ESP Associates, P.A.

June 3, 2009

Speedway Motorsports
6425 Idlewild Road
Building 3, Suite 205
Charlotte, North Carolina 28212

Attention: Mr. Robert Davis

Reference: **REPORT OF ADDITIONAL SUBSURFACE EXPLORATION – REVISION #1**
Morehead Road Overpass of Tram Road
Concord, North Carolina
ESP Project No. E4B-UH26.350

Dear Mr. Davis:

ESP Associates, P.A. (ESP) has completed the subsurface exploration for the proposed cored slab bridge located in the proposed Morehead Road Overpass in Concord, North Carolina. This exploration was performed in general accordance with our Proposal No. E4B-0644, dated July 31, 2006. Authorization to proceed with this study was provided by execution via an e-mail dated May 12, 2008.

The purpose of the exploration was to evaluate the general subsurface conditions within the proposed cored slab bridge area with regard to the design and construction of the foundation systems and bridge support. This report presents our findings, conclusions and recommendations for foundation design, as well as construction considerations for the proposed retaining wall areas. This report was revised to incorporate AASHTO LRFD bridge design specification factored resistance bearing capacities.

ESP appreciates the opportunity to assist you during this phase of the project. If you should have any questions concerning this report, or if we may be of further assistance, please contact us.

Sincerely,

ESP Associates, P.A.

Jason K. Tapia, EI
Project Manager

David A. Bixler, II, PE
NC Registration No. 33560

NORTH CAROLINA PROFESSIONAL SEAL 033560 ENGINEER DAVID A. BIXLER II

6/3/09

JKT/DAB /kc

Copies Submitted: (1)

cc: Mr. Kenneth Smith – Santec Consulting
Mr. Joe Kelvington – Santec Consulting

NORTH CAROLINA LICENSE NO. C-0587 ESP ASSOCIATES, P.A.

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1.0 INTRODUCTION

1.1 SITE AND PROJECT DESCRIPTION

The subject site is located to the south of Lowes Motor Speedway and to the southwest of BFI Landfill located off the existing Morehead Road in Concord, North Carolina, reference "Site Vicinity Map," Figure 1. The site is bordered to the west by an existing parking/campground used by Lowes Motor Speedway, to the east by Sam Bass Art Gallery, and to the north by the existing Morehead Road. At the time of our exploration, the site was moderately wooded and within close proximity of the stream. We understand that plans are to construct a cored slab bridge in the vicinity of the above mentioned stream with associated headwalls/wing walls. ESP also understands that the cored slab bridge will be supported on vertical face abutments, which will be founded on spread footings and that this is not a stream crossing, but rather an overpass of a paved access road with sidewalks. ESP did not perform a scour analysis for the bridge. ESP performed a previous subsurface exploration in the vicinity of the project as part of the Proposed Morehead Road Alignment. For further details, please refer to our report titled "Report of Subsurface Exploration," dated January 8, 2007. No other information is known at this time.

1.2 PURPOSE OF SERVICES

The purpose of the exploration was to evaluate the general subsurface conditions at the proposed cored slab bridge with regard to the design and construction of the foundation systems and bridge support. This report presents our findings, conclusions and recommendations for foundation design, as well as construction considerations for the proposed bridge area. This report also contains a brief description of the field testing and laboratory procedures performed for this study and a discussion of the soil conditions encountered at the bridge.

2.0 EXPLORATION PROCEDURES

2.1 FIELD

Four (4) soil test borings (Borings MH-1 through MH-4) were performed at the approximate locations shown on the attached "Boring Location Plan," Figure 1. Boring locations were chosen by Santec Consulting and were surveyed in the field by ESP. Hand clearing was required to gain access to the boring locations. Borings were offset from the staked locations in the field as required due to access issues and were re-surveyed by ESP. The soil test borings were extended to or resulted in auger refusal at depths ranging between approximately 18.5 and 24 feet below the existing ground surface using a CME 550X drill rig mounted on an ATV carrier. Hollow-stem, continuous flight augers were used to advance the borings into the ground.

Standard Penetration Tests were performed at designated intervals in the soil test borings in general accordance with ASTM D 1586 in order to obtain data for estimating soil strength and consistency. In conjunction with the penetration testing, split-spoon soil samples were recovered for soil classification and laboratory testing. Water level measurements were attempted at, and up to 1 day after, the termination of drilling. A brief description of the field testing procedures is included in the Appendix.

While in the field, a representative of the geotechnical engineer visually examined each sample to evaluate the type of soil encountered, soil plasticity, moisture condition, organic content, presence of lenses and seams, colors and apparent geological origin. The results of the visual soil classifications for the borings, as well as field test results, are presented on the individual "Test Boring Records," included in the Appendix. Similar soils were grouped into strata on the logs. The strata lines represent approximate boundaries between the soil types; however, the actual transition between soil types in the field may be gradual in both the horizontal and vertical directions.

2.2 LABORATORY

Select samples of the on-site soils obtained during the field testing program were tested in the laboratory. Tests performed included Atterberg limits and grain size distribution. The limited testing program was designed to determine selected engineering properties of the on-site soils relative to their use for the project. The results of the soil tests performed for this study, along with a brief description of the laboratory procedures used, are presented in the Appendix. A brief summary of results is provided below in Table I.

TABLE I – SUMMARY OF LABORATORY TESTING

ESP Sample No..	Sample Description & Location	Liquid Limit / Plasticity Index	Percent Passing No. 200 Sieve
S-1	Light Brown Gray Silty SAND (SM) MH-1 (1 to 5 feet)	27/NP	16.9
S-2	Tan Light Brown Silty SAND (SM) MH-3 (3.5 to 5) and MH-4 (1 to 2.5 feet)	25/NP	19.8

3.0 SUBSURFACE CONDITIONS

3.1 PHYSIOGRAPHY AND AREA GEOLOGY

The referenced property is located in Concord, North Carolina which is in the Piedmont Physiographic Province. The Piedmont Province generally consists of hills and ridges which are intertwined with an established system of draws and streams. The Piedmont Province is predominately underlain by igneous rock (formed from molten material) and metamorphic rock (formed by heat, pressure and/or chemical action), which were initially formed during the Precambrian and Paleozoic eras.

The virgin soils encountered in this area are the residual product of in-place chemical weathering of rock which was similar to the rock presently underlying the site. In areas not altered by erosion or disturbed by the activities of man, the typical residual soil profile consists of clayey soils near the surface, where soil weathering is more advanced, underlain by sandy silts and silty sands. The boundary between soil and rock is not sharply defined. This transitional zone termed "partially weathered rock" is normally found overlying the parent bedrock. Partially weathered rock is defined, for engineering purposes, as residual material with Standard Penetration Resistances in excess of 100 blows per foot. Weathering is facilitated by fractures, joints and by the presence of less resistant rock types. Consequently, the profile of the partially weathered rock and hard rock is quite irregular and erratic, even over short horizontal distances. Also, it is common to find lenses and boulders of hard rock and zones of partially weathered rock within the soil mantle, well above the general bedrock level.

