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PRELIMINARY REPORT OF SUBSURFACE EXPLORATION

**PROPOSED PEDESTRIAN BRIDGE
KNOXVILLE SOUTH WATERFRONT
KNOXVILLE, TENNESSEE**

Prepared For:

**City of Knoxville
South Waterfront Development**

Knoxville, Tennessee

Prepared By:

MACTEC ENGINEERING AND CONSULTING, INC.

Knoxville, Tennessee

MACTEC Project 3043081018

July 30, 2009





engineering and constructing a better tomorrow

July 30, 2009

Ms. Susanna Bass
Project Manager – South Waterfront Development
400 Main Street – Room 503
Knoxville, Tennessee 37902

Subject: **Preliminary Report of Subsurface Exploration
Proposed Pedestrian Bridge
Knoxville South Waterfront
Knoxville, Tennessee
MACTEC Project 3043081018**

Dear Ms. Bass:

We at MACTEC Engineering and Consulting, Inc., (MACTEC) are pleased to submit this Preliminary Report of Subsurface Exploration for your project. Our services, as authorized by you were provided in general accordance with the our City of Knoxville Contract CO-09-0056 and our subsequent “Mobilization of Field Efforts for the Geotechnical Investigation” letter addressed to you and dated June 5, 2009.

This report reviews the information provided to us, discusses the site and subsurface conditions, and presents our conclusions and recommendations. The Appendices contain the Field Exploratory Procedures, a Key Sheet and the Test Boring Records, and the Laboratory Test Procedures and Test Results.

We will be pleased to discuss our recommendations with you and would welcome the opportunity to provide the engineering and material testing services needed to successfully complete your project.

Sincerely,

MACTEC ENGINEERING AND CONSULTING, INC.

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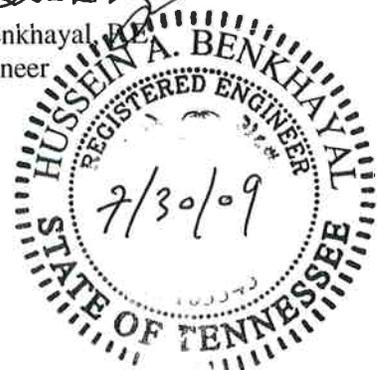


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EXECUTIVE SUMMARY

We were selected by the City of Knoxville to perform a preliminary subsurface exploration for the proposed Pedestrian Bridge. The proposed project site is located near downtown Knoxville, and the bridge will connect the Knoxville South Waterfront Development to the University of Tennessee, Knoxville by spanning the Tennessee River. The exact bridge location is still undetermined, therefore the recommendations provided in this report should be considered as preliminary. The objectives of our exploration were to determine general subsurface conditions and obtain data to aid in providing preliminary foundation recommendations for the proposed pedestrian bridge.

The exploration consisted of drilling four test borings along the proposed location of the Pedestrian Bridge. Two test borings were located on land near the probable area of the bridge abutments, and two test borings were located in the Tennessee River near the probable area of the bridge piers. The land borings were drilled to refusal and then cored approximately ten feet into rock, and the river borings were cored approximately twenty feet into rock. The major findings and recommendations of our subsurface exploration are as follows:

- The land borings generally encountered fill soils with organics and alluvium. Boring LBPB-2 had asphalt and gravel at the surface. The depth of fill ranged from 9 to 11 feet. Organics were encountered in the soil to an approximate depth of 1.5 feet in boring LBPB-1, while boring LBPB-2 had organics down to an approximate depth of 3.5 feet. The consistencies of the fill in both borings ranged from firm to stiff. The alluvium had consistencies ranging from soft to stiff. Auger refusal was encountered at an approximate elevation of 808 ft in boring LBPB-1 and at an approximate elevation of 794 ft in boring LBPB-2. Dolomite was encountered as the bedrock material in LBPB-1, while limestone was the bedrock material in LBPB-2. Cobbles and/or boulders were encountered in both borings near the auger refusal elevation. The rock cores from both land borings showed significant weathering with several voids and cavities. However, the rock core quality encountered in boring LBPB-1 was fair with RQD values ranging from 55 to 60 percent, and the rock core quality was good to excellent below the first two feet of rock coring in boring LBPB-2 with RQD values ranging from 81 to 99 percent.
- The sediment in the river test borings, RBPB-3 and RBPB-4, were not sampled for geotechnical information due to environmental contamination concerns. Sediment thickness ranged from approximately 2.8 feet thick in RBPB-3, while RBPB-4 had approximately 9.0 feet of sediment. Rock was cored in each of the borings. Dolomite and limestone were both encountered as the bedrock material in boring RBPB-3, while boring RBPB-4 encountered limestone as the bedrock material. The rock cores

from both river borings showed some signs of weathering. However, the rock core quality encountered in both borings were good to excellent with RQD values ranging from 75 to 94 percent.

- Ground water was not encountered at the time of drilling in land boring LBPB-1, but ground water was encountered in land boring LBPB-2 at an approximate elevation of 812 ft. Long term measurements for the presence or absence of groundwater were not obtained.
- We recommend the bridge be supported by deep foundations bearing into the bedrock. Drilled piers or micropiles are recommended to support this structure. We recommend an allowable rock bearing pressure of 80 ksf and an allowable concrete/grout to rock skin friction of 70 psi.
- Lightly loaded structures associated with the proposed bridge may be designed to be supported by shallow foundations. Shallow foundations may be sized for a maximum allowable soil bearing pressure of 2,000 psf.
- There is an inherent risk of sinkhole development at sites underlain by calcareous rocks. We do not believe the probability of sinkhole development at this site is greater than on surrounding successfully developed sites in the same geologic setting.

We recommend experienced geotechnical personnel observe subgrades, foundation excavations, fill placement, and other construction procedures. We recommend the owner retain MACTEC to provide these services based on our familiarity with the project, the subsurface conditions, the intent of the recommendations, and our experience in this area. This summary is only an overview and should not be used as a separate document or in place of reading the entire report, including the appendices.

1.0 INTRODUCTION

This report presents the findings of our preliminary subsurface exploration and laboratory testing recently performed for the City of Knoxville. The proposed construction will consist of a pedestrian bridge that spans the Tennessee River. The exact bridge location is still undetermined, but it will be located near downtown Knoxville. Currently, the bridge is expected to connect Clancy Avenue in South Knoxville to a location close to the Thompson Boling Arena at the University of Tennessee. Our services were authorized by Ms. Susanna Bass of City of Knoxville, South Waterfront Development.

2.0 OBJECTIVES OF EXPLORATION

The objectives of our exploration were to determine general subsurface conditions and obtain data to aid in providing preliminary foundation recommendations for the proposed pedestrian bridge. An assessment of site environmental conditions, or an assessment for the presence or absence of pollutants in the soil, bedrock, surface water, or ground water of the site was beyond the proposed objectives of our exploration. Therefore, any statements in this report or attachments regarding color, odor, or unusual items or conditions are for information purposes only.

3.0 SCOPE OF EXPLORATION

The scope of services for this exploration has included a site reconnaissance, layout of the borings using approximate methods, drilling two test borings on land in the general locations of the bridge abutments and two test borings in the Tennessee River near the probable locations of the bridge piers, and visually classifying the soil samples obtained from the standard penetration testing. We cored a total of 24.7 feet of rock in the land borings and 41.3 feet of rock in the river borings to verify the continuity and composition of refusal materials.

We collected four undisturbed samples in conjunction with the drilling for laboratory testing. Three unconfined compression (ASTM D 2166), two Atterberg Limits (ASTM D 4318), and two sieve analyses laboratory tests were conducted on the undisturbed samples. Six moisture (ASTM D 2216) laboratory tests were conducted on selected samples obtained during the standard penetration testing. In addition, eight unconfined compression tests were performed on selected rock core samples. The results of our laboratory testing are attached in Appendix C.

4.0 PROJECT INFORMATION AND SITE CONDITIONS

Project information was provided to us during meetings with the City of Knoxville South Waterfront Development office. The City of Knoxville desires to build a pedestrian bridge spanning the Tennessee River. The bridge has yet to be designed, and its exact location has not been determined. This bridge is presently expected to connect Thompson-Boling Arena on the University of Tennessee, Knoxville's campus with Clancy Avenue in South Knoxville. Since a bridge design with definitive abutment and bridge pier locations is not currently available, the recommendations provided in this report should be considered as preliminary recommendations.

The bridge is currently expected to tie in to a deck structure on the Thompson-Boling Arena on the north side of the Tennessee River. Thompson-Boling Arena is a large structure that hosts entertainment and athletic events. A greenway runs next to the Thompson-Boling Arena, and the pedestrian bridge will ultimately connect greenways from both sides of the river. In addition, there are railroad tracks located between the greenway and Thompson-Boling Arena. The south side abutment will be built on Clancy Avenue, a small dead-end road located between two active industrial sites.

5.0 AREA AND SITE GEOLOGY

Knoxville, Tennessee, is located in the Appalachian Valley and Ridge Physiographic Province. This province extends as a continuous belt from Central Alabama, through Georgia and Tennessee, northward into Pennsylvania. The formations that underlie this province consist primarily of limestone, dolostone, shale, and sandstone, which have been folded and faulted in the geologic past. These formations range in age from Cambrian to Pennsylvanian and have been subjected to at least one extensive period of erosion since their structural deformation. The erosion has produced a series of subparallel, alternating ridges and valleys. The valleys are formed over more soluble bedrock (limestone, dolostone, and shale), whereas bedrock more resistant to solution weathering forms ridges (sandstone, shale, and cherty dolostone).

According to the Geologic Map of the Knoxville quadrangle dated 1958, the subject site is underlain by the Lenoir limestone, the Mosheim member of Lenoir Limestone, and the Newala formations. All three formations underlay the proposed bridge span, with the northern abutment underlain by the Newala formation; the southern abutment underlain by the Lenoir Limestone; and the Mosheim member of the Lenoir limestone in this location is represented as a thin band in the

Tennessee River that the bridge will span. The Lenoir limestone formation typically consists of argillaceous to silty, “nodular” limestone with thin beds of clay or silt impurities. The Lenoir limestone weathers to an orange-red silty clay residuum. The Mosheim member is typically dove-gray limestone with small calcite specks. Finally, the Newala formation typically consists of gray, dense, or fine-grained dolomite and blue or brown limestone in the lower part of its formation, while the upper part of the formation is typically light-gray to cream-colored dolomite with some blue-gray, aphanitic limestone lenses. The Newala formation generally has rounded nodules of chert and lenses of light-colored chert.

Limestone and dolomite, such as the strata underlying this site, are of great geologic age and have been subject to solution weathering for many millions of years. Rainwater falling onto the surface and percolating downward through the soil and into cracks and fissures gradually dissolves the rock, producing insoluble impurities such as chert and clay. Since limestone and dolomite both vary greatly in resistance to weathering, the soil to bedrock contact may be extremely irregular. More soluble bedrock develops a thicker soil cover and a more irregular bedrock surface, with pinnacles and slots, and less soluble bedrock usually develops a thinner soil cover and a less irregular soil-bedrock surface. Because of the geologic history of the area and the difference in weathering, it is not uncommon to encounter rock at depths varying by as much as 50 feet in borings as close as 10 feet apart in some areas.

These large variations in bedrock depth are greatly enhanced by the presence of fractures, bedding planes, and faults, which provide an increased opportunity for a greater influx of percolating water. The weaknesses may form clay-filled cavities or enlarge into caves and may be connected by a network of passageways. If a cave forms close to the bedrock surface, its roof may collapse and the overlying soils may erode into the cave. Once the weight of the overlying soil exceeds the soil’s arching strength, the soil collapses and an open hole or depression may appear at the ground surface. Such a feature is termed a sinkhole. Sinkholes are quite common in areas of East Tennessee underlain by soluble bedrock and, therefore, all sites underlain by soluble bedrock have the potential for sinkhole development.

It has been our experience that those areas underlain by soluble bedrock in which the topsoil and an interval of residual soil have been removed are more susceptible to the downward migration of water and, therefore, such areas have a higher potential for sinkhole development, whether the water originates from rainfall, underground utility lines, or ground water. Additionally,

fluctuations of the ground-water level commonly play a part in the formation of sinkholes.

The borings at the site did not encounter any open voids in the soil or other signs of incipient sinkhole conditions. However, the owner should be aware that there is inherent risk of sinkholes developing at any site underlain by calcareous rocks. We believe the probability of sinkhole development at this site is no greater than on surrounding successfully developed sites in this geologic setting.

