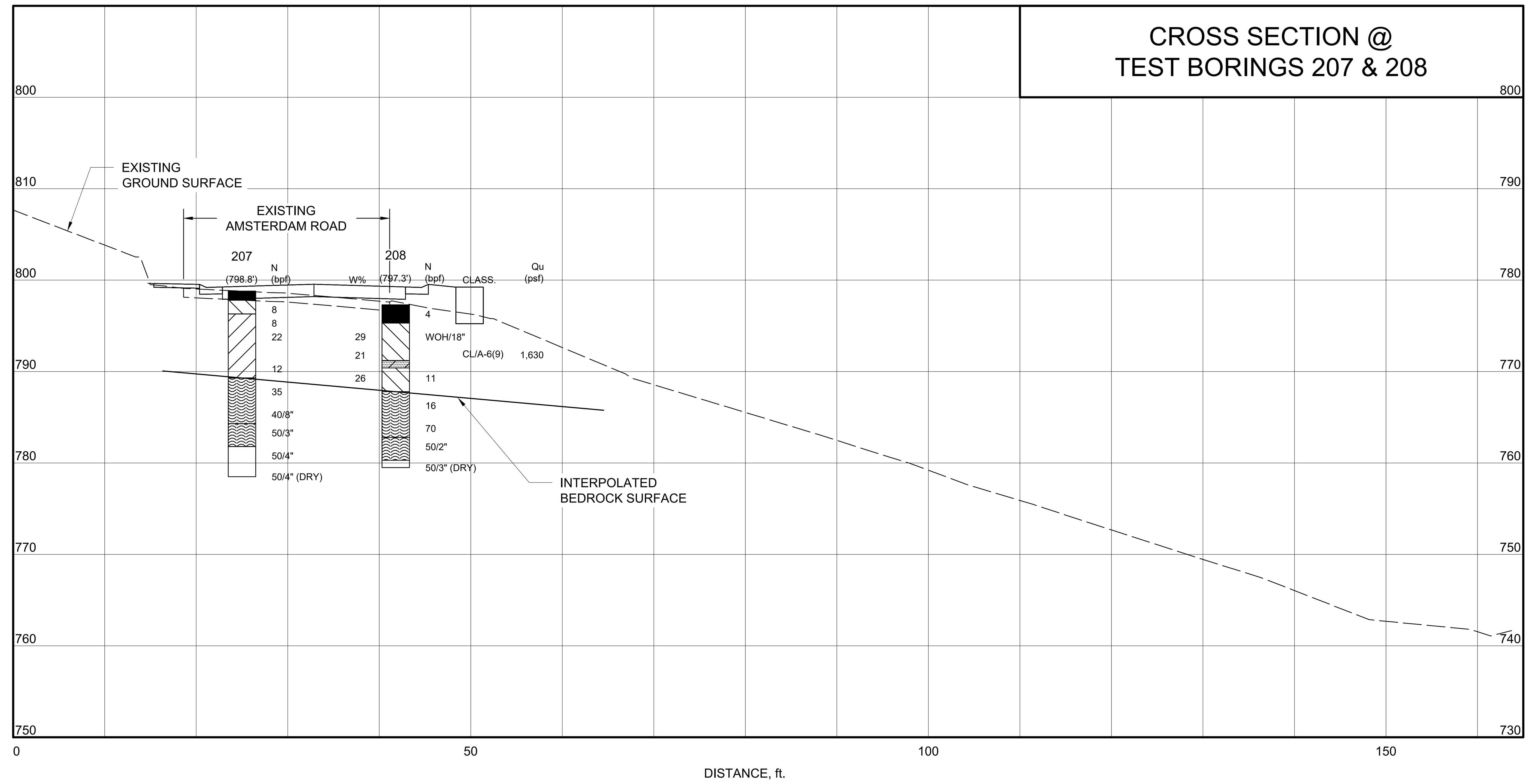
Client: CT Consultants, Inc.

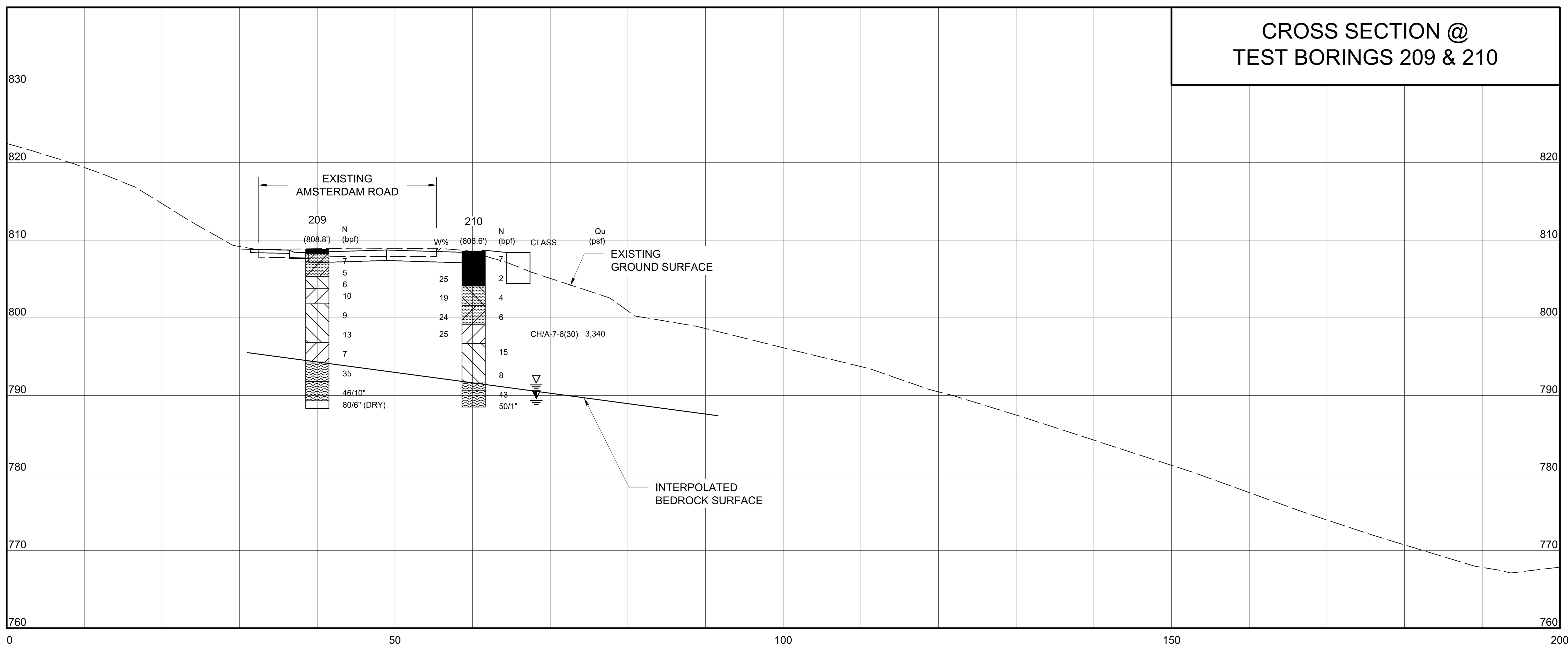
Date: 8/21/2017

Project No.: J029038.01

Drawing No.: 102







LEGEND

- FILL
- SAND
- SILT
- LEAN CLAY
- FAT CLAY
- LEAN CLAY (COLLUVIUM)
- FAT CLAY (COLLUVIUM)
- HIGHLY WEATHERED SHALE AND LIMESTONE BEDROCK
- WEATHERED SHALE AND LIMESTONE BEDROCK
- UNWEATHERED SHALE AND LIMESTONE BEDROCK
- WATER FIRST NOTED
- WATER AT COMPLETION
- (DRY) BOREHOLE DRY AT COMPLETION OF DRILLING

NOTE: PROPOSED GROUND SURFACES ESTIMATED BASED ON SECTIONS PROVIDED ELECTRONICALLY BY CT CONSULTANTS, INC. ON 8/1/2017

Drawing Revisions

Date:

Description:

GEOTECHNOLOGY INC
FROM THE GROUND UP

Project: Amsterdam Road Reconstruction, Phase 1

Location: Fort Wright, Kentucky

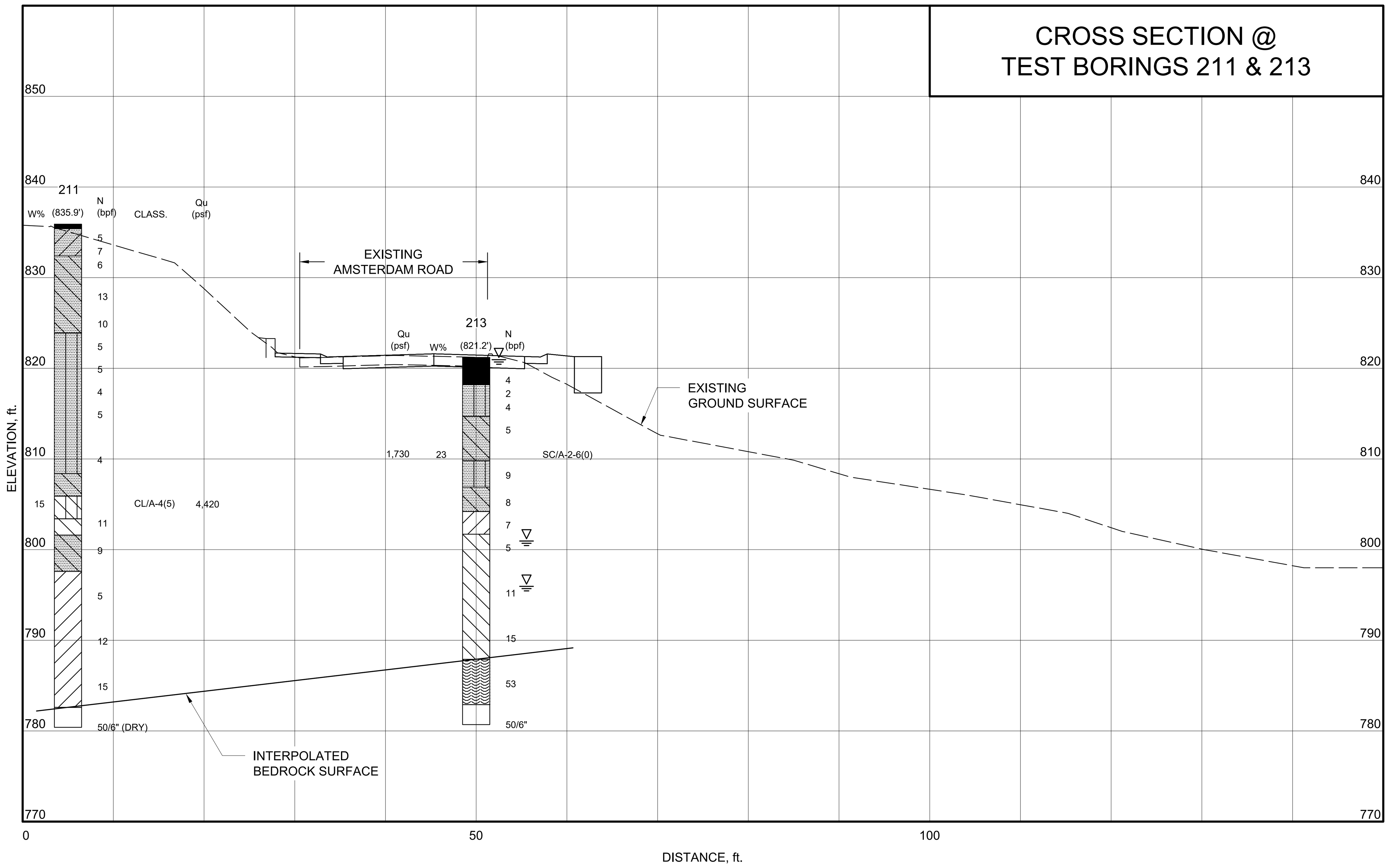
Title: CROSS SECTIONS

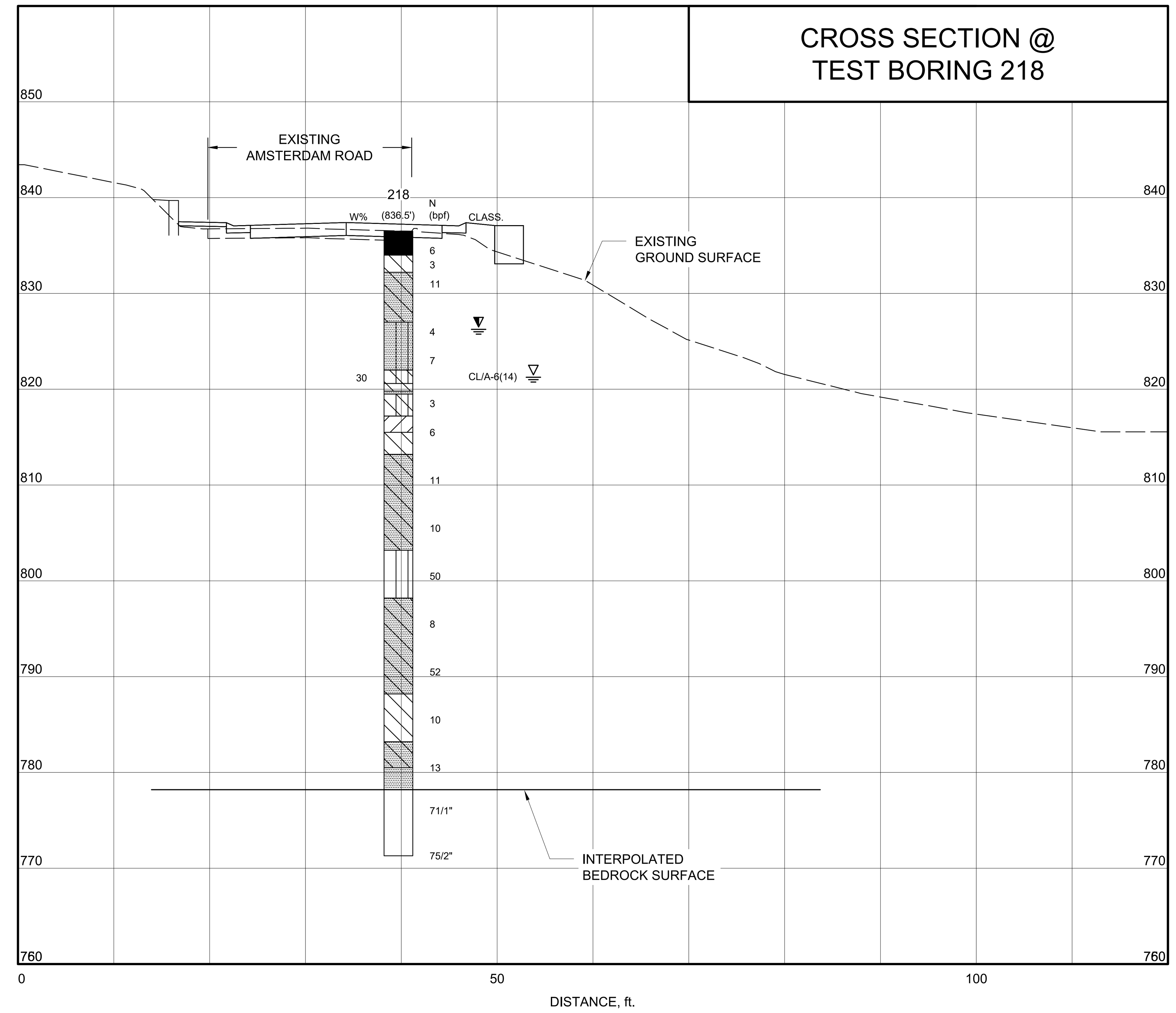
Client: CT Consultants, Inc.

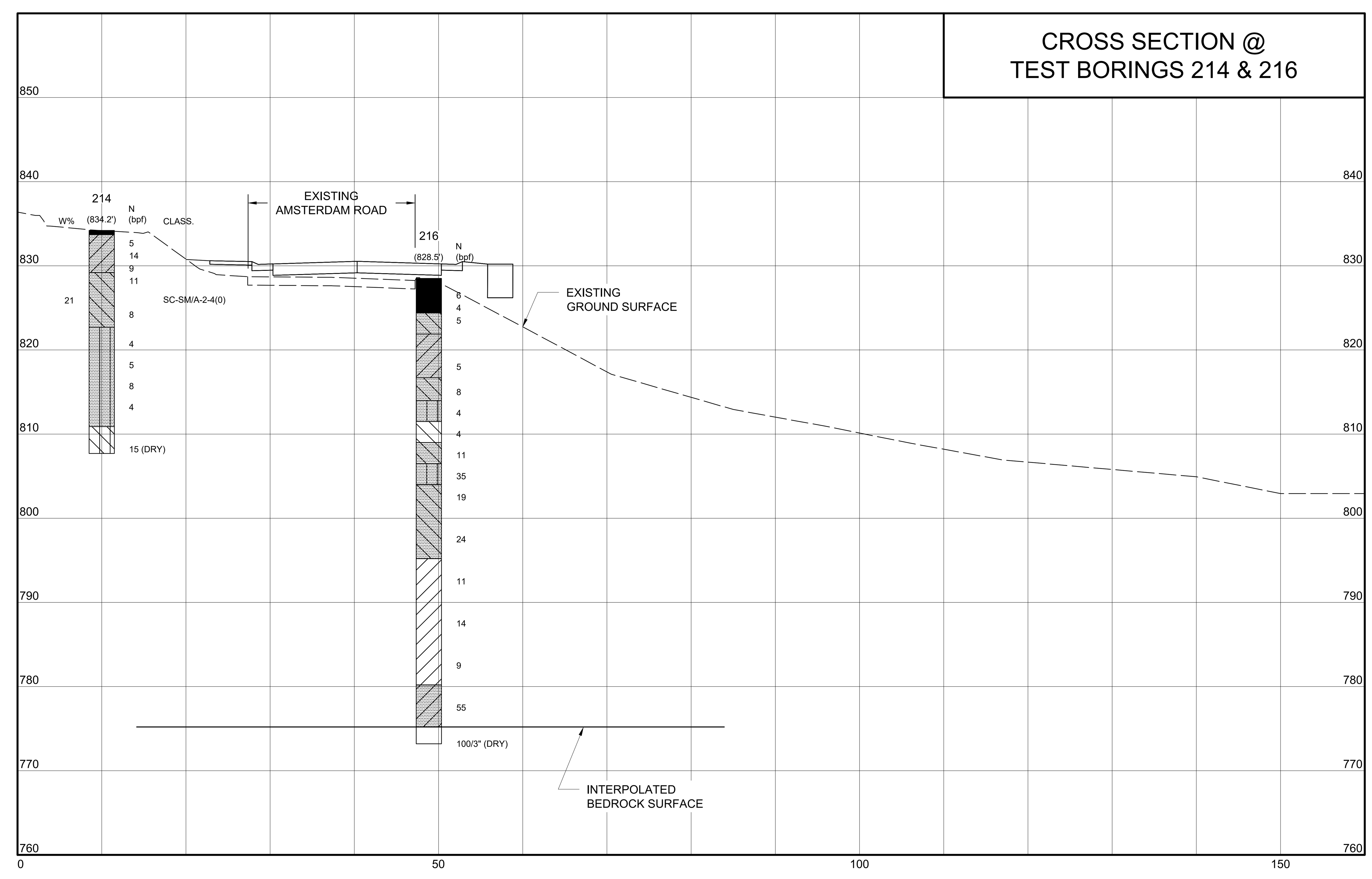
Date: 8/21/2017

Project No.: J029038.01

Drawing No.: 103







LEGEND

- FILL
- SAND
- SILT
- LEAN CLAY
- FAT CLAY
- LEAN CLAY (COLLUVIUM)
- FAT CLAY (COLLUVIUM)
- HIGHLY WEATHERED SHALE AND LIMESTONE BEDROCK
- WEATHERED SHALE AND LIMESTONE BEDROCK
- UNWEATHERED SHALE AND LIMESTONE BEDROCK
- WATER FIRST NOTED
- WATER AT COMPLETION
- (DRY) BOREHOLE DRY AT COMPLETION OF DRILLING

NOTE: PROPOSED GROUND SURFACES ESTIMATED BASED ON SECTIONS PROVIDED ELECTRONICALLY BY CT CONSULTANTS, INC. ON 8/1/2017

Drawing Revisions
Date: Description:

GEOTECHNOLOGY INC
FROM THE GROUND UP

Project: Amsterdam Road Reconstruction, Phase 1
Location: Fort Wright, Kentucky

Title: CROSS SECTIONS
Client: CT Consultants, Inc.

