Connection of Garden City Greenway to Roanoke River Greenway
ITB# 16-06-01

Questions and Answers

1. Is there a specification for the illuminated sign-no turn on red?
   A.

2. Can the Right Turn lane be closed?
   A. Yes, provided that two-way traffic can continue on the roadway.

3. When do you anticipate issuing a Notice to Proceed.
   A. The Notice to Proceed will depend on the speed of execution of the Contract. Notice to Proceed may tentatively occur in May, possibly earlier.

4. Is there any concern about Log Perch?
   A. There are no environmental restrictions in place for this project.

5. What are the requirements for abutment?
   A. Please refer the contractors to Section 401, Structure Excavation, of the VDOT Road and Bridge Specifications section (i) backfilling and City Specifications Section 02315, Fill and Backfill. In summary, fill around the perimeter of abutments shall be placed in 6-8 inch layers to a density of 95%. No provision for drainage since we do not have weep holes in the abutment walls. Cost for backfill is included in the price for the concrete bridge abutments, see Measurement and Payment Section 01200.

6. What are the requirements for road restoration?
   A. Standard City restoration requirements apply.

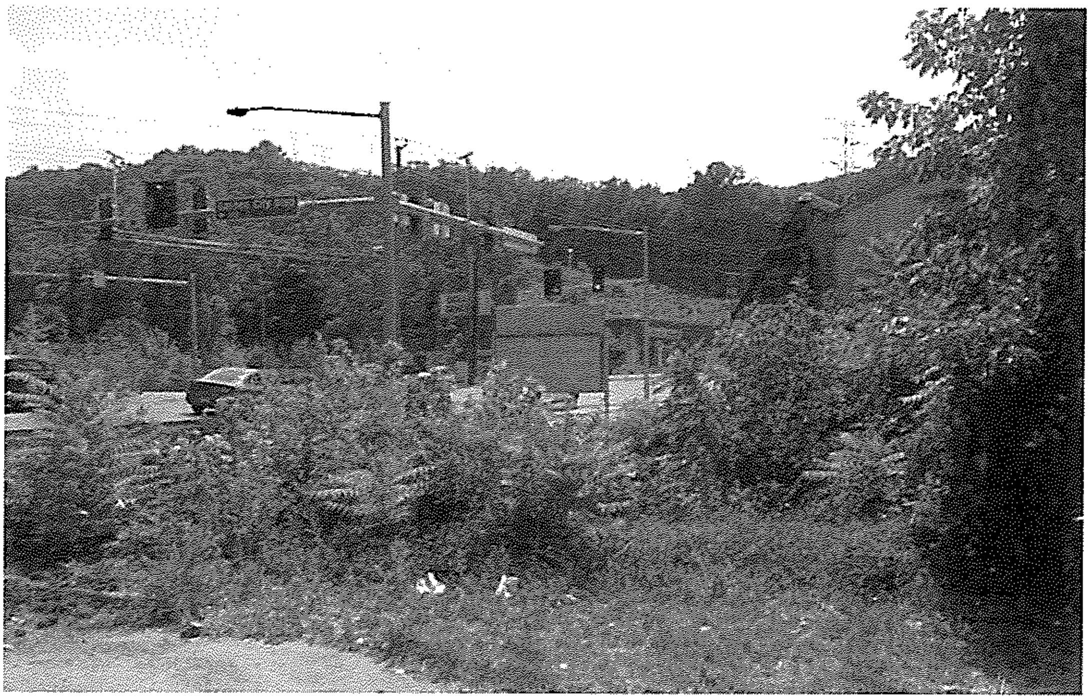
7. Has the City specified paint or thermo?
   A. Thermo

8. Is there a project specific geotechnical report available?
   A. The City originally proposed the bridge on the opposite side of Riverland Road and have a report from that location. We estimate field conditions are similar at the current location for the bridge. See the attached prior geotechnical report, but be mindful it was a different location. Competent rock is anticipated at 30'. The bid form estimate of 600 lf in overburden is representative of that expectation.

9. What is the bridge length?
   A. The plan measures 42.85' however, this is subject to field conditions and the final length of the bridge will be determined once the abutments are set and the field measurements are taken.

GEOTECHNICAL INVESTIGATION AND EVALUATION

Garden City Greenway Connection Project
Roanoke, Virginia



Prepared for

Priscilla Cygielnik
Roanoke Engineering
215 Church Avenue, S.W.
Room 350
Roanoke, VA 24011

CTI Job No. 11G-1236

July, 2014

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Blacksburg, Virginia 24060
Tel: (540) 552-1575
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Email: cnewman@cti-consultants.com

July 27, 2014

Roanoke Engineering
215 Church Avenue, S.W.
Room 350
Roanoke, VA 24011

Attention: Priscilla Cygielnik

Reference: Limited Geotechnical Investigation and Evaluation
Garden City Greenway Connection Project – Pedestrian Bridge, 1400 Riverland Road SE
Roanoke, Virginia
CTI Project No.: 11G-1236

Ms. Cygielnik:

CTI Consultants, Inc. is pleased to submit this Limited Geotechnical Investigation and Evaluation for the above referenced project. These services were provided in accordance with CTI Proposal No. P-11G-610-050714 dated May 7, 2014. Purchase order CT530SCI40515001442-1 was issued as approval of the proposal.

The following report describes the project characteristics, subsurface exploratory program, data obtained from the program, along with the results of our geotechnical evaluations and recommendations. Soil samples obtained from the study will be stored in our laboratory for 90 days, after which time they will be discarded unless you request otherwise.

CTI appreciates the opportunity to provide geotechnical services for you, and will remain available for further consultation during the design and construction of this project.
Should you have any questions concerning this report, or require additional consultation, construction inspection, or testing services then contact our office at (540) 552-1575.

Sincerely,

CTI CONSULTANTS, INC.

COMMONWEALTH OF VIRGINIA
CHARLES J. NEWMAN
Lic. No. 40732
PROFESSIONAL ENGINEER

Digitally signed by Charles J Newman
Date: 2014.07.28 07:37:18 -04'00'

Chuck Newman, P.E.
Principal Engineer

Garden City Greenway Connection Project

Pedestrian Bridge at 1400 Riverland Rd SE

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1.0 EXECUTIVE SUMMARY

CTI has completed a limited geotechnical investigation for the pedestrian bridge greenway connection near the intersection of Riverland Road and Garden City Boulevard in Roanoke, Virginia. The bridge will cross a small continuously flowing creek that is located roughly seven feet below the adjacent roadway and bounded by relatively riprap embankments that are overgrown and brushy. There are a significant number of existing and abandoned utilities in the area of the abutments, which are adjacent to existing paved parking lots. He deck will have a trail width of 10 feet and a conceptual span on the order of 25 feet. Specific loading was not provided, but the analysis in this report assumed 250 psf for determination of abutment support criteria.

Three borings were conducted with an intended termination depth of 20 feet, although one of the borings encountered shallower refusal on cobbles or bedrock at a depth of 17.5 feet. The borings utilized hollow stem augers to allow for frequent standard penetration testing and sampling of the subsurface profile, which was defined by terrace deposits underlain by granular residuum with existing manmade Fill overburden. Relatively competent weathered rock is anticipated within 30 feet below existing grade. Free moisture was noted during drilling at depths of 6, 14, and 17.5 feet with many of the soil samples being visually classified as moist approaching wet. N-values were highly variable with medium-stiff results near surface that decreased substantially with depth and increasing moisture. The values increased significantly near boring termination in the granular material.