6.0 SUBSURFACE CONDITIONS

Subsurface conditions were explored with four widely spaced borings drilled in general accordance with the procedures presented in Appendix A. The boring locations were selected by others. Our geotechnical engineer established the actual boring locations in the field. Boring elevations were estimated by superimposing boring locations onto the topographic site plan provided by Hargreaves Associates and interpolating between contours. River borings elevations were estimated from the Tennessee Valley Authority's (TVA) reservoir information webpage for reservoir elevations given for Fort Loudon Lake on the days barge drilling occurred. The reservoir elevation fluctuates over time, and an average elevation was estimated from a graph showing reservoir elevations over time for each day barge drilling occurred. Therefore, both the boring locations shown on the Boring Location Plan (Figure 2) and the elevations shown on the Test Boring Records in Appendix B, should be considered approximate.

Subsurface conditions encountered at the boring locations are shown on the Test Boring Records in Appendix B. These Test Boring Records represent our interpretation of the subsurface conditions, based on the field logs and visual examination of the field samples by one of our engineers. The lines designating the interfaces between various strata on the Test Boring Records represent the approximate interface locations.

The subsurface conditions of the land borings generally encountered fill soils with organics and alluvium. Fill soils are soils that have been transported to their present location by man. Alluvial soils are soils that have been transported to their present location by running water. Both land test borings were advanced until refusal was encountered. Boring LBPB-1 encountered auger refusal at an approximate elevation of 807.6 ft, while boring LBPB-2 encountered refusal at an approximate elevation of 794.4 ft.

Boring LBPB-2 had asphalt and gravel at the surface. Boring LBPB-1 had fill to a depth of approximately nine feet while boring LBPB-2 had fill to a depth of approximately 11 feet. Organics were encountered in the soil to an approximate depth of 1.5 feet in boring LBPB-1, while boring LBPB-2 had organics down to an approximate depth of 3.5 feet. The fill typically appeared to be red clay in boring LBPB-1, and the fill in boring LBPB-2 typically appeared to be a silty or sandy clay. The fill in boring LBPB-1 had a stiff consistency with standard penetration test (SPT) resistance values ranging from 9 to 14 blows per foot (bpf), while boring LBPB-2 had consistencies in the fill that ranged from firm to stiff with SPT resistance values ranging from 5 to 13 bpf. The alluvium in both borings typically appeared to be a silty or sandy clay with chert and rock fragments in some samples. Boring LBPB-1 had alluvium with consistencies ranging from soft to stiff with SPT resistance values ranging from 3 to 13 bpf, and the alluvium in boring LBPB-2 had consistencies ranging from firm to stiff with SPT resistance values ranging from 7 to 9 bpf. The alluvial soils were a mix of clay, silt, sand, and pebbles/rock fragments.

The river test borings did not have geotechnical sampling of the sediment due to concerns about possible contamination, therefore casing was advanced through the sediment and coring began when rock was encountered. River boring RBPB-3 had a sediment layer of approximately 2.8 feet, and the elevation that coring began on this boring was approximately 783 feet. River boring RBPB-2 had approximately 9.0 feet of sediment, and coring began at an approximate elevation of 784 feet.

Dolomite or limestone was encountered as the bedrock material in all borings. The rock cores from all borings showed significant weathering with several voids and cavities. Generally, the rock quality ranged from fair to excellent except during the first three feet core in boring LBPB-2 where possible boulder/cobbles and pebbles were encountered.

Land test boring LBPB-1 encountered dolomite as the bedrock material. In this boring, the rock quality designation (RQD) values ranged from 55 to 60 percent, and the percent recovery values ranged from 61 to 68 percent. Land test boring LBPB-2 encountered limestone as the bedrock material. This boring showed values of RQD ranging from 0 to 99 percent and percent recovery ranged from 41 to 100 percent, and the lowest RQD and recovery values were obtained in the first three feet of coring. The initial three feet of rock core in boring LBPB-2 was an assortment of cobbles, pebbles, and an apparent boulder.

Dolomite and limestone was encountered as the bedrock material in river boring RBPB-3, while RBPB-4 encountered only limestone as the bedrock material. The rock cores from both river borings showed some signs of weathering with some voids and cavities. Approximately 21.3 feet of rock was cored in boring RBPB-3. This boring had RQD values ranging from 67 to 94 percent and percent recovery ranging from 74 to 97 percent. River boring RBPB-4 had approximately 20.0 feet of rock cored. This boring had RQD values that ranged from 75 to 93 percent while the percent recovery ranged from 90 to 99 percent.

7.0 GROUND-WATER CONDITIONS

Ground water was not observed in land test boring LBPB-1 at the time of drilling, but it was encountered at approximately 13 feet below the ground surface (bgs) in land test boring LBPB-2 at the time of drilling. This depth is equivalent to an approximate elevation of 812 feet. For safety reasons, the borings were backfilled promptly after drilling; consequently, long-term measurements for the presence or absence of ground water were not obtained.

Fluctuations in the ground-water level occur because of variations in rainfall, evaporation, construction activity, surface run-off, and other site-specific factors such as springs or reservoir levels. Since this site is very close to the Tennessee River, ground water may present significant construction problems for this project. Preparations must be made to adequately address potential problems caused by groundwater during construction.

8.0 PRELIMINARY FOUNDATION RECOMMENDATIONS

8.1 Pedestrian Bridge (Deep Foundations)

Considering the nature of the proposed project and the subsurface data obtained, it is our opinion that the use of a deep foundation system to support the proposed pedestrian bridge is appropriate. We have evaluated two deep foundation support alternatives for support of the proposed bridge, drilled piers and micropiles. The owner should make an informed decision with regards to the selected deep foundation alternative based upon estimated cost, schedule, and relative advantages and disadvantages of each system.

Drilled Piers

Drilled piers may be considered for support of the proposed bridge. Based upon the limited subsurface data and our experience in this geologic setting, drilled piers bearing on continuous hard to very hard, slightly weathered to fresh limestone or dolomite may be designed considering a maximum allowable rock end bearing pressure of 80 kips per square foot (ksf). An allowable skin friction value of 70 pounds per square inch (psi) may be used for drilled piers socketed into the recommended bearing materials. Skin friction in the overlying soils should be neglected. A minimum drilled pier diameter of 36 inches is recommended to provide reasonable entry space for cleaning, bottom preparation, and inspection. If the drilled piers are designed to have smaller diameters, the piers may be designed to carry the entire design load by rock to concrete skin friction along a rock socket.

As the drilled pier hole is advanced, a temporary protective steel casing should be installed in the drilled hole. A properly designed steel casing will greatly reduce the possibility of sidewall collapse. Additionally, properly designed steel casing will reduce excessive mud and water intrusion into the excavation and will allow workers to excavate, clean, and inspect the drilled pier.

With the exception subsequently discussed in the next three paragraphs, the protective steel casings may be extracted as the concrete is placed. However, the protective steel casing should not be moved until the concrete is above the ground-water level. A minimum 5-foot head of concrete should be maintained above the bottom of the casing during withdrawal and the contractor should prevent concrete from "hanging up" inside the casing that can cause soils and water intrusion below the casing.

Ground-water conditions at this site may require the use of special procedures to achieve a satisfactory foundation installation if substantial localized flows are encountered. If water is flowing into the drilled pier at less than 20 gallons per minute (gpm), pumps should be used to maintain less than 2 inches of water in the hole during the cleaning and inspection. After verification that adequate bearing is obtained, the pumps should be pulled and concreting commenced immediately.

If more than 20 gpm is flowing into the drilled pier, the water level should be allowed to stabilize before attempting to place concrete. If the water is allowed to stabilize, we recommend the

concrete be placed into the drilled pier through a tremie pipe. The end of the pipe should be lowered to the bottom of the drilled pier so the water will be displaced out the top of the pier during concrete placement.

If water is pumped out of the drilled pier, we recommend the concrete placement be directed through a centering chute or other commonly used methods at the surface so that fall is vertical down the center of the shaft without hitting sides or reinforcing steel. This procedure is required to reduce side flow and segregation of the concrete.

Concrete slumps ranging from five to seven inches are recommended for the drilled pier construction. Concrete slumps in this range will usually fill irregularities along the sides and bottom of the pier and displace water as it is placed.

Based upon the limited subsurface information of this exploration, it appears that the top of rock elevation generally ranges from approximately 783 to 808 feet in the proposed bridge location. However, as noted earlier, the depth to rock can vary greatly in this geologic setting and these values are provided only for preliminary estimation purposes.

An inherent disadvantage to the use of drilled piers at this site is the discontinuous nature of the rock. The discontinuous character of the rock geology causes drilled pier lengths to vary substantially. This makes it difficult to accurately estimate drilled pier lengths, bearing levels, and rock excavation quantities in advance, resulting in an uncertain foundation cost. Furthermore, drilled pier construction at this site would likely be more difficult as compared with other sites in this geologic setting considering the proximity to the Tennessee River. It is likely that the cavities, such as those encountered in the rock core of this exploration, are interconnected and the recharge of water into the drilled pier excavations from the river would require tremie placement of concrete into the piers.

In view of the above, it is imperative that a qualified geotechnical engineer observe the drilled pier construction. The geotechnical engineer will document the shaft diameter, depth, cleanliness, plumbness, and type of suitability of the bearing material for the design bearing pressure. Significant deviations from the specified or anticipated conditions will be reported to the owner's representative and to the foundation designer. Each drilled pier excavation should be observed after the bottom of the pier is level, cleaned of any mud or extraneous material, and dewatered.

In order to verify the availability of end bearing support, we recommend the contractor drill at least one probe hole in the bottom of each drilled pier excavation. The probe hole should be at least 1.5 inches in diameter and should be drilled by the contractor with a pneumatic percussion drill. These probe holes should be drilled to a minimum depth of two times the drilled pier diameter. Each hole should be checked by one of our geotechnical engineers with a steel feeler rod to assess the rock continuity. If this check indicates significant discontinuous rock or compressible seams that could contribute to excessive settlement, the drilled pier should be excavated deeper. Additional probe holes may be required by our geotechnical engineer to check drilled piers supported on marginal materials. Additional borings may be required at specific locations to estimate the pier length.

If the drilled piers are designed to support the entire design load by rock to concrete skin friction along their rock sockets only, the probe hole drilling and inspection will not be required.

Micropiles

Another option for deep foundation support of the proposed pedestrian bridge is the use of micropiles. Since the foundation of the bridge abutments will provide support for more than just a column load, micropiles would be a favorable option for foundation support of the bridge abutments. Micropiles are relatively small diameter drilled and grouted piles. Their diameters are generally larger than 5 inches but rarely larger than 12 inches. They are constructed with equipment similar to that used for anchoring and grouting, unlike conventional piles that need to be driven or bored. Rotary drilling/percussion is used to advance a casing to the surface of the bearing strata (bedrock), then drill rods are advanced through the casing and into the slightly weathered to fresh bedrock to the design bearing depth. Once the design depth is achieved, the drill rods are removed and an inner steel pipe (and a high strength dowel when required) is set through the casing to the bottom of the drilled hole. The inner pipe as well as the annulus between the inner pipe and the rock (bond zone) or casing is filled with grout. Additional grout is pumped through the casing and into the bearing strata prior to and during the outer casing withdrawal. Micropiles have the advantage of being able to penetrate into rock and achieve relatively high load carrying capacities as a result of the drilling process.

Because of the uncertainty as to the amount of load that would actually be transferred to the tip of the pile and the inability to confirm the soundness of the bedrock below the pile tip, we recommend evaluation of micropile capacity be computed based upon rock to grout skin friction only. For evaluation purposes, micropile capacity (in compression and tension) may be based upon an

allowable grout to rock skin friction of 70 psi for hard to very hard, slightly weathered to fresh limestone or dolomite bedrock. Soil skin friction in the overlying soils should be neglected in micropile design.

Bedrock layers of 12 inches or more can be utilized in computing the rock to grout skin friction, provided cavities in the bedrock are soil filled and terminated in at least 5 feet of continuous bedrock. The actual socket length would be determined in the field by the geotechnical engineer.

As mentioned previously, the discontinuous nature of the bedrock makes the estimation of the depth to the bedrock surface and to sound continuous rock difficult. Therefore the actual micropile lengths are expected to be highly variable. The design socket length per pile, with an allowance for cavities in the bedrock, should be added to the estimated drilling footage. Considering the cavities encountered in the rock cored at this site, we recommend a conservative allowance be added to develop the estimated pile socket length.

Micropiles have the advantage of being readily available and relatively easy to cut off or splice to accommodate length variations. This is an important consideration since pile lengths are expected to be highly variable. Ground water inflow, such as expected at this site, is also less of a concern with the installation of micropiles than drilled piers. From a scheduling standpoint, micropiles can typically be installed more quickly than drilled piers, especially considering likely drilled pier construction difficulties related to length variations caused by discontinuous rock and high ground water inflow rates.