Date: 8/21/2017
Project No.: J029038.01
Drawing No.: 104

APPENDIX C – BORING INFORMATION

New and Historic Boring Logs

Soil Classification Sheet

Rock Classification Sheet

GEOTECHNOLOGY INC
FROM THE GROUND UP

LOG OF TEST BORING

CLIENT: CT Consultants, Inc.
BORING #: 201
PROJECT: Amsterdam Road Reconstruction, Phase 1
Fort Wright, Kentucky
PROJECT #: J029038.01
PAGE #: 1 of 1
LOCATION OF BORING: As shown on Boring Plan, Drawing 1

ELEV.	COLOR, MOISTURE, DENSITY, PLASTICITY, SIZE, PROPORTIONS DESCRIPTION	Strata Depth (feet)	Depth Scale (feet)	Sample Condition	Sample Number	Sample Type	SPT* Blows/6" / Rock Core RQD (%)	Recovery (in.)	Recovery (%)
783.4	Ground Surface	0.0	0						
781.4	Mixed brown moist stiff FILL, lean clay, trace topsoil and roots with shale fragments.	2.0		I	1	DS	2-3-6	12	67
778.9	Mixed brown moist medium stiff to stiff FILL, lean clay, trace topsoil and roots with shale and limestone fragments.	4.5		I	2	DS	3-5-7	18	100
773.9	Olive brown and gray moist stiff LEAN CLAY with limestone floaters, trace bedding planes (residual).	9.5	5	I	3	DS	5-25-10	12	67
				I	4	DS	9-29-16	12	67
771.4	Interbedded brown to olive brown moist extremely weak highly weathered SHALE and gray medium strong to very strong unweathered LIMESTONE (bedrock).	12.0	10	I	5	DS	18-20-23	12	67
767.1	Interbedded olive brown moist very weak weathered SHALE and gray medium strong to very strong unweathered LIMESTONE (bedrock).	16.3		I	6	DS	27-50/5"	12	100
			15	I	7	DS	20-38-50/3"	18	100
	Bottom of test boring at 16.3 feet.		20						
			25						
			30						

Datum: NAVD 88
Hammer Weight: 140 lb.
Hole Diameter: 8 in.
Drill Rig: CME-55 TD-5
Surface Elevation: 783.4 ft.
Hammer Drop: 30 in.
Rock Core Diameter: --
Foreman: M. Lozier
Date Started: 4/4/2017
Pipe Size: 2 in. O.D.
Boring Method: HSA-3.25
Engineer: Michelle E. Casto
Date Completed: 4/4/2017

BORING METHOD
HSA = Hollow Stem Augers
CFA = Continuous Flight Augers
DC = Driving Casing
MD = Mud Drilling

SAMPLE TYPE
PC = Pavement Core
CA = Continuous Flight Auger
DS = Driven Split Spoon
PT = Pressed Shelby Tube
RC = Rock Core

SAMPLE CONDITIONS
D = Disintegrated
I = Intact
U = Undisturbed
L = Lost

GROUNDWATER DEPTH
First Noted: None
At Completion: Dry
After: --
Backfilled: Immediately

* SPT = Standard Penetration Test - Driving 2" O.D. Sampler 18" with 140-Pound Hammer Falling 30"; Count Made at 6" Intervals

GEOTECHNOLOGY INC
FROM THE GROUND UP

LOG OF TEST BORING

CLIENT: CT Consultants, Inc.
PROJECT: Amsterdam Road Reconstruction, Phase 1
Fort Wright, Kentucky
LOCATION OF BORING: As shown on Boring Plan, Drawing 1

BORING #: 202
PROJECT #: J029038.01
PAGE #: 1 of 1

ELEV.	COLOR, MOISTURE, DENSITY, PLASTICITY, SIZE, PROPORTIONS DESCRIPTION	Strata Depth (feet)	Depth Scale (feet)	Sample Condition	Sample Number	Sample Type	SPT* Blows/6" / Rock Core RQD (%)	Recovery (in.)	Recovery (%)
775.7	Ground Surface	0.0	0						
775.4	TOPSOIL (3 inches)	0.3							
773.7	Brown, trace gray moist medium stiff LEAN CLAY with limestone floaters, trace bedding planes (residual).	2.0		I	1A 1B	DS	2-4-3	8	44
	Brown, trace gray moist stiff LEAN CLAY, trace roots and oxide stains, trace bedding planes (residual) [CL/A-6(17)].			I	2	DS	7-8-50	12	67
768.7		7.0	5	I	3	DS	8-8-14	18	100
766.2	Interbedded brown moist extremely weak highly weathered SHALE and gray medium strong to very strong unweathered LIMESTONE (bedrock).	9.5		I	4	DS	11-50/4"	18	100
763.7	Interbedded olive brown, trace gray moist very weak weathered SHALE and gray medium strong to very strong unweathered LIMESTONE (bedrock).	12.0	10	I	5	DS	25-36-50/4"	18	100
760.6	Interbedded gray moist very weak unweathered SHALE and gray medium strong to very strong unweathered LIMESTONE (bedrock).	15.1		I	6	DS	10-21-50/2"	12	86
			15	L	7	DS	50/1"	0	0
	Bottom of test boring at 15.1 feet.		20						
			25						
			30						

Datum: NAVD 88	Hammer Weight: 140 lb.	Hole Diameter: 8 in.	Drill Rig: CME-55 TD-5
Surface Elevation: 775.7 ft.	Hammer Drop: 30 in.	Rock Core Diameter: --	Foreman: M. Lozier
Date Started: 4/4/2017	Pipe Size: 2 in. O.D.	Boring Method: HSA-3.25	Engineer: Michelle E. Casto
Date Completed: 4/4/2017			

BORING METHOD
HSA = Hollow Stem Augers
CFA = Continuous Flight Augers
DC = Driving Casing
MD = Mud Drilling

SAMPLE TYPE
PC = Pavement Core
CA = Continuous Flight Auger
DS = Driven Split Spoon
PT = Pressed Shelby Tube
RC = Rock Core

SAMPLE CONDITIONS
D = Disintegrated
I = Intact
U = Undisturbed
L = Lost

GROUNDWATER DEPTH
First Noted: None
At Completion: Dry
After: --
Backfilled: Immediately

* SPT = Standard Penetration Test - Driving 2" O.D. Sampler 18" with 140-Pound Hammer Falling 30"; Count Made at 6" Intervals

GEOTECHNOLOGY INC
FROM THE GROUND UP

LOG OF TEST BORING

CLIENT: CT Consultants, Inc.
PROJECT: Amsterdam Road Reconstruction, Phase 1
Fort Wright, Kentucky
LOCATION OF BORING: As shown on Boring Plan, Drawing 1

BORING #: 203
PROJECT #: J029038.01
PAGE #: 1 of 1

ELEV.	COLOR, MOISTURE, DENSITY, PLASTICITY, SIZE, PROPORTIONS DESCRIPTION	Strata Depth (feet)	Depth Scale (feet)	Sample Condition	Sample Number	Sample Type	SPT* Blows/6" / Rock Core RQD (%)	Recovery (in.)	Recovery (%)
778.6	Ground Surface	0.0	0						
777.6	ASPHALT (12 inches)	1.0							
776.1	Brown moist stiff LEAN CLAY with shale and limestone fragments (colluvium).	2.5		I	1	DS	3-4-8	12	67
772.1	Brown, trace gray moist very stiff LEAN CLAY, trace fat clay seams with limestone fragments, trace bedding planes (residual) [CL/A-6(13)].	6.5		I	2	DS	6-6-3	12	67
			5	I	3	DS	8-28-13	6	33
769.1	Interbedded brown moist extremely weak highly weathered SHALE and gray medium strong to very strong unweathered LIMESTONE (bedrock).	9.5		I	4	DS	18-41-50/5"	6	35
766.6	Interbedded olive brown moist very weak weathered SHALE and gray medium strong to very strong unweathered LIMESTONE (bedrock).	12.0	10	I	5	DS	20-50/4"	6	60
764.1	Interbedded gray, trace brown moist very weak weathered SHALE and gray medium strong to very strong unweathered LIMESTONE (bedrock).	14.5		I	6	DS	15-24/3"	6	67
763.2	Interbedded gray moist very weak unweathered SHALE and gray medium strong to very strong unweathered LIMESTONE (bedrock).	15.4	15	I	7	DS	50/5"	5	100
	Bottom of test boring at 15.4 feet.		20						
			25						
			30						

Datum: NAVD 88
Surface Elevation: 778.6 ft.
Date Started: 4/10/2017
Date Completed: 4/10/2017
Hammer Weight: 140 lb.
Hammer Drop: 30 in.
Pipe Size: 2 in. O.D.
Hole Diameter: 8 in.
Rock Core Diameter: --
Boring Method: HSA-3.25
Drill Rig: CME-55 TD-5
Foreman: J. Franz
Engineer: Michelle E. Casto

BORING METHOD
HSA = Hollow Stem Augers
CFA = Continuous Flight Augers
DC = Driving Casing
MD = Mud Drilling

SAMPLE TYPE
PC = Pavement Core
CA = Continuous Flight Auger
DS = Driven Split Spoon
PT = Pressed Shelby Tube
RC = Rock Core

SAMPLE CONDITIONS
D = Disintegrated
I = Intact
U = Undisturbed
L = Lost

GROUNDWATER DEPTH
First Noted: None
At Completion: Dry
After: --
Backfilled: Immediately

* SPT = Standard Penetration Test - Driving 2" O.D. Sampler 18" with 140-Pound Hammer Falling 30"; Count Made at 6" Intervals

GEOTECHNOLOGY INC
FROM THE GROUND UP

LOG OF TEST BORING

CLIENT: CT Consultants, Inc.
BORING #: 204
PROJECT: Amsterdam Road Reconstruction, Phase 1
Fort Wright, Kentucky
PROJECT #: J029038.01
PAGE #: 1 of 1
LOCATION OF BORING: As shown on Boring Plan, Drawing 1

ELEV.	COLOR, MOISTURE, DENSITY, PLASTICITY, SIZE, PROPORTIONS DESCRIPTION	Strata Depth (feet)	Sample Condition	Sample Number	Sample Type	SPT* Blows/6" / Rock Core RQD (%)	Recovery (in.)	Recovery (%)
778.3	Ground Surface	0.0						
777.3	ASPHALT (12 inches)	1.0						
774.9	Mixed brown moist stiff FILL, fat clay, trace topsoil with limestone fragments.	3.4	I	1	DS	3-4-17	12	67
			L	2	DS	50/3"	0	0
772.8	Mixed brown moist stiff FILL, lean clay, trace topsoil with a layer of limestone floaters.	5.5	I	3	DS	68-11-19	12	67
771.0	Brown to olive brown moist very stiff LEAN CLAY, trace fat clay seams, oxide stains, trace bedding planes (residual).	7.3	I	4	DS	5-5-5	18	100
768.8	Interbedded brown, trace gray moist extremely weak highly weathered SHALE, trace clay seams, and gray medium strong to very strong unweathered LIMESTONE (bedrock).	9.5	I	5	DS	9-7-10	12	67
766.3	Interbedded brown moist extremely weak highly weathered SHALE and gray medium strong to very strong unweathered LIMESTONE (bedrock).	12.0	I	6	DS	13-19-10	12	67
761.8	Interbedded gray moist very weak unweathered SHALE and gray medium strong to very strong unweathered LIMESTONE (bedrock).	16.5	I	7	DS	5-13-27	18	100
			I	8	DS	8-59-44	12	67
	Bottom of test boring at 16.5 feet.							

Depth Scale (feet): 0, 5, 10, 15, 20, 25, 30

Datum: NAVD 88
Surface Elevation: 778.3 ft.
Date Started: 4/5/2017
Date Completed: 4/5/2017
Hammer Weight: 140 lb.
Hammer Drop: 30 in.
Pipe Size: 2 in. O.D.
Hole Diameter: 8 in.
Rock Core Diameter: --
Boring Method: HSA-3.25
Drill Rig: CME-55 TD-5
Foreman: M. Lozier
Engineer: Michelle E. Casto

BORING METHOD
HSA = Hollow Stem Augers
CFA = Continuous Flight Augers
DC = Driving Casing
MD = Mud Drilling

SAMPLE TYPE
PC = Pavement Core
CA = Continuous Flight Auger
DS = Driven Split Spoon
PT = Pressed Shelby Tube
RC = Rock Core

SAMPLE CONDITIONS
D = Disintegrated
I = Intact
U = Undisturbed
L = Lost

GROUNDWATER DEPTH
First Noted 12.0 ft.
At Completion Dry
After --
Backfilled Immediately

* SPT = Standard Penetration Test - Driving 2" O.D. Sampler 18" with 140-Pound Hammer Falling 30"; Count Made at 6" Intervals

GEOTECHNOLOGY INC
FROM THE GROUND UP

LOG OF TEST BORING

CLIENT: CT Consultants, Inc.
BORING #: 205
PROJECT: Amsterdam Road Reconstruction, Phase 1
Fort Wright, Kentucky
PROJECT #: J029038.01
PAGE #: 1 of 1
LOCATION OF BORING: As shown on Boring Plan, Drawing 1

ELEV.	COLOR, MOISTURE, DENSITY, PLASTICITY, SIZE, PROPORTIONS DESCRIPTION	Strata Depth (feet)	Sample Condition	Sample Number	Sample Type	SPT* Blows/6" / Rock Core RQD (%)	Recovery (in.)	Recovery (%)
788.6	Ground Surface	0.0						
787.6	ASPHALT (12 inches)	1.0						
787.3	Mixed brown, green and gray moist stiff FILL, lean clay with oxide stains, trace shale fragments.	1.3	I	1A 1B	DS	2-3-5	6	33
786.6	Brown, trace gray moist stiff FAT CLAY, trace oxide stains (glacial).	2.0	I	2	DS	WOH-2-3	12	67
784.1	Gray, trace brown and orangish brown moist medium stiff FAT CLAY with oxide stains and limestone floaters, trace clayey sand seams (glacial).	4.5	I	3	DS	50/3"	3	100
779.1	Interbedded brown moist extremely weak highly weathered SHALE and gray medium strong to very strong unweathered LIMESTONE (bedrock).	9.5	I	4	DS	17-13-26	12	67
776.6	Interbedded gray, trace live brown moist very weak weathered SHALE and gray medium strong to very strong unweathered LIMESTONE (bedrock).	12.0	I	5	DS	18-35/2"	12	100
775.9	Interbedded gray moist very weak unweathered SHALE and gray medium strong to very strong unweathered LIMESTONE (bedrock).	12.7	I	6	DS	50/2"	6	100
	Bottom of test boring at 12.7 feet.							

Depth Scale (feet): 0, 5, 10, 15, 20, 25, 30

Datum: NAVD 88
Hammer Weight: 140 lb.
Hole Diameter: 8 in.
Drill Rig: CME-55 TD-5
Surface Elevation: 788.6 ft.
Hammer Drop: 30 in.
Rock Core Diameter: --
Foreman: J. Franz
Date Started: 4/10/2017
Pipe Size: 2 in. O.D.
Boring Method: HSA-3.25
Engineer: Michelle E. Casto
Date Completed: 4/10/2017

BORING METHOD
HSA = Hollow Stem Augers
CFA = Continuous Flight Augers
DC = Driving Casing
MD = Mud Drilling

SAMPLE TYPE
PC = Pavement Core
CA = Continuous Flight Auger
DS = Driven Split Spoon
PT = Pressed Shelby Tube
RC = Rock Core

SAMPLE CONDITIONS
D = Disintegrated
I = Intact
U = Undisturbed
L = Lost

GROUNDWATER DEPTH
First Noted: None
At Completion: Dry
After: --
Backfilled: Immediately

* SPT = Standard Penetration Test - Driving 2" O.D. Sampler 18" with 140-Pound Hammer Falling 30"; Count Made at 6" Intervals

GEOTECHNOLOGY INC
FROM THE GROUND UP

LOG OF TEST BORING

CLIENT: CT Consultants, Inc.
BORING #: 206
PROJECT: Amsterdam Road Reconstruction, Phase 1
Fort Wright, Kentucky
PROJECT #: J029038.01
PAGE #: 1 of 1
LOCATION OF BORING: As shown on Boring Plan, Drawing 1

ELEV.	COLOR, MOISTURE, DENSITY, PLASTICITY, SIZE, PROPORTIONS DESCRIPTION	Strata Depth (feet)	Depth Scale (feet)	Sample Condition	Sample Number	Sample Type	SPT* Blows/6" / Rock Core RQD (%)	Recovery (in.)	Recovery (%)
787.5	Ground Surface	0.0	0						
	Mixed black and dark brown moist very loose FILL, asphalt fragments, some coarse gravel, little lean clay.			I	1	DS	2-2-1	6	33
785.5		2.0							
	Mixed brown moist stiff FILL, fat clay, trace roots and oxide stains.			I	2	DS	2-2-3	18	100
783.0		4.5							
	Brown moist very stiff FAT CLAY with oxide stains (glacial) [CH/A-7-6(30)].		5	U	3	PT		9	94
781.7		5.8							
	Brown, trace gray moist very stiff LEAN CLAY with limestone floaters, trace bedding planes (residual).			I	4	DS	24-7-11	6	33
780.1		7.4							
	Interbedded brown moist extremely weak highly weathered SHALE and gray medium strong to very strong unweathered LIMESTONE (bedrock).			I	5	DS	15-10-12	6	33
			10	I	6	DS	50/6"	6	100
775.5		12.0							
	Interbedded olive brown and gray moist very weak weathered SHALE and gray medium strong to very strong unweathered LIMESTONE (bedrock).			I	7	DS	8-50/2"	10	100
			15	I	8	DS	14-50/3"	10	100
770.5		17.0							
	Interbedded gray moist very weak unweathered SHALE and gray medium strong to very strong unweathered LIMESTONE (bedrock).			I	9	DS	35-50/5"	6	55
768.5		19.0							
	Bottom of test boring at 19.0 feet.		20						
			25						
			30						

Datum: NAVD 88
Hammer Weight: 140 lb.
Hole Diameter: 8 in.
Drill Rig: CME-55 TD-5
Surface Elevation: 787.5 ft.
Hammer Drop: 30 in.
Rock Core Diameter: --
Foreman: M. Lozier
Date Started: 4/5/2017
Pipe Size: 2 in. O.D.
Boring Method: HSA-3.25
Engineer: Michelle E. Casto
Date Completed: 4/5/2017

BORING METHOD
HSA = Hollow Stem Augers
CFA = Continuous Flight Augers
DC = Driving Casing
MD = Mud Drilling

SAMPLE TYPE
PC = Pavement Core
CA = Continuous Flight Auger
DS = Driven Split Spoon
PT = Pressed Shelby Tube
RC = Rock Core

SAMPLE CONDITIONS
D = Disintegrated
I = Intact
U = Undisturbed
L = Lost

GROUNDWATER DEPTH
First Noted: None
At Completion: Dry
After: --
Backfilled: Immediately

* SPT = Standard Penetration Test - Driving 2" O.D. Sampler 18" with 140-Pound Hammer Falling 30"; Count Made at 6" Intervals

GEOTECHNOLOGY INC
FROM THE GROUND UP

LOG OF TEST BORING

CLIENT: CT Consultants, Inc.
BORING #: 207
PROJECT: Amsterdam Road Reconstruction, Phase 1
Fort Wright, Kentucky
PROJECT #: J029038.01
PAGE #: 1 of 1
LOCATION OF BORING: As shown on Boring Plan, Drawing 1

ELEV.	COLOR, MOISTURE, DENSITY, PLASTICITY, SIZE, PROPORTIONS DESCRIPTION	Strata Depth (feet)	Sample Condition	Sample Number	Sample Type	SPT* Blows/6" / Rock Core RQD (%)	Recovery (in.)	Recovery (%)
798.8	Ground Surface	0.0						
798.3	ASPHALT (6 inches)	0.5						
797.8	Crushed limestone base (6 inches)	1.0	D	1	CA			
796.3	Mottled brown moist stiff LEAN CLAY, trace oxide stains (glacial).	2.5	I	2	DS	3-3-5	6	33
794.8	Brown, trace gray moist very stiff FAT CLAY, trace oxide stains (glacial).	4.0	I	3	DS	3-3-5	6	33
789.3	Brown, trace gray moist very stiff FAT CLAY with oxide stains, trace bedding planes (residual).	9.5	I	4	DS	4-15-7	12	67
			I	5	DS	4-5-7	18	100
784.3	Interbedded brown moist extremely weak highly weathered SHALE and gray medium strong to very strong unweathered LIMESTONE, trace fat clay seams (bedrock).	14.5	I	6	DS	30-19-16	6	33
			I	7	DS	3-14-26/2"	12	86
781.8	Interbedded olive brown and gray moist very weak weathered SHALE and gray medium strong to very strong unweathered LIMESTONE (bedrock).	17.0	I	8	DS	50/3"	6	100
778.5	Interbedded gray moist very weak unweathered SHALE and gray medium strong to very strong unweathered LIMESTONE (bedrock).	20.3	I	9	DS	50/4"	2	50
			I	10	DS	50/4"	2	50
	Bottom of test boring at 20.3 feet.							