Support of the deck by mass concrete abutments will be problematic due to the depth of bedrock, shallow groundwater, and anticipate poor slope stability. It would be advisable to consider increasing the deck span to allow for shallow foundations at either abutment following soil improvement. Alternatively, deep foundations by means of micropile or driven pile installation would allow for construction of the abutments at their conceptual locations.

Detailed discussion and recommendations regarding these and other topics are included in this report, which must not be separated from this brief executive summary.

2.0 PURPOSE AND SCOPE

At your request, CTI Consultants, Inc. (CTI) performed a limited geotechnical investigation and analysis for the planned pedestrian bridge, which is a portion of the Garden City Greenway Connection Project and will be located at the 1400 block of Riverland Rd SE in Roanoke, Virginia. The investigation was conducted in a manner that would allow for a general description of the subsurface soil conditions along with their suitability to standard construction practices. In order to accomplish the above objectives, the following scope of services was undertaken:

1 Performed three drill rig borings on July 7th, 2014 with standard penetration tests (SPT) and sampling performed in a non-continuous manner.
2 Performed laboratory tests on five representative soil samples to determine pertinent engineering properties.
3 Reviewed and summarized readily available subsurface information for the project site and vicinity.
4 Evaluated the findings of the field investigation in regards to the suitability of the subsurface for the proposed project.
5 Determined general and project specific recommendations regarding geotechnical aspects of development and construction.
6 Prepared a limited geotechnical report detailing the investigation procedures, geologic research findings, subsurface conditions, and geotechnical recommendations in conformance with generally accepted engineering practices for a preliminary site study.

Our proposed scope of services did not include a survey of boring locations, borehole abandonment, preparation of plans or specifications, evaluation of a specific project, structural design, or the identification of environmental aspects of the project site.

3.0 SITE AND PROJECT DESCRIPTION

3.1 Site Location and Conditions

The pedestrian bridge is located on the northern side of Riverland Road SE at its intersection with Garden City Boulevard in Roanoke, Virginia. The bridge will span a small continuously flowing creek that is located roughly seven feet below the adjacent roadway. The channel of the creek is brushy and overgrown with variable and steep slopes that have primarily been stabilized by placement of quarry riprap. The abutment areas border on the adjacent business parking areas, which are relatively flat. Land use in the vicinity is primarily retail with a Piggly Wiggly to the northeast, a CVS to the southwest, and a vacant structure on the western side that previously housed a pharmacy. A substantial quantity of underground and overhead utilities were present in the immediate vicinity of the planned construction and had been marked following a Miss Utility request.

3.2 Project Description

Conceptual bridge plans prepared by the Office of the City Engineer dated 5/1/14 were utilized as a basis for analysis. A Contech "Connector" or equivalent single span steel pedestrian bridge is depicted having a path width of 10 feet and span on the order of 25 feet. The abutments are depicted as mass concrete gravity walls that would be extended and doweled into underlying competent rock. The provided abutment width versus height would only accommodate competent rock depths of less than 15 feet. Grading should be nominal and primarily consist of Fill to match the existing greenway elevation. Abutment loads were not provided, but have been estimated based on 250 psf deck loading, which would be 3,125 plf (width) although the footing load would be notably higher based upon the mass concrete dimensions provided.

4.0 SITE GEOLOGY

The bulk of southwestern Virginia is geologically located in the Valley and Ridge Physiographic Province of the eastern United States. Based upon a review of *Geologic Map of the Garden City (DMME, 2011)*, the subject property is underlain by alluvial fan deposits (Qaf). The formation contains poorly stratified overlapping debris-flow deposits that consist of dark-reddish-brown clay, silt, sand and gravel matrix with well-rounded cobbles and boulders. The project is within a few hundred feet of the Garden City Fault and the geologic map contains a bisecting cross-section roughly 100 feet the east of the site. It depicts relatively thin alluvium on the order of 20 feet, which is underlain by the Erwin (Antietam) Formation (Ꞓe) and Hampton (Harpers) Formation (Ꞓh) of the Chilhowee Group. The Erwin Formation is Medium-gray to pale-yellowish-white, fine- to medium-grained, cross-laminated, very thick bedded, Skolithos-bearing quartzite that forms prominent ledges. It is interbedded with drab, greenish-gray, fine-grained, medium-bedded, sandy to silty, quartz-sericite phyllite. The Hampton Formation is dark-greenish-gray to brownish-gray, quartzchlorite-sericite phyllite and quartzose metasiltstone interbedded with greenish gray, thin- to thick-bedded, metamorphosed lithic sandstone that exhibits spheroidal weathering. Lesser purple to very dark gray, medium- to coarse-grained, ferruginous metasandstone with coarse lithic clasts set in a mica-hematite-chlorite matrix.

Karst formation occurs by the dissolution of carbonate rocks, which are more susceptible to dissolution than other rock types because of the chemical reaction of the carbonates to slightly acidic rain water. The dissolution takes place primarily along bedding planes and joints as water percolates through those features. As the carbonates dissolve, the percolating water carries away the soluble components and leaves behind the insoluble clay minerals and silicates. The remaining soils are often very soft and compressible. The continued dissolution of carbonate rocks can sometimes result in open cavities in the rock.

Carbonate materials solution in water over long periods of time, resulting in loss of rock material. The solution process typically occurs along planes of more soluble material and causes the formation of interconnected seams and cavities within the carbonate formations. The rate of solutioning is also affected by rates of groundwater flow and groundwater chemistry. These seams and cavities are frequently filled with soft material which has not dissolved or materials that have infiltrated into the seam or cavity from above. Sinkholes, or karst features, can result from the collapse of material bridging over the top of caverns formed during the solution process. In some areas, individual sinkholes form in clusters and are separated by narrow ridges of bedrock. Areas having the greatest concentration of karst features are controlled by bedrock stratigraphy and structure as well as proximity to major drainages. Upper Cambrian and Middle Ordovician limestone is more soluble than Cambrian and Lower Ordovician dolomite and shaly dolomite; thus areas of limestone have the greatest number of sinkholes and caves. However, Cambrian and Ordovician dolomite and limestone show enhanced karst development in areas of low bedrock dip; where bedding is intensely folded, cleaved, and jointed; or near major drainages. Typical karst residuum profiles exhibit decreasing strength with depth.

All borings terminated within alluvium or on possible bedrock. Although the bulk of Roanoke is underlain by limestone or dolomite bedrock, the geologic map suggests minimally susceptible sandstone and/or phyllite below the alluvium. Based upon "Selected Karst Features of the Central Valley and Ridge Province, Virginia" (Division of Mineral Resources, 2001), there are several identified sinkholes within a mile of the site, although their frequency in the general vicinity would generally be categorized as low to moderate and associated with nearby carbonate bedrock formations. Based upon this data, it is anticipated that long-term Owner risk relating to karst development will be low.

5.0 FIELD EXPLORATION

5.1 Subsurface Geotechnical Exploration

The subsurface exploration included three drill rig borings that were conducted on July 7th, 2014 by Total Depth Drilling of Knoxville, Tennessee. The borings were performed by a truck-mounted CME rig with a six cylinder gas engine and manual safety hammer. The rig utilized continuous flight hollow stem augers to advance boreholes to a termination depth of 20 feet, although prior refusal on cobbles or rock occurred at one boring. Initially, four borings had been requested with a termination depth of 15 feet each. Based upon a conversation with the City of Roanoke Project Inspector, it was understood that preliminary abutment foundations were intended to reach competent rock, thus the boring quantity was decreased in an effort to better explore the surface of underlying rock. Borings were located by CTI with site features, pacing, and topography utilized to determine the positions depicted in the location plan of Appendix A. All were offset to some degree from the requested locations due to the presence of utilities or site-specific features that prevented access by the drill rig.