It is imperative that one of our geotechnical engineers observe and document the construction of the selected deep foundation system. The on-site engineer should be present to document the pier/pile diameter, depth, cleanliness, plumbness, and type and suitability of the bearing material for the design bearing pressure of drilled piers. The engineer should be present to observe the installation of pier/piles and confirm that the interpretation of suitable bearing rock and socket are consistent with those recommended.

8.2 Lightly Loaded Structures (Shallow Foundations)

Recommendations in this section are provided for lightly loaded structures that may be associated with the bridge.

Considering the nature of the proposed project and the subsurface data obtained, it is our opinion that the use of a shallow foundation system to support some of the lightly loaded structures associated with the bridge will be appropriate. Since the design of the proposed pedestrian bridge has not yet be completed and in an effort to be as inclusive as possible for the future designers, we have included preliminary recommendations for shallow foundations.

We recommend any existing fill and soils containing organic material within the proposed structure areas be undercut and replaced with compacted fill as discussed subsequently in the section titled "Site Preparation Recommendations." After the site is properly prepared in accordance with our subsequent recommendations, foundations may be sized for a maximum allowable soil bearing pressure of 2,000 pounds per square foot (psf).

The minimum column and continuous wall foundation widths should be 24 and 18 inches, respectively, for ease of construction and to avoid a local punching failure of the foundations into the underlying soils. All exterior foundations should be founded at least 18 inches below finished exterior grade to protect against frost heave and to provide protective embedment. Interior foundations may be founded at nominal depths unless the completed foundation subgrade will be exposed to freezing weather or severe evaporation during construction.

We recommend that walls be provided with construction joints at locations of change in support from alluvial soils to compacted fill in order to accommodate differential settlements at such locations. Individual column foundations should be entirely supported by compacted fill.

Exposure to the environment may weaken soils at the foundation bearing level if foundation excavations remain open overnight. Therefore, foundation concrete should be placed as soon as possible after excavations are made. If the excavation must remain open overnight, or if rainfall becomes imminent while bearing soils are exposed, we recommend a 2- to 4-inch-thick "mud-mat" of "lean" concrete be placed on the bearing soils for protection. Bearing soils softened by surface water intrusion or exposure must be removed from the foundation excavation before placement of concrete.

We recommend that one of our geotechnical engineers observe the foundation excavations before the placement of reinforcing steel or concrete. Based on these observations, selected foundation excavations may be tested with a dynamic cone penetrometer to aid in evaluating the suitability of the bearing soils for the design bearing pressure. The foundation bearing area should be level or

suitably benched. It should also be free of loose soil, ponded water, and debris. If soils unsuitable for the design bearing pressure are encountered in the foundation excavations, the condition, along with appropriate recommendations, will be brought to the attention of the owner's representative.

9.0 SITE SEISMIC CLASSIFICATION

Based on the International Building Code (IBC), 2006, and considering the subsurface conditions encountered in the test borings, the rock cores from the site are considered Class A with a Hard Rock Profile, while the soil encountered at the site is classified as a Class E with a Soft Soil Profile. Class A, Hard Rock Profile, may be designed for site coefficients F_a value of 0.8 and F_v value of 0.8 for this geographic location. Class E, Soft Soil Profile, may be designed for site coefficients F_a value of 1.7 and F_v value of 3.5 for this geographic location.

10.0 SITE PREPARATION RECOMMENDATIONS

All topsoil, vegetation, debris, surface soil containing organic material, and existing fill should be stripped and removed from any associated structures or pavement areas. If suitable, topsoil can be reused in areas to be landscaped. Topsoil or soil containing organic material was encountered in borings LBPB-1 and LBPB-2 to depths ranging from 1.5 to 3.5 feet, respectively. The thickness of topsoil may be greater or less in unexplored areas.

After stripping and before placing fill, we recommend the exposed subgrade in any associated structures and pavement areas be proofrolled to detect unsuitable soil conditions. Proofrolling should be done after a suitable period of dry weather to avoid degrading an otherwise acceptable subgrade. Proofrolling should be performed with a heavily-loaded dump truck or with similar approved construction equipment. The proofrolling equipment should make at least four passes over each section, with the last two passes perpendicular to the first two.

We recommend the exposed subgrade and proofrolling operation be observed and documented by our personnel. If unstable conditions are encountered at the subgrade level, our geotechnical engineer will make appropriate recommendations to the owner's representative for dealing with the conditions. Soft, organic, highly plastic, wet soils, or soils that pump, rut, or wave, during site grading or proofrolling operations should be excavated and replaced with compacted fill or evaluated for stabilization alternatives.

Based on our understanding of the project requirements and the subsurface conditions encountered, we expect subgrade stabilization may be required in the undercut excavations as well as the parking areas. Such stabilization, if required, can typically be achieved using one of the following stabilization alternatives:

- Undercutting poor subgrade soils to expose competent soils and then backfilling with compacted soil fill to planned (or proposed) subgrade levels.
- Undercutting poor subgrade soils to a depth sufficient to allow the placement of a "bridging layer" of soil or stone backfill upon which an interval of compacted soil fill can be constructed for pavement subgrade support.
- Undercutting poor subgrade soils 2 to 3 feet below the pavement subgrade elevation and then placing a high-modulus geotextile and/or a layer of aggregate for stabilization.
- Undercutting poor subgrade soils 1 to 2 feet below the pavement subgrade elevation and then placing a non-woven geotextile and one or more layers of biaxial geogrid in combination with aggregate for stabilization.

We recommend subgrade stabilization requirements be determined at the time of site preparation, based on the stability of the subgrades as determined by proofrolling. The construction budget should include contingency moneys for this purpose.

During the undercutting and rough grading, positive surface drainage should be maintained to prevent the accumulation of water. If the exposed subgrade becomes excessively wet or frozen, or if conditions different from those described previously in this report are encountered, our geotechnical engineer should be contacted.

11.0 COMPACTED FILL RECOMMENDATIONS

We recommend all compacted fill be constructed by spreading acceptable soil in loose layers not more than 8 inches thick. The soils used within the proposed construction areas should be compacted in lifts to at least 98 percent of the standard Proctor maximum dry density (ASTM D 698). The upper 24 inches of fill beneath pavements and upper 12 inches beneath grade slabs should be compacted to at least 100 percent of standard Proctor maximum dry density.

As a general rule, the moisture content of the fill soils compacted to 98 percent standard Proctor should be maintained within +3 to -3 percentage points of the optimum moisture content as determined from the standard Proctor compaction test. This provision may require the contractor to dry soils during periods of wet weather or to wet soils during the hot summer months. The fill soils should have a plasticity index (PI) of less than 30, and a maximum dry density of no less than 90 pounds per cubic foot (pcf).

If variably weathered shale or other degradable rock materials are to be used as engineered fill, it is imperative this material be reduced to a soil/gravel gradation during compaction. If the material size is not adequately reduced, it may subsequently degrade when exposed to water causing losses in soil volume and strength that could adversely effect the proposed structure.

The fill surface must be adequately maintained during construction in order to achieve an acceptable compacted fill. We recommend the fill surface be sloped to achieve sufficient drainage and to prevent ponding of water on the fill. If precipitation is expected while fill construction is temporarily halted, the surface should be rolled with rubber-tired or steel-drummed equipment to improve surface run-off. If the surface soils become excessively wet or frozen, fill operations should be halted and we should be consulted for guidance.

Before final grading of a fill slope, the edge of the compacted fill should extend at least 10 feet horizontally beyond the outside edge of the building foundations, beyond areas of proposed future building expansion, and beyond paved areas. Fill slopes should be grassed to protect from erosion. Based on our experience, we recommend compacted fill slopes be constructed at 2-1/2H:1V or flatter. Fill slopes of 3H:1V or flatter are more desirable for mowing.

Before filling operations begin, representative samples of the proposed fill material should be collected and tested to determine the maximum dry density, optimum moisture content, natural moisture content, and the soil plasticity. These tests are needed to determine if the fill material is acceptable and for construction quality control during compaction operations.

We recommend the fill placement and compaction be observed and documented by our engineering personnel. To verify compaction level obtained, we recommend frequent field density tests of fill soils as they are placed. Significant deviations, either from the project specifications or from good construction practice, will be brought to the attention of the owner's representative along with appropriate recommendations.

12.0 SOIL PLASTICITY CONSIDERATIONS

According to published data for a climate similar to that of East Tennessee, soils with plasticity indexes (PIs) lower than 30 are slightly susceptible to volume changes, and soils with PIs higher than 50 are generally highly susceptible to volume changes. Soils with PIs between these limits have moderate volume change potential. Two plasticity (Atterberg limits) tests were conducted on soils collected at this site with the results of a non-plastic sand and a clay with a PI of 41.

Shrinking and swelling problems are generally not as severe in the East Tennessee area as in other areas, since extended periods of excessively wet or excessively dry weather do not normally occur. Therefore, changes in the moisture content of foundation soils are usually minimal. However, it is not uncommon for significant drying of soils to occur if grading is performed during dry weather. If these soils resaturate after completion of foundation construction, there is the potential for significant structural distress to structures supported on shallow foundations. Therefore, the volume change potential of the soils at this site should be considered for any structures supported on shallow foundations, and the following construction precautions are recommended:

- Foundation construction should be completed as rapidly as possible to prevent damage of foundation soils by exposure to the elements. It is most desirable to complete concreting of individual foundation excavations the same day they are made.
- Subgrades of floor slabs should be protected from excessive drying or wetting by covering the subgrade prior to floor slab construction. This can be done by leaving the floor subgrade several inches high and then making the final excavation to subgrade shortly before floor construction.
- Low plasticity clay should be used for backfill beneath floor slabs and other structural elements whenever possible.
- The site should be graded to promote rapid drainage of surface water during construction.

In addition to these construction precautions, we recommend that the following considerations be incorporated into design:

- Floor slabs should be liberally jointed to control cracking in the event volume changes occur.
- Roof drains should discharge well away from the building area to prevent ponding of water near foundations.

- Heat sources should be isolated from foundation soils to minimize drying and the resultant shrinkage of foundation soils.
- Plantings, shrubs, and trees with high moisture demands should not be placed adjacent to foundations.

13.0 BASIS OF RECOMMENDATIONS

The recommendations provided herein are based on the subsurface conditions and on project information provided to us; they apply only to the specific project and site discussed in this report. If the project information section in this report contains incorrect information or if additional information becomes available, you should convey the corrected or additional information to us and retain us to review our recommendations. We will then modify them if the new information has rendered them inappropriate for the proposed project. Additionally, since the design of the bridge is not currently available, our recommendations must be considered as preliminary.

Regardless of the thoroughness of a geotechnical exploration, there is always a possibility that conditions between test borings will differ from those at specific test boring locations, and that conditions may not be as anticipated by the designers or contractors. In addition, the construction process may itself alter soil conditions. Therefore, experienced geotechnical personnel should observe and document the construction procedures used and the conditions encountered. Unanticipated conditions and inappropriate procedures will be reported to the design team, along with timely recommendations to solve the problems created. We recommend that the owner retain MACTEC to provide this service, based upon our familiarity with the subsurface conditions, the project design, and the intent of the recommendations.

Our exploration services include storing the collected samples and making them available for inspection for a period of 30 days. The samples are then discarded unless you request otherwise.

