GEOTECHNICAL ENGINEERING SERVICES

Renovation of the East Schuylkill River Wall
Kelly Drive
Philadelphia, PA

Submitted To:

City of Philadelphia
Philadelphia Parks & Recreation
1515 Arch Street, 11th Floor
Philadelphia, PA 19102

Submitted By:

Pennoni Associates Inc.
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Philadelphia, PA

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Geotechnical Project Engineer

PHILP18003

January 8, 2020

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January 8, 2020

PHILP18003

Mr. Bernard McFadden
City of Philadelphia
Philadelphia Parks & Recreation
1515 Arch Street, 11th Floor
Philadelphia, PA 19102

RE: Geotechnical Engineering Services
Renovation of the East Schuylkill River Wall
Philadelphia, PA

Mr. McFadden:

We are pleased to submit our geotechnical engineering report for the proposed renovation of the East Schuylkill River Wall in Philadelphia, PA. Work was initiated in general accordance with the scope of work presented in our proposal dated February 26, 2019, (revised on October 11, 2019) and your subsequent authorization to proceed.

We trust that the information presented in this report is what you require at this time and we thank you for the opportunity to assist you with this project. If you have any questions, or if you need any further assistance with this project, please contact this office at your earliest convenience.

Respectfully yours,

PENNONI ASSOCIATES INC.

Elisabetta Iannetti
Graduate Engineer

Daniel P. Marano Jr., PE
Geotechnical Project Engineer

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1. EXECUTIVE SUMMARY

Pennoni has completed our geotechnical evaluation for the proposed rehabilitation of the East Schuylkill River Wall in Philadelphia, PA. The purpose of our evaluation was to perform geotechnical field and laboratory testing to classify the subsurface soils in the area of the proposed construction and provide conclusions and recommendations related to the construction of the retaining wall.

The rehabilitation of the wall was deemed necessary due to the many deficiencies observed during diving investigation. The deficiencies include deterioration of the existing timber cribbing mat and supporting timber piles. The proposed rehabilitation will consist of removing the existing retaining wall along with the timber cribbing mat and then installing/constructing a new foundation system which will support a new concrete footing and stone masonry wall. The wall will be approximately 6 to 6.5 ft tall.

From October 30 to November 5, 2019, four geotechnical borings were drilled at the site. The borings generally disclosed that the near surface subsoils are a very loose to medium dense, Sand and Silt layer, which is underlain by clayey silt, decomposed mica schist, and rock layers, respectively. Groundwater was encountered in all the test borings at depths ranging from approximately of 7 to 13 ft below existing grades. This is consistent with the river elevation compared to the elevation at which the borings were performed.

Based on the results of our field exploration, laboratory testing, engineering analyses, and our experience, we conclude that the reconstruction of the river wall is feasible. In our professional opinion, deep foundation systems such as timber piles, steel pipe piles or H-piles can be considered for the support of the river wall foundation. Detrimental long-term post-construction settlements are not expected if the recommendations presented in this report are followed. The clean inert portions of Strata F and 2 can be used in compacted load bearing fills.

This report provides a more detailed summary of the field and laboratory testing programs as well as a discussion of the conclusions and recommendations pertaining to foundation design and construction.

2. INTRODUCTION

2.1. LOCATION AND SURFACE FEATURES

The existing river wall is located near the reviewing stands between the Schuylkill River and Kelly Drive in Philadelphia, Pennsylvania, approximately 1000 ft upstream from the Columbia Bridge.

The project area is currently occupied by a parking lot with a masonry wall adjacent to the river's edge. The existing wall is approximately 6 ft tall and is supported on timber piles. The ground surface of the site generally consists of bituminous concrete and grass between the river wall and the parking lot. Evidence of overhead utilities was observed on the site.

2.2. PREVIOUS INVESTIGATIONS/PROJECT HISTORY

In late October 2011 and March 2012, the City of Philadelphia tasked Pennoni to perform an underwater survey to determine the possible causes of the collapse along the failed section of the wall. The survey concluded that the wall exhibited several locations of scour, undermining of the timber cribbing mat, deterioration, and section loss of the timber members.