Depth Scale (feet): 0, 5, 10, 15, 20, 25, 30

Datum: NAVD 88
Hammer Weight: 140 lb.
Hole Diameter: 8 in.
Drill Rig: CME-55 TD-5
Surface Elevation: 798.8 ft.
Hammer Drop: 30 in.
Rock Core Diameter: --
Foreman: J. Franz
Date Started: 4/10/2017
Pipe Size: 2 in. O.D.
Boring Method: HSA-3.25
Engineer: Michelle E. Casto
Date Completed: 4/10/2017

BORING METHOD
HSA = Hollow Stem Augers
CFA = Continuous Flight Augers
DC = Driving Casing
MD = Mud Drilling

SAMPLE TYPE
PC = Pavement Core
CA = Continuous Flight Auger
DS = Driven Split Spoon
PT = Pressed Shelby Tube
RC = Rock Core

SAMPLE CONDITIONS
D = Disintegrated
I = Intact
U = Undisturbed
L = Lost

GROUNDWATER DEPTH
First Noted: None
At Completion: Dry
After: --
Backfilled: Immediately

* SPT = Standard Penetration Test - Driving 2" O.D. Sampler 18" with 140-Pound Hammer Falling 30"; Count Made at 6" Intervals

GEOTECHNOLOGY INC
FROM THE GROUND UP

LOG OF TEST BORING

CLIENT: CT Consultants, Inc.
PROJECT: Amsterdam Road Reconstruction, Phase 1
Fort Wright, Kentucky
LOCATION OF BORING: As shown on Boring Plan, Drawing 1

BORING #: 208
PROJECT #: J029038.01
PAGE #: 1 of 1

ELEV.	COLOR, MOISTURE, DENSITY, PLASTICITY, SIZE, PROPORTIONS DESCRIPTION	Strata Depth (feet)	Sample Condition	Sample Number	Sample Type	SPT* Blows/6" / Rock Core RQD (%)	Recovery (in.)	Recovery (%)
797.3	Ground Surface	0.0						
795.3	Mixed dark brown moist soft FILL, fat clay, some sand and gravel with asphalt fragments.	2.0	I	1	DS	4-2-2	6	33
792.8	Dark brown moist very soft LEAN CLAY, trace fine sand (glacial).	4.5	I	2	DS	WOH/18"	12	67
791.2	Orangish brown moist medium stiff sandy LEAN CLAY with oxide concretions (glacial) [CL/A-6(9)].	6.1	U	3	PT		19	79
790.4	Orangish brown moist very stiff sandy FAT CLAY with oxide concretions (glacial).	6.9						
787.8	Brown moist stiff LEAN CLAY with oxide concretions and shale fragments (glacial).	9.5	I	4	DS	3-6-5	18	100
785.3	Interbedded brown moist extremely weak highly weathered SHALE and gray medium strong to very strong unweathered LIMESTONE with fat clay seams (bedrock).	12.0	I	5	DS	8-5-11	18	100
782.8	Interbedded brown moist extremely weak highly weathered SHALE and gray medium strong to very strong unweathered LIMESTONE (bedrock).	14.5	I	6	DS	11-29-41	18	100
780.3	Interbedded olive brown and gray moist very weak weathered SHALE and gray medium strong to very strong unweathered LIMESTONE (bedrock).	17.0	I	7	DS	9-50/2"	12	100
779.5	Interbedded gray moist very weak unweathered SHALE and gray medium strong to very strong unweathered LIMESTONE (bedrock).	17.8	I	8	DS	50/3"	3	100
	Bottom of test boring at 17.8 feet.							

Datum: NAVD 88
Surface Elevation: 797.3 ft.
Date Started: 4/5/2017
Date Completed: 4/5/2017
Hammer Weight: 140 lb.
Hammer Drop: 30 in.
Pipe Size: 2 in. O.D.
Hole Diameter: 8 in.
Rock Core Diameter: --
Boring Method: HSA-3.25
Drill Rig: CME-55 TD-5
Foreman: M. Lozier
Engineer: Michelle E. Casto

BORING METHOD
HSA = Hollow Stem Augers
CFA = Continuous Flight Augers
DC = Driving Casing
MD = Mud Drilling

SAMPLE TYPE
PC = Pavement Core
CA = Continuous Flight Auger
DS = Driven Split Spoon
PT = Pressed Shelby Tube
RC = Rock Core

SAMPLE CONDITIONS
D = Disintegrated
I = Intact
U = Undisturbed
L = Lost

GROUNDWATER DEPTH
First Noted: None
At Completion: Dry
After: --
Backfilled: Immediately

* SPT = Standard Penetration Test - Driving 2" O.D. Sampler 18" with 140-Pound Hammer Falling 30"; Count Made at 6" Intervals

GEOTECHNOLOGY INC
FROM THE GROUND UP

LOG OF TEST BORING

CLIENT: CT Consultants, Inc.
BORING #: 209
PROJECT: Amsterdam Road Reconstruction, Phase 1
Fort Wright, Kentucky
PROJECT #: J029038.01

LOCATION OF BORING: As shown on Boring Plan, Drawing 1

ELEV.	COLOR, MOISTURE, DENSITY, PLASTICITY, SIZE, PROPORTIONS DESCRIPTION	Strata Depth (feet)	Sample Condition	Sample Number	Sample Type	SPT* Blows/6" / Rock Core RQD (%)	Recovery (in.)	Recovery (%)
808.8	Ground Surface	0.0						
808.3	ASPHALT (6 inches)	0.5						
806.8	Orangish brown moist stiff sandy FAT CLAY, trace roots and oxide stains (glacial).	2.0	I	1	DS	3-4-3	12	67
805.3	Orangish brown moist soft sandy FAT CLAY, trace oxide stains with limestone fragments (glacial).	3.5	I	2	DS	2-2-3	12	67
803.8	Brown, trace light gray moist stiff LEAN CLAY with oxide stains (glacial).	5.0	I	3	DS	4-3-3	18	100
801.8	Brown, trace light gray moist very stiff FAT CLAY, trace sand with oxide stains (glacial).	7.0	I	4	DS	4-4-6	18	100
799.3	Mottled brown moist stiff LEAN CLAY with oxide stains (glacial).	9.5	I	5	DS	3-4-5	12	67
796.8	Brown moist very stiff LEAN CLAY with oxide stains, trace bedding planes (residual).	12.0	I	6	DS	3-5-8	18	100
794.3	Mottled brown moist very stiff FAT CLAY, trace sand with oxide stains, trace bedding planes (residual).	14.5	I	7	DS	4-3-4	18	100
791.8	Interbedded brown moist extremely weak highly weathered SHALE and gray medium strong to very strong unweathered LIMESTONE (bedrock).	17.0	I	8	DS	20-10-25	6	33
789.3	Interbedded olive brown moist very weak weathered SHALE and gray medium strong to very strong unweathered LIMESTONE (bedrock).	19.5	I	9	DS	3-11-35/4"	6	38
788.3	Interbedded gray moist very weak unweathered SHALE and gray medium strong to very strong unweathered LIMESTONE (bedrock).	20.5	I	10	DS	80/6"	6	100
	Bottom of test boring at 20.5 feet.							

Depth Scale (feet): 0, 5, 10, 15, 20, 25, 30

Datum: NAVD 88	Hammer Weight: 140 lb.	Hole Diameter: 8 in.	Drill Rig: CME-55 TD-5
Surface Elevation: 808.8 ft.	Hammer Drop: 30 in.	Rock Core Diameter: --	Foreman: J. Franz
Date Started: 4/12/2017	Pipe Size: 2 in. O.D.	Boring Method: HSA-3.25	Engineer: Michelle E. Casto
Date Completed: 4/12/2017			

BORING METHOD
HSA = Hollow Stem Augers
CFA = Continuous Flight Augers
DC = Driving Casing
MD = Mud Drilling

SAMPLE TYPE
PC = Pavement Core
CA = Continuous Flight Auger
DS = Driven Split Spoon
PT = Pressed Shelby Tube
RC = Rock Core

SAMPLE CONDITIONS
D = Disintegrated
I = Intact
U = Undisturbed
L = Lost

GROUNDWATER DEPTH
First Noted: None
At Completion: Dry
After: --
Backfilled: Immediately

* SPT = Standard Penetration Test - Driving 2" O.D. Sampler 18" with 140-Pound Hammer Falling 30"; Count Made at 6" Intervals

GEOTECHNOLOGY INC
FROM THE GROUND UP

LOG OF TEST BORING

CLIENT: CT Consultants, Inc. BORING #: 210
PROJECT: Amsterdam Road Reconstruction, Phase 1 PROJECT #: J029038.01
Fort Wright, Kentucky PAGE #: 1 of 1
LOCATION OF BORING: As shown on Boring Plan, Drawing 1

ELEV.	COLOR, MOISTURE, DENSITY, PLASTICITY, SIZE, PROPORTIONS DESCRIPTION	Strata Depth (feet)	Depth Scale (feet)	Sample Condition	Sample Number	Sample Type	SPT* Blows/6" / Rock Core RQD (%)	Recovery (in.)	Recovery (%)
808.6	Ground Surface	0.0	0						
	Mixed brown moist medium stiff FILL, lean clay, trace fine gravel with roots and brick fragments.			I	1	DS	2-3-4	8	44
806.6		2.0							
	Mixed dark brown moist soft FILL, lean clay, trace roots.			I	2	DS	1-1-1	15	83
804.1		4.5							
	Orangish brown moist medium stiff sandy LEAN CLAY, trace oxide concretions (glacial).		5	I	3	DS	2-2-2	18	100
801.6		7.0							
	Brown moist medium stiff sandy FAT CLAY (glacial).			I	4	DS	2-3-3	18	100
799.1		9.5							
	Brown, trace gray moist stiff FAT CLAY, little sand with oxide stains (glacial) [CH/A-7-6(30)].		10	U	5	PT		22	92
796.7		11.9							
	Brown moist stiff LEAN CLAY with oxide stains, trace bedding planes (residual).			I	6	DS	5-8-7	18	100
794.1		14.5							
	Brown moist very stiff LEAN CLAY, trace oxide stains with bedding planes (residual).		15	I	7	DS	4-4-4	6	33
791.6		17.0							
	Interbedded brown, trace gray moist extremely weak highly weathered SHALE and gray medium strong to very strong unweathered LIMESTONE (bedrock).								
790.6		18.0							
	Interbedded gray, trace olive brown moist very weak weathered SHALE and gray medium strong to very strong unweathered LIMESTONE (bedrock).			I	8A 8B	DS	WOH-7-36	18	100
788.5		20.1	20	L	9	DS	50/1"	0	0
	Bottom of test boring at 20.1 feet.								
			25						
			30						

Datum: NAVD 88 | Hammer Weight: 140 lb. | Hole Diameter: 8 in. | Drill Rig: CME-55 TD-5
Surface Elevation: 808.6 ft. | Hammer Drop: 30 in. | Rock Core Diameter: -- | Foreman: M. Lozier
Date Started: 4/5/2017 | Pipe Size: 2 in. O.D. | Boring Method: HSA-3.25 | Engineer: Michelle E. Casto
Date Completed: 4/5/2017

BORING METHOD
HSA = Hollow Stem Augers
CFA = Continuous Flight Augers
DC = Driving Casing
MD = Mud Drilling

SAMPLE TYPE
PC = Pavement Core
CA = Continuous Flight Auger
DS = Driven Split Spoon
PT = Pressed Shelby Tube
RC = Rock Core

SAMPLE CONDITIONS
D = Disintegrated
I = Intact
U = Undisturbed
L = Lost

GROUNDWATER DEPTH
First Noted 17.0 ft.
At Completion 19.0 ft.
After --
Backfilled Immediately

* SPT = Standard Penetration Test - Driving 2" O.D. Sampler 18" with 140-Pound Hammer Falling 30"; Count Made at 6" Intervals

GEOTECHNOLOGY INC
FROM THE GROUND UP

LOG OF TEST BORING

CLIENT: CT Consultants, Inc.
BORING #: 211
PROJECT: Amsterdam Road Reconstruction, Phase 1
Fort Wright, Kentucky
PROJECT #: J029038.01
PAGE #: 1 of 2
LOCATION OF BORING: As shown on Boring Plan, Drawing 1

ELEV.	COLOR, MOISTURE, DENSITY, PLASTICITY, SIZE, PROPORTIONS DESCRIPTION	Strata Depth (feet)	Depth Scale (feet)	Sample Condition	Sample Number	Sample Type	SPT* Blows/6" / Rock Core RQD (%)	Recovery (in.)	Recovery (%)
835.9	Ground Surface	0.0	0						
835.7	ASPHALT (2.5 inches)	0.2		D	1	CA			
835.4	Crushed limestone base (3.5 inches)	0.5		I	2	DS	1-2-3	12	67
832.4	Reddish brown moist soft sandy FAT CLAY, trace oxide concretions (glacial).	3.5		I	3	DS	5-5-2	12	67
830.9	Reddish brown moist very loose clayey SAND, trace oxide concretions (glacial).	5.0	5	I	4	DS	2-2-4	12	67
828.9	Orangish brown moist dense clayey SAND with oxide concretions (glacial).	7.0		U	5	PT		24	100
826.6	Orangish brown moist medium dense clayey SAND with oxide concretions (glacial).	9.3		I	6	DS	5-6-7	12	67
823.9	Orangish brown moist loose clayey SAND with oxide stains (glacial).	12.0	10	D	7	DS	5-5-5	12	67
821.4	Orangish brown moist very loose silty SAND, trace fat clay with oxide stains (glacial).	14.5		D	8	DS	3-2-3	12	67
	Brown moist very loose sandy SILT, trace oxide stains (glacial).		15	D	9	DS	2-2-3	12	67
				D	10	DS	2-2-2	18	100
812.1		23.8	20	D	11	DS	2-2-3	18	100
808.4	Brown moist very loose sandy SILT (glacial).	27.5	25	I	12	DS	2-2-2	6	33
805.9	Gray moist very stiff sandy LEAN CLAY, little fine to coarse gravel with limestone fragments (glacial).	30.0	30						

Datum: NAVD 88
Hammer Weight: 140 lb.
Hole Diameter: 8 in.
Drill Rig: CME-55 TD-5
Surface Elevation: 835.9 ft.
Hammer Drop: 30 in.
Rock Core Diameter: --
Foreman: J. Franz
Date Started: 4/11/2017
Pipe Size: 2 in. O.D.
Boring Method: HSA-3.25
Engineer: Michelle E. Casto
Date Completed: 4/11/2017

BORING METHOD
HSA = Hollow Stem Augers
CFA = Continuous Flight Augers
DC = Driving Casing
MD = Mud Drilling

SAMPLE TYPE
PC = Pavement Core
CA = Continuous Flight Auger
DS = Driven Split Spoon
PT = Pressed Shelby Tube
RC = Rock Core

SAMPLE CONDITIONS
D = Disintegrated
I = Intact
U = Undisturbed
L = Lost

GROUNDWATER DEPTH
First Noted: None
At Completion: Dry
After: --
Backfilled: Immediately

* SPT = Standard Penetration Test - Driving 2" O.D. Sampler 18" with 140-Pound Hammer Falling 30"; Count Made at 6" Intervals

GEOTECHNOLOGY INC
FROM THE GROUND UP

LOG OF TEST BORING

CLIENT: CT Consultants, Inc.
BORING #: 211
PROJECT: Amsterdam Road Reconstruction, Phase 1
Fort Wright, Kentucky
PROJECT #: J029038.01

LOCATION OF BORING: As shown on Boring Plan, Drawing 1

ELEV.	COLOR, MOISTURE, DENSITY, PLASTICITY, SIZE, PROPORTIONS DESCRIPTION	Strata Depth (feet)	Depth Scale (feet)	Sample Condition	Sample Number	Sample Type	SPT* Blows/6" / Rock Core RQD (%)	Recovery (in.)	Recovery (%)
			30						
803.4	Gray moist very stiff clayey SILT, little sand, trace fine to coarse gravel with limestone fragments (glacial) [CL/A-4(5)].	32.5		U	13	PT		24	100
801.6	Gray moist stiff LEAN CLAY, trace sand and gravel with shale fragments (glacial).	34.3		I	14A 14B	DS	4-5-6	12	67
797.6	Gray moist stiff sandy LEAN CLAY, trace gravel with shale and limestone fragments (glacial).	38.3	35	I	15	DS	4-4-5	18	100
792.6	Gray moist stiff FAT CLAY with limestone fragments (glacial).	43.3	40	I	16	DS	3-1-4	18	100
787.6	Gray, trace olive moist very stiff FAT CLAY, trace bedding planes (residual).	48.3	45	I	17	DS	5-4-8	18	100
782.6	Olive brown, trace gray moist stiff FAT CLAY, trace oxide concretions, trace bedding planes (residual).	53.3	50	I	18	DS	6-6-9	18	100
780.4	Interbedded gray moist very weak unweathered SHALE and gray medium strong to very strong unweathered LIMESTONE (bedrock).	55.5	55	I	19	DS	50/6"	6	100
	Bottom of test boring at 55.5 feet.		60						

Datum: NAVD 88
Hammer Weight: 140 lb.
Hole Diameter: 8 in.
Drill Rig: CME-55 TD-5
Surface Elevation: 835.9 ft.
Hammer Drop: 30 in.
Rock Core Diameter: --
Foreman: J. Franz
Date Started: 4/11/2017
Pipe Size: 2 in. O.D.
Boring Method: HSA-3.25
Engineer: Michelle E. Casto
Date Completed: 4/11/2017

BORING METHOD
HSA = Hollow Stem Augers
CFA = Continuous Flight Augers
DC = Driving Casing
MD = Mud Drilling

SAMPLE TYPE
PC = Pavement Core
CA = Continuous Flight Auger
DS = Driven Split Spoon
PT = Pressed Shelby Tube
RC = Rock Core

SAMPLE CONDITIONS
D = Disintegrated
I = Intact
U = Undisturbed
L = Lost

GROUNDWATER DEPTH
First Noted: None
At Completion: Dry
After: --
Backfilled: Immediately

* SPT = Standard Penetration Test - Driving 2" O.D. Sampler 18" with 140-Pound Hammer Falling 30"; Count Made at 6" Intervals

GEOTECHNOLOGY INC
FROM THE GROUND UP

LOG OF TEST BORING

CLIENT: CT Consultants, Inc.
PROJECT: Amsterdam Road Reconstruction, Phase 1
Fort Wright, Kentucky
LOCATION OF BORING: As shown on Boring Plan, Drawing 1

BORING #: 213
PROJECT #: J029038.01
PAGE #: 1 of 2

ELEV.	COLOR, MOISTURE, DENSITY, PLASTICITY, SIZE, PROPORTIONS DESCRIPTION	Strata Depth (feet)	Sample Condition	Sample Number	Sample Type	SPT* Blows/6" / Rock Core RQD (%)	Recovery (in.)	Recovery (%)
821.2	Ground Surface	0.0						
820.2	ASPHALT (12 inches)	1.0						
819.7	Crushed limestone base (6 inches)	1.5	D	1	CA			
818.2	Mixed orangish brown and black moist very soft FILL, sandy fat clay, trace gravel with oxide concretions.	3.0	I	2	DS	1-2-2	12	67
814.7	Brown moist very loose sandy SILT, trace gravel and oxide concretions (glacial).	6.5	D	3	DS	1-1-1	6	33
			I	4	DS	2-2-2	18	100
811.9	Reddish brown moist very loose clayey SAND, trace oxide concretions (glacial).	9.3	I	5	DS	2-2-3	12	67
809.8	Orangish brown moist medium stiff clayey SAND, little silt with oxide concretions (glacial) [SC/A-2-6(0)].	11.4	U	6	PT		23	96
809.3	Orangish brown moist dense silty SAND with oxide concretions (glacial).	11.9						
806.9	Orangish brown moist loose sandy SILT, trace fat clay with oxide concretions (glacial).	14.3	I	7	DS	3-4-5	18	100
804.2	Brown moist very stiff sandy LEAN CLAY with oxide stains and shale fragments (glacial).	17.0	I	8	DS	2-3-5	18	100
801.7	Brown moist very stiff FAT CLAY with oxide stains and shale fragments (glacial).	19.5	I	9	DS	2-4-3	18	100
797.9	Brown moist soft to medium stiff LEAN CLAY, trace fat clay seam, trace oxide concretions (glacial).	23.3	I	10	DS	2-1-4	18	100
792.9	Grayish brown moist very stiff LEAN CLAY with oxide stains (glacial).	28.3	I	11	DS	3-5-6	18	100
791.2	Brown moist stiff LEAN CLAY with oxide stains, partially layered (glacial).	30.0						

Datum: NAVD 88
Surface Elevation: 821.2 ft.
Date Started: 4/5/2017
Date Completed: 4/5/2017
Hammer Weight: 140 lb.
Hammer Drop: 30 in.
Pipe Size: 2 in. O.D.
Hole Diameter: 8 in.
Rock Core Diameter: --
Boring Method: HSA-3.25
Drill Rig: CME-55 TD-5
Foreman: M. Lozier
Engineer: Michelle E. Casto

BORING METHOD
HSA = Hollow Stem Augers
CFA = Continuous Flight Augers
DC = Driving Casing
MD = Mud Drilling

SAMPLE TYPE
PC = Pavement Core
CA = Continuous Flight Auger
DS = Driven Split Spoon
PT = Pressed Shelby Tube
RC = Rock Core

SAMPLE CONDITIONS
D = Disintegrated
I = Intact
U = Undisturbed
L = Lost

GROUNDWATER DEPTH
First Noted: Trace @20.0 ft., 25.0 ft.
At Completion: Dry
After: --
Backfilled: Immediately

* SPT = Standard Penetration Test - Driving 2" O.D. Sampler 18" with 140-Pound Hammer Falling 30"; Count Made at 6" Intervals

GEOTECHNOLOGY INC
FROM THE GROUND UP

LOG OF TEST BORING

CLIENT: CT Consultants, Inc.
BORING #: 213
PROJECT: Amsterdam Road Reconstruction, Phase 1
Fort Wright, Kentucky
PROJECT #: J029038.01