3.2 SUBSURFACE CONDITIONS

Subsurface conditions as indicated by the borings generally consist of topsoil underlain by residual soils and partially weathered rock. The generalized subsurface conditions at each of the boring locations are described below. For more detailed soil descriptions and stratifications at a particular boring location, the respective "Test Boring Record" should be reviewed. The "Test Boring Records" are included in the Appendix. The ground surface elevations indicated on the "Test Boring Records" were surveyed by ESP.

Surface: A topsoil/organic layer approximately 2 to 4 inches thick was present at each of the boring locations.

Residuum: Underlying the topsoil in Borings MH-1, MH-2, and MH-4 and as a lens within the Partially Weathered Rock (PWR) in Boring MH-3, residual soils were encountered. The residuum consists of loose to very dense micaceous silty sand. N-values in the residuum varied from 7 to 76 blows per feet (bpf), with greater than 19 bpf.

Partially Weathered Rock: Underlying the residuum in Borings MH-1, MH-2, and MH-4, and at the surface in MH-3, PWR was encountered. PWR is defined as residual soils exhibiting N-values in excess of 100 bpf. When sampled, the PWR breaks down into micaceous silty sand. Borings MH-3 and MH-4 were terminated in the PWR at approximately 24 feet below the existing ground surface. Borings MH-1 and MH-2 were terminated in the PWR upon **auger**

refusal at depths ranging from approximately 18.5 to 20 feet. Auger refusal is defined as material that could not be penetrated with the drill rig equipment used on the project. Auger refusal material may consist of large boulders, rock ledges, lenses, seams or the top of parent bedrock. Core drilling techniques would be required to evaluate the character and continuity of the refusal material. Core drilling was not included in this scope of work.

3.3 SUBSURFACE WATER

Groundwater was encountered during drilling in Borings B-2 at a depth of approximately 15.2 feet below existing grade. All remaining borings were dry at the termination of drilling. Subsequent water levels were measured in each of the borings at 1 day at depths ranging between approximately 7.0 and 15.0 feet beneath the ground surface. Hole cave-in depths ranged between approximately 16.2 and 22.7 feet below the existing ground surface. Hole cave-in depths may provide an indication of water present.

Subsurface water levels tend to fluctuate with seasonal and climatic variations, as well as with some types of construction operations. Therefore, water may be encountered during construction at depths not indicated during this study.

4.0 CONCLUSIONS AND RECOMMENDATIONS

4.1 GENERAL

Our conclusions and recommendations are based on the project information previously discussed and on the data obtained from the field testing program. If the structural loading, geometry, or proposed cored slab bridge locations are changed or significantly differ from that discussed, or if conditions are encountered during construction that differ from those encountered by the borings, ESP requests the opportunity to review our recommendations based on the new information and make any necessary changes. Additional evaluations may be appropriate once final design plans are available.

4.2 SITE DEVELOPMENT

The results of the field testing program and analyses indicate the property appears to be suitable for constructing the proposed cored slab bridge, provided the following measures are considered.

A) Lower consistency residual soils were encountered in boring MH-1. Based on the results of our borings, these soils extend to an approximate depth of 3 feet below existing grade. **The loose soils are not suitable for the proposed cored slab bridge or the associated wing walls support.** ESP recommends that these materials be removed from the culvert alignments. Detailed recommendations for removal are provided in subsequent sections of this report.

B) Each of the test borings encountered PWR at depths ranging from approximately 3.5 to 6 feet below existing ground surface. In addition, two of the test borings (MH-1 and MH-2) encountered **auger refusal** within the residual soils at depths ranging from approximately 18.5 to 20 feet below existing ground surface. Therefore, difficult excavation should be anticipated in areas where excavations extend into PWR and/or refusal materials. Difficult excavation is discussed in more detail in subsequent sections.

C) Water was observed in each of the borings at depths ranging from 7 to 15 feet beneath the ground surface after approximately one day. Due to construction taking place within the existing streambed, temporary dewatering will be required. Temporary dewatering likely will need to consist of a combination of stream diversion and strategically placed sump pump excavations. Detailed recommendations for temporary dewatering are provided in subsequent sections of this report.

4.3 LOOSE CONSISTENCY SOILS

Loose consistency soils (N-values less than 8 bpf) were encountered in boring MH-1 at existing ground surface to approximately 3 feet below the existing ground surface. If these materials area encountered within the upper 5 feet of final design grades, ESP anticipates undercutting and/or reworking of the soft material will be required in order to obtain a stable subgrade for the wing walls/retaining walls. ESP anticipates that the cored slab bridge and associated wing walls are to bear on PWR and that all lower consistency soils beneath structural areas are to be undercut. A representative of the geotechnical engineer should evaluate the extent of material to be reworked and/or undercut while monitoring construction activities.

4.4 PARTIALLY WEATHERED ROCK

PWR was encountered beneath the residuum in borings MH-1, MH-2, MH-3 and MH-4 at depths ranging from existing ground surface to approximately 3.5 to 13.5 feet below the existing ground surface. In addition, a lens of partially weathered rock was encountered within the residual soils in boring MH-4 at a depth of approximately 3.5 feet below existing ground surface.

The presence of rock and/or weathered rock should be considered when establishing design grades. The depth to, and thickness of, weathered rock and rock lenses and seams can vary dramatically in short distances and between soil testing boring locations; therefore, weathered rock or bedrock may be encountered during locations and depths between soil test borings not examined during this exploration.

4.5 FOUNDATION SUPPORT

As previously mentioned, undercutting of loose and soft soils will be required during foundation construction. The following table is a summary of the borings and depths to PWR. PWR is defined as residual soils exhibiting N-Value design in excess of 100 blows per feet. We are providing the following table of available factored resistance bearing capacities, which assumes shallow foundations for the proposed cored slab bridge and associated wing walls bearing on PWR.

Table II – Factored Resistance Bearing Capacity

Boring ID	Boring Location (Northing, Easting)	Approximate Elevation of Top of PWR (feet)	Approximate Elevation of End of Exploration (feet)	Elevation of Groundwater at 1 Day After Completion of Boring (feet)	Factored Resistance Bearing Capacity (psf)
MH-1	584842.59 1498265.26	655.8	641.8	654.8	10,000
MH-2	584827.03 1498258.49	658.1	645.6	649.1	10,000
MH-3	584761.69 1498243.60	659.5	641.6	650.5	10,000
MH-4	584746.79 1498240.11	651.7	641.2	657.0	10,000

We recommend that the subgrade soils be observed by a representative of ESP prior to foundation installation. This is to assess the subgrade soils suitability for foundation support and confirm their consistency with the conditions upon which our recommendations are based. We recommend that foundations maintain a minimum dimension of 24 inches to reduce the possibility of a localized, punching-type shear failure.

4.6 CUT AND FILL SLOPES

For landscaping and mowing concerns, final project slopes should be designed to be 3 horizontal to 1 vertical or flatter. Slopes can be designed as steep as 2 horizontal to 1 vertical; however, soil erosion, slope sloughing and slope maintenance should be expected. If designing slopes steeper than 3 horizontal to 1 vertical, a slope stability analysis should be performed to verify stability of the slope. The tops and bases of all slopes should be located a minimum of 5 feet from structural and pavement limits. The fill slopes should be adequately compacted as outlined below, and all slopes should be seeded and maintained after construction.