Standard penetration tests (SPT) were performed in general accordance with ASTM D1586, Standard Test Method for Penetration Test and Split-Barrel Sampling of Soils. Four split spoon samples were obtained within the first 10 feet while subsequent samples were removed at five foot intervals thereafter. In brief, this procedure is conducted with a two inch outside diameter split barrel sampler driven a distance of 18 inches in advance of the auger by a 140 pound hammer falling 30 inches. The number of blows required to drive the sampler through each six inch increment was recorded in the field logs. The blow count for the final two increments were totaled and termed the SPT N-value, which is indicated on the attached boring logs. The N-value can be used as a qualitative index of relative density of non-cohesive or granular soils. In a more approximate way, the N-value can be used to describe the consistency of cohesive soils. While the SPT test does not allow for direct evaluation of many significant soil parameters, generalized correlations have been empirically developed to allow for soil modulus, undrained friction angle, and shear strength that are needed for development of foundation design parameters. These correlations are primarily based upon the N60 value that is factored to account for hammer efficiency and existing overburden pressure. A correction factor of 1.1 was assumed for the truck rig based upon the use of a manual safety hammer and a split spoon sampler that was free of significant distress. It should be noted that all references to N-value within this report are in terms of field counts.

The soil horizons were categorized as per the Unified Soils Classification System (USCS) with additional notes regarding any soft, moist, or unsuitable soils. The presence and depth of subsurface water was estimated during drilling and measured after completion of each boring. The descriptions and classifications contained within the boring logs of Appendix B were determined by visual observation of a Geotechnical Engineer during drilling unless laboratory testing was also performed.

5.2 Laboratory Analysis

A single representative soil samples was selected for laboratory analysis of its natural moisture and expansion index. The Expansion Index testing conformed to ASTM D4829 while natural moisture content was per ASTM D2216. An additional four samples were analyzed for natural moisture content. Summarized results for the samples are located in section 6.2, with detailed results attached to this report in Appendix C.

6.0 SUBSURFACE CONDITIONS

6.1 *Generalized Subsurface Strata*

The specific descriptions of the subsurface soils and conditions are contained in the boring logs of Appendix B. The transitions between the actual soil strata are less distinct than what is shown in the logs. Although the logs are representative of the subsurface conditions at the borehole locations on the dates shown, they are not necessarily indicative of subsurface conditions at other locations or at other times. A generalized description of typical subgrade materials are summarized in the following paragraphs.

6.1.1 Stratum A: Manmade Fill Soils

Apparent manmade Fill was encountered in each of the borings with a total thickness of 8 – 11 feet. Pavement and aggregate subbase layers were sampled in the existing western parking lot as well as under the relatively recent Fill placed just beyond current termination of the Greenway. The remainder of the Fill consisted of variable soil material with primarily clay or sand classifications along with quarry and natural gravel as well as sporadic organic content and odor. The Fill was not placed in a controlled manner and was erratic in terms of moisture content and consistency. The layered Fill material returned N-values ranging from 0 – 12 blows per foot except when the sample included gravel or pavement material and the majority of blow counts were five or less blows per foot. Free moisture was noted within the Fill at a depth of six feet in boring B-1 near the western abutment. Borings B-2 and B-3 contained layers of moist to very moist soil, but free moisture was not observed prior to the underlying alluvial strata.

Table 1: Summary of Manmade Fill Depths at Boring Locations

Bore	Depth	USCS	Range of N-values
B-1	9.5'	CL/SC	0 – 12
B-2	11'	SC	2 – 9
B-3	8'	SC/GC	3 – 5

It should be noted that an unknown soily substance was encountered in boring B-3 in the range of 6.5 – 8 feet below grade. It was visually and texturally comparable to moistened chalk, although it could not be specifically identified and previously had not been encountered in drilling throughout the project region. While similar in color to natural asbestos, it lacked the fibrous composition, although this was not verified by laboratory analysis.

6.1.2 Stratum B: Alluvium

Alluvial material consisting of layered clay, sand, and gravel continued to boring termination at depths of 17.5 – 20 feet. Free moisture was noted on the drill rods at depths of 14 feet at B-2 and 17.5 feet at B-3. N-values ranged from 1 – 38 blows per foot with the majority of results falling at the extreme ends of the range. The horizon was moist approaching wet, thus samples of Lean Clay ("CL") required only a few blows to penetrate while those with significant gravel content generally needed 30 or more blows for collection. The encountered material was significantly variable despite their proximity as borings B-2 and B-3, which were within 20 feet of each other, contained notably different classifications. This could be due to greater than categorized depths of manmade Fill, but is also relatively common in the vicinity of known faults.

A sample of the alluvium from boring B-3 at a depth of nine feet returned an expansion index result of 41, which would be categorized as low by the test standard, although it is approaching the borderline 50 value of medium expansive potential. The 2009 international building code (IBC) categorizes expansion index values of 20 or more as being prone to shrink-swell behavior, although there is no further classification in regards to the severity of the potential. The natural moisture of the soil sample was approximately equivalent to that which a sample prepared at 50 percent saturation achieved after 24 hours inundation. As such, it is unlikely that a foundation supported by the represented horizon would be subject to heave, but could undergo bearing loss due to shrinkage if ground moisture levels decreased substantially.

6.1.3 Weathered Rock / River Jack

The scope of the field investigation did not include rock coring, thus the nature of refusal at boring B-1 could not be explored. Alluvium is classically underlain by subrounded gravel to cobble sized particles and coarse sand that has cyclically been transported by flooding and current from an upstream source. Generally, the "river jack" layer can be penetrated to some degree by hollow stem augers, although refusal will eventually occur due to encountering a boulder or as a result of the augers skewing as they are diverted by larger particles. This was not true at B-1, thus it appears more likely that refusal occurred on the upper weathered surface of the Quartzose or Sandstone of the geology underlying the alluvium.

6.1.4 Water Level Observations

Free moisture was encountered at each of the borings during drilling at depths of 6, 14, and 17.5 feet. The natural moisture content of alluvial samples ranged from 22.8 – 48.8 percent with values primarily in the thirties, indicating the soils are moist approaching saturated. The manmade Fill was also overly moist, although generally above the elevation of free moisture encountered during drilling. It should be understood that groundwater levels fluctuate as a result of normal variations in precipitation, perched water conditions, surface runoff amounts and other site specific factors that are not always evident. The borings were backfilled upon completion due to safety concerns, thus stabilized levels were not determined.

6.2 Laboratory Test Results

The subsequent table summarizes the results of the representative laboratory samples with detailed data in appendix C. Abbreviations used in the table are defined on the following page.

Table 2: Summary of Physical Laboratory Test Results

Bore	Lab #	Depth	EI	% NM
B-3	1297-14	9'	41	48.8
B-1	-	14'	-	32.6
B-2	-	4'	-	28.9
B-2	-	9'	-	23.4
B-3	-	4'	-	36.7
B-3	-	14'	-	22.8

NM: Natural moisture: defined as the moisture content of the in place sampled soil.

EI: Expansion Index: the relative change in height of a specimen during the initial 24 hour period of submersion during which it is subject to a surcharge load. The test provides a performance based analysis of uplift during brief cyclic changes in moisture content.

7.0 ENGINEERING EVALUATION AND RECOMMENDATIONS

7.1 General

The following evaluation and recommendations are based on our observations at the site, interpretations of the field and laboratory data obtained during the exploration, and experience with similar subsurface conditions. Corrected soil penetration data has been used to estimate allowable soil bearing pressure based upon established empirical correlations utilizing theoretical foundation loads and settlement tolerances. Subsurface conditions in unexplored locations may vary from those encountered. When structure locations, loading, or elevations are determined, CTI or another geotechnical firm should perform a supplementary investigation specific to the project characteristics, which will allow for presentation of final recommendations.