LOCATION OF BORING: As shown on Boring Plan, Drawing 1

ELEV.	COLOR, MOISTURE, DENSITY, PLASTICITY, SIZE, PROPORTIONS DESCRIPTION	Strata Depth (feet)	Depth Scale (feet)	Sample Condition	Sample Number	Sample Type	SPT* Blows/6" / Rock Core RQD (%)	Recovery (in.)	Recovery (%)
	Brown moist stiff LEAN CLAY with oxide stains, partially layered (glacial).		30	I	12	DS	6-7-8	18	100
787.9		33.3							
	Interbedded brown moist extremely weak highly weathered SHALE and gray medium strong to very strong unweathered LIMESTONE with fat clay seams (bedrock).		35	I	13	DS	19-32-21	12	67
782.9		38.3							
	Interbedded gray moist very weak unweathered SHALE and gray medium strong to very strong unweathered LIMESTONE (bedrock).								
780.7		40.5	40	I	14	DS	50/6"	6	100
	Bottom of test boring at 40.5 feet.		45						
			50						
			55						
			60						

Datum: NAVD 88
Hammer Weight: 140 lb.
Hole Diameter: 8 in.
Drill Rig: CME-55 TD-5
Surface Elevation: 821.2 ft.
Hammer Drop: 30 in.
Rock Core Diameter: --
Foreman: M. Lozier
Date Started: 4/5/2017
Pipe Size: 2 in. O.D.
Boring Method: HSA-3.25
Engineer: Michelle E. Casto
Date Completed: 4/5/2017

BORING METHOD
HSA = Hollow Stem Augers
CFA = Continuous Flight Augers
DC = Driving Casing
MD = Mud Drilling

SAMPLE TYPE
PC = Pavement Core
CA = Continuous Flight Auger
DS = Driven Split Spoon
PT = Pressed Shelby Tube
RC = Rock Core

SAMPLE CONDITIONS
D = Disintegrated
I = Intact
U = Undisturbed
L = Lost

GROUNDWATER DEPTH
First Noted: Trace @20.0 ft., 25.0 ft.
At Completion: Dry
After: --
Backfilled: Immediately

* SPT = Standard Penetration Test - Driving 2" O.D. Sampler 18" with 140-Pound Hammer Falling 30"; Count Made at 6" Intervals

GEOTECHNOLOGY INC
FROM THE GROUND UP

LOG OF TEST BORING

CLIENT: CT Consultants, Inc.
PROJECT: Amsterdam Road Reconstruction, Phase 1
Fort Wright, Kentucky
LOCATION OF BORING: As shown on Boring Plan, Drawing 1

BORING #: 214
PROJECT #: J029038.01
PAGE #: 1 of 1

ELEV.	COLOR, MOISTURE, DENSITY, PLASTICITY, SIZE, PROPORTIONS DESCRIPTION	Strata Depth (feet)	Depth Scale (feet)	Sample Condition	Sample Number	Sample Type	SPT* Blows/6" / Rock Core RQD (%)	Recovery (in.)	Recovery (%)
834.2	Ground Surface	0.0	0						
834.0	ASPHALT (2 inches)	0.2		D	1	CA			
833.7	Crushed limestone base (4 inches)	0.5		I	2	DS	3-2-3	18	100
832.2	Orangish brown moist soft sandy FAT CLAY, trace oxide stains (glacial).	2.0							
	Reddish brown moist stiff sandy FAT CLAY, trace oxide stains (glacial).			I	3	DS	4-6-8	18	100
829.2		5.0		I	4	DS	2-4-5	18	100
	Reddish brown moist medium dense SAND, trace oxide stains (glacial).		5	I	5	DS	3-5-6	18	100
827.4		6.8							
	Orangish brown moist medium dense clayey SAND, trace silt with oxide concretions (glacial) [SC-SM/A-2-4(0)].			U	6	PT		24	100
825.2		9.0							
	Orangish brown loose clayey SAND, little silt with oxide concretions (glacial).		10	I	7	DS	5-3-5	18	100
822.7		11.5							
	Brown moist very loose silty SAND, trace fat clay, trace oxide stains (glacial).			I	8	DS	3-2-2	6	33
819.7		14.5							
	Brown moist very loose sandy SILT with oxide concretions (glacial).		15	I	9	DS	2-2-3	12	67
817.2		17.0							
	Brown moist loose sandy SILT, trace fat clay and oxide stains (glacial).			I	10	DS	2-4-4	12	67
814.7		19.5							
	Brown moist very loose sandy SILT, trace fat clay and oxide stains (glacial).		20	I	11	DS	2-2-2	12	67
810.9		23.3							
	Brown, trace gray moist medium stiff clayey SILT with shale fragments and limestone floaters, trace bedding planes (glacial).		25	I	12	DS	6-9-6	12	67
807.7		26.5							
	Bottom of test boring at 26.5 feet.		30						

Datum: NAVD 88
Surface Elevation: 834.2 ft.
Date Started: 4/11/2017
Date Completed: 4/11/2017
Hammer Weight: 140 lb.
Hammer Drop: 30 in.
Pipe Size: 2 in. O.D.
Hole Diameter: 8 in.
Rock Core Diameter: --
Boring Method: HSA-3.25
Drill Rig: CME-55 TD-5
Foreman: J. Franz
Engineer: Michelle E. Casto

BORING METHOD
HSA = Hollow Stem Augers
CFA = Continuous Flight Augers
DC = Driving Casing
MD = Mud Drilling

SAMPLE TYPE
PC = Pavement Core
CA = Continuous Flight Auger
DS = Driven Split Spoon
PT = Pressed Shelby Tube
RC = Rock Core

SAMPLE CONDITIONS
D = Disintegrated
I = Intact
U = Undisturbed
L = Lost

GROUNDWATER DEPTH
First Noted: None
At Completion: Dry
After: --
Backfilled: Immediately

* SPT = Standard Penetration Test - Driving 2" O.D. Sampler 18" with 140-Pound Hammer Falling 30"; Count Made at 6" Intervals

GEOTECHNOLOGY INC
FROM THE GROUND UP

LOG OF TEST BORING

CLIENT: CT Consultants, Inc.
PROJECT: Amsterdam Road Reconstruction, Phase 1
Fort Wright, Kentucky
LOCATION OF BORING: As shown on Boring Plan, Drawing 1

BORING #: 216
PROJECT #: J029038.01
PAGE #: 1 of 2

ELEV.	COLOR, MOISTURE, DENSITY, PLASTICITY, SIZE, PROPORTIONS DESCRIPTION	Strata Depth (feet)	Sample Condition	Sample Number	Sample Type	SPT* Blows/6" / Rock Core RQD (%)	Recovery (in.)	Recovery (%)
828.5	Ground Surface	0.0						
827.4	ASPHALT (13 inches)	1.1						
825.9	Mixed brown moist medium stiff FILL, sandy lean clay, trace topsoil, trace oxide stains.	2.6	I	1	DS	2-3-3	18	100
824.4	Mixed dark brown moist medium stiff FILL, lean clay, trace sand, trace topsoil with oxide stains.	4.1	I	2	DS	2-2-2	18	100
821.9	Brown moist medium stiff sandy LEAN CLAY, trace oxide stains (glacial).	6.6	I	3	DS	2-3-2	18	100
819.0	Orangish brown, trace light gray moist stiff sandy FAT CLAY with oxide concretions (glacial).	9.5	U	4	PT		13	54
816.7	Mottled orangish brown moist medium stiff sandy FAT CLAY (glacial).	11.8	I	5	DS	2-2-3		
814.0	Mottled orangish brown moist loose clayey SAND with oxide concretions (glacial).	14.5	I	6	DS	3-4-4	18	100
811.5	Mottled brown moist very loose sandy SILT, little fat clay with oxide concretions (glacial).	17.0	D	7	DS	2-2-2	18	100
809.0	Mottled brown, trace light gray moist stiff LEAN CLAY, trace fat clay with oxide stains, partially layered (glacial).	19.5	I	8	DS	2-2-2	18	100
806.5	Brown and brownish gray moist stiff sandy LEAN CLAY, trace gravel and oxide concretions (glacial).	22.0	I	9	DS	3-5-6	18	100
804.0	Brown wet dense sandy SILT, trace gravel (glacial).	24.5	D	10	DS	11-16-19	18	100
800.2	Gray moist very stiff sandy LEAN CLAY, some gravel (glacial till).	28.3	I	11	DS	10-10-9	15	83
	Gray moist stiff sandy LEAN CLAY, trace gravel with clayey silt seams (glacial).							

Depth Scale (feet): 0, 5, 10, 15, 20, 25, 30

Datum: NAVD 88
Surface Elevation: 828.5 ft.
Date Started: 4/7/2017
Date Completed: 4/7/2017
Hammer Weight: 140 lb.
Hammer Drop: 30 in.
Pipe Size: 2 in. O.D.
Hole Diameter: 8 in.
Rock Core Diameter: --
Boring Method: HSA-3.25
Drill Rig: CME-55 TD-5
Foreman: M. Lozier
Engineer: Michelle E. Casto

BORING METHOD
HSA = Hollow Stem Augers
CFA = Continuous Flight Augers
DC = Driving Casing
MD = Mud Drilling

SAMPLE TYPE
PC = Pavement Core
CA = Continuous Flight Auger
DS = Driven Split Spoon
PT = Pressed Shelby Tube
RC = Rock Core

SAMPLE CONDITIONS
D = Disintegrated
I = Intact
U = Undisturbed
L = Lost

GROUNDWATER DEPTH
First Noted: Trace @ 22.5 ft.
At Completion: --
After: --
Backfilled: Immediately

* SPT = Standard Penetration Test - Driving 2" O.D. Sampler 18" with 140-Pound Hammer Falling 30"; Count Made at 6" Intervals

GEOTECHNOLOGY INC
FROM THE GROUND UP

LOG OF TEST BORING

CLIENT: CT Consultants, Inc.
BORING #: 216
PROJECT: Amsterdam Road Reconstruction, Phase 1
Fort Wright, Kentucky
PROJECT #: J029038.01

LOCATION OF BORING: As shown on Boring Plan, Drawing 1

ELEV.	COLOR, MOISTURE, DENSITY, PLASTICITY, SIZE, PROPORTIONS DESCRIPTION	Strata Depth (feet)	Depth Scale (feet)	Sample Condition	Sample Number	Sample Type	SPT* Blows/6" / Rock Core RQD (%)	Recovery (in.)	Recovery (%)
	Gray moist stiff sandy LEAN CLAY, trace gravel with clayey silt seams (glacial).		30	I	12	DS	12-13-11	15	83
795.2		33.3							
	Gray moist very stiff FAT CLAY with blue azurite deposits (glacial).		35	I	13	DS	3-4-7	18	100
790.2		38.3							
	Gray, trace olive brown moist very stiff FAT CLAY (glacial).		40	I	14	DS	6-7-7	18	100
785.2		43.3							
	Gray and olive brown moist stiff FAT CLAY with shale fragments, trace bedding planes (residual).		45	I	15	DS	4-4-5	18	100
780.2		48.3							
	Reddish brown moist very stiff sandy FAT CLAY, trace silt with oxide concretions, trace bedding planes (residual).		50	I	16	DS	12-21-34	18	100
775.2		53.3							
	Interbedded gray moist very weak unweathered SHALE and gray medium strong to very strong unweathered LIMESTONE (bedrock).								
773.2		55.3	55	I	17	DS	100/3"	3	100
	Bottom of test boring at 55.3 feet.		60						

Datum: NAVD 88	Hammer Weight: 140 lb.	Hole Diameter: 8 in.	Drill Rig: CME-55 TD-5
Surface Elevation: 828.5 ft.	Hammer Drop: 30 in.	Rock Core Diameter: --	Foreman: M. Lozier
Date Started: 4/7/2017	Pipe Size: 2 in. O.D.	Boring Method: HSA-3.25	Engineer: Michelle E. Casto
Date Completed: 4/7/2017			

BORING METHOD
HSA = Hollow Stem Augers
CFA = Continuous Flight Augers
DC = Driving Casing
MD = Mud Drilling

SAMPLE TYPE
PC = Pavement Core
CA = Continuous Flight Auger
DS = Driven Split Spoon
PT = Pressed Shelby Tube
RC = Rock Core

SAMPLE CONDITIONS
D = Disintegrated
I = Intact
U = Undisturbed
L = Lost

GROUNDWATER DEPTH
First Noted: Trace @ 22.5 ft.
At Completion: --
After: --
Backfilled: Immediately

* SPT = Standard Penetration Test - Driving 2" O.D. Sampler 18" with 140-Pound Hammer Falling 30"; Count Made at 6" Intervals

GEOTECHNOLOGY INC

FROM THE GROUND UP

LOG OF TEST BORING

CLIENT: CT Consultants, Inc.

PROJECT: Amsterdam Road Reconstruction, Phase 1
Fort Wright, Kentucky

LOCATION OF BORING: As shown on Boring Plan, Drawing 1

BORING #: 218

PROJECT #: J029038.01

ELEV.	COLOR, MOISTURE, DENSITY, PLASTICITY, SIZE, PROPORTIONS DESCRIPTION	Strata Depth (feet)	Depth Scale (feet)	Sample Condition	Sample Number	Sample Type	SPT* Blows/6" / Rock Core RQD (%)	Recovery (in.)	Recovery (%)
836.5	Ground Surface	0.0	0						
	ASPHALT (15 inches)								
835.2		1.3							
	Mixed dark brown moist medium stiff FILL, sandy lean clay, little topsoil.			I	1	DS	3-3-3	12	67
834.0		2.5							
	Mottled brown moist soft LEAN CLAY, trace sand with oxide stains (glacial).			I	2	DS	WOH-1-2	6	33
832.2		4.3							
	Orangish brown moist medium dense clayey SAND with oxide stains (glacial).		5	I	3	DS	3-5-6	18	100
				U	4	PT		24	100
827.0		9.5							
	Orangish brown moist very loose sandy SILT, some fat clay, trace oxide stains (glacial).		10	I	5	DS	2-2-2		
824.5		12.0							
	Orangish brown moist loose sandy SILT, trace fat clay, trace oxide stains (glacial).			I	6	DS	3-3-4	18	100
822.0		14.5							
	Orangish brown wet loose clayey SILT, trace sand with oxide stains (glacial) [CL/A-6(14)].		15						
820.6		15.9							
	Brown to orangish brown moist stiff to very stiff LEAN CLAY, trace sand with oxide stains (glacial).			U	7	PT		24	100
819.8		16.7							
819.5		17.0							
	Orangish brown wet loose silty SAND, trace fat clay with oxide stains (glacial).								
	Brown moist soft clayey SILT, trace fat clay pockets, trace oxide stains (glacial).			I	8	DS	2-1-2	18	100
817.2		19.3							
	Mottled brown moist stiff FAT CLAY with lean clay seams, trace oxide stains (glacial).		20						
815.5		21.0		I	9A 9B	DS	3-3-3	6	33
	Brown moist soft to medium stiff LEAN CLAY with oxide stains, partially laminated (glacial).								
813.2		23.3							
	Gray moist medium stiff sandy LEAN CLAY, little gravel with silt seams (glacial).		25	I	10	DS	3-5-6	18	100
808.2		28.3							
	Gray moist stiff sandy LEAN CLAY, little gravel with silt seams (glacial).		30						

Datum: NAVD 88
Hammer Weight: 140 lb.
Hole Diameter: 8 in.
Drill Rig: CME-55 TD-5
Surface Elevation: 836.5 ft.
Hammer Drop: 30 in.
Rock Core Diameter: --
Foreman: J. Franz
Date Started: 4/10/2017
Pipe Size: 2 in. O.D.
Boring Method: HSA-3.25
Engineer: Michelle E. Casto
Date Completed: 4/10/2017

BORING METHOD
HSA = Hollow Stem Augers
CFA = Continuous Flight Augers
DC = Driving Casing
MD = Mud Drilling

SAMPLE TYPE
PC = Pavement Core
CA = Continuous Flight Auger
DS = Driven Split Spoon
PT = Pressed Shelby Tube
RC = Rock Core

SAMPLE CONDITIONS
D = Disintegrated
I = Intact
U = Undisturbed
L = Lost

GROUNDWATER DEPTH
First Noted 15.0 ft.
At Completion 10.0 ft.
After --
Backfilled Immediately

* SPT = Standard Penetration Test - Driving 2" O.D. Sampler 18" with 140-Pound Hammer Falling 30"; Count Made at 6" Intervals

GEOTECHNOLOGY INC
FROM THE GROUND UP

LOG OF TEST BORING

CLIENT: CT Consultants, Inc.
BORING #: 218
PROJECT: Amsterdam Road Reconstruction, Phase 1
Fort Wright, Kentucky
PROJECT #: J029038.01

LOCATION OF BORING: As shown on Boring Plan, Drawing 1

ELEV.	COLOR, MOISTURE, DENSITY, PLASTICITY, SIZE, PROPORTIONS DESCRIPTION	Strata Depth (feet)	Depth Scale (feet)	Sample Condition	Sample Number	Sample Type	SPT* Blows/6" / Rock Core RQD (%)	Recovery (in.)	Recovery (%)
	Gray moist stiff sandy LEAN CLAY, little gravel with silt seams (glacial).		30	I	11	DS	2-5-5	12	67
803.2		33.3							
	Gray moist dense SILT, trace sand, trace fat clay (glacial).		35	I	12	DS	13-23-27	18	100
798.2		38.3							
	Gray moist soft to medium stiff sandy LEAN CLAY, little gravel (glacial).		40	I	13	DS	3-3-5	18	100
793.2		43.3							
	Gray moist soft sandy LEAN CLAY, little gravel with wood, trace gray shale fragments (glacial).		45	I	14	DS	2-2-50	18	100
788.2		48.3							
	Gray moist stiff LEAN CLAY, trace fat clay, trace blue azurite deposits and oxide stains (glacial).		50	I	15	DS	3-4-6	18	100
783.2		53.3							
	Blue, trace dark brown moist medium dense clayey SAND, trace gravel (glacial).		55						
780.5		56.0		I	16A 16B	DS	3-5-8		
	Black, some blue wet medium dense fine to coarse SAND, trace fat clay (glacial).								
778.2		58.3							
	Interbedded gray moist very weak unweathered SHALE and gray medium strong to very strong unweathered LIMESTONE (bedrock).		60						

Datum: NAVD 88
Hammer Weight: 140 lb.
Hole Diameter: 8 in.
Drill Rig: CME-55 TD-5
Surface Elevation: 836.5 ft.
Hammer Drop: 30 in.
Rock Core Diameter: --
Foreman: J. Franz
Date Started: 4/10/2017
Pipe Size: 2 in. O.D.
Boring Method: HSA-3.25
Engineer: Michelle E. Casto
Date Completed: 4/10/2017

BORING METHOD
HSA = Hollow Stem Augers
CFA = Continuous Flight Augers
DC = Driving Casing
MD = Mud Drilling

SAMPLE TYPE
PC = Pavement Core
CA = Continuous Flight Auger
DS = Driven Split Spoon
PT = Pressed Shelby Tube
RC = Rock Core

SAMPLE CONDITIONS
D = Disintegrated
I = Intact
U = Undisturbed
L = Lost

GROUNDWATER DEPTH
First Noted 15.0 ft.
At Completion 10.0 ft.
After --
Backfilled Immediately

* SPT = Standard Penetration Test - Driving 2" O.D. Sampler 18" with 140-Pound Hammer Falling 30"; Count Made at 6" Intervals

GEOTECHNOLOGY INC
FROM THE GROUND UP

LOG OF TEST BORING

CLIENT: CT Consultants, Inc.
BORING #: 218
PROJECT: Amsterdam Road Reconstruction, Phase 1
Fort Wright, Kentucky
PROJECT #: J029038.01
PAGE #: 3 of 3
LOCATION OF BORING: As shown on Boring Plan, Drawing 1

ELEV.	COLOR, MOISTURE, DENSITY, PLASTICITY, SIZE, PROPORTIONS DESCRIPTION	Strata Depth (feet)	Depth Scale (feet)	Sample Condition	Sample Number	Sample Type	SPT* Blows/6" / Rock Core RQD (%)	Recovery (in.)	Recovery (%)
			60	I	17	DS	71/1"	1	100
	Interbedded gray moist very weak unweathered SHALE and gray medium strong to very strong unweathered LIMESTONE (bedrock).								
771.3		65.2	65	I	18	DS	75/2"	2	100
	Bottom of test boring at 65.2 feet.								
			70						
			75						
			80						
			85						
			90						

Datum: NAVD 88
Hammer Weight: 140 lb.
Hole Diameter: 8 in.
Drill Rig: CME-55 TD-5
Surface Elevation: 836.5 ft.
Hammer Drop: 30 in.
Rock Core Diameter: --
Foreman: J. Franz
Date Started: 4/10/2017
Pipe Size: 2 in. O.D.
Boring Method: HSA-3.25
Engineer: Michelle E. Casto
Date Completed: 4/10/2017

BORING METHOD
HSA = Hollow Stem Augers
CFA = Continuous Flight Augers
DC = Driving Casing
MD = Mud Drilling

SAMPLE TYPE
PC = Pavement Core
CA = Continuous Flight Auger
DS = Driven Split Spoon
PT = Pressed Shelby Tube
RC = Rock Core