TABLES

Table 1
 Summary of Soil Index Properties

Boring	Sample Depth	Sample Type	Natural Moisture Content (%)	Dry Unit Weight (pcf)	Atterberg Limits			Percent Finer Than No. 200 Sieve	USCS Classification
					Liquid Limit	Plastic Limit	Plasticity Index		
LBPB-1	1.5-3.0	SPT	21.8	-	-	-	-	-	
LBPB-1	4.0-6.0	UD	25.8	99.3	-	-	-	-	
LBPB-1	6.0-7.5	SPT	26.6	-	-	-	-	-	
LBPB-1	14.0-16.0	UD	33.3	90.3	67	26	82.8	CH	
LBPB-1	16.0-17.5	SPT	33.0	-	-	-	-	-	
LBPB-2	5.5-7.0	SPT	30.6	-	-	-	-	-	
LBPB-2	10.5-12.0	SPT	23.5	-	-	-	-	-	
LBPB-2	12.0-14.0	UD	22.4	106.3	NV	NP	47.6	SM	
LBPB-2	18.5-20.0	SPT	21.9	-	-	-	-	-	

Prepared by: *CFE*

Date: 7/30/09

Checked By: *AAS*

Date: 7/30/09

Table 2
 Summary of Soil Strength Properties

Boring	Depth (ft bgs)	Dry Density (pcf)	Unconfined Compressive Strength (ksf)	Undrained Shear Strength (ksf)
LBPB-1	4.0-6.0	99.3	2.62	1.31
LBPB-1	14.0-16.0	90.3	1.44	0.72
LBPB-2	12.0-14.0	106.3	1.55	0.78

Prepared by: *GTZ* Date: *7/30/2009* Checked by: *HAB* Date: *7/30/09*

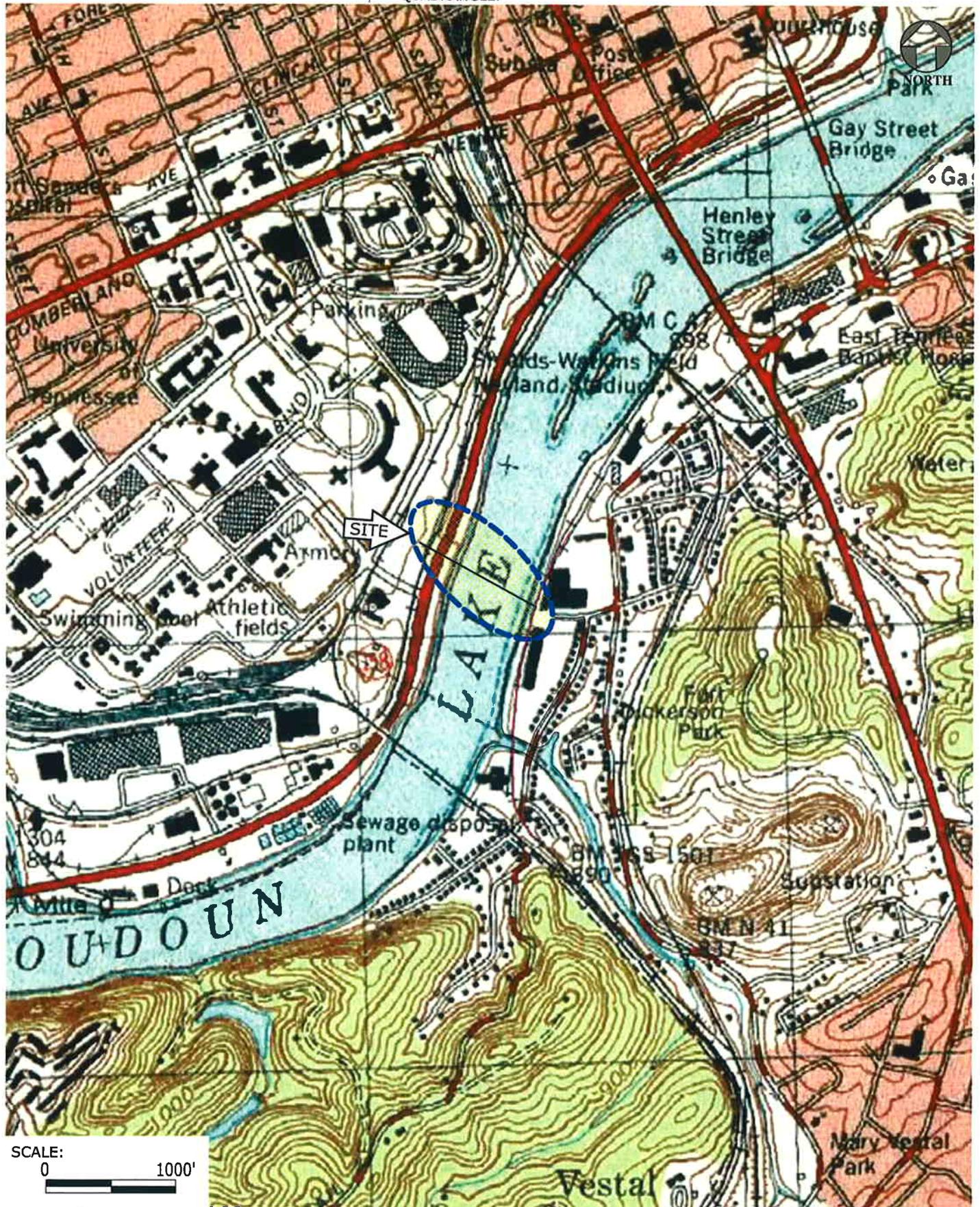
Table 3
 Summary of Rock Strength Properties

Boring	Depth (ft bgs)	Unit Weight (pcf)	Load at Failure (lbs)	Ultimate Compressive Strength (psi)	Ultimate Compressive Strength (ksf)
LBPB-1	44.7-45.1	173.0	50,000	18,402	2,650
LBPB-1	50.8-51.2	172.3	70,450	25,651	3,694
LBPB-2	31.2-31.6	167.7	37,260	13,713	1,975
LBPB-2	40.5-40.9	167.4	38,030	13,996	2,015
RPPB-3	34.0-34.4	175.0	62,275	22,919	3,300
RPPB-3	40.8-41.1	173.1	58,840	21,424	3,085
RPPB-4	33.0-33.4	165.5	32,790	11,939	1,719
RPPB-4	43.4-43.8	165.9	32,820	11,950	1,721

Prepared by: *GTZ* Date: *7/30/2009* Checked by: *HAB* Date: *7/30/09*

FIGURES

SOURCE: USGS TOPOGRAPHIC MAP OF THE KNOXVILLE, TN QUADRANGLE.



PREPARED BY: *RSE* DATE: 09 Jul 2009 CHECKED BY: *HAR* DATE: 09 Jul 2009

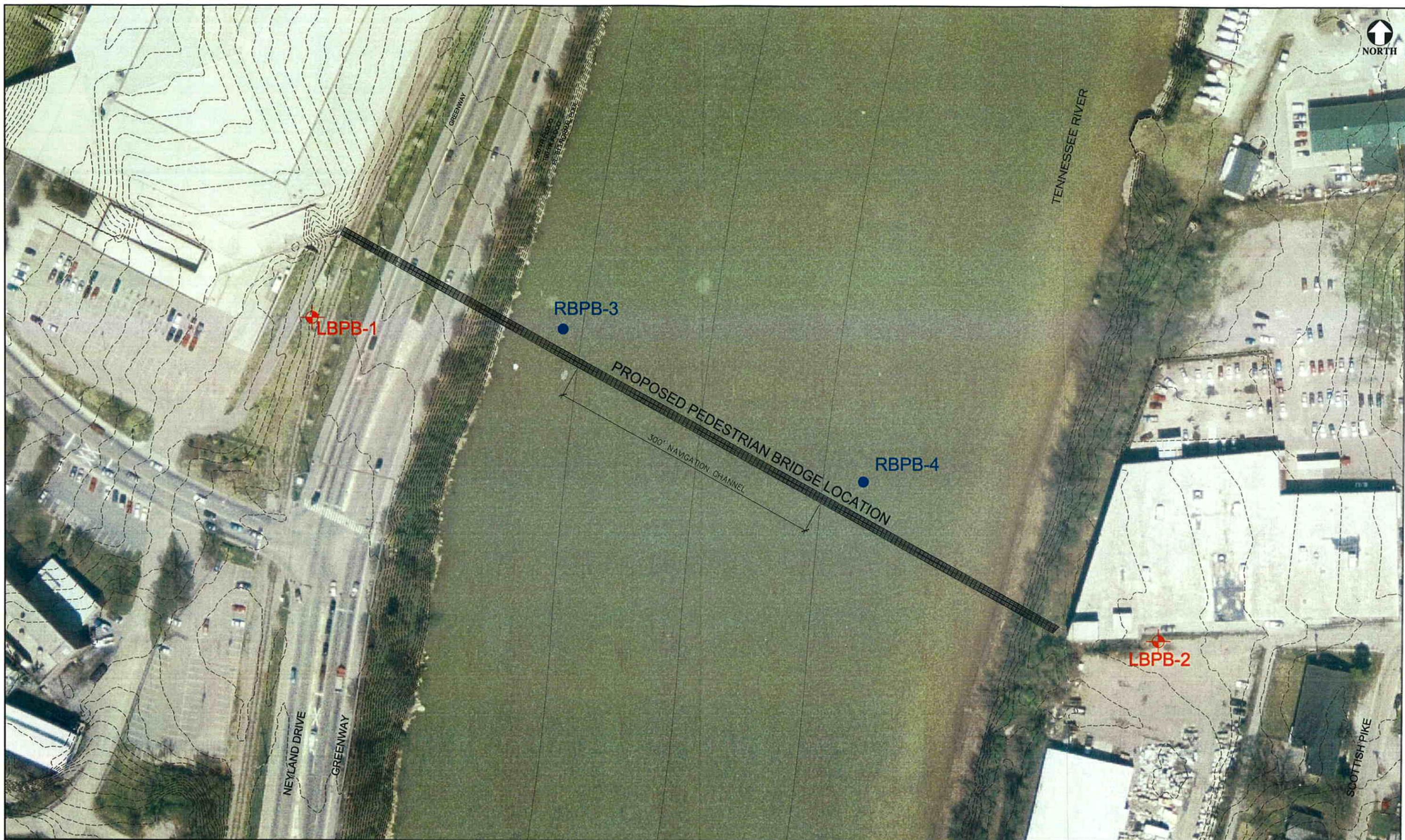
CITY OF KNOXVILLE
SOUTH WATERFRONT
DEVELOPMENT DEPARTMENT
CITY-COUNTY BUILDING, 400 MAIN STREET
KNOXVILLE, TENNESSEE

 **MACTEC**
9725 Cogdill Road
Knoxville, TN 37932
Phone: 865-588-8544 Fax: 865-588-8026

SITE LOCATION MAP
KNOXVILLE SOUTH WATERFRONT
PEDESTRIAN BRIDGE
KNOXVILLE, TENNESSEE

PROJECT NO. 3043-08-1018

FIGURE 1



LEGEND

RBPB-3 ● APPROXIMATE RIVER BORING LOCATION

LBPB-1 ⊕ APPROXIMATE LAND BORING LOCATION

NOTE: THIS DRAWINGS WAS ADAPTED FROM A DRAWING PROVIDED BY HARGREAVES ASSOCIATES INC.

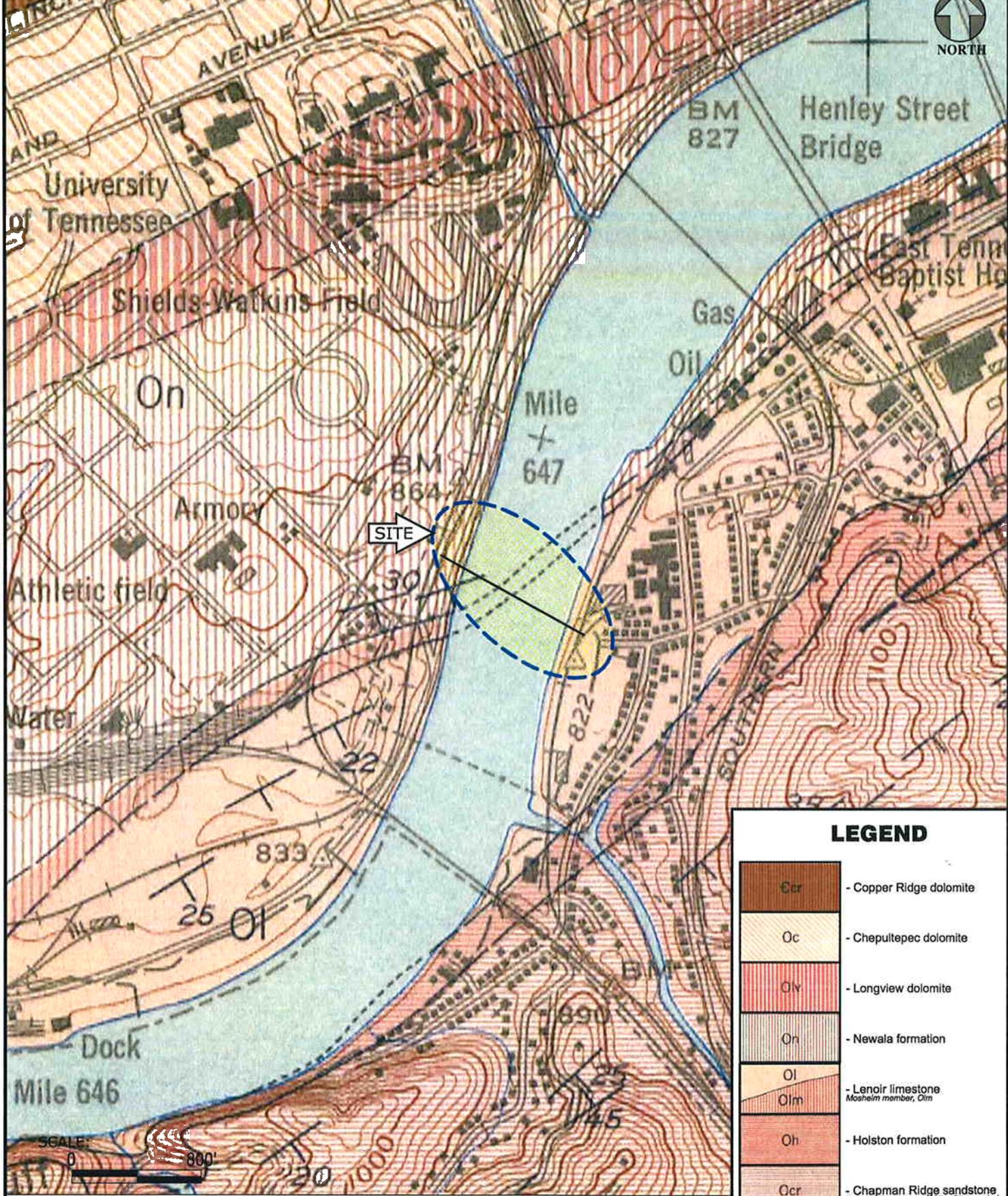
DRAFTING BY & DATE
RSE 09 Jul 2009
PREPARED BY & DATE
GTE 7/24/09
CHECKED BY & DATE
HAB 7/24/09