4.7 WALL PARAMETERS

The soil unit weight and lateral earth pressure values needed to design the walls will depend on the type of material used as backfill behind the walls. The values for these parameters vary for different material/soil types, meaning that clays exhibit different unit weights and exert different lateral pressures than do sands. As such, it would be ideal to know or be provided with the proposed backfill materials prior to the wall designer designing the wall. Given the size of the site and the magnitude of development planned, it is anticipated that the contractor will prefer to use on-site soils as backfill materials behind the walls.

This exploration has confirmed that soils at this site primarily consist of silty sand. Granular soils are most desirable for use as fill behind walls since they typically exhibit higher long-term shear strengths, and are more freely draining than lower permeability silts or clays. Proper drainage must be provided behind the walls to prevent build-up of excessive hydrostatic pressures, or the walls must be designed to resist these additional stresses. Also, walls with clay or silt as the backfill should be designed more conservatively because these soils exhibit lower shear strength characteristics.

If on-site soils are used as backfill, we recommend that the generalized parameters provided below in Table III for "Silty Soil" under General Backfill Soil Type be used for preliminary design of the wall since these values are more conservative. Once a definite source for the wall backfill is identified, the prevailing material type should be determined by the geotechnical engineer. This will require a site visit by the geotechnical engineer and additional laboratory testing. If it is determined that the prevailing material type is granular in nature, and the project schedule and budget allows, the wall design can be modified consistent with such backfill materials. In any event, laboratory testing of proposed backfill/borrow soils, once determined, should be conducted and include natural moisture, material index (Atterberg Limits and grain size analysis), triaxial shear, and unit weight testing. This additional laboratory testing should also be planned for in the construction schedule and project budget.

Table III - Recommended Preliminary Design Parameters for Header/Retaining Walls

General Backfill Soil Type	Recommended Unit Weights (pcf)		Recommended Lateral Earth Pressure Coefficients		
	Moist	Saturated	At Rest, Ko	Active, Ka	Passive, Kp
Sandy Soil[1]	115	130	0.47	0.31	3.25
Clayey Soil[2]	110	120	0.53	0.36	2.77
Silty Soil[3]	105	115	0.56	0.39	2.56

1. Based on an effective friction angle, $\phi' = 32°$
2. Based on an effective friction angle, $\phi' = 28°$
3. Based on an effective friction angle, $\phi' = 26°$

Regardless of the backfill material type, we recommend that a drainage medium consisting of clean sand (ASTM C33) or washed stone (No. 57 or 67) totally encapsulated in a filter fabric be placed along the backside of the walls. Weep holes and/or drain piping must be placed at the base of the drainage medium to allow for proper drainage.

We recommend the backfill placed within five feet laterally behind the walls be compacted with hand operated compaction equipment. Large compaction equipment should not be used within this distance since it may overstress the walls. The wall backfill should be compacted to at least 95 percent of the Standard Proctor maximum dry density, and the moisture content should be maintained within three percent of optimum. Use of hand operated compaction equipment will require placement of thinner fill lifts in order to achieve the proper compaction.

It should be noted the earth pressure coefficients in Table III assume a relatively flat backfill surface (less than 1/2 percent) behind the walls. If a sloping backfill surface is to be used, revisions to these earth pressure coefficients will be required.

Retaining wall analyses were not part of our scope of services for this exploration. Any walls designed must be properly analyzed by the wall design engineer with respect to global stability analysis and other design parameters. In addition, the analyses of wall stability should also include any measurable loading that the walls may be subject to, such as from vehicular traffic and adjacent structures. Lastly, any retaining wall should be designed using appropriate soil parameters based on the site and construction conditions, which is the basis for our above recommendation of additional laboratory testing once definitive backfill/borrow soils are determined.

5.0 CONSTRUCTION CONSIDERATIONS

5.1 TEMPORARY DEWATERING

Based on existing ground surface elevations, proximity to drainage features and stabilized groundwater levels, we anticipate that **temporary dewatering will be required at the bridge area during construction.** The first step in the dewatering process should be diversion of the existing streams from the proposed bridge alignments. Stream diversion typically can be achieved by piping the stream around the construction area or excavating a temporary streambed around the construction area that is a minimum of 3 feet below the proposed excavation depth. Additionally, pumping from sumps excavated at least 3 feet below the bottom of the excavations may be required. Pumping from the sumps should be maintained until concrete placement in the foundations is complete. At no time should pumping be performed directly beneath the exposed foundation subgrade elevation since this could result in disturbance of the bearing materials and a loss of soil strength and increased settlement.

5.2 SITE PREPARATION

The entire buildings and pavement areas should be stripped of all topsoil, high plasticity near surface soils, trash, debris and other organic materials to a minimum of 10 feet beyond the structural and pavement limits. Upon completion of the stripping operations, the exposed subgrade in areas to receive fill should be proofrolled with a loaded dump truck or similar pneumatic tired vehicle (minimum loaded weight of 20 tons) under the observation of a representative of the geotechnical engineer. The proofrolling procedures should consist of four complete passes of the exposed areas, with two of the passes being in a direction perpendicular to the preceding ones. After excavation of the site has been completed, the exposed subgrade in cut areas should also be proofrolled as previously described. Any areas which deflect, rut or pump excessively during proofrolling or fail to improve sufficiently after successive passes should be undercut to suitable soils and replaced with structural fill.

Some undercutting of the soft near surface soils in the portions of the site should be anticipated. The extent of the undercut required should be evaluated in the field by an experienced representative of the geotechnical engineer while monitoring construction activity. The evaluation should consist of a comprehensive proofrolling program and thorough field evaluation during construction. After the proofrolling operation has been completed and approved, final site grading should proceed immediately. If construction progresses during wet weather, the proofrolling operation should be repeated with at least one pass in each direction immediately prior to placing base course in the parking areas. If unstable conditions are exposed during this operation, then undercutting should be performed

5.3 UNDERCUTTING FOUNDATIONS

Based on the depth to competent PWR, undercutting of loose and soft soils will likely be required for the retaining wall foundations. As previously discussed, it may be feasible to lower the retaining wall foundation bearing elevations to the undercut depth needed to obtain the desired bearing capacity. If this is not feasible, the excavation should be backfilled with washed stone wrapped in a non-woven geotextile filter fabric such as Mirafi 140N (or equivalent). If undercutting and replacement with washed stone is selected, the excavation should be widened one foot for every vertical foot of undercut. Some caving of the excavation walls should be expected. Proper shoring and/or bracing should be utilized to reduce caving concerns. As an alternative, undercut excavations could be backfilled with a lean concrete (i.e. minimum 28-day strength of 2,000 psi) to the design foundation elevation.

5.4 TEMPORARY EXCAVATION STABILITY

Excavations greater than four feet in depth should be sloped or shored in accordance with local, state, and federal regulations, including OSHA "Construction Standard for Excavations" (29 CFR Part 1926.650-652). The contractor usually is solely responsible for site safety. This information is provided only as a service and under no circumstances should ESP be assumed to be responsible for construction site safety.