SAMPLE CONDITIONS
D = Disintegrated
I = Intact
U = Undisturbed
L = Lost

GROUNDWATER DEPTH
First Noted 15.0 ft.
At Completion 10.0 ft.
After --
Backfilled Immediately

* SPT = Standard Penetration Test - Driving 2" O.D. Sampler 18" with 140-Pound Hammer Falling 30"; Count Made at 6" Intervals

GEOTECHNOLOGY INC
FROM THE GROUND UP

LOG OF TEST BORING

CLIENT: CT Consultants, Inc.
BORING #: 219
PROJECT: Amsterdam Road Reconstruction, Phase 1
Fort Wright, Kentucky
PROJECT #: J029038.01
PAGE #: 1 of 1
LOCATION OF BORING: As shown on Boring Plan, Drawing 1

ELEV.	COLOR, MOISTURE, DENSITY, PLASTICITY, SIZE, PROPORTIONS DESCRIPTION	Strata Depth (feet)	Depth Scale (feet)	Sample Condition	Sample Number	Sample Type	SPT* Blows/6" / Rock Core RQD (%)	Recovery (in.)	Recovery (%)
849.4	Ground Surface	0.0	0						
848.6	ASPHALT (10 inches)	0.8							
848.4	Crushed limestone base (2 inches)	1.0		D	1	CA			
846.9	Orangish brown moist very loose clayey SAND, trace gravel (glacial).	2.5		I	2	DS	3-2-3	18	100
	Orangish brown moist loose clayey SAND, trace gravel with oxide concretions and shale fragments (glacial).			I	3	DS	3-5-4	18	100
843.9		5.5	5	I	4	DS	4-4-5	18	100
	Bottom of test boring at 5.5 feet.		10						
			15						
			20						
			25						
			30						

Datum: NAVD 88
Hammer Weight: 140 lb.
Hole Diameter: 8 in.
Drill Rig: CME-55 TD-5
Surface Elevation: 849.4 ft.
Hammer Drop: 30 in.
Rock Core Diameter: --
Foreman: J. Franz
Date Started: 4/12/2017
Pipe Size: 2 in. O.D.
Boring Method: HSA-3.25
Engineer: Michelle E. Casto
Date Completed: 4/12/2017

BORING METHOD
HSA = Hollow Stem Augers
CFA = Continuous Flight Augers
DC = Driving Casing
MD = Mud Drilling

SAMPLE TYPE
PC = Pavement Core
CA = Continuous Flight Auger
DS = Driven Split Spoon
PT = Pressed Shelby Tube
RC = Rock Core

SAMPLE CONDITIONS
D = Disintegrated
I = Intact
U = Undisturbed
L = Lost

GROUNDWATER DEPTH
First Noted: None
At Completion: Dry
After: --
Backfilled: Immediately

* SPT = Standard Penetration Test - Driving 2" O.D. Sampler 18" with 140-Pound Hammer Falling 30"; Count Made at 6" Intervals

GEOTECHNOLOGY INC
FROM THE GROUND UP

LOG OF TEST BORING

CLIENT: CT Consultants, Inc.
BORING #: 220
PROJECT: Amsterdam Road Reconstruction, Phase 1
Fort Wright, Kentucky
PROJECT #: J029038.01
PAGE #: 1 of 1
LOCATION OF BORING: As shown on Boring Plan, Drawing 1

ELEV.	COLOR, MOISTURE, DENSITY, PLASTICITY, SIZE, PROPORTIONS DESCRIPTION	Strata Depth (feet)	Depth Scale (feet)	Sample Condition	Sample Number	Sample Type	SPT* Blows/6" / Rock Core RQD (%)	Recovery (in.)	Recovery (%)
858.1	Ground Surface	0.0	0						
857.1	ASPHALT (12 inches)	1.0							
856.1	Crushed limestone base (12 inches)	2.0		D	1	CA			
854.6	Orangish brown moist stiff sandy FAT CLAY, trace oxide stains (glacial).	3.5		I	2	DS	2-2-3	12	67
853.1	Orangish brown moist very stiff FAT CLAY with oxide stains and concretions (glacial).	5.0	5	I	3	DS	3-4-7	12	67
851.6	Brown moist very stiff FAT CLAY, trace sand with oxide stains and concretions (glacial).	6.5		I	4	DS	6-8-9	12	67
	Bottom of test boring at 6.5 feet.		10, 15, 20, 25, 30						

Datum: NAVD 88
Hammer Weight: 140 lb.
Hole Diameter: 8 in.
Drill Rig: CME-55 TD-5
Surface Elevation: 858.1 ft.
Hammer Drop: 30 in.
Rock Core Diameter: --
Foreman: J. Franz
Date Started: 4/12/2017
Pipe Size: 2 in. O.D.
Boring Method: HSA-3.25
Engineer: Michelle E. Casto
Date Completed: 4/12/2017

BORING METHOD
HSA = Hollow Stem Augers
CFA = Continuous Flight Augers
DC = Driving Casing
MD = Mud Drilling

SAMPLE TYPE
PC = Pavement Core
CA = Continuous Flight Auger
DS = Driven Split Spoon
PT = Pressed Shelby Tube
RC = Rock Core

SAMPLE CONDITIONS
D = Disintegrated
I = Intact
U = Undisturbed
L = Lost

GROUNDWATER DEPTH
First Noted: None
At Completion: Dry
After: --
Backfilled: Immediately

* SPT = Standard Penetration Test - Driving 2" O.D. Sampler 18" with 140-Pound Hammer Falling 30"; Count Made at 6" Intervals

GEOTECHNOLOGY INC
FROM THE GROUND UP

LOG OF TEST BORING

CLIENT: CT Consultants, Inc.
BORING #: 221
PROJECT: Amsterdam Road Reconstruction, Phase 1
Fort Wright, Kentucky
PROJECT #: J029038.01
PAGE #: 1 of 1
LOCATION OF BORING: As shown on Boring Plan, Drawing 1

ELEV.	COLOR, MOISTURE, DENSITY, PLASTICITY, SIZE, PROPORTIONS DESCRIPTION	Strata Depth (feet)	Depth Scale (feet)	Sample Condition	Sample Number	Sample Type	SPT* Blows/6" / Rock Core RQD (%)	Recovery (in.)	Recovery (%)
861.6	Ground Surface	0.0	0						
860.6	ASPHALT (12 inches)	1.0							
859.6	Crushed limestone base (12 inches)	2.0		D	1	CA			
858.1	Orangish brown moist stiff clayey SILT, little sand, trace oxide stains (glacial).	3.5		I	2	DS	3-5-5	18	100
856.6	Orangish brown moist stiff sandy LEAN CLAY, trace oxide stains (glacial) [CL/A-7-6(19)].	5.0	5	I	3	DS	6-8-12	18	100
855.1	Orangish brown moist medium stiff LEAN CLAY, little sand, trace oxide stains (glacial).	6.5		I	4	DS	11-11-10	12	67
	Bottom of test boring at 6.5 feet.		10						
			15						
			20						
			25						
			30						

Datum: NAVD 88
Hammer Weight: 140 lb.
Hole Diameter: 8 in.
Drill Rig: CME-55 TD-5
Surface Elevation: 861.6 ft.
Hammer Drop: 30 in.
Rock Core Diameter: --
Foreman: J. Franz
Date Started: 4/12/2017
Pipe Size: 2 in. O.D.
Boring Method: HSA-3.25
Engineer: Michelle E. Casto
Date Completed: 4/12/2017

BORING METHOD
HSA = Hollow Stem Augers
CFA = Continuous Flight Augers
DC = Driving Casing
MD = Mud Drilling

SAMPLE TYPE
PC = Pavement Core
CA = Continuous Flight Auger
DS = Driven Split Spoon
PT = Pressed Shelby Tube
RC = Rock Core

SAMPLE CONDITIONS
D = Disintegrated
I = Intact
U = Undisturbed
L = Lost

GROUNDWATER DEPTH
First Noted: None
At Completion: Dry
After: --
Backfilled: Immediately

* SPT = Standard Penetration Test - Driving 2" O.D. Sampler 18" with 140-Pound Hammer Falling 30"; Count Made at 6" Intervals

GEOTECHNOLOGY INC
FROM THE GROUND UP

LOG OF TEST BORING

CLIENT: CT Consultants, Inc.
PROJECT: Amsterdam Road Reconstruction, Phase 1
Fort Wright, Kentucky
LOCATION OF BORING: As shown on Boring Plan, Drawing 1

BORING #: 222
PROJECT #: J029038.01

ELEV.	COLOR, MOISTURE, DENSITY, PLASTICITY, SIZE, PROPORTIONS DESCRIPTION	Strata Depth (feet)	Depth Scale (feet)	Sample Condition	Sample Number	Sample Type	SPT* Blows/6" / Rock Core RQD (%)	Recovery (in.)	Recovery (%)
862.9	Ground Surface	0.0	0						
862.2	ASPHALT (8 inches)	0.7							
861.7	Crushed limestone base (7.5 inches)	1.2		D	1	CA			
860.2	Mixed gray damp medium dense FILL, limestone floaters, trace sandy clay.	2.7		I	2	DS	28-12-9	3	17
858.7	Orangish brown moist stiff sandy FAT CLAY, trace oxide concretions (glacial).	4.2		I	3	DS	3-3-7	18	100
857.2	Orangish brown moist loose SAND, trace fat clay, trace oxide stains (glacial).	5.7	5	I	4	DS	8-7-9	18	100
	Bottom of test boring at 5.7 feet.		10						
			15						
			20						
			25						
			30						

Datum: NAVD 88
Surface Elevation: 862.9 ft.
Date Started: 4/12/2017
Date Completed: 4/12/2017
Hammer Weight: 140 lb.
Hammer Drop: 30 in.
Pipe Size: 2 in. O.D.
Hole Diameter: 8 in.
Rock Core Diameter: --
Boring Method: HSA-3.25
Drill Rig: CME-55 TD-5
Foreman: J. Franz
Engineer: Michelle E. Casto

BORING METHOD
HSA = Hollow Stem Augers
CFA = Continuous Flight Augers
DC = Driving Casing
MD = Mud Drilling

SAMPLE TYPE
PC = Pavement Core
CA = Continuous Flight Auger
DS = Driven Split Spoon
PT = Pressed Shelby Tube
RC = Rock Core

SAMPLE CONDITIONS
D = Disintegrated
I = Intact
U = Undisturbed
L = Lost

GROUNDWATER DEPTH
First Noted: None
At Completion: Dry
After: --
Backfilled: Immediately

* SPT = Standard Penetration Test - Driving 2" O.D. Sampler 18" with 140-Pound Hammer Falling 30"; Count Made at 6" Intervals

THELEN ASSOCIATES, INC.
Geotechnical • Testing Engineers
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www.thelenassoc.com

LOG OF TEST BORING

CLIENT: City of Ft. Mitchell **BORING # :** 9

PROJECT: Consulting Services, Amsterdam Road Repair, Ft. Mitchell, Kentucky **JOB # :** 070087E

LOCATION OF BORING: As shown on Site Plan, Drawing 070087E-1

ELEV.	SOIL DESCRIPTION COLOR, MOISTURE, DENSITY, PLASTICITY, SIZE, PROPORTIONS	STRATA DEPTH (feet)	SAMPLE Cond	Blows/6"	No.	Type	Rec. (Inches)
764.0	SURFACE	0.0					
762.5	ASPHALT	1.5					
760.8	Mixed brown moist medium stiff to stiff FILL, silty clay, trace topsoil.	3.2	I	2/7/5	1	DS	10
760.1	Mixed dark brown moist medium stiff FILL, silty clay, some topsoil.	3.9	I	4/4/6	2A 2B	DS	10
			I	5/5/15	3	DS	12
757.0	Brown, trace gray moist stiff SILTY CLAY with shale fragments (colluvium).	7.0	I	22/25/32	4	DS	18
752.0	Interbedded brown moist very soft highly weathered SHALE and gray hard LIMESTONE (bedrock).	12.0	I	50/4"	5	DS	4
749.5	Interbedded olive brown moist soft weathered SHALE and gray hard LIMESTONE (bedrock).	14.5	I	50/3"	6	DS	3
748.5	Interbedded gray moist soft SHALE and gray hard LIMESTONE (bedrock).	15.5	I	50/6"	7	DS	6

Split spoon refusal and bottom of test boring at 15.5 feet.

Datum: Est. MSL	Hammer Wt.: 140 lbs.	Hole Diameter: 5 in.	Foreman: ML
Surf. Elev.: 764.0 ft.	Hammer Drop: 30 in.	Rock Core Dia.: - in.	Engineer: MES
Date Started: 4/7/08	Pipe Size: - in.	Boring Method: CFA	Date Completed: 4/7/08

SAMPLE CONDITIONS
D - DISINTEGRATED
I - INTACT
U - UNDISTURBED
L - LOST

SAMPLE TYPE
DS - DRIVEN SPLIT SPOON
PT - PRESSED SHELBY TUBE
CA - CONTINUOUS FLIGHT AUGER
RC - ROCK CORE

GROUNDWATER DEPTH
FIRST NOTED None ft.
AT COMPLETION Dry ft.
AFTER - hrs. - ft.
BACKFILLED Immed. hrs.

BORING METHOD
HSA - HOLLOW STEM AUGERS
CFA - CONTINUOUS FLIGHT AUGERS
DC - DRIVING CASING
MD - MUD DRILLING

STANDARD PENETRATION TEST - DRIVING 2" O.D. SAMPLER 1' WITH 140# HAMMER FALLING 30"; COUNT MADE AT 6" INTERVALS

THELEN ASSOCIATES, INC.
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LOG OF TEST BORING

CLIENT: City of Ft. Mitchell **BORING #:** 10

PROJECT: Consulting Services, Amsterdam Road Repair, Ft. Mitchell, Kentucky **JOB #:** 070087E

LOCATION OF BORING: As shown on Site Plan, Drawing 070087E-1

ELEV.	SOIL DESCRIPTION COLOR, MOISTURE, DENSITY, PLASTICITY, SIZE, PROPORTIONS	STRATA DEPTH (feet)	DEPTH SCALE (feet)	SAMPLE Cond	Blows/6"	No.	Type	Rec. (Inches)
764.2	SURFACE	0.0						
763.2	ASPHALT	1.0		I	4/50/6"	1	DS	6
761.7	Brown moist stiff SILTY CLAY with limestone floaters, trace bedding planes (residual).	2.5		I	14/31/14	2	DS	12
759.6	Interbedded brown moist very soft highly weathered SHALE and gray hard LIMESTONE with clay layers (bedrock).	4.6	5	I	20/27/44	3	DS	18
757.7	Interbedded olive brown moist soft weathered SHALE and gray hard LIMESTONE (bedrock).	6.5						
	Bottom of test boring at 6.5 feet.		10					
			15					
			20					
			25					

Datum Est. MSL Hammer Wt. 140 lbs. Hole Diameter 5 in. Foreman ML

Surf. Elev. 764.2 ft. Hammer Drop 30 in. Rock Core Dia. - in. Engineer MES

Date Started 4/3/08 Pipe Size - in. Boring Method CFA Date Completed 4/3/08

SAMPLE CONDITIONS	SAMPLE TYPE	GROUNDWATER DEPTH	BORING METHOD
D - DISINTEGRATED	DS - DRIVEN SPLIT SPOON	FIRST NOTED None ft.	HSA - HOLLOW STEM AUGERS
I - INTACT	PT - PRESSED SHELBY TUBE	AT COMPLETION Dry ft.	CFA - CONTINUOUS FLIGHT AUGERS
U - UNDISTURBED	CA - CONTINUOUS FLIGHT AUGER	AFTER - hrs. - ft.	DC - DRIVING CASING
L - LOST	RC - ROCK CORE	BACKFILLED Immed. hrs.	MD - MUD DRILLING

STANDARD PENETRATION TEST - DRIVING 2" O.D. SAMPLER 1' WITH 140# HAMMER FALLING 30"; COUNT MADE AT 6" INTERVALS

CIVIL ENGINEERS

G. J. Thelen & Associates, Inc.

☑ 516 Enterprise Drive / Covington, Kentucky 41017-1595 / 606-341-1322 / Fax 606-341-0832
☐ 1310 Kemper Meadow Drive, Suite 600 / Forest Park, Ohio 45240-1651 / 513-825-4350 / Fax 513-825-4756

LOG OF TEST BORING

CLIENT Kenton County Fiscal Court BORING # 104
PROJECT Landslide Exploration, Amsterdam Road, Kenton County, Kentucky JOB # 91018E
LOCATION OF BORING As shown on contract documents prepared by David E. Estes Engineering, Inc.

ELEV.	SOIL DESCRIPTION COLOR, MOISTURE, DENSITY, PLASTICITY, SIZE, PROPORTIONS	STRA. DEPTH	DEPTH SCALE	SAMPLE Cond	Blows/6"	No.	Type	Rec.
770.0	SURFACE							
769.0	Mixed brown, black and gray moist medium stiff FILL, silty clay, topsoil, sand and gravel.	1.0		I	5/4/5	1	DS	14"
				I	3/4/3	2	DS	16"
765.5	Mixed dark gray and black moist medium stiff FILL, silty clay with decayed roots.	4.5						
764.5	Dark brown moist medium stiff to stiff SILTY CLAY, trace roots.	5.5	5	I	6/8/8	3A 3B	DS	15"
762.1	Mottled brown, trace gray moist medium stiff to stiff CLAY with limestone fragments and floaters.	7.9		I	6/6"	4	DS	6"
				I	8/10/15	5	DS	18"
760.0	Brown and gray moist stiff SILTY CLAY with limestone floaters and bedding planes.	10.0	10	I	10/10/14	6	DS	18"
757.2	Brown, trace gray moist very soft highly weathered SHALE and thinly bedded LIMESTONE (bedrock).	12.8		I	50/4"	7	DS	2"
	Bottom of test boring at 12.8 feet.		15					
			20					
			25					

Datum USGS Hammer Wt. 140 Lbs. Hole Diameter 5" Foreman GB
Surf. Elev. 770.0 Ft. Hammer Drop 30 In. Rock Core Dia. ___ Engineer JWK/TWV
Date Started 2/3/95 Pipe Size O.D.2 In. Boring Method CFA Date Completed 2/3/95

SAMPLE CONDITIONS	SAMPLER TYPE	GROUND WATER DEPTH	BORING METHOD
D – DISINTEGRATED	DS – DRIVEN SPLIT SPOON	FIRST NOTED None FT.	HSA – Hollow Stem Augers
I – INTACT	PT – PRESSED SHELBY TUBE	AT COMPLETION Dry FT.	CFA – Continous Flight Augers
U – UNDISTURBED	CA – CONTINUOUS FLIGHT AUGER	AFTER ___ HRS. ___ FT.	DC – Driving Casing
L – LOST	RC – ROCK CORE	BACKFILLED Immed. HRS.	MD – Mud Drilling

SOIL CLASSIFICATION SHEET

NON COHESIVE SOILS
(Silt, Sand, Gravel and Combinations)

Density

Very Loose	- 5 blows/ft. or less
Loose	- 6 to 10 blows/ft.
Medium Dense	- 11 to 30 blows/ft.
Dense	- 31 to 50 blows/ft.
Very Dense	- 51 blows/ft. or more

Relative Properties

Descriptive Term	Percent
Trace	1 – 10
Little	11 – 20
Some	21 – 35
And	36 – 50

Particle Size Identification

Boulders	- 8 inch diameter or more	
Cobbles	- 3 to 8 inch diameter	
Gravel	- Coarse	- 3/4 to 3 inches
	- Fine	- 3/16 to 3/4 inches
Sand	- Coarse	- 2mm to 5mm (dia. of pencil lead)
	- Medium	- 0.45mm to 2mm (dia. of broom straw)
	- Fine	- 0.075mm to 0.45mm (dia. of human hair)
Silt		- 0.005mm to 0.075mm (Cannot see particles)

COHESIVE SOILS
(Clay, Silt and Combinations)

Consistency	Field Identification	Unconfined Compressive Strength (tons/sq. ft.)
Very Soft	Easily penetrated several inches by fist	Less than 0.25
Soft	Easily penetrated several inches by thumb	0.25 – 0.5
Medium Stiff	Can be penetrated several inches by thumb with moderate effort	0.5 – 1.0
Stiff	Readily indented by thumb but penetrated only with great effort	1.0 – 2.0
Very Stiff	Readily indented by thumbnail	2.0 – 4.0
Hard	Indented with difficulty by thumbnail	Over 4.0

Classification on logs are made by visual inspection.

Standard Penetration Test – Driving a 2.0" O.D., 1 3/8" I.D., sampler a distance of 1.0 foot into undisturbed soil with a 140 pound hammer free falling a distance of 30 inches. It is customary to drive the spoon 6 inches to seat into undisturbed soil, then perform the test. The number of hammer blows for seating the spoon and making the tests are recorded for each 6 inches of penetration on the drill log (Example – 6/8/9). The standard penetration test results can be obtained by adding the last two figures (i.e. 8+9=17 blows/ft.). Refusal is defined as greater than 50 blows for 6 inches or less penetration.

Strata Changes – In the column "Soil Descriptions" on the drill log, the horizontal lines represent strata changes. A solid line (————) represents an actually observed change; a dashed line (— — — —) represents an estimated change.