CITY OF KNOXVILLE
 SOUTH WATERFRONT
 DEVELOPMENT DEPARTMENT
 CITY-COUNTY BUILDING, 400 MAIN STREET
 KNOXVILLE, TENNESSEE

MACTEC
 MACTEC Engineering and Consulting, Inc.
 9725 Cogdill Road
 Knoxville, Tennessee 37932
 865-588-8544

BORING LOCATION PLAN
 FOR PROPOSED PEDESTRIAN BRIDGE
 WATERFRONT PROPERTIES
 KNOXVILLE, TENNESSEE

SCALE: 0 100'
PROJECT NUMBER: 3043-08-1018
FIGURE 2 CADD FILE: 3043081018_Agbs.psd brtpg.dwg
PLOT DATE: Thu, 09 Jul 2009



LEGEND

Ccr	- Copper Ridge dolomite
Oc	- Chepultepec dolomite
Olv	- Longview dolomite
On	- Newala formation
Ol	- Lenoir limestone
Olm	- Lenoir limestone <i>Mosheim member, Olm</i>
Oh	- Holston formation
Ocr	- Chapman Ridge sandstone <i>Includes lower tongue of the Ottosea shale</i>

PREPARED BY: *[Signature]* DATE: 09 Jul 2009 CHECKED BY: *[Signature]* DATE: 09 Jul 2009

CITY OF KNOXVILLE
SOUTH WATERFRONT
DEVELOPMENT DEPARTMENT
CITY-COUNTY BUILDING, 400 MAIN STREET
KNOXVILLE, TENNESSEE

MACTEC
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SITE GEOLOGY
KNOXVILLE SOUTH WATERFRONT
PEDESTRIAN BRIDGE
KNOXVILLE, TENNESSEE
PROJECT NO. 3043-08-1018 **FIGURE 3**

APPENDIX A

FIELD EXPLORATORY PROCEDURES

FIELD EXPLORATORY PROCEDURES

Soil Test Boring (Hollow Stem)

All boring and sampling operations were conducted in general accordance with ASTM D 1586. The borings were advanced by mechanically turning continuous steel hollow-stem auger flights into the ground. At regular 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 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 required to drive the sampler the final foot of penetration was recorded and is designated the "standard penetration test (SPT) resistance." Proper evaluation of the penetration resistance provides an index to the soil's strength, density, and ability to support foundations.

Representative portions of the soil samples obtained from the split-tube sampler were sealed in glass jars and transported to our laboratory, where they were examined by our engineer to verify the driller's field classifications. Test Boring Records are attached, graphically showing the soil descriptions and penetration resistances.

Boring Backfill

The borings were backfilled shortly after drilling for safety purposes. We backfilled the borings with auger cuttings. If necessary, borings were patched with asphaltic concrete cold mix or Portland cement in paved areas.

You are advised that, even with this backfill technique, there is the possibility of future borehole subsidence depending on actual subsurface conditions, surface drainage, etc. The property owner should monitor the boring locations over time to discover subsidence and make any necessary repairs.

Rock Coring

Prior to coring, casing is set in the hole drilled through the overburden soils, if necessary, to keep the hole from caving. Refusal materials are then cored according to ASTM D 2113, using a diamond-studded bit fastened to the end of a hollow, double-tube core barrel. This device is rotated at high

speeds, and the cuttings are brought to the surface by circulating water. Core samples of the material penetrated are protected and retained in the swivel-mounted inner tube. Upon completion of each core run, the core barrel is brought to the surface, the core recovery is measured, the samples are removed, and the core is placed in boxes for transportation and storage.

The core samples are returned to the laboratory where the refusal material is identified, and the percent core recovery and rock quality designation are determined by a soils engineer or geologist. The percent core recovery is the ratio of the sample length obtained to the depth drilled, expressed as a percent. The rock quality designation (RQD) is obtained by summing up the length of core recovered, including only the pieces of core that are 4 inches or longer, and divided by the total length drilled. The percent core recovery and RQD are related to the soundness and continuity of the refusal material. Refusal material descriptions, recoveries, and the bit size used are shown on the "Test Boring Records."

The NQ and NX sizes designate bits that obtain rock cores 1-7/8 and 2-1/8 inches in diameter, respectively.

Undisturbed Sampling

The relatively undisturbed samples were obtained by pushing a section of 3-inch O.D., 16-gauge steel tubing into the soil at the desired sampling level. The sampling procedure is described by ASTM D-1587. The tube, together with the encased soils, was carefully removed from the ground, made airtight, and transported to our laboratory.

APPENDIX B

KEY TO SYMBOLS AND DESCRIPTIONS

SOIL TEST BORING RECORDS

GROUP SYMBOLS	TYPICAL NAMES	GROUP SYMBOLS	TYPICAL NAMES	Undisturbed Sample 1.5-2.0 = Recovered (ft) / Pushed (ft)
	TOPSOIL		CONCRETE	Split Spoon Sample Auger Cuttings
	ASPHALT		DOLOMITE	Rock Core 60-100 = RQD / Recovery Dilatometer
	GRAVEL		LIMESTONE	No Sample Crandall Sampler
	FILL		SHALE	Rotary Drill Pressure Meter
	SUBSOIL		LIMESTONE/SHALE- Limestone with shale interbeds	Water Table at time of drilling No Recovery
	ALLUVIUM		SANDSTONE	Water Table after 24 hours
	COLLUVIUM		SILTSTONE	
	RESIDUUM - Soft to firm		AUGER BORING	
	RESIDUUM - Stiff to very hard		UNDISTURBED SAMPLE ATTEMPT	

Correlation of Penetration Resistance with Relative Density and Consistency

SAND & GRAVEL		SILT & CLAY	
No. of Blows	Relative Density	No. of Blows	Consistency
0 - 4	Very Loose	0 - 2	Very Soft
5 - 10	Loose	3 - 4	Soft
11 - 20	Firm	5 - 8	Firm
21 - 30	Very Firm	9 - 15	Stiff
31 - 50	Dense	16 - 30	Very Stiff
Over 50	Very Dense	31 - 50	Hard
		Over 50	Very Hard

KEY TO SYMBOLS AND DESCRIPTIONS

BOUNDARY CLASSIFICATIONS: Soils possessing characteristics of two groups are designated by combinations of group symbols.

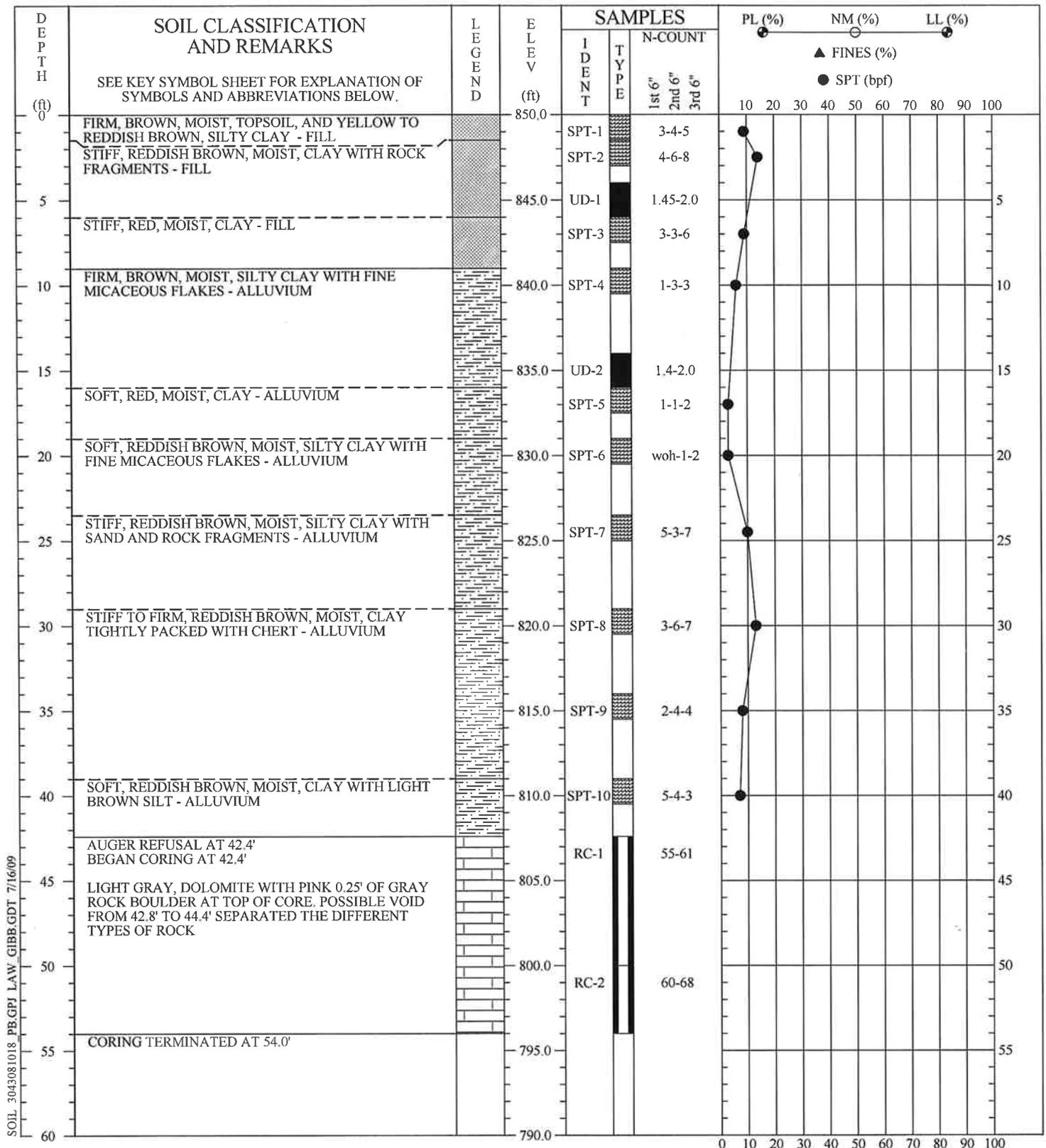
SILT OR CLAY	SAND			GRAVEL			Cobbles/Boulders
	Fine	Medium	Coarse	Fine	Coarse		
	No.200	No.40	No.10	No.4	3/4"	3"	12"

U.S. STANDARD SIEVE SIZE



MACTEC Engineering and Consulting of Georgia, Inc.
9725 Cogdill Road
Knoxville, Tennessee 37932
865-588-8544 • Fax: 865-588-8026

Reference: The Unified Soil Classification System, Corps of Engineers, U.S. Army Technical Memorandum No. 3-357, Vol. I, March, 1953 (Revised April, 1960)