5.5 DIFFICULT EXCAVATION

Based on the results of the soil test borings, it appears that the majority of the general excavation will be in loose to dense residual soils. We anticipate that these soils can be excavated using pans, scrapers, backhoes and front end loaders. Borings MH-1 and MH-2 were terminated upon **auger refusal** at depths ranging from approximately 18.5 to 20 feet below the existing ground surface. Based on the foundation elevations provided by Santec Consulting and the results of our borings, we anticipate that partially weathered rock, intermittent rock lenses and/or boulders will be encountered during general site grading and excavation for the installation of bridge footings and utilities. It should be noted that ripping of rock is dependent upon finding the right combination of equipment and techniques used, as well as the operator's skill and experience. The success of the ripping operation is dependent on finding the proper combinations for the conditions encountered. Excavation of the weathered rock typically is much more difficult in confined excavations such as the culvert footings. Jack-hammering or blasting is anticipated to be required for materials having N-values in excess of 50 blows per 0.2 foot.

The depth to, and thickness of, PWR and rock lenses or seams, can vary dramatically in short distances and between boring locations; therefore, PWR or bedrock may be encountered during construction at locations or depths between boring locations, not encountered during this exploration.

5.6 FILL MATERIAL AND PLACEMENT

All fill used for site grading operations should consist of a clean (free of organics and debris), lower plasticity soil (Plasticity Index less than 30). The proposed fill should have a maximum dry density of at least 90 pounds per cubic foot as determined by a Standard Proctor compaction test (ASTM D 698). **Fill planned for use in the foundation, reinforced and retained zones around the proposed walls should comply with the specification of the design plans regardless of whether a modular wall system or cast-in-place concrete is used.** All fill should be placed in loose lifts not exceeding 8 inches in thickness and compacted to a minimum of 95 percent of its Standard Proctor maximum dry density, with 100 percent at the surface. We recommend that field density tests, including one-point Proctor verification tests, be performed on the fill as it is being placed at a frequency determined by an experienced geotechnical engineer to verify the compaction criteria.

6.0 LIMITATIONS OF REPORT

This report has been prepared in accordance with generally accepted geotechnical engineering practice with regard to the specific conditions and requirements of this site. The conclusions and recommendations contained in this report were based on the applicable standards of our practice in this geographic area at the time this report was prepared. No other warranty, expressed or implied, is made.

The analysis and recommendations submitted herein are based, in part, upon the data obtained from the subsurface exploration. The nature and extent of variations between the borings will not be known until construction is underway. If variations appear evident, then we request the opportunity to re-evaluate the recommendations of this report. In the event that any changes in the nature, design, or location of the structures are planned, the conclusions and recommendations contained in this report will not be considered valid unless the changes are reviewed and conclusions modified or verified in writing by ESP.

In order to verify that earthwork and foundation design recommendations are properly interpreted and implemented, we recommend that ESP be provided the opportunity to review the final plans and specifications. Any concerns observed will be brought to our client's attention in writing.

FIELD EXPLORATION PROCEDURES

Soil Test Boring: Four (4) soil test borings were drilled at the approximate locations shown on the attached "Boring Location Plan," Figure 1. Soil sampling and penetration testing were performed in accordance with ASTM D 1586.

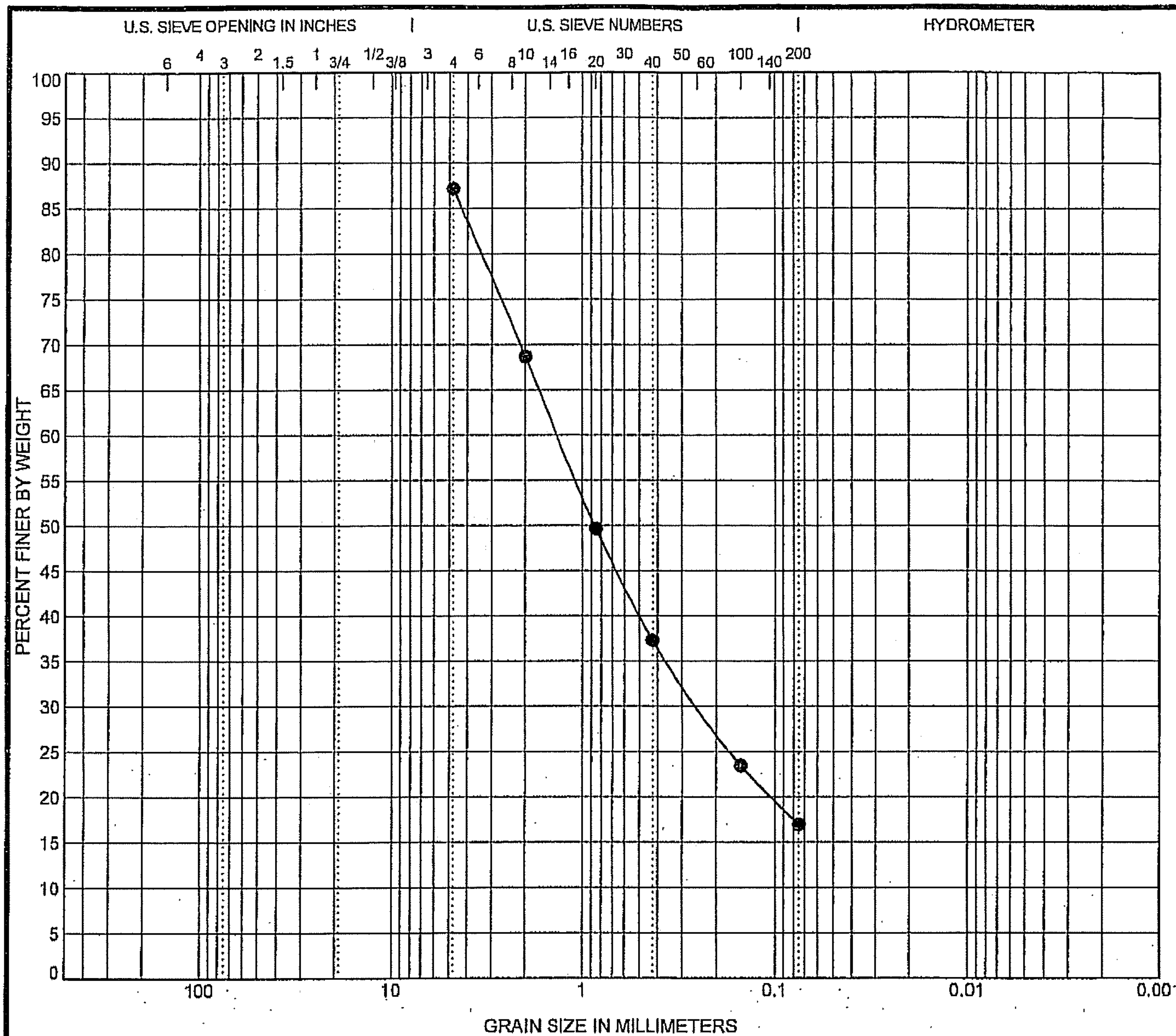
The borings were advanced with hollow-stem augers and, at standard intervals, soil samples were obtained with a standard 1.4-inch I.D., 2-inch O.D., split-tube sampler. The sampler was first seated six (6) inches to penetrate any loose cuttings, and then driven an additional foot with blows of a 140-pound hammer falling 30 inches. The number of hammer blows is designated the "Standard Penetration Resistance." When properly evaluated, the Standard Penetration Resistances provide an index to soil strength, relative density, and ability to support foundations.