Groundwater observations were made at the times indicated. Porosity of soil strata, weather conditions, site topography, etc., may cause changes in the water levels indicated on the logs.

ROCK CLASSIFICATION SHEET

ROCK WEATHERING

Descriptions	Field Identification
Unweathered	No visible sign of rock material weathering, perhaps slight discoloration on major discontinuity surfaces.
Weathered	Discoloration indicates weathering of rock material and discontinuity surfaces. All the rock material may be discolored by weathering and may be somewhat weaker externally than it its fresh condition.
Highly Weathered	Less than half of the rock material is decomposed and/or disintegrated to a soil. Fresh or discolored rock is present either as a discontinuous framework or as corestones.
Residual Soil	All rock material is decomposed and/or disintegrated to soil. The original mass structure is still largely intact with bedding planes visible, and the soil has not been significantly transported.

ROCK STRENGTH

Descriptions	Field Identification	Uniaxial Compressive Strength (psi)
Extremely Weak	Indented by thumbnail	40-150
Very Weak	Crumbles under firm blows with point of geological hammer, can be peeled by a pocket knife.	150-700
Weak	Can be peeled by a pocket knife with difficulty, shallow indentations made by firm blow with point of geological hammer.	700-4,000
Medium Strong	Cannot be scraped or peeled with a pocket knife, specimen can be fractured with a single blow of a geological hammer.	4,000-7,000
Strong	Specimen requires more than one blow of a geological hammer to fracture.	7,000-15,000
Very Strong	Specimen requires many blows with a geological hammer to fracture.	15,000-36,000
Extremely Strong	Specimen can only be chipped with geological hammer.	>36,000

BEDDING

Descriptive Term	Bed Thickness
Massive	> 4 ft.
Thick	2 to 4 ft.
Medium	2 in. to 2 ft.
Thin	< 2 in.

APPENDIX D – LABORATORY TEST DATA

Tabulation of Laboratory Tests

Particle-Size Analysis Test Forms

Soil Unconfined Compressive Strength Test Forms

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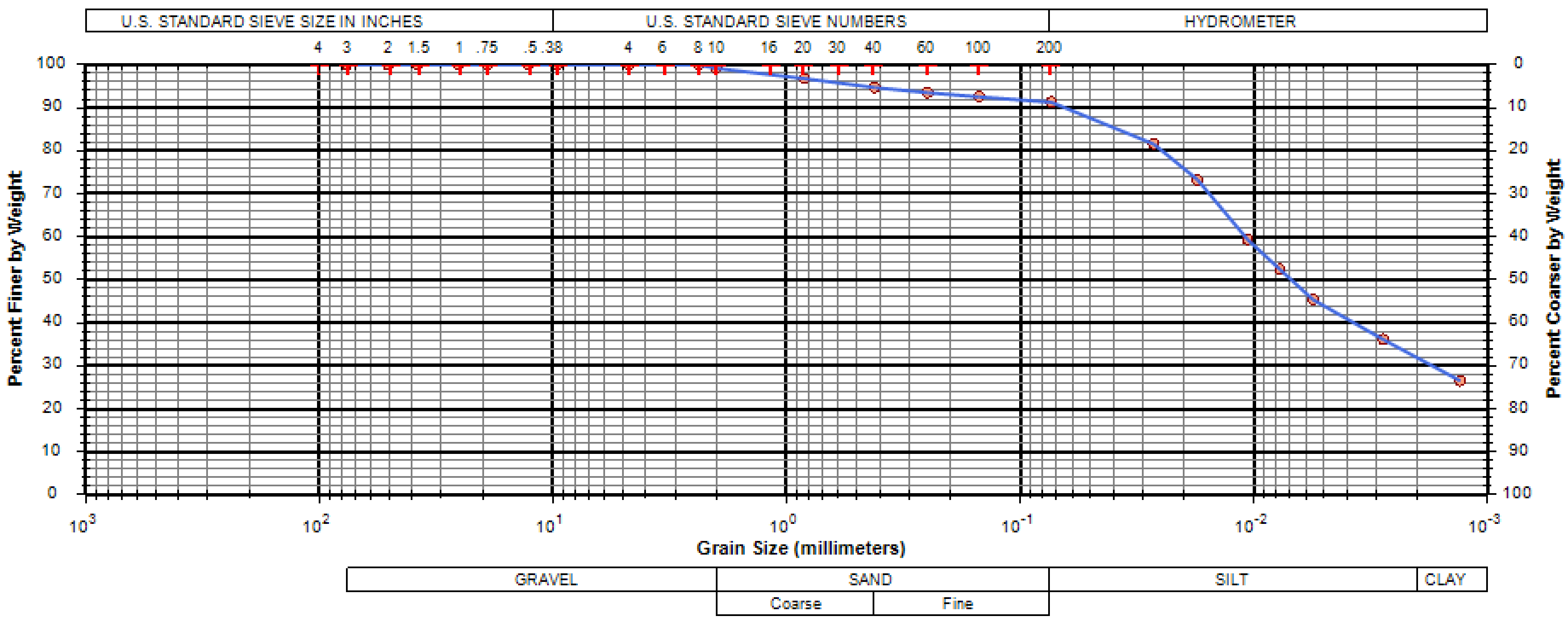
TABULATION OF LABORATORY TESTS

Boring No.	Sample No.	Depth (ft.) From	Depth (ft.) To	Moisture Content (%)	Dry Unit Weight (pcf)	Atterberg Limits (%) LL	PL	PI	Gradation Analysis (%) Gravel	Sand	Silt	Clay	USCS/AASHTO Classification	Unconfined Compressive Strength (psf)
201	2	2.5	4.0	23.3										
201	4	7.5	9.0	25.8										
201	5	10.0	11.5	14.8										
201	6	12.5	14.0	5.7										
201	7	15.0	16.3	14.9										
202	2	2.5	4.0	18.4		38	20	18	0.8	7.9	59.4	31.9	CL/A-6	
203	1	1.0	2.5	19.5										
203	2	2.5	4.0	19.6		38	23	15	3.3	14.0	52.2	30.5	CL/A-6	
203	3	4.0	5.5	20.5										
203	4	7.5	8.9	11.9										
203	5	10.0	10.8	14.9										
203	6	12.5	13.3	10.1										
203	7	15.0	15.4	3.5										
205	1	1.0	2.5	21.8										
205	2	2.5	4.0	24.7										
205	3	4.0	4.3	17.6										
205	4	7.5	9.0	19.5										
205	5	10.0	10.7	8.8										
205	6	12.5	12.7	3.8										
206	PT-3	5.3	5.8	31.4		59	29	30	0.5	11.1	36.1	52.3	CH/A-7-6	
208	2	2.5	4.0	29.0										
208	PT-3	5.5	6.0	21.0	105.9	34	16	18	0.1	34.0	38.0	27.9	CL/A-6	1,630
208	4	7.0	8.5	25.6										
210	2	2.5	4.0	25.2										
210	3	5.0	6.5	19.4										

PARTICLE-SIZE ANALYSIS OF SOILS AASHTO T88

Client:	CT Consultants, Inc.							Project No.:	J029038.01
Project:	Geotechnical Exploration, Amsterdam Road Reconstruction, Phase 1, Fort Wright, KY							Date:	04/26/2017
Boring No.:	202	Sample No.:	2	Depth (ft.):	2.5 - 4.0	Gravel (%)	Sand (%)	Silt (%)	Clay (%)	AASHTO
Sample Description:		Brown, trace gray moist stiff LEAN CLAY, trace roots and oxide stains, trace bedding planes (residual)				0.8	7.9	59.4	31.9	A-6
						LL	PL	PI	Group Index	WC (%)
						38	20	18	17	18.4



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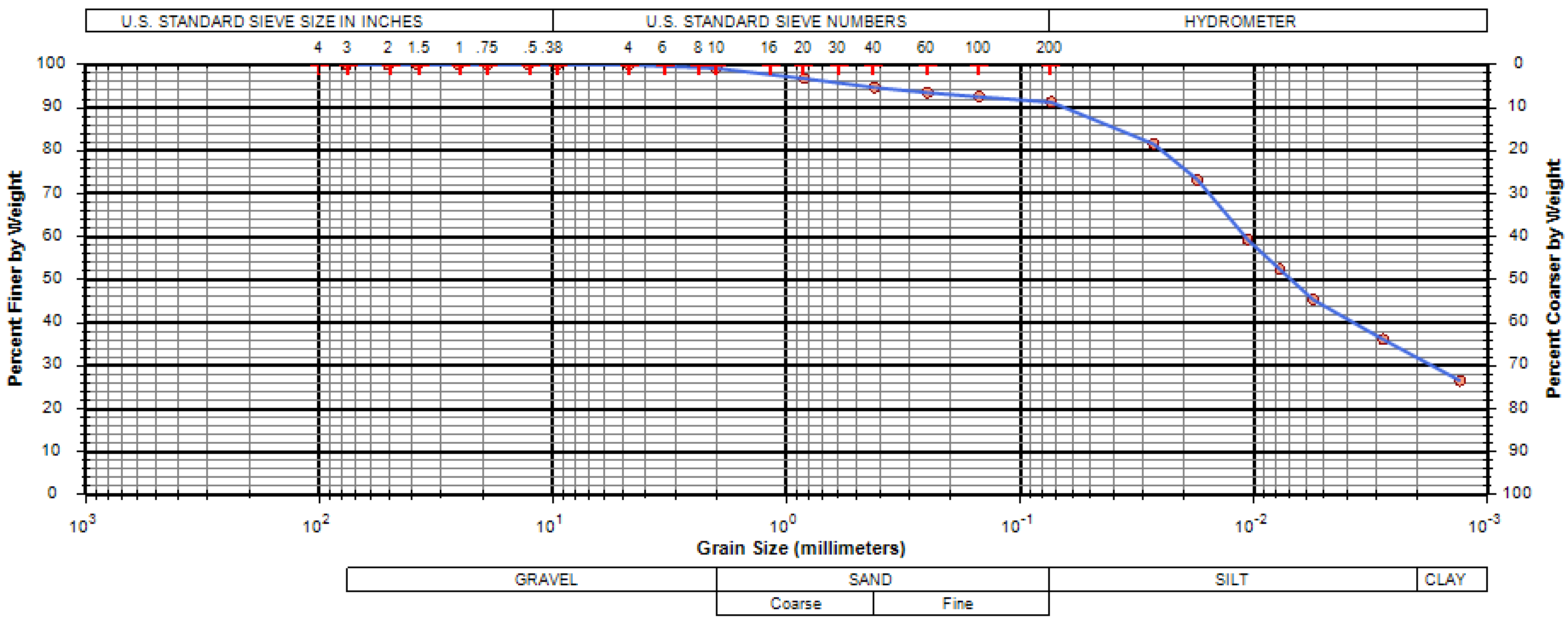
PARTICLE-SIZE ANALYSIS OF SOILS AASHTO T88

Client:	CT Consultants, Inc.							**Project No.:**	J029038.01
Project:	Geotechnical Exploration, Amsterdam Road Reconstruction, Phase 1, Fort Wright, KY							**Date:**	04/26/2017
Boring No.:	202	**Sample No.:**	2	**Depth (ft.):**	5.0 - 6.5	**Gravel (%)**	**Sand (%)**	**Silt (%)**	**Clay (%)**	**AASHTO**
Sample Description:		Brown, trace gray moist stiff LEAN CLAY, trace roots and oxide stains, trace bedding planes (residual)				0.8	7.9	59.4	31.9	A-6
						LL	**PL**	**PI**	**Group Index**	**WC (%)**
						38	20	18	17	18.4



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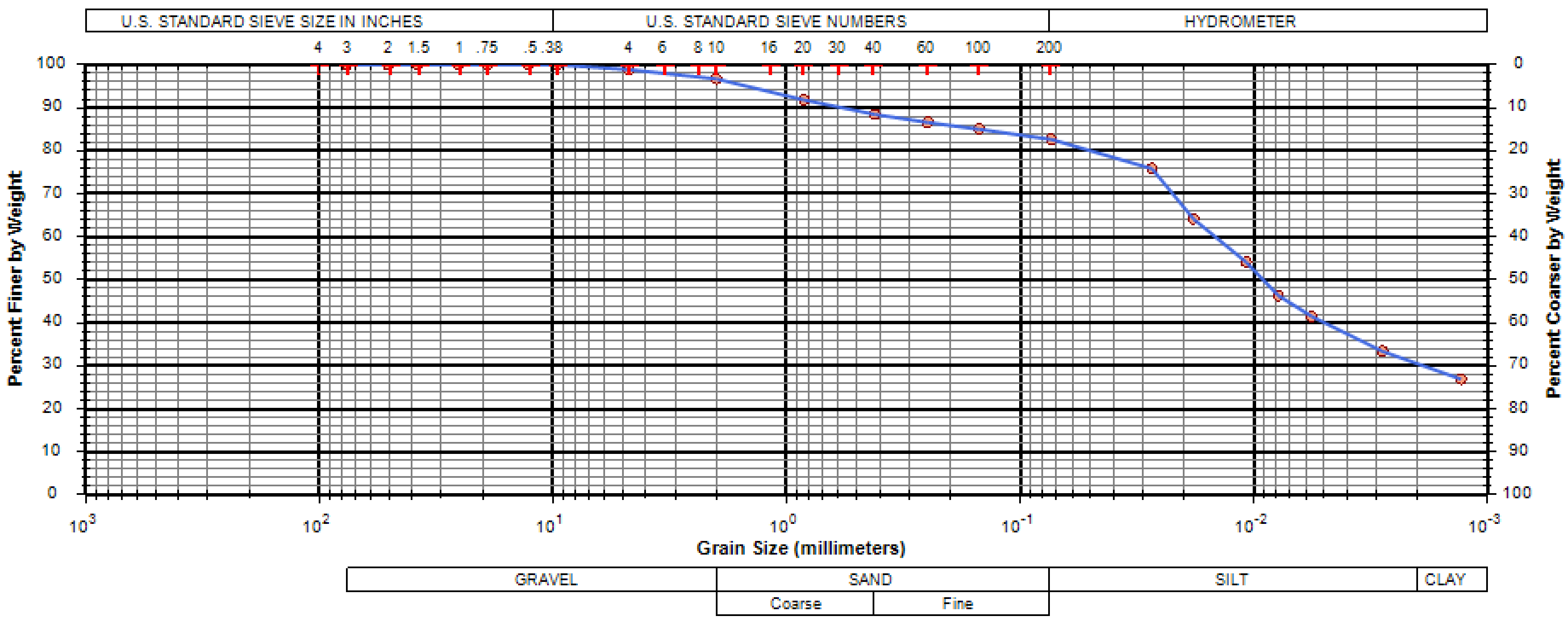
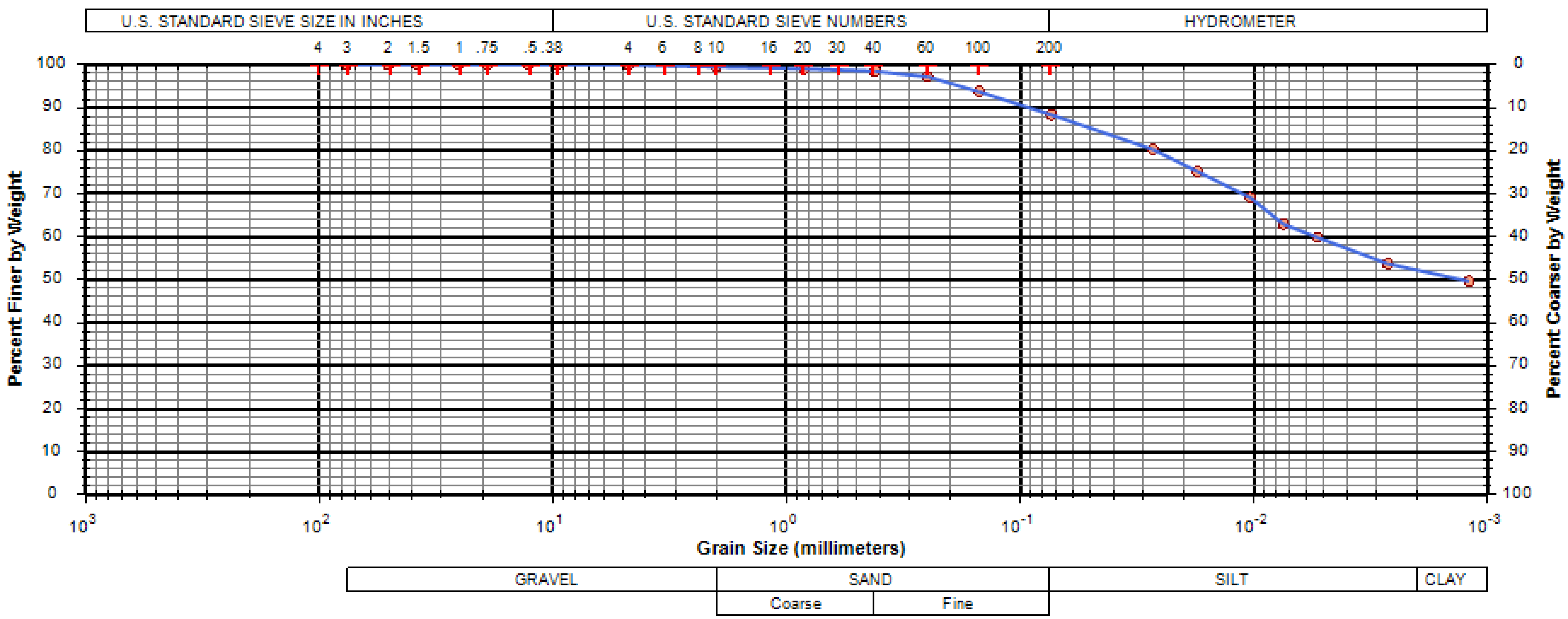
PARTICLE-SIZE ANALYSIS OF SOILS AASHTO T88

Client:	CT Consultants, Inc.							**Project No.:**	J029038.01
Project:	Geotechnical Exploration, Amsterdam Road Reconstruction, Phase 1, Fort Wright, KY							**Date:**	04/26/2017
Boring No.:	203	**Sample No.:**	2	**Depth (ft.):**	2.5 - 4.0	**Gravel (%)**	**Sand (%)**	**Silt (%)**	**Clay (%)**	**AASHTO**
Sample Description:	Brown trace gray moist very stiff LEAN CLAY, trace fat clay seams with limestone fragments, trace bedding planes (residual)					3.3	14.0	52.2	30.5	A-6
						LL	**PL**	**PI**	**Group Index**	**WC (%)**
						38	23	15	13	19.6



PARTICLE-SIZE ANALYSIS OF SOILS AASHTO T88

Client:	CT Consultants, Inc.							**Project No.:**	J029038.01
Project:	Geotechnical Exploration, Amsterdam Road Reconstruction, Phase 1, Fort Wright, KY							**Date:**	04/26/2017
Boring No.:	206	**Sample No.:**	PT-3	**Depth (ft.):**	5.3 - 5.8	**Gravel (%)**	**Sand (%)**	**Silt (%)**	**Clay (%)**	**AASHTO**
Sample Description:		Brown moist very stiff FAT CLAY with oxide stains (glacial)				0.5	11.1	36.1	52.3	A-7-6
						LL	**PL**	**PI**	**Group Index**	**WC (%)**
						59	29	30	30	31.4



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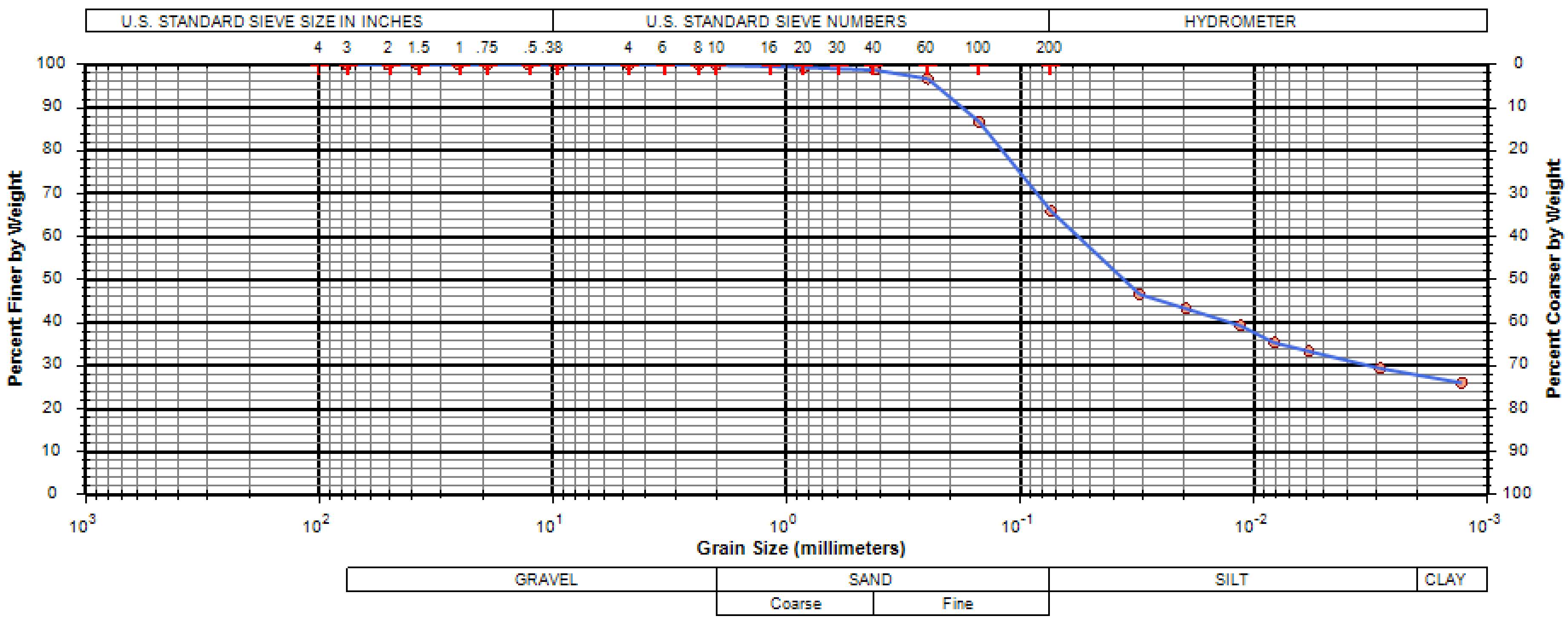
PARTICLE-SIZE ANALYSIS OF SOILS AASHTO T88