SOIL 3043081018 PB.GPJ LAW_GIBB.GDT 7/16/09

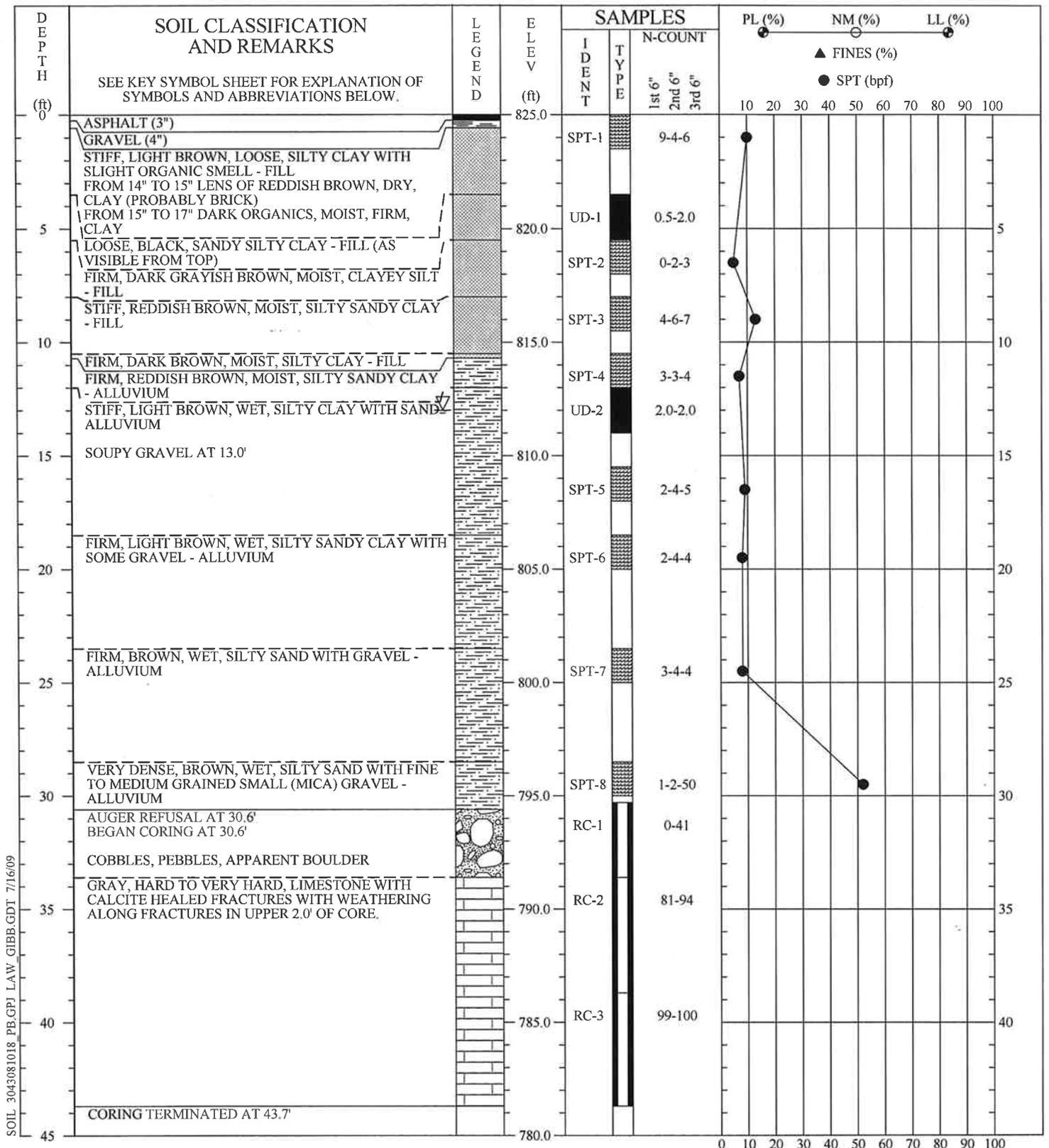
REMARKS: STANDARD PENETRATION RESISTANCE TESTING PERFORMED USING AN AUTOMATIC HAMMER. NO GROUND WATER ENCOUNTERED AT TIME OF EXPLORATION.

SOIL TEST BORING RECORD	
PROJECT: Knoxville South Waterfront - Pedestrian Bridge	BORING NO.: LBPB-1
DRILLED: June 15, 2009	
PROJ. NO.: 3043-08-1018	PAGE 1 OF 1

THIS RECORD IS A REASONABLE INTERPRETATION OF SUBSURFACE CONDITIONS AT THE EXPLORATION LOCATION. SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND AT OTHER TIMES MAY DIFFER. INTERFACES BETWEEN STRATA ARE APPROXIMATE. TRANSITIONS BETWEEN STRATA MAY BE GRADUAL.

Driller : Tri-State
 Logged By: G. T. Z.
 Checked By: H.A.B.





REMARKS: STANDARD PENETRATION RESISTANCE TESTING PERFORMED USING AN AUTOMATIC HAMMER.

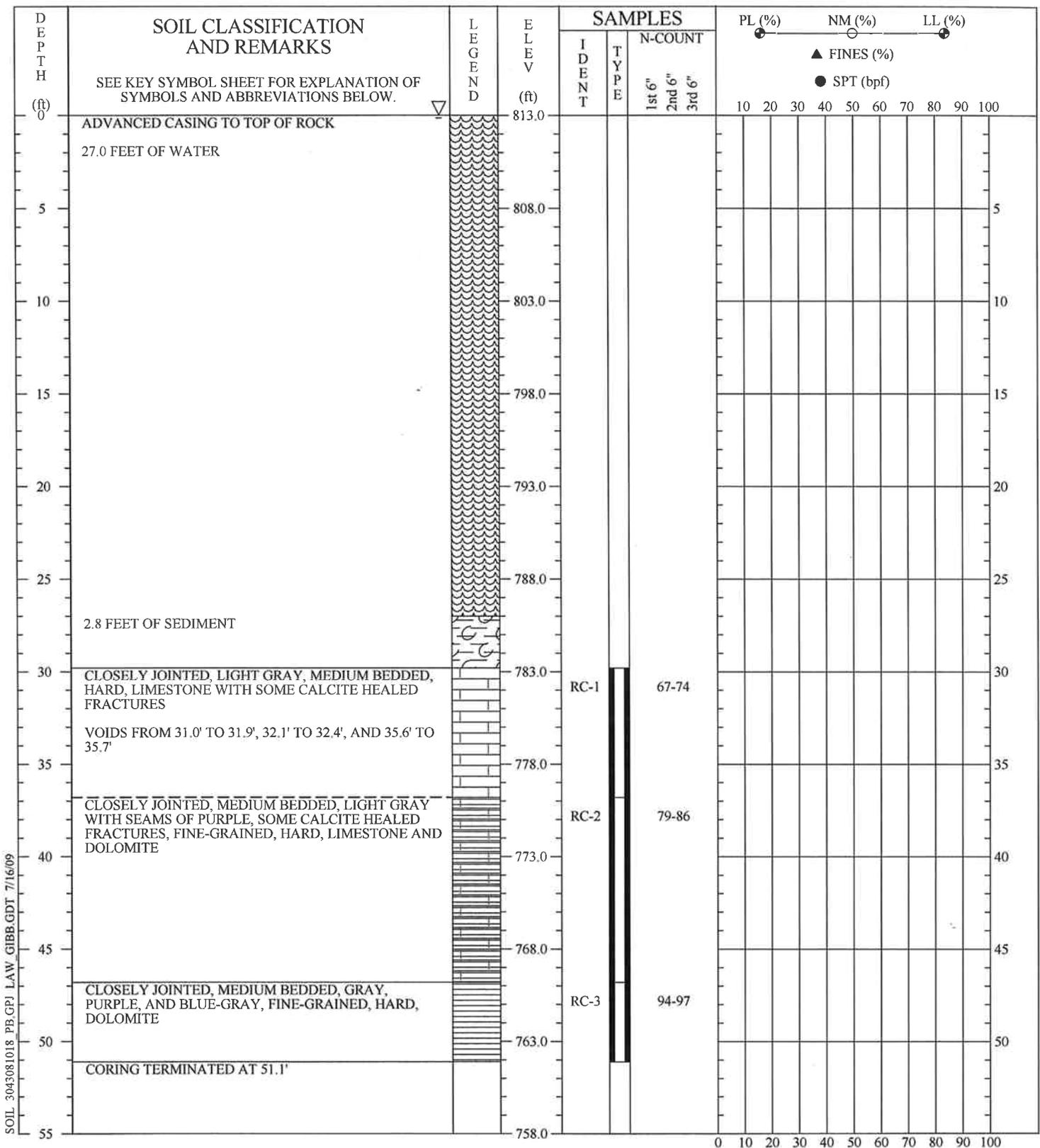
SOIL TEST BORING RECORD

PROJECT: Knoxville South Waterfront - Pedestrian Bridge
DRILLED: June 12, 2009 **BORING NO.:** LBPB-2
PROJ. NO.: 3043-08-1018 **PAGE 1 OF 1**

THIS RECORD IS A REASONABLE INTERPRETATION OF SUBSURFACE CONDITIONS AT THE EXPLORATION LOCATION. SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND AT OTHER TIMES MAY DIFFER. INTERFACES BETWEEN STRATA ARE APPROXIMATE. TRANSITIONS BETWEEN STRATA MAY BE GRADUAL.

Driller : Tri-State
 Logged By: J. E. S.
 Checked By: H.A.B.





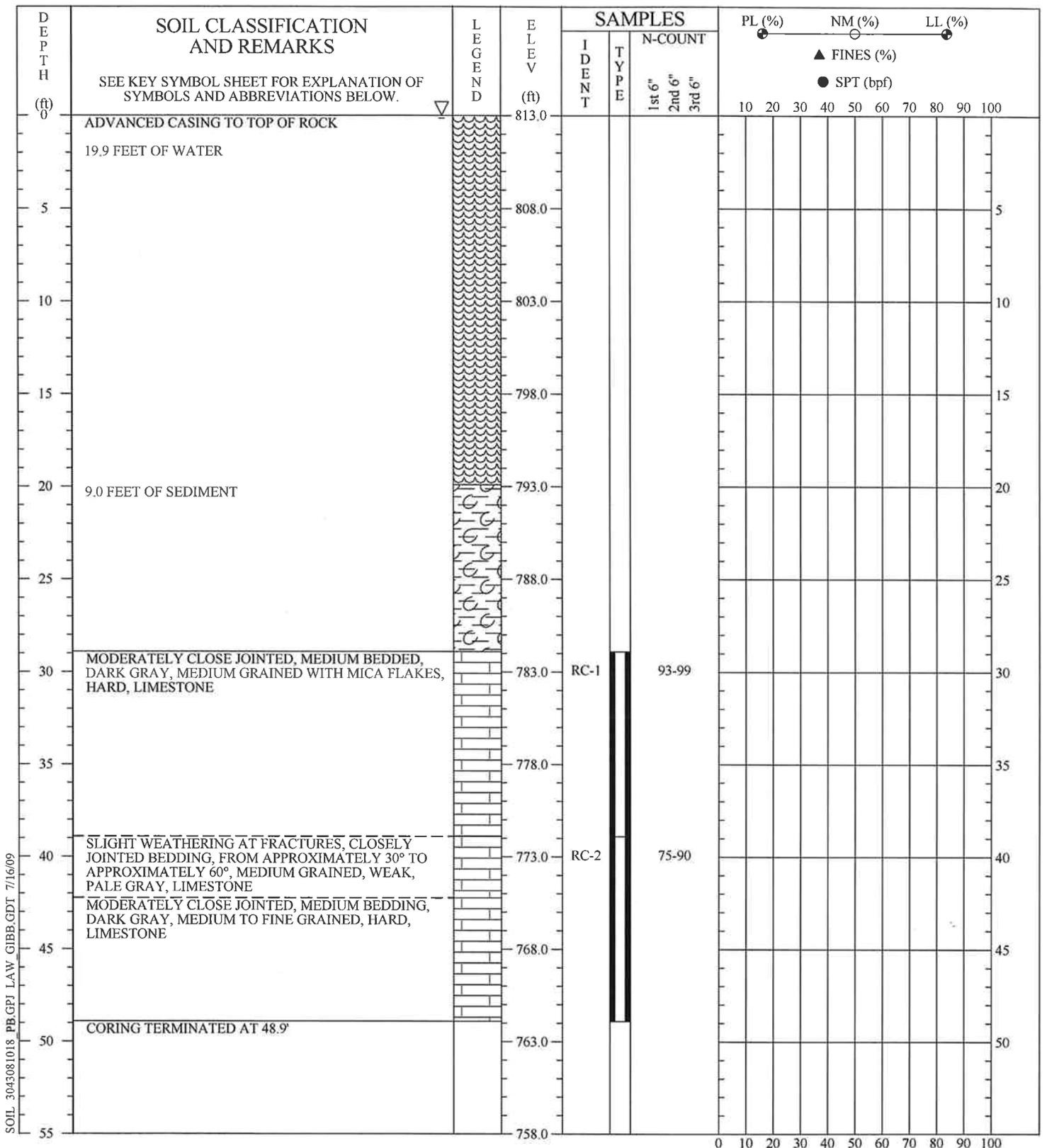
SOIL 3043081018 PB.GPJ LAW_GIBB.GDT 7/16/09

REMARKS: DEPTHS WERE MEASURED FROM WATER SURFACE. RESERVOIR WATER ELEVATIONS FLUCTUATE, ELEVATIONS SHOWN REPRESENT AN APPROXIMATE ELEVATION FOR THE DATE DRILLED.