In February 2018, Pennoni performed an underwater pre-design survey of the 400 ft long wall to provide an update on the findings presented in the 2012 report. The portion of the wall surveyed was undermined the full length, exposing the timber cribbing mat and portions of the timber piles. Numerous exterior timber piles also exhibited section losses ranging from 40 % to 60% in the top 1-ft among other deteriorations.

Additionally, in July 2019 Pennoni performed another underwater condition survey of the wall. This was done to evaluate larger extents of the wall where structural deficiencies similar to the previous findings were suspected. Deficiencies were found to extend both north and south from the area surveyed. At the time of the July 2019 survey, most of the wall was undermined up to 2-feet laterally and up to 8-feet deep below the timber cribbing mat and portions of the timber piles that were exposed. Six drainage outlets were also found to be clogged and/or collapsed.

2.3. PROPOSED CONSTRUCTION

The rehabilitation of the wall was deemed necessary due to the many deficiencies observed during diving investigation. The deficiencies include deterioration of the existing timber cribbing mat and supporting timber piles. The proposed rehabilitation will consist of removing the existing retaining wall along with the timber cribbing mat and then installing/constructing a new foundation system which will support a new concrete footing and stone masonry wall. The wall will be approximately 6 to 6.5 ft tall. We understand that there are currently 3 pile configurations being considered for support of the new retaining wall. The first pile configurations are presented in Table 1 on the next page. All options were analyzed in Section 5.3.1 of this report.

TABLE 1: Pile Configurations

Pile Configuration	Number of Piles per Row	Spacing (ft. o.c.)
No. 1	3	5
No. 2	2	5
No. 3	3	10

2.4. OBJECTIVES

The objectives of this geotechnical study were to determine subsurface conditions at the project site, evaluate these conditions with respect to the proposed construction, and present our conclusions and recommendations regarding:

- foundation design, including a discussion of alternate solutions, if applicable, allowable bearing capacity and anticipated total and differential settlement amounts;
- lateral load capacity analysis of pile foundations using LPILE;
- design frost depth;
- "general procedure" Seismic Soil Site Classification based on applicable IBC requirements;
- evaluation and determination of the earthwork requirements, including material selection and placement operations;
- ground water conditions;
- lateral earth pressure parameters;
- suitability of on-site material for re-use as fill for the project;
- removal or treatment of objectionable material, and;
- quality assurance, field-testing, and observation during construction.

3. FIELD AND LABORATORY WORK

3.1. FIELD WORK

From October 30 to November 5, 2019, four geotechnical Standard Penetration Test (SPT) borings were drilled by CGC Geoservices, LLC, at the approximate locations presented on Drawing No. LP-1, enclosed in Appendix A. Representative soil samples were obtained in general accordance with ASTM D 1586 and ASTM D 2113 methods. The boring locations were selected and established by Pennoni personnel. Boring logs are presented in Appendix A.

Our D. Marano, PE directed the field work; our E. Brinker, N. Rex, and E. Iannetti, provided full-time observation of the drilling operations.

3.2. LABORATORY WORK

The soil samples collected during our field study were delivered to our laboratory. Representative samples were selected and tested to determine moisture contents, plasticity indices, and gradation characteristics of the subsoils, and unconfined shear strength on several rock core samples. Appendix B includes the laboratory testing results and a list of testing procedures.

4. SITE CHARACTERISTICS

4.1. GEOLOGY

The project site is located within the Lowland and Intermediate Upland section of the Atlantic Coastal Plain Province. The dominant topographic features of this section include very low local relief and a flat upper terrace surface cut by narrow, steep-sided to open valleys, shallow valleys; includes the Delaware River floodplain. The underlying subsurface material types consist of unconsolidated to poorly consolidated sand and gravel deposits, underlain by very complex, faulted and folded schist, gneiss, and other metamorphic rocks.

Available geological data indicates that the subject site is underlain by the Wissahickon Formation. The Wissahickon Formation consists of a coarsely crystalline, excessively micaceous schist. Fracturing results in a well-developed, platy pattern. This Formation is fissile to thinly bedded, moderately resistant to weathering, and often highly weathered to a moderate depth.

4.2. SUBSOILS

The borings revealed a topsoil surface layer that varies from 4 to 5 in. thick. The underlying subsoils have been grouped into four principal strata based on their engineering properties and our interpretation of their origin. Brief strata descriptions are presented below.