Client:	CT Consultants, Inc.							**Project No.:**	J029038.01
Project:	Geotechnical Exploration, Amsterdam Road Reconstruction, Phase 1, Fort Wright, KY							**Date:**	04/26/2017
Boring No.:	208	**Sample No.:**	PT-3	**Depth (ft.):**	5.5 - 6.0	**Gravel (%)**	**Sand (%)**	**Silt (%)**	**Clay (%)**	**AASHTO**
Sample Description:		Orangish brown moist medium stiff sandy LEAN CLAY with oxide concretions (glacial)				0.1	34.0	38.0	27.9	A-6
						LL	**PL**	**PI**	**Group Index**	**WC (%)**
						34	16	18	9	21.0



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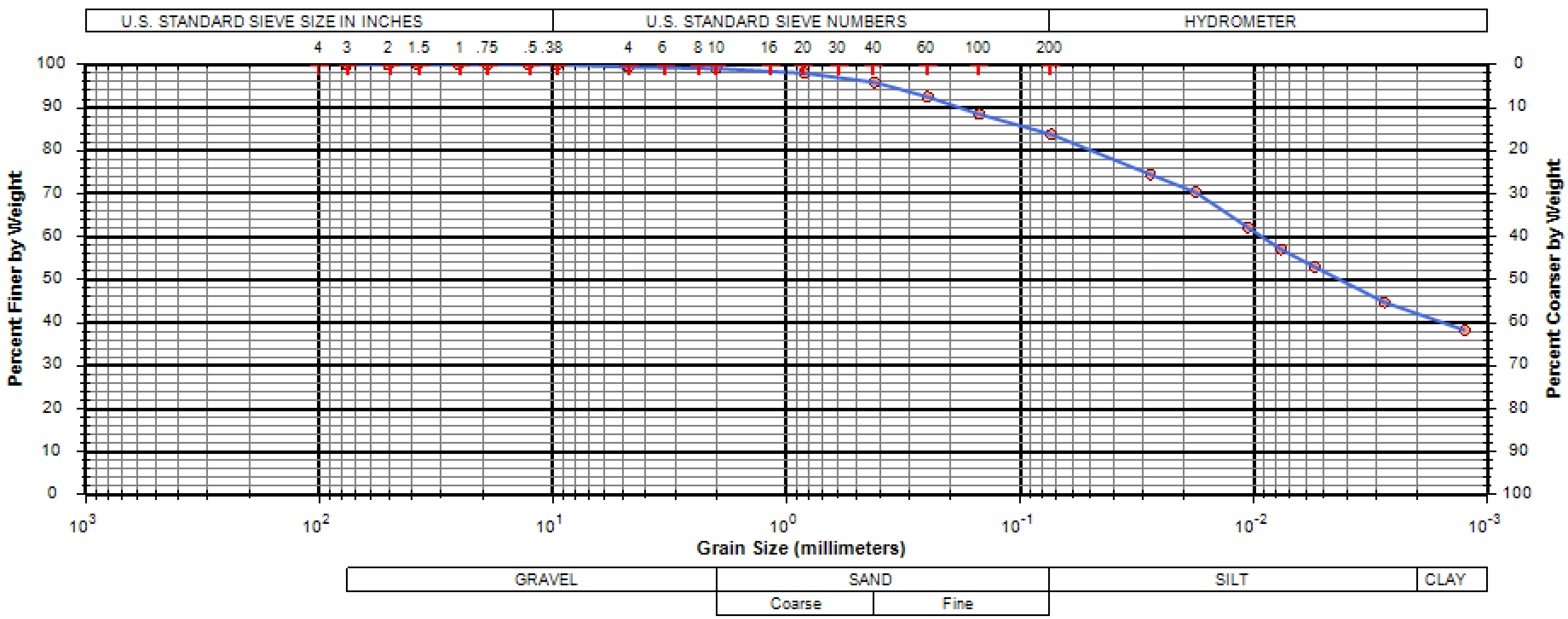
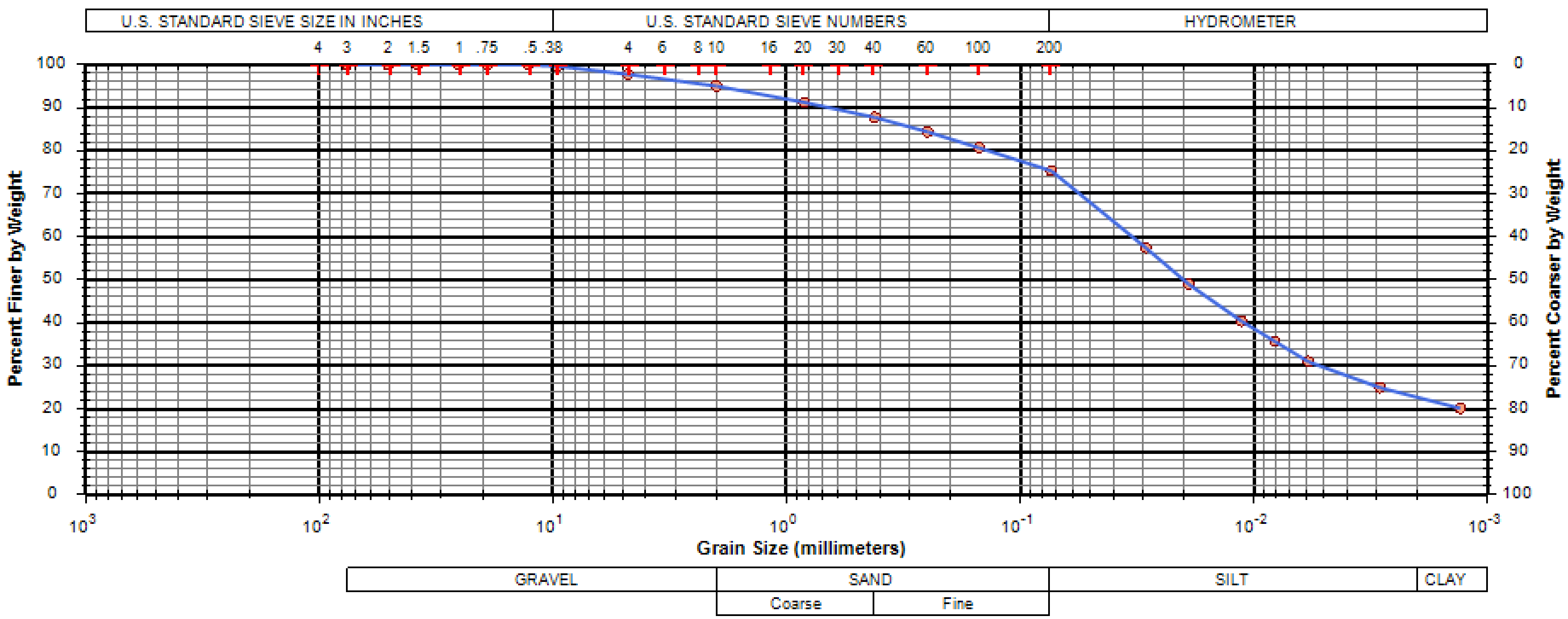
PARTICLE-SIZE ANALYSIS OF SOILS AASHTO T88

Client:	CT Consultants, Inc.							**Project No.:**	J029038.01
Project:	Geotechnical Exploration, Amsterdam Road Reconstruction, Phase 1, Fort Wright, KY							**Date:**	04/26/2017
Boring No.:	210	**Sample No.:**	PT-5B	**Depth (ft.):**	10.8 - 11.3	**Gravel (%)**	**Sand (%)**	**Silt (%)**	**Clay (%)**	**AASHTO**
Sample Description:		Brown, trace gray moist stiff FAT CLAY, little sand with oxide stains (glacial)				0.9	15.3	41.7	42.1	A-7-6
						LL	**PL**	**PI**	**Group Index**	**WC (%)**
						57	24	33	30	25.4



PARTICLE-SIZE ANALYSIS OF SOILS AASHTO T88

Client:	CT Consultants, Inc.							**Project No.:**	J029038.01
Project:	Geotechnical Exploration, Amsterdam Road Reconstruction, Phase 1, Fort Wright, KY							**Date:**	04/26/2017
Boring No.:	211	**Sample No.:**	PT-13B	**Depth (ft.):**	30.9 - 31.4	**Gravel (%)**	**Sand (%)**	**Silt (%)**	**Clay (%)**	**AASHTO**
Sample Description:	Gray moist very stiff clayey SILT, little sand, trace fine to coarse gravel with limestone fragments (glacial)					4.9	19.7	52.7	22.7	A-4
						LL	**PL**	**PI**	**Group Index**	**WC (%)**
						26	17	9	5	14.8



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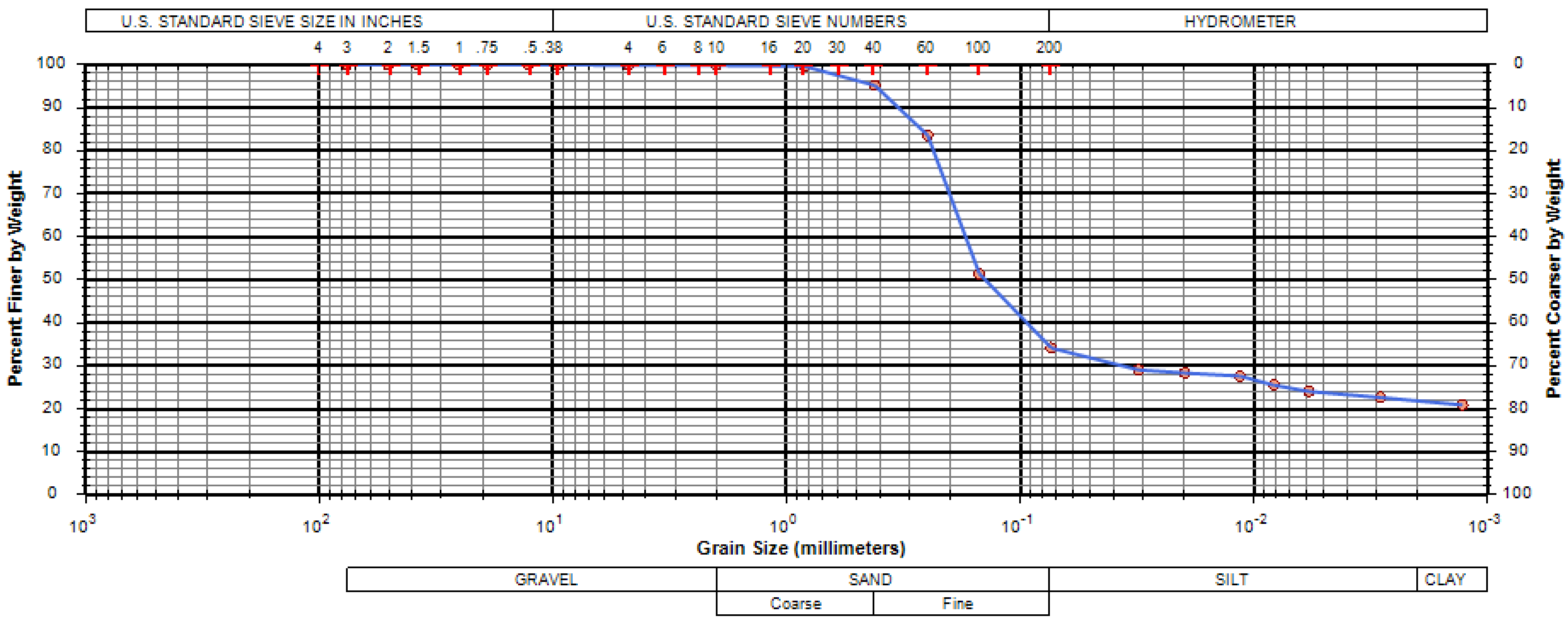
PARTICLE-SIZE ANALYSIS OF SOILS AASHTO T88

Client:	CT Consultants, Inc.							**Project No.:**	J029038.01
Project:	Geotechnical Exploration, Amsterdam Road Reconstruction, Phase 1, Fort Wright, KY							**Date:**	04/26/2017
Boring No.:	213	**Sample No.:**	PT-6B	**Depth (ft.):**	10.8 - 11.3	**Gravel (%)**	**Sand (%)**	**Silt (%)**	**Clay (%)**	**AASHTO**
Sample Description:		Orangish brown moist medium stiff clayey SAND, little silt with oxide concretions (glacial)				0.1	65.8	12.2	21.9	A-2-6
						LL	**PL**	**PI**	**Group Index**	**WC (%)**
						31	20	11	0	23.0



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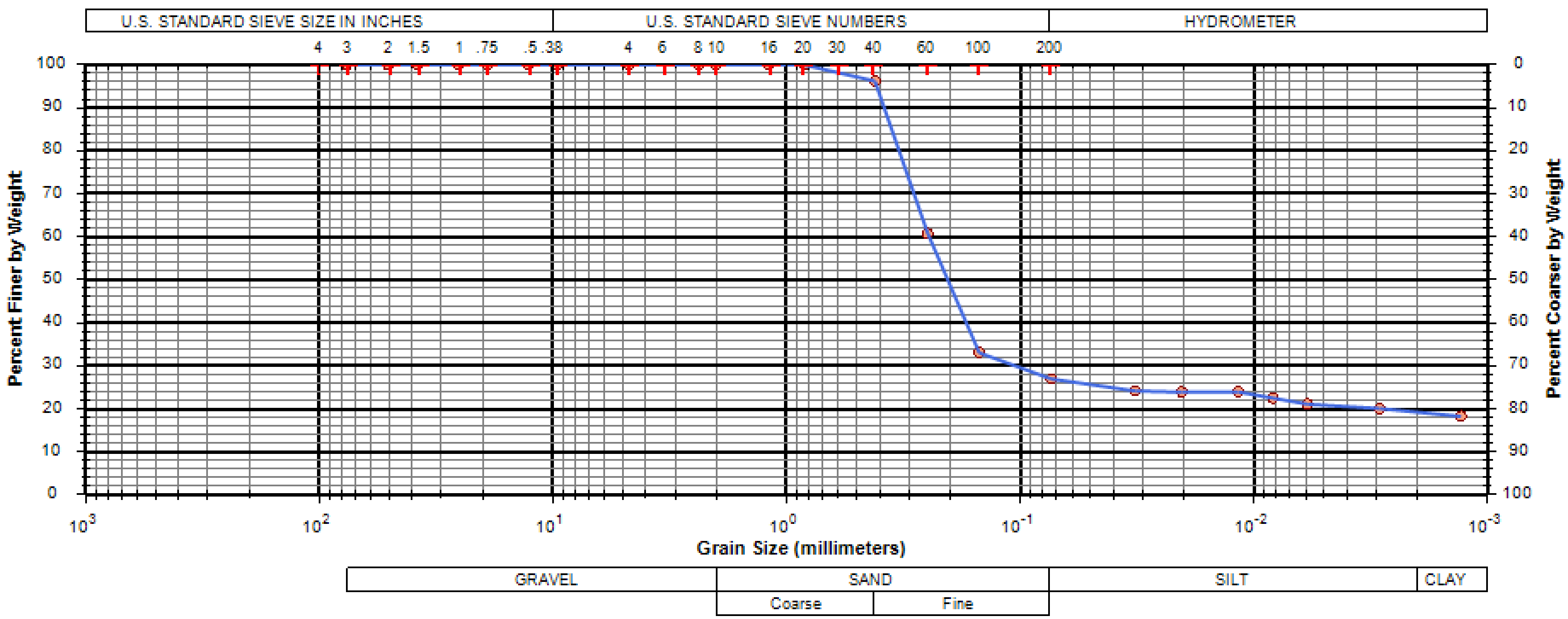
PARTICLE-SIZE ANALYSIS OF SOILS AASHTO T88

Client:	CT Consultants, Inc.							**Project No.:**	J029038.01
Project:	Geotechnical Exploration, Amsterdam Road Reconstruction, Phase 1, Fort Wright, KY							**Date:**	04/26/2017
Boring No.:	214	**Sample No.:**	PT-6	**Depth (ft.):**	8.3 - 8.8	**Gravel (%)**	**Sand (%)**	**Silt (%)**	**Clay (%)**	**AASHTO**
Sample Description:	Orangish brown moist medium dense clayey SAND, trace silt with oxide concretions (glacial)					0.0	73.0	7.8	19.2	A-2-4
						LL	**PL**	**PI**	**Group Index**	**WC (%)**
						28	22	6	0	20.6



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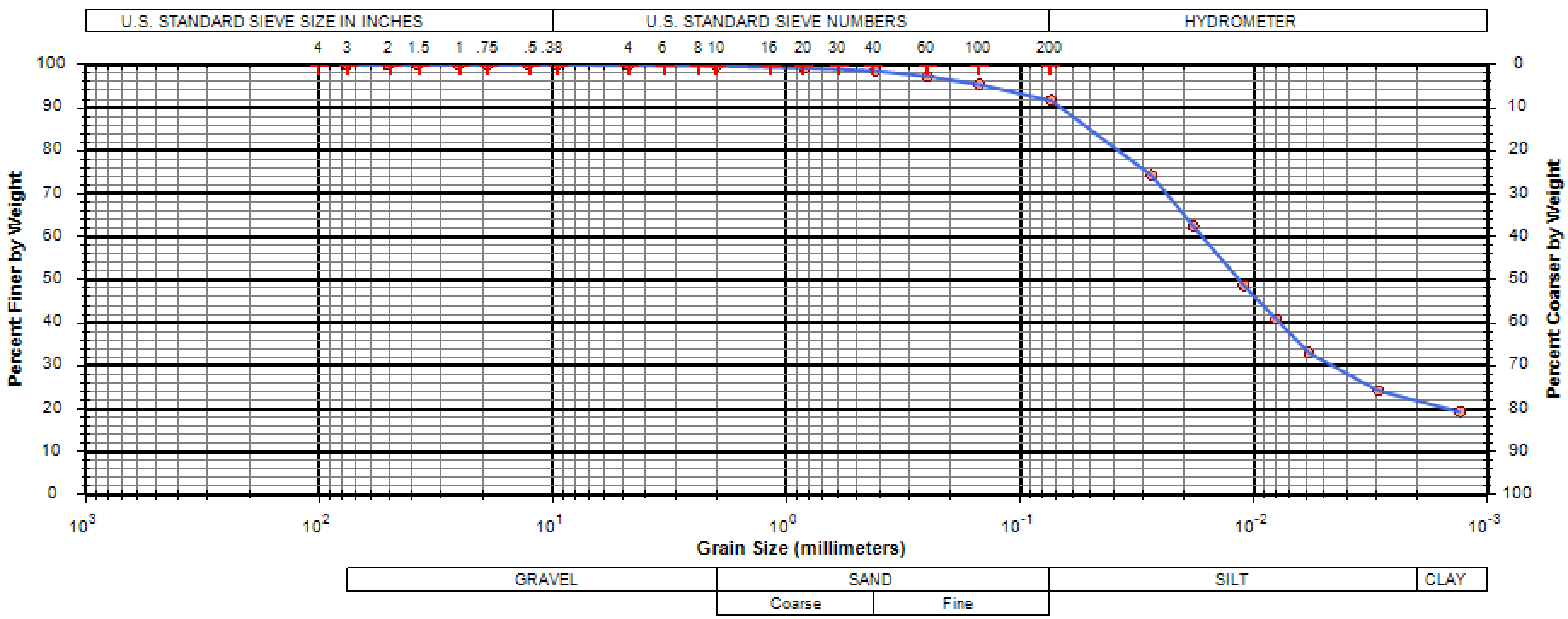
PARTICLE-SIZE ANALYSIS OF SOILS AASHTO T88