THIS RECORD IS A REASONABLE INTERPRETATION OF SUBSURFACE CONDITIONS AT THE EXPLORATION LOCATION. SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND AT OTHER TIMES MAY DIFFER. INTERFACES BETWEEN STRATA ARE APPROXIMATE. TRANSITIONS BETWEEN STRATA MAY BE GRADUAL.

Driller : Tri-State
 Logged By: J. E. T.
 Checked By: H.A.B.

SOIL TEST BORING RECORD	
PROJECT: Knoxville South Waterfront - Pedestrian Bridge	
DRILLED: June 19, 2009	BORING NO.: RBPB-3
PROJ. NO.: 3043-08-1018	PAGE 1 OF 1
	



SOIL 3043081018 RBPB GPJ LAW GIBB GDT 7/16/09

REMARKS: DEPTHS WERE MEASURED FROM WATER SURFACE. RESERVOIR WATER ELEVATIONS FLUCTUATE, ELEVATIONS SHOWN REPRESENT AN APPROXIMATE ELEVATION FOR THE DATE DRILLED.

SOIL TEST BORING RECORD	
PROJECT: Knoxville South Waterfront - Pedestrian Bridge	BORING NO.: RBPB-4
DRILLED: June 22, 2009	PROJ. NO.: 3043-08-1018
PAGE 1 OF 1	

THIS RECORD IS A REASONABLE INTERPRETATION OF SUBSURFACE CONDITIONS AT THE EXPLORATION LOCATION. SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND AT OTHER TIMES MAY DIFFER. INTERFACES BETWEEN STRATA ARE APPROXIMATE. TRANSITIONS BETWEEN STRATA MAY BE GRADUAL.

Driller : Tri-State
 Logged By: J. E. T.
 Checked By: H.A.B.



APPENDIX C

LABORATORY TEST PROCEDURES

LABORATORY TEST RESULTS

LABORATORY TEST PROCEDURES

Atterberg Limits

Originally, the Atterberg Limits consisted of seven "limits of consistency" of fine-grained soils. In current engineering usage, the term usually refers only to the liquid limit (LL) and plastic limit (PL). The LL (between the liquid and plastic states) is the water content at which a trapezoidal groove of specified shape, cut in moist soil held in a special cup, is closed after 25 taps on a hard rubber plate. The PL (between plastic and semi-solid states) is the water content at which the soil crumbles when rolled into threads of 1/8 inch in diameter.

The LL has been found to be proportional to the compressibility of the normally consolidated soil. The PI is the calculated difference in water contents between the LL and the PL. Together the LL and PI are used to classify silts and clays according to the Unified Soil Classification System (ASTM D 2487). The PI is used to predict the potential for volume changes in confined soils beneath foundations or grade slabs. The LL, PL, and PI are determined in accordance with ASTM D 4318.

Moisture Content

The moisture content in a given mass of soil is the ratio, expressed as a percentage, of the weight of the water to the weight of the solid particles. This test was conducted in accordance with ASTM D 2216.

Unconfined Compression of Soil

The unconfined compression test is an unconsolidated-undrained shear test with no lateral confining pressure. This test is used to determine the undrained shear strength of a clayey soil. An unconfined compression test is conducted in general accordance with ASTM D 2166 on a single section of an undisturbed sample extruded from a sampling tube. The sample is trimmed to a length of between 2- and 2-1/2-times its diameter and placed in the testing device. The sample is loaded at a constant strain rate until the sample fails. Strain measurements are made during the testing on some samples and the results are plotted and reported as stress-strain curves. The results from our unconfined compression tests are provided in this report.

Unconfined Compression of Rock Core

The unconfined compression test of rock core is performed using a concrete compression test machine. The rock core is prepared so that it has a diameter-to-height ratio of at least 1 to 2. The ends of the core are then capped with capping compound. The prepared core sample is then tested to failure in compression, and the compressive strength is calculated.

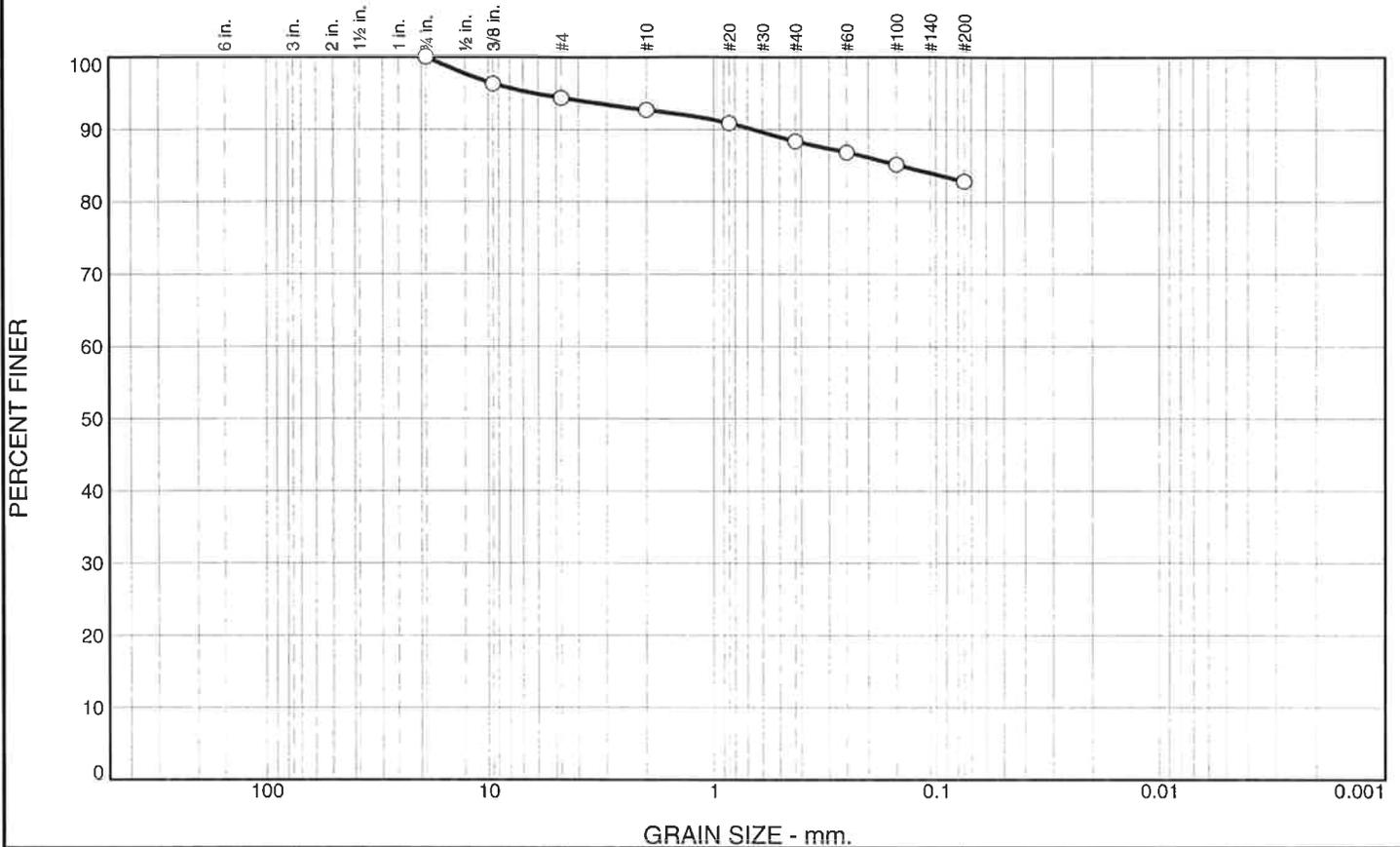
Grain Size Distribution

Grain Size Tests are performed to aid in determining the soil classification and the grain size distribution. The soil samples are prepared for testing according to ASTM D 421 (dry preparation) or ASTM D 2217 (wet preparation). If only the grain size distribution of soils coarser than a number 200 sieve (0.074-mm opening) is desired, the grain size distribution is determined by washing the sample over a number 200 sieve and, after drying, passing the samples through a standard set of nested sieves. If the grain size distribution of the soils finer than the number 200 sieve is also desired, the grain size distribution of the soils coarser than the number 10 sieve is determined by passing the sample through a set of nested sieves. Materials passing the number 10 sieve are dispersed with a dispersing agent and suspended in water, and the grain size distribution calculated from the measured settlement rate of the particles. These tests are conducted in accordance with ASTM D 422.

MACTEC, Inc.
Natural Moisture Content Test Results
South Knoxville Waterfront
Project Number 3043081018.01

Boring Number	Sample Number	Sample Type	Sample Depth (Feet bgs)	Moisture Content (%)
LBPB-1	2	SPT	1.5-3.0	21.8
LBPB-1	3	SPT	6.0-7.5	26.6
LBPB-1	5	SPT	16.0-17.5	33.0
LBPB-2	2	SPT	5.5-7.0	30.6
LBPB-2	4	SPT	10.5-12.0	23.5
LBPB-2	6	SPT	18.5-20.0	21.9
SPT - Standard Penetration Test				
Prepared By <u>RES</u> Date <u>7-16-09</u> Checked By <u>JL</u> Date <u>7-16-09</u>				

Particle Size Distribution Report



% Cobbles	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	5.6	1.7	4.3	5.6	82.8	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
.75	100.0		
.375	96.3		
#4	94.4		
#10	92.7		
#20	90.8		
#40	88.4		
#60	86.8		
#100	85.1		
#200	82.8		

Material Description

Orange brown fat clay with sand

Atterberg Limits

PL= 26 LL= 67 PI= 41

Coefficients

D₉₀= 0.6664 D₈₅= 0.1446 D₆₀=
D₅₀= D₃₀= D₁₅=
D₁₀= C_u= C_c=

Classification

USCS= CH AASHTO= A-7-6(37)

Remarks

DNS - Data Not Submitted; NT - No Test

* (no specification provided)

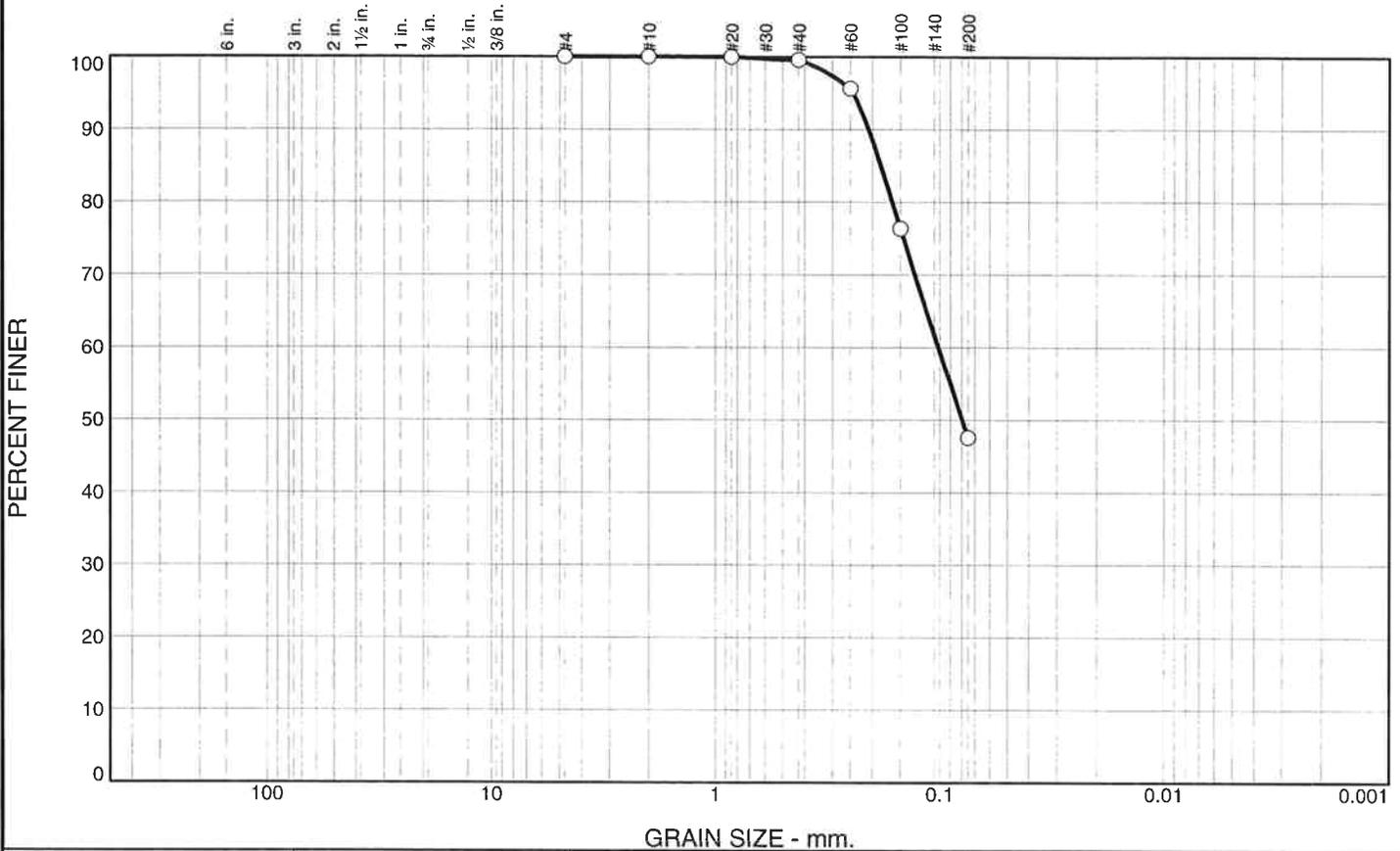
Location: Boring LBPB-1 Depth: 14-16'
Sample Number: UD-2