STRATUM	DESCRIPTION
F	FILL: Medium to fine to coarse SAND and SILT, trace fine gravel size Rock Fragments; very loose to medium dense
1	CLAYEY SILT, some medium to fine Sand, trace fine Gravel; very soft to medium
2	Coarse to medium to fine SAND, little fine Gravel and Silt; very loose to very dense
3	MICA SHIST, slightly to moderately weathered, medium to strong hardness, close fracture spacing.

Auger refusal of the drilling equipment was encountered in all borings at depths ranging from 17.9 to 30 feet below existing grade. Refusal to further penetration of the drilling and sampling equipment generally infers top of bedrock.

4.3. GROUNDWATER

Observations for groundwater were made in each boring during sampling and shortly after completion of drilling. Evidence of groundwater was encountered in all borings at depths ranging from approximately 7 to 13 ft below existing grade. This is consistent with the river elevation compared to the elevation at which the borings were performed. Groundwater observations are for the times and locations noted and may not be indicative of seasonal, daily fluctuations in groundwater levels.

5. ANALYSES AND RECOMMENDATIONS

5.1. SEISMIC SITE CLASSIFICATION

The borings disclosed subsurface conditions generally described according to the Table 20.3-1 of ASCE 7 and referenced in Section 1613.3 of the 2018 International Building Code (IBC) as having a soil-profile corresponding to Site Class E – a soft soil profile. Site Class determination is based on the properties in the upper 100 feet of the ground surface. The borings performed herein were advanced to a maximum depth of 38 feet. Values beyond 38 feet were estimated based on our local experience in this area.

5.2. EARTHWORK

A comparison of the existing grades with the retaining wall grading elevation indicates that cuts as deep as 10 ft will be required to reach subgrade to drive piles. Additionally, fills on the order of 6 feet are anticipated to backfill the walls after construction.

Prior to the placement of new fills and construction of foundations, all existing concrete, asphalt, topsoil and vegetation located within the proposed construction footprint should be removed. Any existing utilities located within the proposed construction areas should be abandoned and relocated. Any existing utility line abandoned in-place should be grouted or the line should be removed from the trench and appropriately backfilled.

Subgrade soils should be manually probed in an attempt to disclose unstable surface areas. Any unstable surface areas (soft, yielding, etc.) found should be stabilized by excavating and replacing those soils with suitable soil that is adequately compacted. This can be accomplished by properly adjusting the moisture content of the subgrade soils and compacting them, or by other methods (placing a geotextile and stone layer, etc. or soil exchange).

Our experience indicates that the clean/inert portions of the on-site soils of Strata F and 2 can be reused for earthwork construction, provided all organics and debris larger than 3 inches in its greatest dimension be removed prior to reuse. Laboratory test results indicate that the present moisture content (19.6 % to 40.2%) of these soils are higher than the optimum moisture content normally associated with these soils to achieve desired degree of compaction. Drying "wet" soil is difficult during wet periods and during lower temperatures. In addition, based on our experience the on-site soils of Stratum F were observed to contain a significant amount of fine-grained material (silts). These types of soils are sensitive to moisture and may therefore require wetting or drying prior to compaction. Therefore, depending on the season that the earthwork operations are taking place, adjusting the moisture content of these on-site soils before use in any compacted fills and/or subgrade preparation may be required. Provisions for importing structural fill should be included in the contract documents.

If necessary, imported structural fill should be selected from suitable borrow sources and be approved by the Geotechnical Engineer well in advance of fill construction. Granular fill ideally should consist of well-graded material with not more than 20 percent passing the No. 200 sieve and have a plasticity index not greater than 8 percent; PennDOT 2A processed aggregate or recycled concrete with a gradation similar to that described above with a maximum particle size of 3 in. can be considered. Other gradations can be considered based on laboratory testing and at the discretion of the Geotechnical Engineer.

Fine grained and granular fills should be placed in layers not exceeding 8 to 10 in. and 10 to 12 in. loose thickness, respectively. This criterion might be adjusted by the geotechnical engineer in the field depending on the conditions present at the time of construction, on the compaction equipment used, and on the fill materials selected.