7.2 Foundation Analysis

Soil profiles encountered across the project site were defined by terrace deposits underlain by granular residuum with existing manmade Fill overburden. For the purposes of providing foundation recommendations, it has been assumed that the upper interface of bedrock is located at a depth of 30 feet or less below existing grade, although only one boring encountered refusal and the others were terminated at 20 feet. Loads utilized to perform settlement calculations were provided previously in this report and have not been verified by the Designer. Specific parameters pertaining to each foundation option are outlined in the underlying sections.

7.2.1 Shallow Foundations

The mass concrete abutments depicted in the provided schematic drawings will likely prove difficult to construct. Excavation will need to extend beyond the maximum height of 12 feet provided in the dimensional selection table, thus a wider abutment base would be needed. Exposure of competent rock will almost certainly require installation of soldier piles to restrain intermediate sheeting as the soft soil material encountered in each boring may not be stable at conservatively terraced slopes of 2H:1V, which would be very problematic and otherwise potentially affect the adjacent roadway if underlain by similar strata. Support of the abutments on the shallower alluvium is also doubtful due to load-bearing, differential settlement, and scour. As such, it is recommended that shallow foundations be considered only if the other underlying options are not acceptable.

7.2.2 Subsurface Improvement / Shallow Foundations

The mass gravity concrete abutments would be possible with improvement of the subsurface soils to a degree that a typical shallow foundation design may be adequately supported. Firms such as TerraSystems, GeoStructures, and Hayward Baker should be considered based upon previous personal experience as well as their established history of successful and efficient implementation of improvement programs in the project region. There are many other Specialty Geotechnical Contractors (SGC) and some may provide opinions that vary from those included in this report. Prior to serious consideration of any method that is not included within this report or which is questioned herein, it is critical that CTI be provided the opportunity to provide guidance regarding the Owner's long term benefits and accepted risks.

All of the underlying options were based upon improvement of the existing fill following rough grading to anticipated abutment/wingwall subgrade. Minimum criteria for consideration included an improved allowable bearing capacity of at least 4,000 psf and adherence to the previously provided settlement tolerances. Consideration of stockpiling the saturated spoils and augured hole stability would need to be addressed in the design submittal along with procedures to limit vibration, which could affect adjacent structures.

7.2.2.1 Modulus Improvement Aggregate Columns

The previously identified SGC all have proprietary versions of this improvement methodology branded as TerraPiers, Geopiers, and VibroPiers. Regardless of the name, each is intended to increase the composite modulus parameter of material supporting the foundation elements, which reduces settlements by up to 50 percent and has some potential to allow for improvement of a previously borderline seismic site classification. This category of remediation has become increasingly popular over the last decade and has supplanted stone trenches and undercut replacement as the most common measure when marginal fill material or residuum are encountered.

In general, the procedure for installation requires fully or partially augered holes extended through the strata to be improved and backfilled with lifts of crushed aggregate that are densified at regular intervals by means of vibration or impact. The vertical forces applied during densification cause the stone to both compact and expand laterally, producing consolidation of the soils surrounding the augered holes. Ultimately each lift becomes bulbous in nature, increasing the shear strength and modulus of the column while also acting to stiffen soils that are intermediate between the piers. It should be noted that each method requires 20 or more feet of overhead clearance and temporary stockpiles of soil and stone, which may not be feasible for this project. The improvement would also need to occur away from the creek embankment to eliminate scour as well as allowing for access by skidsteers to place the aggregate.

7.2.2.2 Soil Matrix Replacement / Consolidation

Another method of improvement involves replacement of marginal subsurface material with an engineered product having significantly better consolidation parameters. The simplest approaches involve auger borings that are subsequently backfilled with low strength grout to provide average subsurface parameters that are sufficient to reduce settlement values to within tolerable limits. The more complex and expensive procedures are based upon pressurized injection of cementitious materials through a closely spaced pattern of small-diameter holes to form semi-rigid slender columns while consolidating and stabilizing the intermediate soil material.

Due to the variability of possible approaches and materials, a specific analysis of this methodology was not performed for this report. Soil matrix replacement may be a viable option if the deck span is increased to place the abutments away from the embankment. 20 – 25 feet of overhead clearance would still be required, but material stockpiles would be limited to the spoils.

7.2.3 Deep Foundation – Micropiles

It is recommended that micro pile support of the abutments be seriously considered. There relatively small diameter will limit spoils while readily supporting the deck loads. They can also be installed with relatively small equipment, which will limit concerns with overhead/lateral restrictions and reduce mobilization costs. Micropiles are generally six to nine inches in diameter and installed by drilling through soil into competent bedrock to create a socket that is backfilled under pressure with portland cement grout. A combination of vertical and battered micropiles would be utilized to support the abutment and provide lateral restraint at its base. Although drilling does create noise, dust, and discharge of drilling water, it does not tend to cause vibration or other disturbance that would impact surrounding structures. Casing would be to prevent caving of the soft soil and center reinforcement provides uplift capacity.

The limited size of the micropiles reduces total capacity of each bearing element, which is somewhat preferred in the localized geology since they are less likely to compress solution seams and have less difficulty initiating the rock socket as steeply sloping bedrock tends to cause larger bits to "skip" along the face. Compressive axial capacity of a representative quantity of piles is generally verified under the observation of a Geotechnical Engineer using procedures in accordance with ASTM D1143 per the optional quick load test method for individual piles. Given the anticipated relatively light loading per element, this may be waived to reduce installation cost if the design produces a sufficient factor of safety.

To provide a uniform basis for bid preparation, it is recommended that preliminary micropile configuration utilize an allowable rock socket bond strength working stress of 7,500 psf, which is inclusive of a factor of safety equal to two, and exclude overburden soil skin friction. Due to the small hole diameter and inability to assess the conditions at and below shaft termination, it is recommended that end bearing capacity be ignored. If the final design utilizes a higher bond strength value or end bearing capacity, then it is recommended that verification tests be required at the time of installation

Micropiles should be individually monitored by a Geotechnical Engineer or qualified representative to determine the adequacy of the encountered material and to verify that they conform to diameter, depth, location, and plumbness. Specimens of the grout should be sampled on a daily basis to verify that batching is consistent and that the pumped material conforms to the requirements incorporated into the project specifications. The SGC design submittal should include consideration of spoils and discharged grout, which would need to be contained within the relatively small work area.

Socketed micropiles do not require significant deflection to develop their full load capacity as would friction piles or shallow foundations. Because of this, differential settlement may be assumed to be less than one-quarter inch with total settlement of no more than one-half inch.

7.2.4 Deep Foundation – Driven Piles

Driven piles may also be valid for support of the structure due to the greatly limited spoils and greater lateral load capacity. Installation of relatively small H-piles would allow for use of a smaller hammer, thus limiting vibration energy transferred to adjacent structures. The piles would be installed both vertical and battered to support and restrain the abutment. Overhead clearance may be an issue, as the hammer is most commonly attached to a crane. There will also need to be a relatively larger laydown area for the piles and several pieces of equipment. Production of spoils will essentially be limited to excavation of the foundations as the piles do not require pre-excavation unless shallow refusal on boulders occurs. Since the presence of river jack was not confirmed by the borings, it is recommended that the production driving criteria be developed based upon installation of at least two test piles as monitored by dynamic pile analysis (PDA). The PDA processes data from strain gauges and accelerometers mounted to the pile top to assess the hammer energy and pile stresses. This allows for real-time determination of possible unforeseen pile damage and a relatively accurate estimate of pile capacity.