Client:	CT Consultants, Inc.							**Project No.:**	J029038.01
Project:	Geotechnical Exploration, Amsterdam Road Reconstruction, Phase 1, Fort Wright, KY							**Date:**	04/26/2017
Boring No.:	218	**Sample No.:**	PT-7	**Depth (ft.):**	15.3 - 15.8	**Gravel (%)**	**Sand (%)**	**Silt (%)**	**Clay (%)**	**AASHTO**
Sample Description:	Orangish brown wet loose clayey SILT, trace sand with oxide stains (glacial)					0.3	8.0	69.8	21.9	A-6
						LL	**PL**	**PI**	**Group Index**	**WC (%)**
						37	22	15	14	30.2



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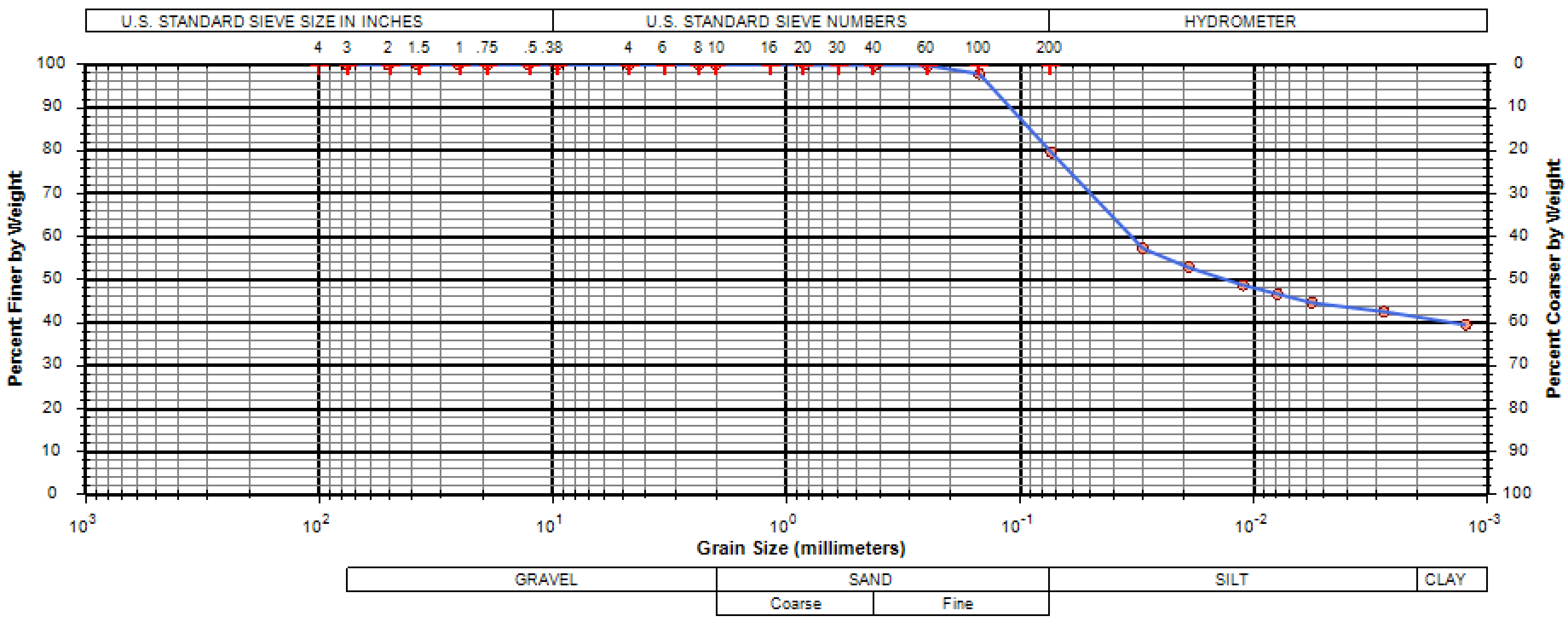
PARTICLE-SIZE ANALYSIS OF SOILS AASHTO T88

Client:	CT Consultants, Inc.							**Project No.:**	J029038.01
Project:	Geotechnical Exploration, Amsterdam Road Reconstruction, Phase 1, Fort Wright, KY							**Date:**	04/26/2017
Boring No.:	221	**Sample No.:**	3	**Depth (ft.):**	3.5 - 5.0	**Gravel (%)**	**Sand (%)**	**Silt (%)**	**Clay (%)**	**AASHTO**
Sample Description:		Orangish brown moist stiff sandy LEAN CLAY, trace oxide stains (glacial)				0.0	20.4	38.2	41.4	A-7-6
						LL	**PL**	**PI**	**Group Index**	**WC (%)**
						49	27	22	19	29.1



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UNCONFINED COMPRESSIVE STRENGTH OF COHESIVE SOILS
AASHTO T 208 (KM 64-522)

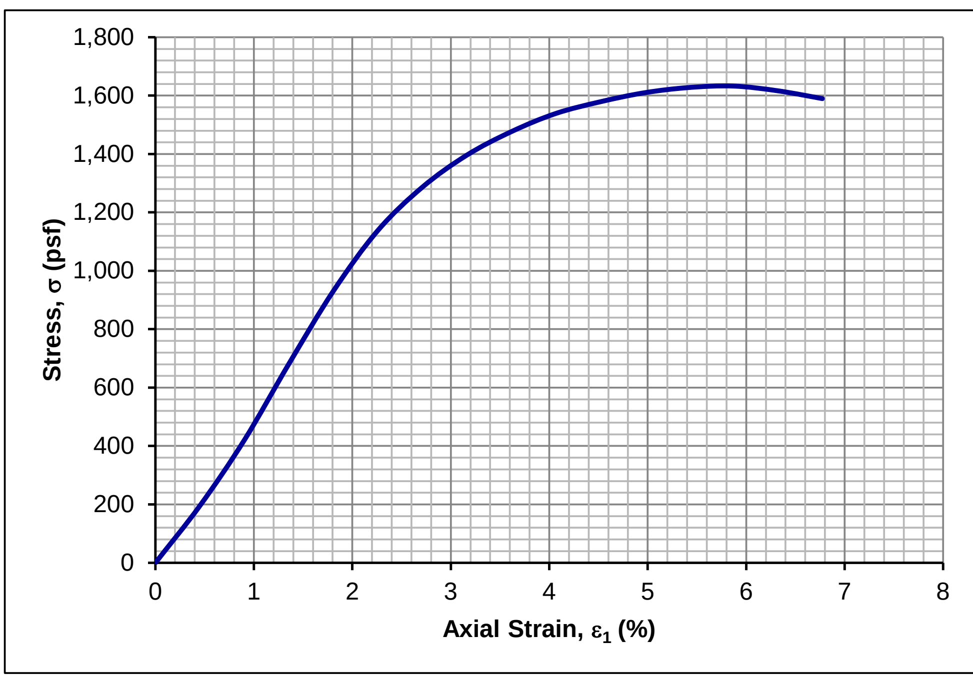
CLIENT : CT Consultants, Inc. DATE: 4/18/2017
PROJECT NO.: J029038.01
PROJECT: Amsterdam Road Reconstruction, Phase 1
LOCATION: Fort Wright, KY

BORING NO.: 208 SAMPLE NO.: PT-3 DEPTH (ft.): 5.5-6.0
SAMPLE OBTAINED BY: Shelby Tube CONDITION: Undisturbed
SAMPLE DESCRIPTION: Orangish brown moist medium stiff sandy LEAN CLAY with oxide concretions (glacial)

LIQUID LIMIT (%): 34 PLASTIC LIMIT (%): 16 PLASTICITY INDEX (%): 18 AASHTO: A-6
GRAVEL (%): 0.1 SAND (%): 34.0 SILT (%): 38.0 CLAY (%): 27.9
SPECIFIC GRAVITY OF SOLIDS: 2.75 (Assumed) LOAD CELL NO.: 1059

SAMPLE DATA	
DIAMETER (in.):	2.85
HEIGHT (in.):	5.54
HEIGHT TO DIAMETER RATIO:	1.94
WET UNIT WEIGHT (pcf):	128.1
DRY UNIT WEIGHT (pcf):	105.9
VOID RATIO:	0.62
MOISTURE CONTENT (%)*:	21.0
DEGREE OF SATURATION (%):	93

FAILURE DATA	
AVERAGE RATE OF AXIAL STRAIN TO FAILURE (%/min.):	**1.0**
AXIAL STRAIN AT FAILURE (%):	**5.9**
TIME TO FAILURE (min.):	**5.8**
UNCONFINED COMPRESSIVE STRENGTH, q_u (psf):	**1,630**
UNDRAINED SHEAR STRENGTH, s_u (psf):	**815**
SENSITIVITY, S_t:	**-**
STRAIN AT 50% OF UCS, ε_{50} (%):	**1.60**



FAILURE SHAPES

FRONT VIEW

SIDE VIEW

REMARKS :

*Moisture content determined after shear from entire sample.

GEOTECHNOLOGY INC
FROM THE GROUND UP

UNCONFINED COMPRESSIVE STRENGTH OF COHESIVE SOILS
AASHTO T 208 (KM 64-522)

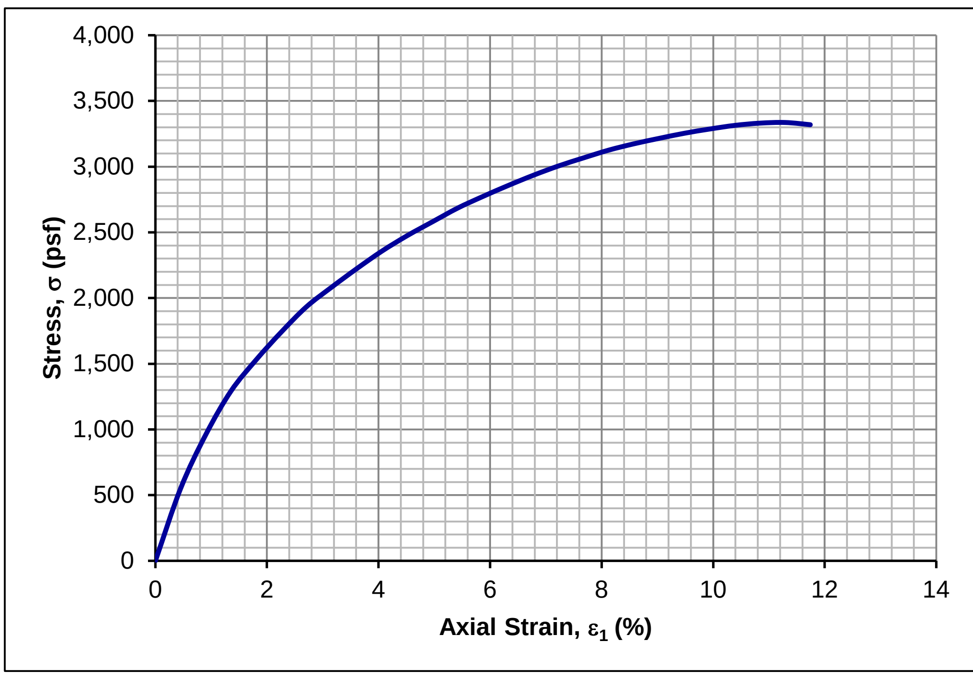
CLIENT : CT Consultants, Inc. DATE: 4/19/2017
PROJECT NO.: J029038.01
PROJECT: Amsterdam Road Reconstruction, Phase 1
LOCATION: Fort Wright, KY

BORING NO.: 210 SAMPLE NO.: PT-5A DEPTH (ft.): 10.2-10.7
SAMPLE OBTAINED BY: Shelby Tube CONDITION: Undisturbed
SAMPLE DESCRIPTION: Brown, trace gray moist stiff FAT CLAY, little sand with oxide stains (glacial)

LIQUID LIMIT (%): 57 PLASTIC LIMIT (%): 24 PLASTICITY INDEX (%): 33 AASHTO: A-7-6
GRAVEL (%): 0.9 SAND (%): 15.3 SILT (%): 41.7 CLAY (%): 42.1
SPECIFIC GRAVITY OF SOLIDS: 2.75 (Assumed) LOAD CELL NO.: 1059

SAMPLE DATA	
DIAMETER (in.):	2.86
HEIGHT (in.):	5.54
HEIGHT TO DIAMETER RATIO:	1.93
WET UNIT WEIGHT (pcf):	122.9
DRY UNIT WEIGHT (pcf):	97.9
VOID RATIO:	0.75
MOISTURE CONTENT (%)*:	25.4
DEGREE OF SATURATION (%):	93

FAILURE DATA	
AVERAGE RATE OF AXIAL STRAIN TO FAILURE (%/min.):	**1.0**
AXIAL STRAIN AT FAILURE (%):	**11.3**
TIME TO FAILURE (min.):	**11.2**
UNCONFINED COMPRESSIVE STRENGTH, q_u (psf):	**3,340**
UNDRAINED SHEAR STRENGTH, s_u (psf):	**1,670**
SENSITIVITY, S_t:	**-**
STRAIN AT 50% OF UCS, ε_{50} (%):	**2.10**



FAILURE SHAPES

FRONT VIEW

SIDE VIEW

REMARKS :

*Moisture content determined after shear from entire sample.

UNCONFINED COMPRESSIVE STRENGTH OF COHESIVE SOILS
AASHTO T 208 (KM 64-522)

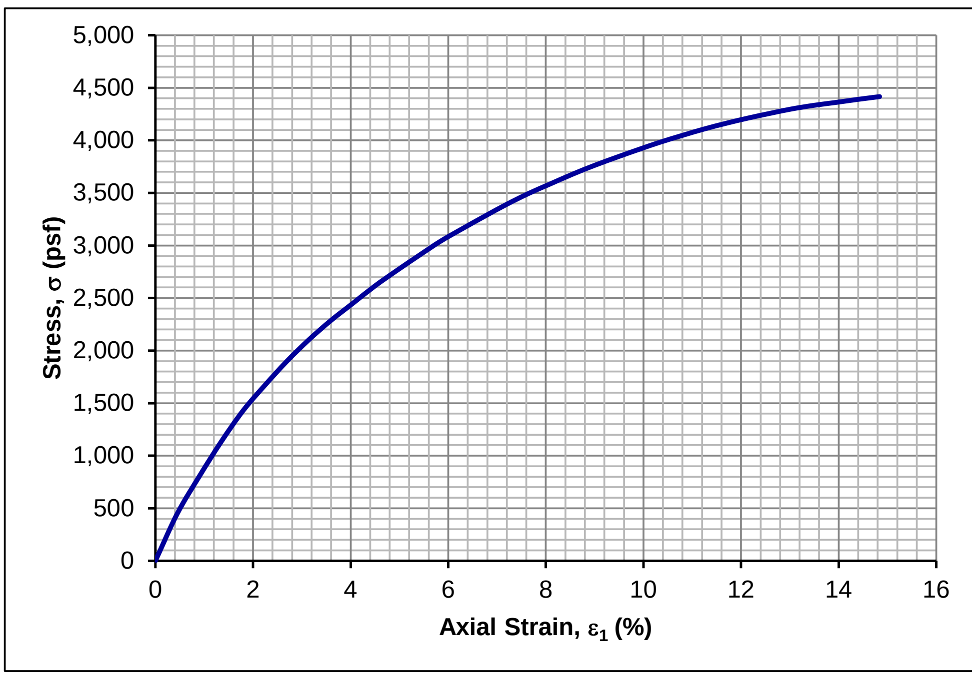
CLIENT : CT Consultants, Inc. DATE: 4/19/2017
PROJECT NO.: J029038.01
PROJECT: Amsterdam Road Reconstruction, Phase 1
LOCATION: Fort Wright, KY

BORING NO.: 211 SAMPLE NO.: PT-13A DEPTH (ft.): 30.3-30.8
SAMPLE OBTAINED BY: Shelby Tube CONDITION: Undisturbed
SAMPLE DESCRIPTION: Gray moist stiff LEAN CLAY, trace sand and gravel with shale fragments (glacial)

LIQUID LIMIT (%): 26 PLASTIC LIMIT (%): 17 PLASTICITY INDEX (%): 9 AASHTO: A-4
GRAVEL (%): 4.9 SAND (%): 19.7 SILT (%): 52.7 CLAY (%): 22.7
SPECIFIC GRAVITY OF SOLIDS: 2.75 (Assumed) LOAD CELL NO.: 1059

SAMPLE DATA	
DIAMETER (in.):	2.86
HEIGHT (in.):	5.56
HEIGHT TO DIAMETER RATIO:	1.94
WET UNIT WEIGHT (pcf):	139.7
DRY UNIT WEIGHT (pcf):	121.7
VOID RATIO:	0.41
MOISTURE CONTENT (%)*:	14.8
DEGREE OF SATURATION (%):	99

FAILURE DATA	
AVERAGE RATE OF AXIAL STRAIN TO FAILURE (%/min.):	**1.0**
AXIAL STRAIN AT FAILURE (%):	**14.8**
TIME TO FAILURE (min.):	**14.7**
UNCONFINED COMPRESSIVE STRENGTH, q_u (psf):	**4,420**
UNDRAINED SHEAR STRENGTH, s_u (psf):	**2,210**
SENSITIVITY, S_t:	**-**
STRAIN AT 50% OF UCS, ε_{50} (%):	**3.40**



FAILURE SHAPES

FRONT VIEW

SIDE VIEW

REMARKS :

*Moisture content determined after shear from entire sample.

UNCONFINED COMPRESSIVE STRENGTH OF COHESIVE SOILS
AASHTO T 208 (KM 64-522)

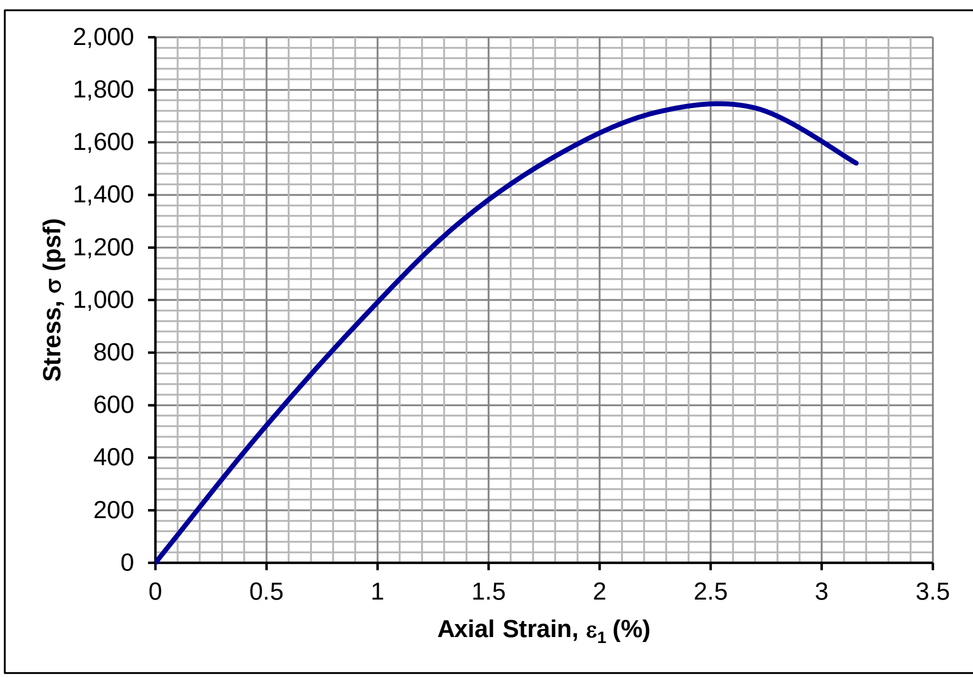
CLIENT : CT Consultants, Inc. DATE: 4/19/2017
PROJECT NO.: J029038.01
PROJECT: Amsterdam Road Reconstruction, Phase 1
LOCATION: Fort Wright, KY

BORING NO.: 213 SAMPLE NO.: PT-6A DEPTH (ft.): 10.2-10.7
SAMPLE OBTAINED BY: Shelby Tube CONDITION: Undisturbed
SAMPLE DESCRIPTION: Orangish brown moist medium stiff clayey SAND, trace oxide concretions (glacial)

LIQUID LIMIT (%): 31 PLASTIC LIMIT (%): 20 PLASTICITY INDEX (%): 11 AASHTO: A-2-6
GRAVEL (%): 0.1 SAND (%): 65.8 SILT (%): 12.2 CLAY (%): 21.9
SPECIFIC GRAVITY OF SOLIDS: 2.75 (Assumed) LOAD CELL NO.: 1059

SAMPLE DATA	
DIAMETER (in.):	2.87
HEIGHT (in.):	5.55
HEIGHT TO DIAMETER RATIO:	1.94
WET UNIT WEIGHT (pcf):	116.8
DRY UNIT WEIGHT (pcf):	95.0
VOID RATIO:	0.81
MOISTURE CONTENT (%)*:	23.0
DEGREE OF SATURATION (%):	78

FAILURE DATA	
AVERAGE RATE OF AXIAL STRAIN TO FAILURE (%/min.):	**1.0**
AXIAL STRAIN AT FAILURE (%):	**2.7**
TIME TO FAILURE (min.):	**2.7**
UNCONFINED COMPRESSIVE STRENGTH, q_u (psf):	**1,730**
UNDRAINED SHEAR STRENGTH, s_u (psf):	**865**
SENSITIVITY, S_t:	**-**
STRAIN AT 50% OF UCS, ε_{50} (%):	**0.86**



FAILURE SHAPES


FRONT VIEW


SIDE VIEW

REMARKS :

*Moisture content determined after shear from entire sample.