Select portions of each soil sample were placed in sealed containers and taken to our office. The samples were examined by a representative of the geotechnical engineer for classification. "Test Boring Records" are attached showing the soil descriptions and Standard Penetration Resistances.

LABORATORY PROCEDURES

Grain Size Test: Grain size tests were performed to determine the particle size and distribution of the samples tested. The grain size distribution of soils coarser than a No. 200 sieve was determined by passing the samples through a set of nested sieves. The soil particles passing the No. 200 sieve were suspended in solution and the grain size distribution determined from the rate of settlement. The results are presented on the attached Grain Size Distribution Sheets.

Soil Plasticity Tests (Atterberg Limits Test): Select samples were identified for Atterberg Limits testing to determine the soil's plasticity characteristics. The Plasticity Index (PI) is representative of this characteristic and is determined utilizing the Liquid Limit (LL) and the Plastic Limit (PL). The Liquid Limit is the moisture content at which the soil will flow as a heavy viscous fluid and is determined in accordance with ASTM D 4318. The Plastic Limit is the moisture content at which the soil begins to lose its plasticity and is determined in accordance with ASTM D4318.



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Specimen Identification	Classification	LL	PL	PI	Cc	Cu
●	S-1	**SILTY SAND SM**	27	NP	-		
		Sampled from Boring MH-1 from 1 to 5 feet below existing ground elevation.					

	Specimen Identification	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	S-1	4.75	1.354	0.246		12.9	70.2	16.9	

US GRAIN SIZE UH26.350 MOREHEAD ROAD CORED SLAB BRIDGE.GPJ LOG-LAB.GDT 5/28/08

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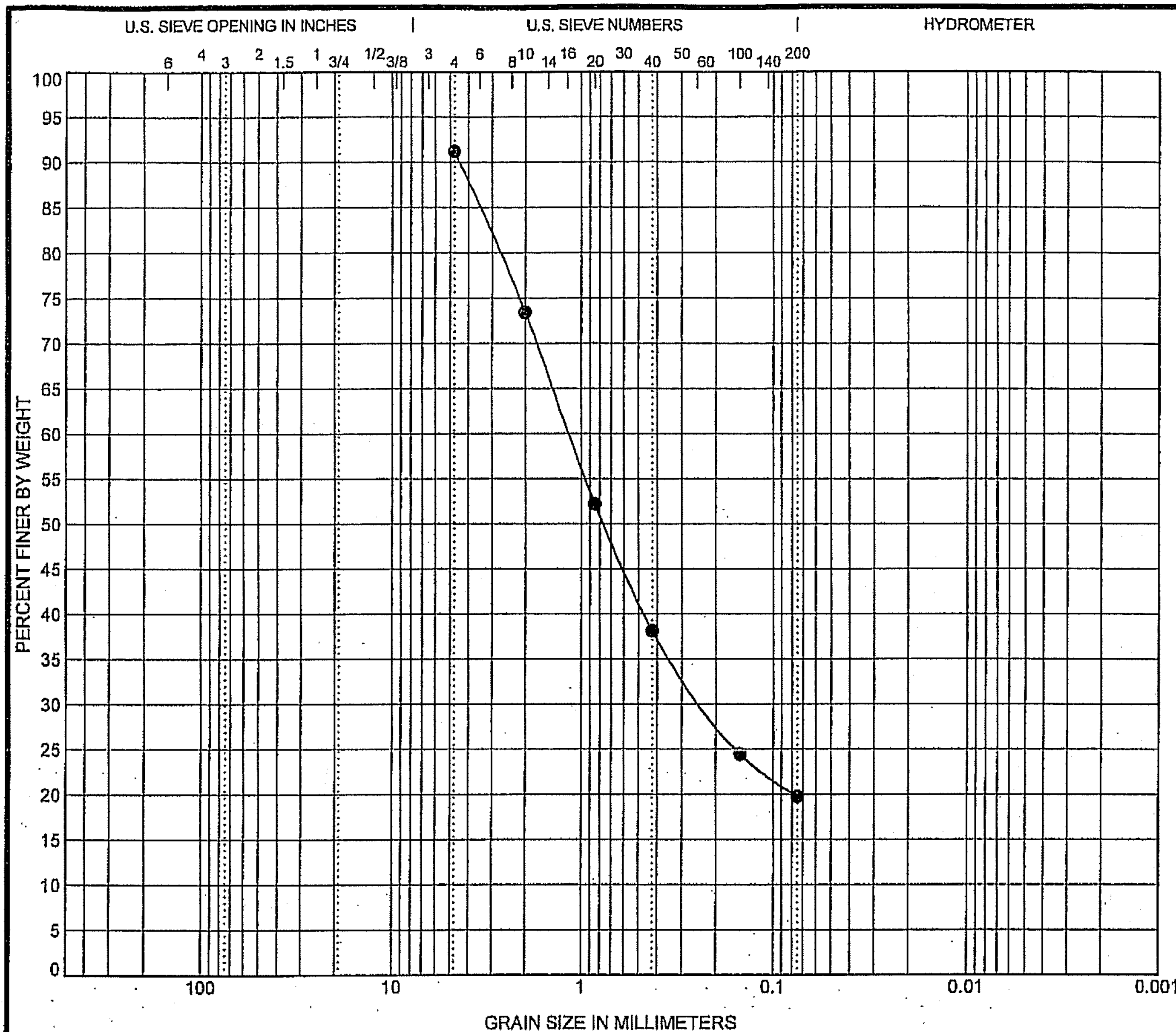
GRAIN SIZE DISTRIBUTION

Project: Morehead Road Overpass fo Tram Road

Location: Lowes Motor Speedway, Concord, NC

Number: UH26.350



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Specimen Identification	Classification	LL	PL	PI	Cc	Cu
●	S-2	**SILTY SAND SM**	**25**	**NP**	**-**		
		Sampled from Borings MH-3 from 3.5 to 5 feet below					
		existing ground elevation and MH-4 from 1 to 2.5 feet					
		below existing ground elevation.					