7.3 Seismic Site Classification

In accordance with section 1613.5.2 of the 2009 International Building Code, a seismic site classification must be assigned for the project. This requires either a seismic conditions survey or performance of a 100 foot deep boring from which N-Values and sample classifications are analyzed. With soft to stiff consistency soils encountered in the borings and probable rock within 50 feet at most locations, it appears reasonable to project a weighted average N-value in excess of 50 bpf for the overall profile. Specific criterion pertaining to isolated zones of soft, moist, or plastic soil were applicable for the residual profile at some locations. Because of this, a conservative default seismic site classification of "D" should be assumed. Direct measurement of shear wave velocities utilizing geophysical methods tends to provide a less conservative seismic site classification and will likely improve the site classification.

7.4 Lateral / Surcharge Pressures

Earth pressures on walls and foundation elements below grade are influenced by their structural design, conditions of restraint, and methods of construction as well as compaction and characteristics of the materials being restrained. The most common conditions assumed for earth retaining wall design are the active and at-rest conditions. Active conditions apply to relatively flexible earth retention structures, such as free-standing cantilever concrete or segmental block wingwalls that withstand slight rotation and movement to relieve lateral pressures without affecting their function. Basement, pit, elevator, and abutment walls are rigidly constrained and should be designed utilizing at-rest conditions. A passive condition also exists to represent the maximum possible pressure that may be developed by soils resisting the forces exerted by the active or at-rest conditions. The magnitude of movement required to completely mobilize the passive forces is often beyond aesthetic and/or structural design tolerances.

To prevent unforeseen increases in lateral loading, large vehicular and heavy excavation equipment should not operate within a lateral distance equal to the wall height or five feet, whichever is greater. If surcharge loads will be applied within this zone, then they should be transposed as factored (K_O or K_A) as an additional equivalent lateral pressure in the initial design. Grading during construction should be maintained to meet the intent of the final design, thus preventing channeled drainage toward partially complete retaining wall structures that could result in delay or damage. This may require diversion dikes, level spreaders, or berms that are not depicted on the erosion and sediment control plan. It is highly recommended that these changes be discussed with the civil design firm to verify that they will not overload the stormwater management facilities or sediment control measures.

Table 3 provides typical parameters for imported select fill soils as well as VDOT #57 crushed stone. A bulk unit weight has been provided for structural backfill, such as would be required for areas where site concrete or roadways fall within the active wedge. The residual unit weight should be used for "green" areas of backfill or undisturbed trench sidewalls. If crushed stone parameters are utilized, then the #57 backfill width should extend beyond the active wedge that is assumed to be oriented in a linear fashion from the outer wall/footing interface at a slope of roughly 60 percent (0.6H:1V). Thus, the overexcavation of the wall at any point would equal 60 percent of the unbalanced wall height at that elevation. The minimum allowable width would occur at the wall base and should be two feet or match the footing projection if the latter measure is greater.

The values assigned to the select fill soil are based upon prior experience within the vicinity of the site. Some suitable soils may be encountered on-site, which would include Sand ("SC"), Silty Sand ("SM"), and Silty Gravel ("GM"). Select fill would include these categories and CBR 30 material or rock dust from a quarry as well as Clayey Gravel ("GC") after verification of plasticity and gradation. The underlying parameters preclude use of fine-grained soils such as Lean Clay ("CL"), Silt ("ML"), or highly plastic occurrences of the aforementioned suitable soils as wall backfill.

Table 3: Below-Grade Wall Design Parameters

Earth Pressure Condition	VDOT #57	On-Site Soils (Select Fill)*
Active (K_A)	0.25	0.31
At-Rest (K_O)	0.40	0.47
Passive (K_P)	2.03	3.25
Structural Moist Unit Weight – (γ_s)	110 pcf	130
Residual Moist Unit Weight – (γ_r)	110 pcf	110
Cohesion (C)	0 psf	0 psf
Angle of Internal Friction (ϕ)	40°	32°
Sliding Coefficient of Soil-Concrete (tan δ)	0.36	0.36

* Field/lab classified as Clayey Sand ("SC"), Silty Sand ("SM"), or better

In specific regards to analysis of wall stability relating to sliding failure, the parameters have been provided with the intent of being incorporated in the following formula:

$R_{SL} + C_A = W * \tan\delta * B$

R_{SL}:	Total resistance to sliding per unit width
W:	Sum of all dead loads (concrete, soil over heel, and dead load of supported floors)
tan δ:	Sliding coefficient based upon δ = 20° for a medium-stiff sandy clay against concrete or δ = 17° for a soft clay
C_A:	Subgrade adhesion (or remolded shear strength in this case) = C / 1.5 (regardless of backfill)
B:	Footing width in feet

The provided formula doesn't include passive pressure along the foundation face, which is generally only considered for keyways. With passive pressure negated, a FS≥1.5 relative to the lateral forces should be sufficiently conservative. Use of the parameters assumes that a full-height drainage system has been installed and maintained during construction and throughout the life of the structure.

7.5 Site and Subgrade Preparation

Areas proposed for grading or construction should be stripped and grubbed of all topsoil, vegetation, roots, organics, organic contaminated soils, soft on-site soils, and pavement before placing structural Fill. Following preparation of exposed subgrades, accessible portions of the building pad subgrade should be proofrolled with a loaded 20 ton tandem axle dump truck and witnessed by the Geotechnical Engineer or qualified representative. The purpose of the proofrolling will be to locate any isolated soft, unstable or "pumping" pockets of soil, which should be excavated or otherwise stabilized as directed by the Geotechnical Engineer. Proper site drainage should be maintained at all times to prevent ponding of water at the site during construction. If the soils do become wet, care should be taken to minimize heavy construction equipment from operating on the prone subgrade.

7.6 Structural Fill

Structural fill materials should be free of organic matter, debris and deleterious materials. Fill sources containing particles larger than three inches (cobbles) should not be allowed within three feet of the surface unless they are screened or raked following loose placement. Proposed fill material should be subjected to laboratory tests consisting of, but not necessarily limited to, Proctor moisture/density determination, Atterberg limits, and sieve analysis. If undercuts are required prior to structural fill placement and coarse quarry stone backfill is utilized, then it will necessarily deviate from the gradation requirement, eliminate lab analysis, and require only visual monitoring of placement.

It should be noted that natural moisture samples of the residual material frequently exceeded anticipated optimum, thus scarification and aeration will be necessary if on-site soil material is utilized as structural fill or backfill. The project specifications should directly address this situation and delineate degree and duration of moisture treatment that will be expected as a portion of the contract to limit material "waste" while providing the Contractor a measurable basis for schedule extension due to site conditions that can't readily be predicted. Any imported soil fill required to balance the site should adhere to the following parameters unless specifically accepted in writing by the Geotechnical Engineer at time of placement:

Maximum Dry Density (ASTM D698)	≥ 95 pcf
Liquid Limit	≤ 60
Plasticity Index	≤ 35
Expansive Index	≤ 50

The structural fill should be placed in maximum four inch lifts if a plate compacter is utilized for compaction, six inch lifts for a jumping-jack compactor, and eight inch lifts for a vibratory trench or rideon roller. The referred to lift thicknesses are based upon compacted depth and thus the loose lift thickness prior to compaction may be estimated as roughly 30 percent greater. Thicker lifts may be approved at the discretion of the Geotechnical Engineer, but in no case may they exceed the maximum measurement depth of the test method utilized to verify appropriate moisture and adequate compaction. Lifts should be compacted to a minimum of 95 percent of the maximum dry density of ASTM D698, Standard Proctor, to planned grades. Moisture content should be within one-fifth of optimum ($M \pm 0.2M$) as determined by D698. Field density tests should be utilized to verify that sufficient compaction has been achieved with systematic verification of moisture content by means of supplementary burn-offs. These tests should be performed by a Soils Inspector under the direction of a Geotechnical Engineer and at a frequency of every 2,500 square feet of mass grading and at 50 lineal foot intervals for trench and wall backfill. No less than two tests should be conducted for each lift of fill material.