Specifications should indicate that the percentage of maximum dry density attained in the field is not the only criteria to be used for assessing fill compaction. Observation of the behavior of the fill under the loads of construction equipment should also be used. If the test results indicate that the percentage of compaction is being achieved, but the soil mass is moving under the equipment, placement of additional fill should not be continued until the movement is stabilized. Otherwise, settlement of the fill may occur.

We recommend that all site preparation and earthwork operations be carried out in the full-time presence of a qualified representative of the Geotechnical Engineer.

5.3. FOUNDATIONS

Based on the structural loading provided, the results of our field exploration and our experience with similar projects it our professional opinion that the reconstruction of the retaining wall is feasible. The borings disclosed a very soft, compressible silt layer, which will experience differential settlements with load application. Therefore, a shallow foundation system cannot be considered for the new retaining wall. We have evaluated several deep foundation alternatives such as timber piles, steel pipe piles, or H-piles. Below is a brief description of each of the pile types evaluated, and at the end of this section is a table summarizing the anticipated lengths of each pile type, axial load, and lateral capacities based on the anticipated subsurface conditions.

5.3.1. DRIVEN PILES

Timber piles

Chromated Copper Arsenate (CCA) treated timber piles should conform to ASTM 25-99 and AWPA C3-03 Specifications and should have minimum tip and butt diameters of 8 and 12 in., respectively. The estimated length is on the order of 20 to 30 ft below the existing grades with the pile tips bearing in the medium dense sand to very dense deposit (Stratum 2). We estimate an allowable pile capacity on the order of 40 kips/pile. Based on the axial loads provided to us for each pile configuration and assuming a maximum deflection of 1 inch, the calculated lateral capacity is 10.5 kips. The minimum pile spacing should be at least three pile diameters (3D), center to center.

Timber piles will require Pile Dynamic Analysis (PDA) testing during installation to confirm axial load carrying capacities. All pile installation, including the test piles, should be carried out in the full-time presence of a qualified representative of the Geotechnical Engineer who will evaluate and correlate the driving data and depth of penetration of each pile with the results of the test piles, our static analyses, and the boring log data. The Geotechnical Engineer's representative should be onsite to ensure that the required driving resistance of each pile is attained.

Open end, Concrete Filled Steel Pipe Piles

Open end, concrete-filled pipe piles with nominal diameters of 12 in., and wall thickness of 1/2 in. can be considered for support of the wall. We estimate an allowable pile capacity of 55 kips/pile. Based on the axial loads provided to us for each pile configuration and assuming a maximum deflection of 1 inch, the calculated lateral capacity is 15.5 kips. The estimated length is on the order of 20 to 30 ft below existing grades. A minimum pile spacing of 3 times the pile diameter should be maintained between piles.

H-Piles

Driven steel H-piles can also be considered for support of the retaining wall. We estimate an allowable pile capacity for an HP12X84 section of 75 kips/pile. Based on the axial loads provided to us for each pile configuration and assuming a maximum deflection of 1 inch, the calculated lateral capacity in the strong direction is 18.5 kips. The H-piles should be extended to relative refusal at approximate depths of 20 to 30 ft below existing grades. Pile spacing of 2.5 times the pile diameter or 3 ft (whichever is greater) is recommended.

General Driven Pile Recommendations

The piles should be "seated" into the bearing stratum using criterion developed based on an acceptable dynamic driving formula - Wave Equation, Engineering News Formula, etc. The timber and steel pipe piles should extend to the dense to very dense sands and gravels encountered in each of the test borings. The estimated length of the piles is expected to be on the order of 45 to 55 ft below existing grades.

The piles should be protected during driving; the heads should be wrapped to prevent deformation, and pile tips should be used. The Wave Equation analysis should be used to determine the suitability of the proposed driving equipment. The contractor should incorporate the results of the wave equation analysis within any submittals that are due prior to construction for approval. Consideration should be given to performing dynamic monitoring on a minimum of 4 piles using a Pile Driving Analyzer (PDA). The PDA will provide information on the actual driving stresses, verification of ultimate geotechnical resistance, energy transfer efficiency, pile damage assessments, and verify the refusal criteria during pile installation. A minimum factor of safety of 2.25 should be used during the PDA testing to confirm the recommended installed pile capacity.

We recommend that the installation of each pile should be monitored and documented by geotechnical personnel under the direct supervision of a professional engineer.