	Specimen Identification	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	S-2	**4.75**	**1.164**	**0.229**		**8.8**	**71.4**	**19.8**	

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GRAIN SIZE DISTRIBUTION

Project: Morehead Road Overpass of Tram Road

Location: Lowes Motor Speedway, Concord, NC

Number: UH26.350

LEGEND TO SOIL CLASSIFICATION AND SYMBOLS

SOIL TYPES
(Shown in Graphic Log)

Asphalt / Concrete	Organic	Sandy Silt
Topsoil	Sandy	Clayey Silt
Gravel	Silty	Sandy Clay
Sand	Clayey	Silty Clay
Silt	Silty Sand	Partially Weathered Rock
Clay	Clayey Sand	Cored Rock

SAMPLER TYPES
(Shown in Samples Column)

- Shelby Tube
- Split Spoon
- Rock Core
- No Recovery

CONSISTENCY OF COHESIVE SOILS

CONSISTENCY	STD. PENETRATION RESISTANCE BLOWS / FOOT
Very Soft	0 to 2
Soft	3 to 4
Firm	5 to 8
Stiff	9 to 15
Very Stiff	16 to 30
Hard	31 to 50
Very Hard	Over 50

WATER LEVELS
(Shown in Water Level Column)

- ∇ = Water Level at Termination of Boring
- ▼ = Water Level Taken After 24 Hours
- ◄ = Loss of Drilling Water
- HC = Hole Cave

CONSISTENCY OF COHESIONLESS SOILS

CONSISTENCY	STD. PENETRATION RESISTANCE BLOWS / FOOT
Very Loose	0 to 4
Loose	5 to 10
Medium Dense	11 to 30
Dense	31 to 50
Very Dense	Over 50

TERMS

Standard Penetration Resistance - The Number of Blows of a 140 lb. Hammer Falling 30 in. Required to Drive a 1.4 in I.D. Split Spoon Sampler 1 Foot (N-Value) As Specified in ASTM D-1586.

REC - Total Length of Rock Recovered in the Core Barrel Divided by the Total Length of the Core Run Times 100 (expressed as a percentage).

RQD - Total Length of Sound Rock Segments Recovered that are Longer Than or Equal to 4" (mechanical breaks included) Divided by the Total Length of the Core Run Times 100 (expressed as a percentage).

Dynamic Cone Penetrometer Test Data - The Number of Blows of a 15 lb. Hammer Falling 20 in. Required to Drive a Cone Point 1 3/4 in. When Properly Evaluated, it can be compared to the Standard Penetration Resistance.

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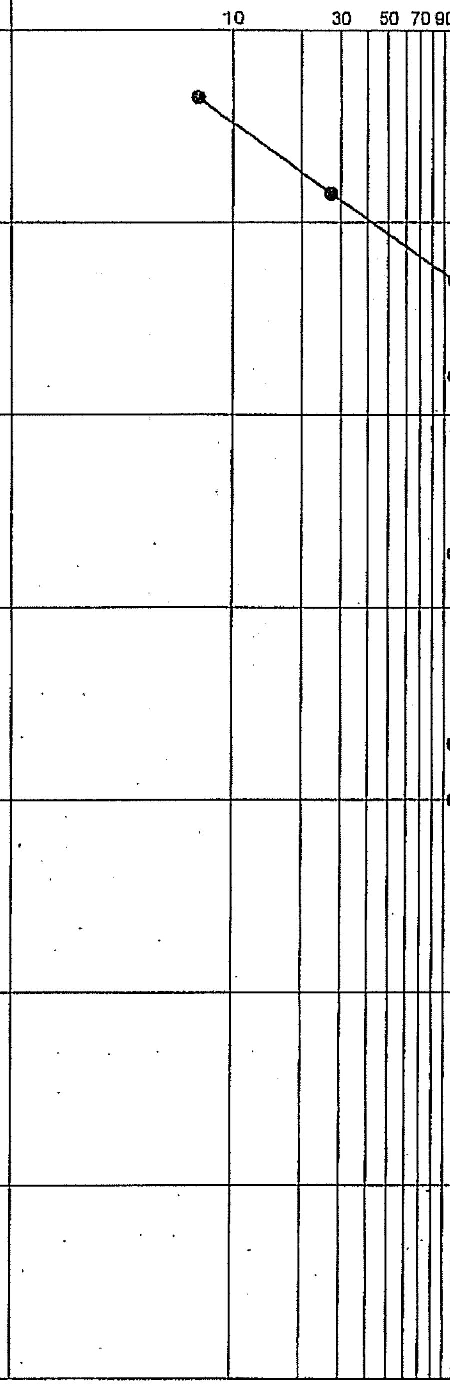
PROJECT: Morehead Road Overpass of Tram Road Lowes Motor Speedway, Concord, NC	**TEST BORING RECORD MH-1**

PROJECT No.:	ELEVATION:	DRILLING METHOD:	NOTES:
UH26.350	661.8 FEET	2-1/4" ID hollow setm	Drilling services provided by Ameridrill
LOGGED BY:	BORING DEPTH:	DRILL RIG:	Northing 584842.59
John Abernathy	20 FEET	CME550X (ATV)	Easting 1498265.26
DATE DRILLED:	WATER LEVEL:		
05/14/08	Dry @ TOB	▼ 7.0 @ 1 day	

DEPTH (ft)	SOIL DESCRIPTION	WATER LEVEL	ELEV. (ft)	BPF
0	TOPSOIL			
	RESIDUUM: Loose to Medium Dense Orangeish Brown and Tan Silty Fine Grained SAND trace Mica, and Rock Fragments			7
				27
5			656.8	
6	PARTIALLY WEATHERED ROCK: Orangeish Brown Silty Fine to Medium Grained SAND trace, and Rock Fragments	▼		50/0.5
				50/0.5
10			651.8	
				50/0.2
15			646.8	
		HC		
				50/0.1
20	Auger refusal encountered at 20 feet. Boring was terminated at 20 feet. Hole cave-in was observed at 16.9 feet.		641.8	50/0.0
25			636.8	
30			631.8	
35			626.8	

STANDARD PENETRATION TEST DATA (Blows/ft)



DEPTH MEASUREMENTS ARE SHOWN TO ILLUSTRATE THE GENERAL ARRANGEMENTS OF THE SOIL TYPES ENCOUNTERED AT THE BORING LOCATIONS.

DO NOT USE ELEVATIONS OR DEPTH MEASUREMENTS FOR DETERMINATION OF DISTANCES OR QUANTITIES.

ESP
ESP Associates, P.A.
4601 Corporate Drive, NW
Concord, North Carolina 28027 704.793.9855