7.7 Pavement Design

Unspecified design life, traffic counts/allocations, and axle loads prevented evaluation of specific pavement sections. For flexible pavement, a preliminary resilient modulus (M_r) value of 5,000 psi is suggested while rigid pavement should utilize a *k* value of 125 pci. These values presume that the design will conform to the *1993 AASHTO Guide for Design of Pavement Structures.*

7.8 Groundwater Control

Shallow foundation excavations on the order of five feet should not encounter seepage, but significantly elevated soil moisture should be anticipated. Deeper excavations will encounter intermittent or continuous seepage in conjunction with moist yielding granular soils. Standard de-watering practices utilizing a mid-sized trash pump in conjunction with perimeter "tail ditches" and collector sump holes should be sufficient to prevent saturation of exposed subgrades exposed to perched water and above the phreatic water table. Excavation to depths 15 feet or more will require customized dewatering systems.

8.0 REMARKS AND LIMITATIONS

8.1 General Comments

This geotechnical engineering report has been prepared for this project by CTI Consultants, Inc. This report is for informational purposes only and should not be considered part of the contract documents. The conclusions and opinions expressed in this report are those of the geotechnical engineer and represent an interpretation of the subsoil conditions, tests, and results of analyses performed for this investigation. Should the data contained in this report not be adequate for the contractor's purposes, the contractor may conduct additional investigations, tests, and analyses prior to bidding.

8.2 Limitations of Study

Information and recommendations contained in this report are partially based upon data obtained from a limited number of test borings performed at the site. This report has been prepared in accordance with generally accepted geotechnical engineering practices. No other warranty, expressed or implied, is made as to the professional advice included in this report. This report does not reflect variations that may occur at different locations. The nature and extent of these variations may not become evident until during excavation of the new foundation.

This report has been prepared to assist the design professionals in the design of this project. It is intended for use with the specific project as described herein. Any substantial changes in design should be brought to our attention so that CTI may determine any affect on the recommendations rendered herein. This report should be made available to bidders prior to submitting their proposals, and to the successful contractor and subcontractors for their information only, and to supply them with facts relative to the localized geology, subsurface investigation, soil laboratory testing, and geotechnical recommendations.

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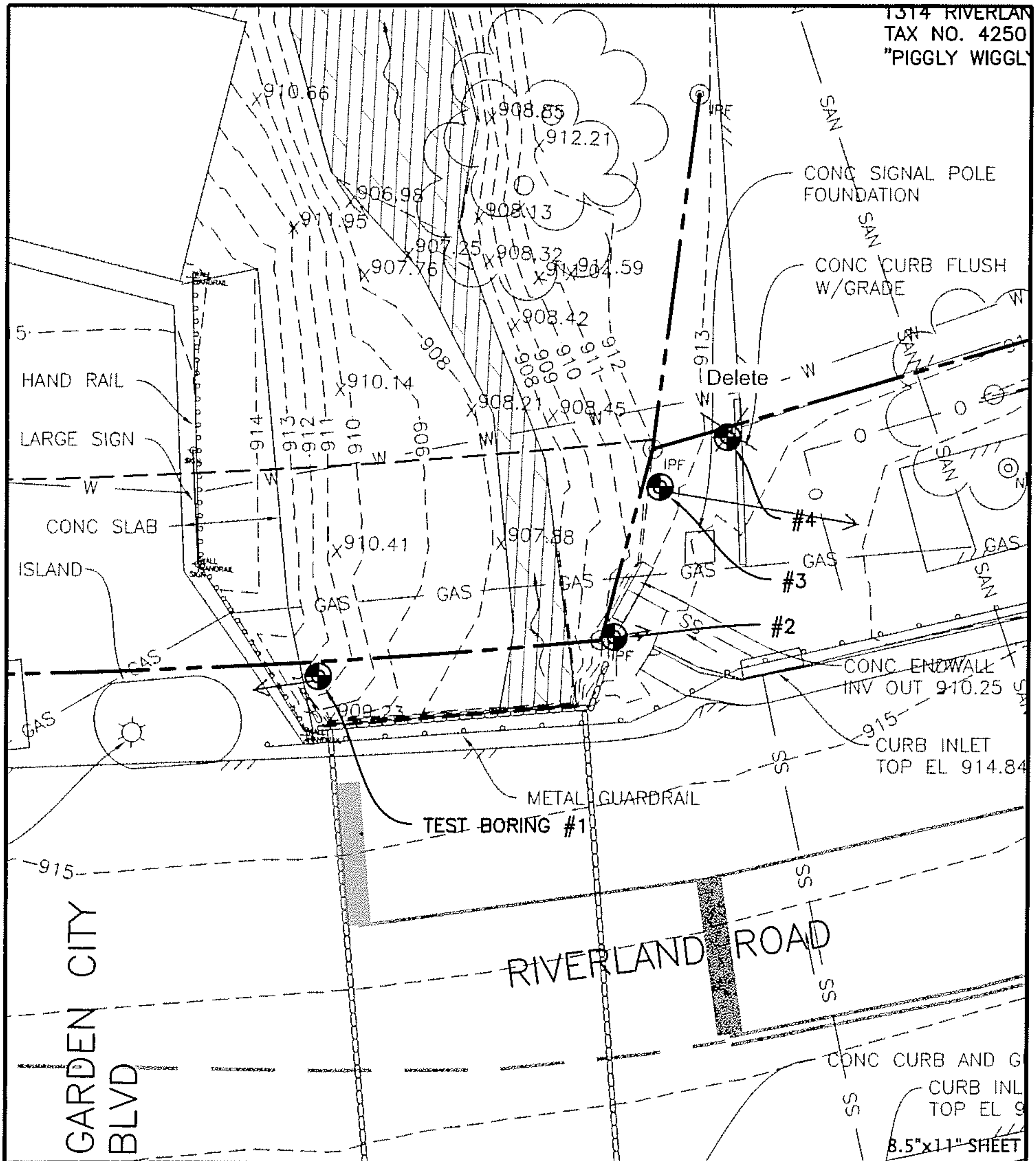
Appendix A:

Boring Locations



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ROANOKE

TEST BORING EXHIBIT

GARDEN CITY GREENWAY CONNECTION PROJECT

SCALE: 1"=10'

DATE: 05/07/2014

SHEET 1 OF 1

Appendix B:

Boring Logs

USCS/Boring Key

CTI CONSULTANTS INC
1348 South Main Street (540) 552-1575
Blacksburg, Virginia 24060 fax (540) 552-2965

SOIL BORING LOG: B-1

Project Number: 11G-1236

Client:	Office of The Engineer	Boring Contractor:	Total Depth Drilling
Project Name:	Garden City Greenway Pedestrian Bridge	Foreman:	A Waller
Project Location:	Riverland Road SE at Garden City Blvd; Roanoke, VA	Boring Method:	Hollow Stem Auger
Boring Location:	Western Abutment Area	Inspector:	C Newman
Total Depth: 17.50 ft / 5.33 m; Elev.: 914.2 ft / 278.6 m	Referenced Datum: Estimated from Boring Location Plan	Completion Date:	July 7, 2014