Date: 7/16/2009

MACTEC Engineering and Consulting, Inc.	Client: City of Knoxville Project: South Knoxville Waterfront	
Knoxville, TN	Project No: 3043081018	Figure LBPB-1

Tested By: BJ 7-16-09 Checked By: HAB 7/24/09

Particle Size Distribution Report



% Cobbles	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.0	0.4	52.0	47.6	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#10	100.0		
#20	100.0		
#40	99.6		
#60	95.6		
#100	76.4		
#200	47.6		

Material Description

Brown alluvial silty sand

Atterberg Limits

PL= NP LL= NV PI= NP

Coefficients

D₉₀= 0.2077 D₈₅= 0.1830 D₆₀= 0.1019
 D₅₀= 0.0797 D₃₀= D₁₅=
 D₁₀= C_u= C_c=

Classification

USCS= SM AASHTO= A-4(0)

Remarks

DNS - Data Not Submitted; NT - No Test; UD - UnDistrubed Tube

* (no specification provided)

Location: Boring LBPB-2
Sample Number: UD-2 **Depth:** 12-14'

Date: 7/16/2009

MACTEC Engineering and Consulting, Inc. Knoxville, TN	Client: City of Knoxville Project: South Knoxville Waterfront Project No: 3043081018
Figure LBPB-2, UD2	

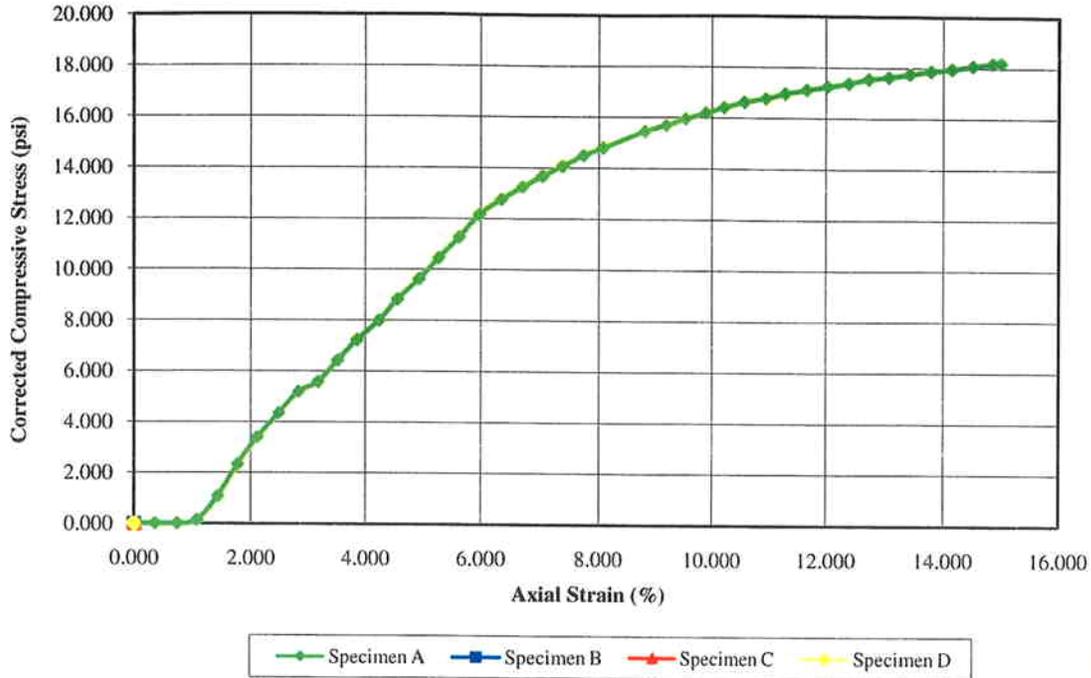
Tested By: RSJ 7-16-09 **Checked By:** HAB 7/24/09

MACTEC, Inc
Unconfined Compression Test Report (ASTM D2166)

Date 7/16/09

Checked By HAB

Compressive Stress Axial Strain Curve



Date 7.14.09

Tested By JLH

Before Test	Specimen			
	A	B	C	D
Water Content (%)	25.84			
Dry Density (pcf)	99.311			
Saturation (%)	98.01			
Void Ratio	0.72			
Diameter (in)	2.823			
Height (in)	5.577			
Test Data	A	B	C	D
Unconfined Strength (ksf)	2.62			
Undrained Shear Strength (tsf)	0.09			
Undrained Shear Strength (ksf)	1.31			
Rate of Strain (in/min)	0.080000			
Description		Project Information		
Project Num	3043081018	Specimen A	Moist stiff silty orange clay	
Project	South Knoxville Waterfront	Specimen B		
Sampling Date	06/15/09	Specimen C		
Sample #	LBPB-1: UD-1: 4'-6'	Specimen D		
Client	City of Knoxville	Test Variables		
		Specific Gravity	2.74	
		Liquid Limit:		
		Plastic Limit:		

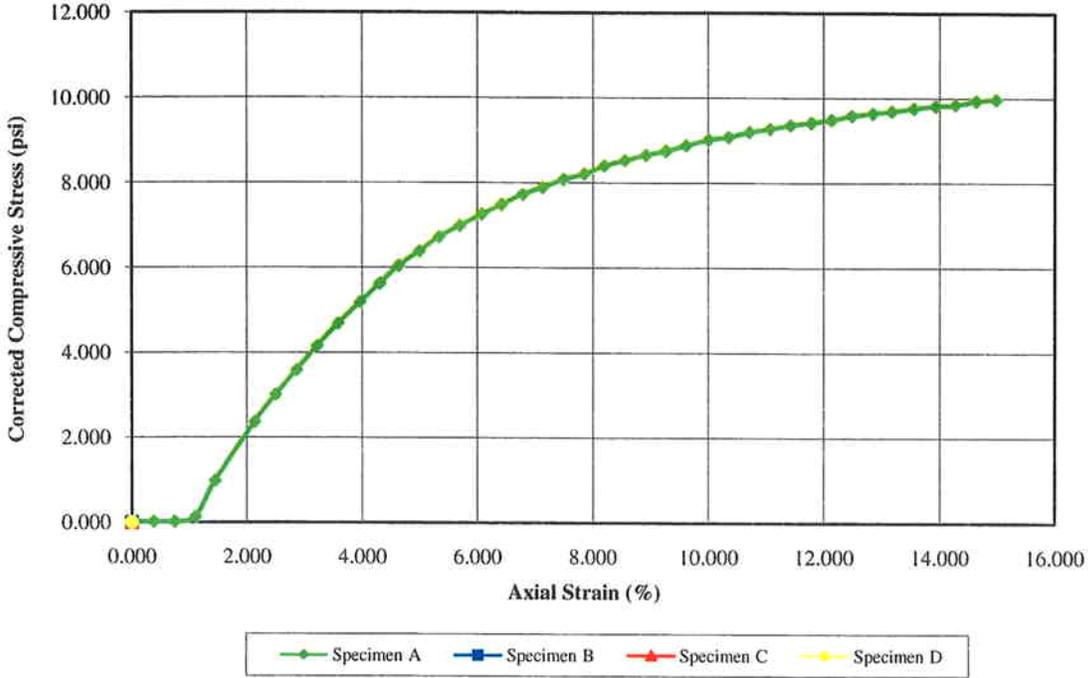
Remarks Sample was uniform throughout

MACTEC, Inc
Unconfined Compression Test Report (ASTM D2166)

Date 7/16/09

Checked By HABS

Compressive Stress Axial Strain Curve



Before Test	Specimen			
	A	B	C	D
Water Content (%)	33.25			
Dry Density (pcf)	90.301			
Saturation (%)	100.00			
Void Ratio	0.89			
Diameter (in)	2.823			
Height (in)	5.520			
Test Data	A	B	C	D
Unconfined Strength (ksf)	1.44			
Undrained Shear Strength (tsf)	0.05			
Undrained Shear Strength (ksf)	0.72			
Rate of Strain (in/min)	0.080000			

Date 7.14.09

Project Information		Specimen Description	
Project Num	3043081018	Specimen A	Moist soft orange clay
Project	South Knoxville Waterfront	Specimen B	
Sampling Date	06/15/09	Specimen C	
Sample #	LBPB-1: UD-2: 14'-16'	Specimen D	
Client	City of Knoxville	Test Variables	
		Specific Gravity	2.74
		Liquid Limit:	
		Plastic Limit:	

Tested By JLH

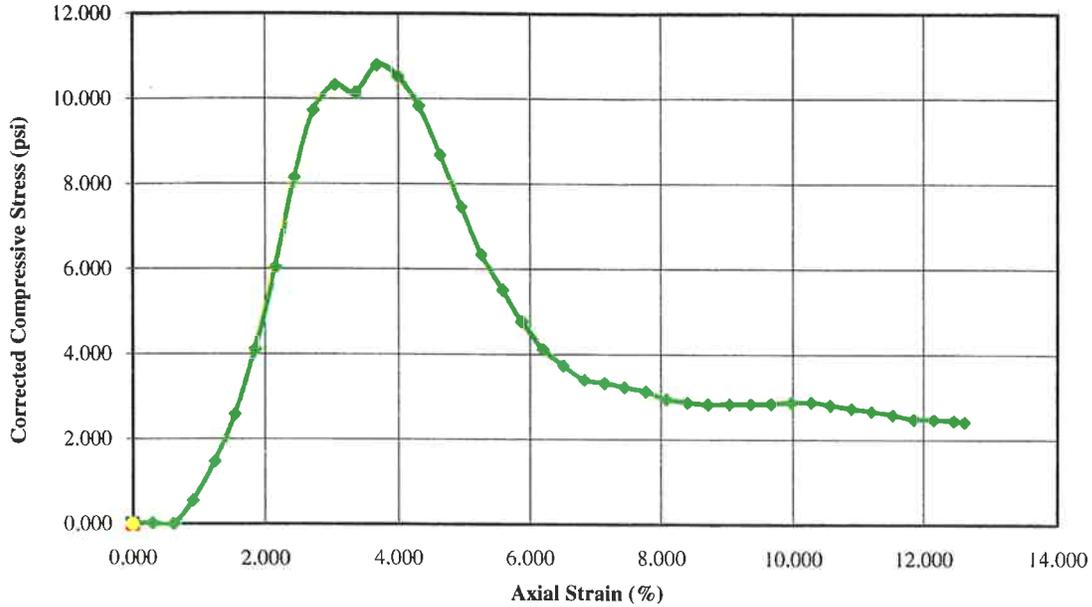
Remarks

MACTEC, Inc
Unconfined Compression Test Report (ASTM D2166)

Date 7/16/09

Checked By AAB

Compressive Stress Axial Strain Curve



◆ Specimen A
 ■ Specimen B
 ◆ Specimen C
 ◆ Specimen D

Before Test	Specimen			
	A	B	C	D
Water Content (%)	22.41			
Dry Density (pcf)	106.347			
Saturation (%)	100.00			
Void Ratio	0.61			
Diameter (in)	2.840			
Height (in)	5.553			
Test Data	A	B	C	D
Unconfined Strength (ksf)	1.55			
Undrained Shear Strength (tsf)	0.06			
Undrained Shear Strength (ksf)	0.78			
Rate of Strain (in/min)	0.080000			
Description				
Project Information		Specimen Description		
Project Num	3043081018	Specimen A	Moist, stiff brown sandy silty clay	
Project	South Knoxville Waterfront	Specimen B		
Sampling Date	6/12/09	Specimen C		
Sample #	LBPB-2: UD-2 : 12'-14'	Specimen D		
Client	City of Knoxville	Test Variables		
		Specific Gravity	2.74	
		Liquid Limit:		
		Plastic Limit:		

Date 7-14-09

Tested By JLH

Remarks