TBR 3 UH26.350 MOREHEAD ROAD CORED SLAB BRIDGE.GPJ LOG-LAB.GDT 6/3/08

PROJECT: Morehead Road Overpass of Tram Road
Lowes Motor Speedway, Concord, NC

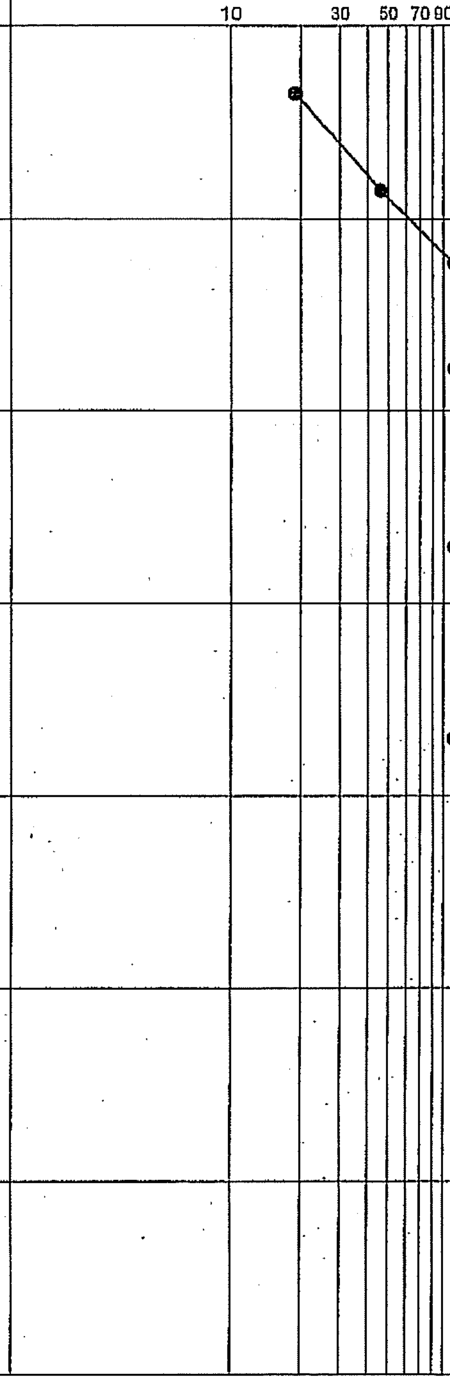
TEST BORING RECORD MH-2

PROJECT No.:	ELEVATION:	DRILLING METHOD:
UH26.350	664.1 FEET	2-1/4" ID hollow setm
LOGGED BY:	BORING DEPTH:	DRILL RIG:
John Abernathy	18.5 FEET	CME550X (ATV)
DATE DRILLED:	WATER LEVEL:	
05/14/08	▽ 15.2 @ TOB	▼ 15.0 @ 1 day

NOTES:
Drilling services provided by Ameridrill
Northing 584827.03
Easting 1498258.49

DEPTH (ft)	SOIL DESCRIPTION	WATER LEVEL	ELEV. (ft)	BPF
0	TOPSOIL			
	RESIDUUM: Medium Dense to Dense Light Brown and Tan Silty Fine to Medium Grained **SAND** with Rock Fragments, trace Mica			19
				46
5			659.1	
	PARTIALLY WEATHERED ROCK: Light Brown and Tan Silty Medium Grained **SAND** with Rock Fragments, and Mica			50/0.3
				50/0.3
10			654.1	
				50/0.1
15		▼ ▽ HC	649.1	
				50/0.0
	Auger refusal encountered at 18.5 feet. Boring was terminated at 18.5 feet. Hole cave-in was observed at 16.2 feet.			
20			644.1	
25			639.1	
30			634.1	
35			629.1	

STANDARD PENETRATION TEST DATA (Blows/ft)



DEPTH MEASUREMENTS ARE SHOWN TO ILLUSTRATE THE GENERAL ARRANGEMENTS OF THE SOIL TYPES ENCOUNTERED AT THE BORING LOCATIONS.

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TBR 3 UH26.350 MOREHEAD ROAD CORED SLAB BRIDGE.GPJ LOG-LAB.GDT 6/3/08

PROJECT: Morehead Road Overpass of Tram Road Lowes Motor Speedway, Concord, NC	TEST BORING RECORD MH-3

PROJECT No.:	ELEVATION:	DRILLING METHOD:	NOTES:
UH26.350	665.5 FEET	2-1/4" ID hollow setm	Drilling services provided by Ameridrill
LOGGED BY:	**BORING DEPTH:**	**DRILL RIG:**	Northing 584761.69
John Abernathy	23.9 FEET	CME550X (ATV)	Easting 1498243.60
DATE DRILLED:	**WATER LEVEL:**		
05/14/08	Dry @ TOB	▼ 15.0 @ 1 day	

DEPTH (ft)	SOIL DESCRIPTION	WATER LEVEL	ELEV. (ft)	BPF
0	**TOPSOIL**		665.5	
	PARTIALLY WEATHERED ROCK: Light Brown and Tan Silty **SAND** with Mica, and Rock Fragments			60/0.5
5	**RESIDUUM:** Very Dense Light Brown and Tan Silty Fine to Medium Grained **SAND** with Mica, and Rock Fragments		660.5	59
	PARTIALLY WEATHERED ROCK: Light Brown and Tan Silty Fine Grained **SAND** with Rock Fragments, and Mica			60/0.3
				60/0.3
10			655.5	
				60/0.5
15		▼	650.5	
	RESIDUUM: Very Dense Light Brown and Tan Silty Fine Grained **SAND** with Rock Fragments, and Mica			76
20		HC	645.5	
	PARTIALLY WEATHERED ROCK: Light Brown and Tan Silty Fine Grained **SAND** with Rock Fragments, and Mica			60/0.4
25	Boring was terminated at 23.9 feet. Hole cave-in was observed at 21 feet.		640.5	
30			635.5	
35			630.5	

STANDARD PENETRATION TEST DATA (Blows/ft): 10 30 50 70 90

DEPTH MEASUREMENTS ARE SHOWN TO ILLUSTRATE THE GENERAL ARRANGEMENTS OF THE SOIL TYPES ENCOUNTERED AT THE BORING LOCATIONS.

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TBR 3 UH26.350 MOREHEAD ROAD CORED SLAB BRIDGE.GPJ LOG-LAB.GDT 6/3/08

PROJECT: Morehead Road Overpass of Tram Road Lowes Motor Speedway, Concord, NC	**TEST BORING RECORD MH-4**

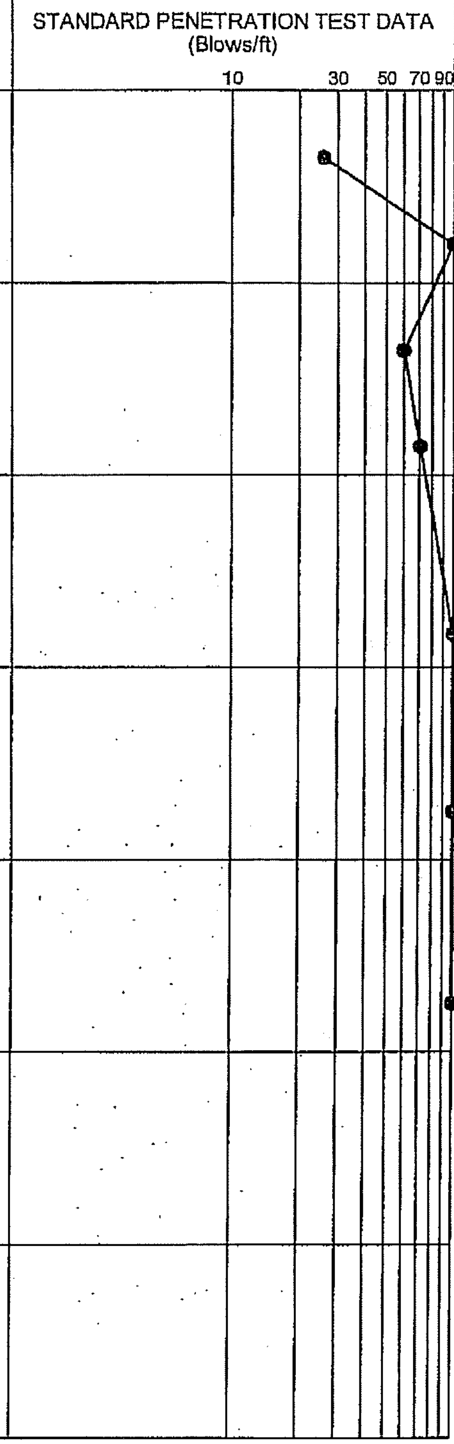
PROJECT No.:	ELEVATION:	DRILLING METHOD:
UH26.350	665.2 FEET	2-1/4" ID hollow setm
LOGGED BY:	BORING DEPTH:	DRILL RIG:
John Abernathy	24 FEET	CME550X (ATV)
DATE DRILLED:	WATER LEVEL:	
05/14/08	Dry @ TOB	▼ 8.2 @ 1 day