Elev (ft)	Depth (ft)	DESCRIPTION OF MATERIALS	Stratum Depth(ft)	Sample Number	Sample Blows	N Value	REMARKS
		Asphalt	0.1				
		Dense-graded aggregate	0.5				
		FILL consisting of clayey soil and bituminous asphalt, moist, medium-stiff, moderate plasticity; reddish-brown	1.7	SS1	2-2-6	8	
		Sandy lean clay FILL, moist, medium-stiff, moderate plasticity, brownish-red	4.0	SS2	2-5-5	10	
910.0	5	Silty gravel FILL, moist+, loose, low plasticity; black and gray; contains asphalt, quarry gravel, some organics, and possibly minor coal	7.0	SS3	2-W-W	0	Free moisture on spoon at 6'
905.0		Clayey sand FILL, wet, loose, low plasticity; brown and black; includes substantial organic content	9.5	SS4	6-6-14	20	
900.0	10	Probable undisturbed alluvium: CLAYEY SAND, moist+, loose, moderate plasticity; yellow, brown, and pale gray;	14.5	SS5	3-9-1	10	
	15	CLAYEY GRAVEL, wet, loose, low to moderate plasticity; yellow and brown; subangular fine Shale gravel	17.5				
895.0	20	Auger refusal on apparent weathered rock interface or river jack					Auger refusal on apparent bedrock at 17.5'

WATER LEVEL OBSERVATIONS

Noted on Rods:	6.0 ft
On Completion:	ft

SOIL BORING LOG: B-2

CTI CONSULTANTS INC
1348 South Main Street (540) 552-1575
Blacksburg, Virginia 24060 fax (540) 552-2965

Project Number: 11G-1236

Client:	Office of The Engineer	Boring Contractor:	Total Depth Drilling
Project Name:	Garden City Greenway Pedestrian Bridge	Foreman:	A Waller
Project Location:	Riverland Road SE at Garden City Blvd; Roanoke, VA	Boring Method:	Hollow Stem Auger
Boring Location:	Northeast Area of Eastern Abutment	Inspector:	C Newman
Total Depth 20.00 ft 6.10 m; Elev.: 913.0 ft 278.3 m	Referenced Datum: Estimated from Boring Location Plan	Completion Date:	July 7, 2014

Elev (ft)	Depth (ft)	DESCRIPTION OF MATERIALS	Stratum Depth(ft)	Sample Number	Sample Blows	N Value	REMARKS
		FILL containing intermingled surficial and clayey sandy soil as well as gravel, moist, medium-stiff, moderate plasticity; brown, dark brown, and red; contains root matter and distinguishable organic odor	2.0	SS1	5-5-4	9	
910.0	5	Clayey sand FILL, moist+, loose, low plasticity; brown and yellow-brown, some black; no organics	5.5	SS2	2-2-2	4	
		Sandy lean clay FILL, moist+, very soft, low to moderate plasticity; brown and pale brown	7.5	SS3	1-1-1	2	
905.0	10	FILL consisting of clayey sand and clayey gravel, moist, loose, low to moderate plasticity; brown, pale brown, yellow, and pale gray; contains subangular and subrounded fine gravel from Limestone/Dolomite and Shale	11.0	SS4	7-31-18	14	Cobble at 9.1' resulted in excessively high blow counts through remainder of split spoon length
900.0	15	Probable undisturbed alluvium: LEAN CLAY, wet, very soft, moderate plasticity; yellow-brown		SS5	W-2-3	5	Free moisture on split spoon at 14' Augers sank from 13.5 - 15' under own weight
895.0	20		20.0	SS6	1-W-1	1	
		Boring extended to planned termination depth at 20'					

WATER LEVEL OBSERVATIONS

Noted on Rods: 14.0 ft

On Completion: ft

CTI CONSULTANTS INC

1348 South Main Street (540) 552-1575
Blacksburg, Virginia 24060 fax (540) 552-2965

SOIL BORING LOG: B-3

Project Number: 11G-1236

Client:	Office of The Engineer	Boring Contractor:	Total Depth Drilling
Project Name:	Garden City Greenway Pedestrian Bridge	Foreman:	A Waller
Project Location:	Riverland Road SE at Garden City Blvd; Roanoke, VA	Boring Method:	Hollow Stem Auger
Boring Location:	3' Beyond End of Existing Greenway	Inspector:	C Newman
Total Depth 20.00 ft 6.10 m; Elev.: 914.0 ft 278.6 m; Referenced Datum:	Estimated from Boring Location Plan	Completion Date:	July 7, 2014

Elev (ft)	Depth (ft)	DESCRIPTION OF MATERIALS	Stratum Depth(ft)	Sample Number	Sample Blows	N Value	REMARKS
		Surficial soil and respread topsoil, fine root matter	1.2				
				SS1	50/5	100	
		Bituminous asphalt and dense-graded aggregate	2.5				
		FILL consisting of clayey sand and clayey gravel, moist+, loose, low plasticity; brown, black, and gray; some quarry aggregate		SS2	3-2-1	3	
910.0			5.0				
	5	FILL containing gravel, limited sample recovery	6.5	SS3	15-3-2	5	
		Probable FILL containing unknown silty material, moist, medium-stiff, moderate plasticity; primarily pure white and pale yellow; opaque fine blocky crystalizations can be manually pulverized to form fine friable powder, "chalky" with some plasticity. May be a manmade substance, not encountered previously in drilling throughout region.	8.0				Odd soil/substance sampled from 6.5 - 8'
		Alluvium, possibly disturbed: CLAYEY SAND, moist+, loose, low to moderate plasticity; brown, gray-brown, yellow-brown, and black, contains fine subangular Shale gravel		SS4	10-10-13	23	
905.0	10		12.0				
		CLAYEY GRAVEL, moist, loose, moderate plasticity; yellow-brown and brown; medium and fine subangular and blocky gravel from sedimentary parent material, becoming increasingly coarse with depth		SS5	2-8-19	27	
900.0	15						Free moisture on rods at roughly 17.5'
				SS6	7-15-23	38	
895.0			20.0				
	20	Boring extended to planned termination depth at 20'					

WATER LEVEL OBSERVATIONS

Noted on Rods:	17.5 ft
On Completion:	ft

UNIFIED SOIL CLASSIFICATION SYSTEM

CTI CONSULTANTS INC

UNIFIED SOIL CLASSIFICATION AND SYMBOL CHART

COARSE-GRAINED SOILS
(more than 50% of material is larger than No. 200 sieve size.)

	Symbol	Description
		Clean Gravels (Less than 5% fines)
GRAVELS More than 50% of coarse fraction larger than No. 4 sieve size	GW	Well-graded gravels, gravel-sand mixtures, little or no fines
	GP	Poorly-graded gravels, gravel-sand mixtures, little or no fines
		Gravels with fines (More than 12% fines)
	GM	Silty gravels, gravel-sand-silt mixtures
	GC	Clayey gravels, gravel-sand-clay mixtures
		Clean Sands (Less than 5% fines)
SANDS 50% or more of coarse fraction smaller than No. 4 sieve size	SW	Well-graded sands, gravelly sands, little or no fines
	SP	Poorly graded sands, gravelly sands, little or no fines
		Sands with fines (More than 12% fines)
	SM	Silty sands, sand-silt mixtures
	SC	Clayey sands, sand-clay mixtures

FINE-GRAINED SOILS
(50% or more of material is smaller than No. 200 sieve size.)