NOTES:
Drilling services provided by Ameridrill

Northing 584746.79

Easting 1498240.11

DEPTH (ft)	SOIL DESCRIPTION	WATER LEVEL	ELEV. (ft)	BPF
0	TOPSOIL			
	RESIDUUM: Medium Dense Orangeish Brown and Tan Silty Fine Grained **SAND** trace Mica, and Rock Fragments			26
	PARTIALLY WEATHERED ROCK: Orangeish Brown and Tan Silty **SAND** trace Mica, and Rock Fragments			50/0.5
5			660.3	
	RESIDUUM: Very Dense Orangeish Brown and Tan Silty Fine Grained **SAND** trace Mica, and Rock Fragments	▼		60
				71
10			655.3	
	PARTIALLY WEATHERED ROCK: Orange and Brown Silty Fine to Medium Grained **SAND** trace Mica, and Rock Fragments			50/0.3
15			650.3	
				50/0.5
20			645.3	
		HC		
				50/0.5
25	Boring was terminated at 24 feet. Hole cave-in was observed at 22.7 feet.		640.3	
30			635.3	
35			630.3	



DEPTH MEASUREMENTS ARE SHOWN TO ILLUSTRATE THE GENERAL ARRANGEMENTS OF THE SOIL TYPES ENCOUNTERED AT THE BORING LOCATIONS.

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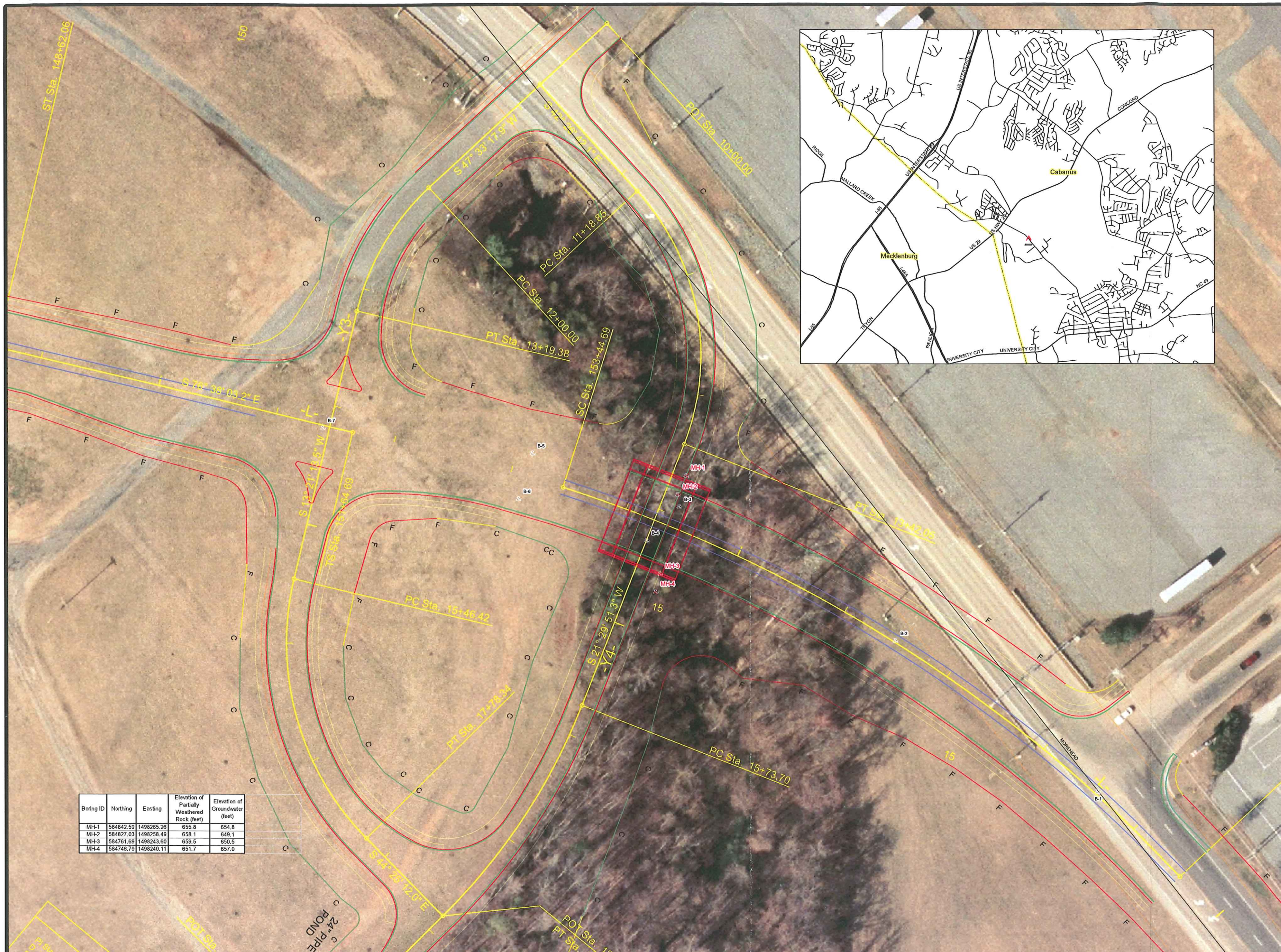
TBR 3 UH26.350 MOREHEAD ROAD CORED SLAB BRIDGE.GPJ LOG-LAB.GDT 6/3/08



Boring ID	Northing	Easting	Elevation of Partially Weathered Rock (feet)	Elevation of Groundwater (feet)
MH-1	584842.59	1498265.26	655.8	654.8
MH-2	584827.03	1498258.49	658.1	649.1
MH-3	584761.69	1498243.60	659.5	650.5
MH-4	584746.79	1498240.11	651.7	657.0

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www.espassociates.com

Legend

Previously Performed Borings
Please reference our report titled,
"Report of Subsurface Exploration," dated January 8, 2007.

< Approximate Boring Locations

Roads

Existing Parcels

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CLIENT

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6425 Idlewild Road
Building 3, Suite 205
Charlotte, NC 28212

SHEET TITLE

BORING LOCATION PLAN

THIS DRAWING IS INTENDED TO SHOW APPROXIMATE BORING LOCATIONS ONLY. NO OTHER INFORMATION IS EXPRESSED OR IMPLIED.

PROJECT

Morehead Road Overpass of Tram Road

PROJECT LOCATION *Cabarrus County*

PROJECT NUMBER	UH26.350
DRAWING NAME	Morehead_rd_realign.mxd
DATE	5.22.08
DRAWN BY	DMC
CHECKED BY	DAB

NO.	DATE	BY	REVISION
1	5.22.08	DMC	CREATE

FIGURE 1