	Symbol	Description
SILTS AND CLAYS Liquid limit less than 50%	ML	Inorganic silts and very fine sands, rock flour, silty of clayey fine sands or clayey silts with slight plasticity
	CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
	OL	Organic silts and organic silty clays of low plasticity
SILTS AND CLAYS Liquid limit 50% or greater	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts
	CH	Inorganic clays of high plasticity, fat clays
	OH	Organic clays of medium to high plasticity, organic silts
HIGHLY ORGANIC SOILS	PT	Peat and other highly organic soils

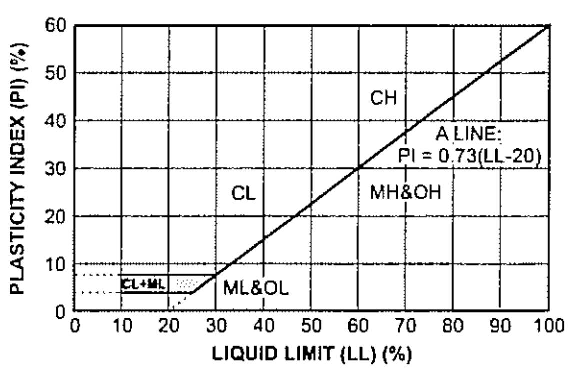
LABORATORY CLASSIFICATION CRITERIA

GW	$C_u = \frac{D_{60}}{D_{10}}$ greater than 4; $C_c = \frac{D_{30}}{D_{10} \times D_{60}}$ between 1 and 3	
GP	Not meeting all gradation requirements for GW	
GM	Atterberg limits below "A" line or P.I. less than 4	Above "A" line with P.I. between 4 and 7 are borderline cases requiring use of dual symbols
GC	Atterberg limits above "A" line with P.I. greater than 7	
SW	$C_u = \frac{D_{60}}{D_{10}}$ greater than 4; $C_c = \frac{D_{30}}{D_{10} \times D_{60}}$ between 1 and 3	
SP	Not meeting all gradation requirements for GW	
SM	Atterberg limits below "A" line or P.I. less than 4	Limits plotting in shaded zone with P.I. between 4 and 7 are borderline cases requiring use of dual symbols.
SC	Atterberg limits above "A" line with P.I. greater than 7	

Determine percentages of sand and gravel from grain-size curve. Depending on percentage of fines (fraction smaller than No. 200 sieve size), coarse-grained soils are classified as follows:

Less than 5 percent .... GW, GP, SW, SP
More than 12 percent .... GM, GC, SM, SC
5 to 12 percent .... Borderline cases requiring dual symbols

PLASTICITY CHART



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Appendix C:

Detailed Laboratory Results

putting excellence
to the test

1348 South Main Street
Blacksburg, Virginia 24060

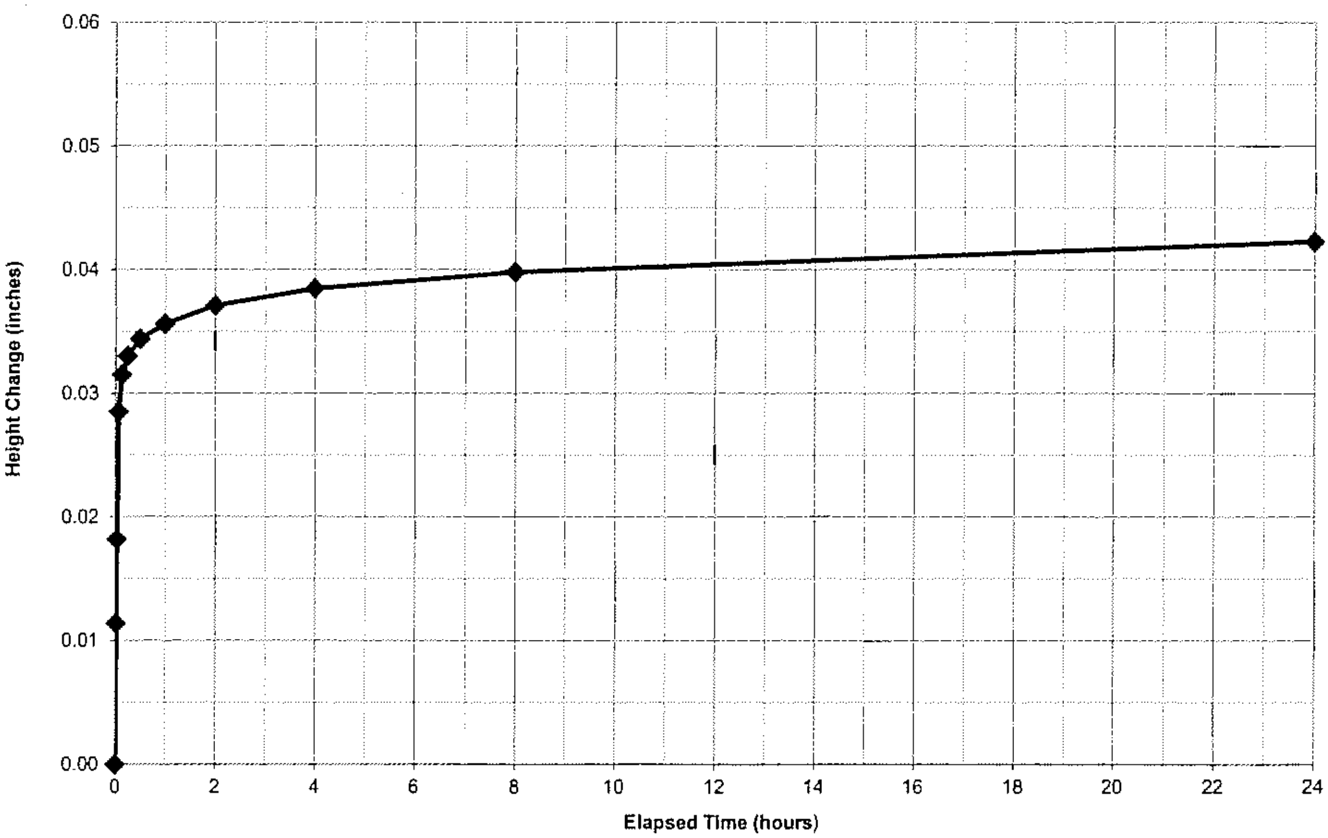
Project Name:	Grden City Greenway Connection Project	Sampled by:	C. Newman
Project Number:	11G-1236	Tested by:	J. Epifanio
Laboratory Number:	1297-14	Surcharge Weight:	1 psi
Date Received:	07/16/14	Number of Layers:	2
Report Date:	07/19/14	Blows per layer:	13
Soil Description:	Clayey Sand; brown, gray-brown, yellow-brown, & black	Specific Gravity:	2.70
Material Source:	Boring B-3	Sample Thickness:	1.00 inches

SAMPLE NUMBER	SAMPLE DEPTH (ft.)	SAMPLE TYPE	USCS CLASS.	AASHTO CLASS.	NATURAL MOISTURE (%)	ATTERBERG LIMITS		
						LL	PL	PI
1297-14	9	Bulk	N/A	N/A	48.8	N/A	N/A	N/A

COBBLES	GRAVEL	SAND	SILT & CLAY (FINES)
N/A	N/A	N/A	N/A

DRY DENSITY	INITIAL MOISTURE	SATURATION	FINAL MOISTURE
72.4	23.8%	48	48.1%

INITIAL DIAL	FINAL DIAL	INDEX VALUE	POTENTIAL EXPANSION
0.055	0.098	41	Low



Test Procedures: *Expansion Index, ASTM D4829*
Natural Moisture Content, ASTM D2216