5.4. SETTLEMENT

Provided the structure is supported on deep foundations total and differential settlement values are expected to be less than or equal to 1 in. total and ½ in. differential. Detrimental post-construction settlements are not expected, if the recommendations presented herein are followed.

5.5. LATERAL EARTH PRESSURE PARAMETERS

The soil parameters presented in Table 2 can be used to estimate lateral earth pressures to design below grade structures and temporary shoring. If the top of the structure is restrained from movement, thereby preventing the mobilization of active soil pressures, the structure should be designed using the at-rest pressure coefficient, k_o.

The earth pressure coefficients are based on the assumption of vertical walls, horizontal backfill, no surcharges, no wall friction, and a safety factor of 1.0. Hydrostatic pressures associated with seepage must also be considered in the design unless a drain and drainage stone layer are provided behind the wall.

TABLE 2: Lateral Earth Pressure Parameters

Parameter	*Fill*	*Stratum 1*	*Stratum 2*	*Engineered Granular Fill (PennDOT 2A)*
Unit Weight, pcf	120	100	125	135
Angle of Internal Friction, degrees	28	0	36	40
Cohesion, psf	0	200	0	0
Friction Factor (concrete)	0.34	0	0.45	0.60
k_a	0.36	1.0	0.26	0.22
k_o	0.53	1.0	0.41	0.36
k_p	2.77	1.0	3.85	4.60

If the contractor is responsible for the design of temporary or permanent retaining structures, then the contract documents should clearly require that a competent registered engineer performs the design and that the responsibility for satisfactory earth support is solely the contractor's. Furthermore, the contract documents should require the contractor to notify the engineer immediately if differing or unforeseen subsurface conditions are encountered during construction.

6. RECOMMENDATIONS FOR FURTHER GEOTECHNICAL SERVICES

Our experience on numerous construction projects is that the interests of the project team are best served by retaining the Geotechnical Engineer to provide construction observations during earthwork and foundation construction operations. To determine if soils, other materials, and ground water conditions encountered during construction are similar to those encountered in the borings, and that they have comparable engineering properties or influences on the design of the structure, we recommend that Pennoni should provide field observation services during excavation; preparation of foundation subgrades; and installation/construction of foundations. Pennoni's Geotechnical Technology should review specifications for earthwork and foundation design/construction when they are prepared.

7. LIMITATIONS

This work has been done in accordance with our authorized scope of work and in accordance with generally accepted professional practice in the fields of geotechnical and foundation engineering. This warranty is in lieu of all other warranties either expressed or implied. Our conclusions and recommendations are based on the data revealed by this exploration. We are not responsible for any conclusions or opinions drawn from the data included herein, other than those specifically stated, nor are the recommendations presented in this report intended for direct use as construction specifications. This report is intended for use with regard to the specific project described herein; any changes in loads, structures, or locations should be brought to our attention so that we may determine how they may affect our conclusions. An attempt has been made to provide for normal contingencies, but the possibility remains that unexpected conditions may be encountered during construction. If this should occur, or if additional or contradictory data are revealed in the future, we should be notified so that modifications to this report can be made, if necessary. If we do not review relevant construction documents and witness the relevant construction operations, then we cannot be responsible for any problems that may result from misinterpretation or misunderstanding of this report or failure to comply with our recommendations.

APPENDICES

APPENDIX A – FIELD DATA

TEST BORING LOG

TEST BORING B-1

LATITUDE 39.9864

LONGITUDE -75.2015

CLIENT City of Philadelphia Department of Public Property

PROJECT NAME Renovation of the East Schuylkill River Wall

PROJECT NUMBER PHILP 18003-03

PROJECT LOCATION Reviewing Stands, Kelly Drive, Philadelphia, PA 19130

DATE STARTED 10/30/19 **COMPLETED** 10/30/19

GROUND ELEVATION ---

DRILLING CONTRACTOR CGC Geoservices, LLC

WATER ENCOUNTERED:

DRILLING METHOD Hollow Stem Auger

DURING DRILLING ---

DRILLER / HELPER Jay/Bryant

AT END OF DRILLING ---

LOGGED BY E. Brinker **CHECKED BY** D. Marano

AFTER DRILLING 13.40'

DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY (in.) (RQD)	BLOW COUNTS	STRATA	Depth	DESCRIPTION	REMARKS
0	1	11	4-6-6-6	T	0.25	Topsoil	Location was 20 feet from face of wall
	2	3	5-4-6-5	F		FILL: Black and Dark Brown F/C SAND, some Silt, trace C gravel size Rock Fragments (Angular to Subrounded)	
5	3	17	4-3-3-4			FILL: Brown SILT, some F/M Sand	
	4	20	2-2-2-3		8.00		
10	5	24	WOH/6"-1-1-1	1		Dark Brown CLAYEY SILT, some F Sand	Cohesive Material From 8 feet to 27 Feet
	ST-1	24	REC=100%				
15	6	20	WOH/12"-1-1				
20	7	22	1-1-1-1			Dark Brown CLAYEY SILT, trace F Sand	
						Dark Brown CLAYEY SILT, some F Sand	
25	8	16	1-1-2-4				Spoon Wet
					28.00		Roller Bit to 28 feet Auger refusal at 28 ft
30	R-1	59 (30)	REC=99% RQD=50%	3		Dark Gray MICA SCHIST, Slightly Weathered, Moderately Hard, Multiple Fractures, Moderately Strong	
35	R-2	59 (18)	REC=98% RQD=30%		33.00	Dark Gray MICA SCHIST, Slightly Weathered, Moderately Hard, Multiple Fractures, Slightly Weak to Moderately Strong	
					38.00		

Borehole terminated at 38.00 feet.

NOTES:

Pennoni

TEST BORING LOG

TEST BORING B-2

CLIENT City of Philadelphia Department of Public Property

PROJECT NAME Renovation of the East Schuylkill River Wall

PROJECT NUMBER PHILP 18003-03

PROJECT LOCATION Reviewing Stands, Kelly Drive, Philadelphia, PA 19130

DATE STARTED 11/1/19 **COMPLETED** 11/1/19

GROUND ELEVATION ---

DRILLING CONTRACTOR CGC Geoservices, LLC

WATER ENCOUNTERED:

DRILLING METHOD Hollow Stem Auger

∇ **DURING DRILLING** 9.00'

DRILLER / HELPER Chris Lang

AT END OF DRILLING ---

LOGGED BY N. Rex **CHECKED BY** D. Marano

AFTER DRILLING ---

DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY (in.)	BLOW COUNTS	GRAPHIC LOG	STRATA	Depth	DESCRIPTION	Elev.	REMARKS
0	1	18	1-1-2-3		T	0.33	Topsoil		12 ft from river wall Organics throughout S-1
	2	16	3-3-2-2		F		FILL: Brown to Black SILT, some F/M/C Sand, little F Gravel, loose to medium density, damp		
5	3	8	2-2-2-2						
	4	6	2-2-1-2				FILL: Gray to Black SILT, some M/F Sand, trace subangular gravel loose to medium density, damp to moist		
∇ 10	5	14	2-2-2-2						
						13.00			
15	6	24	WOH/24"		1		Gray to Black SILT, little Clay, little Fine Sand, loose density, damp to moist		
20	7	16	1-WOH/6"-2-WOH/6"						
						23.00			
25	8	18	1-1-2-1		2		Tan to Gray C/M SAND, little Silt, loose density, moist		
30	9	18	20-19-14-7			30.00	Tan to Gray M/F SAND		Auger refusal at 30 ft

Borehole terminated at 30.00 feet.

NOTES:

Pennoni

TEST BORING LOG

CLIENT City of Philadelphia Department of Public Property
PROJECT NAME Renovation of the East Schuylkill River Wall
PROJECT NUMBER PHILP 18003-03
PROJECT LOCATION Reviewing Stands, Kelly Drive, Philadelphia, PA 19130
DATE STARTED 11/1/19 **COMPLETED** 11/1/19
GROUND ELEVATION ---
DRILLING CONTRACTOR CGC Geoservices, LLC
WATER ENCOUNTERED:
DRILLING METHOD Hollow Stem Auger
DURING DRILLING 7.00'
DRILLER / HELPER Chris Lang
AT END OF DRILLING ---
LOGGED BY N. Rex **CHECKED BY** D. Marano
AFTER DRILLING ---

DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY (in.) (RQD)	BLOW COUNTS	STRATA	Depth	DESCRIPTION	REMARKS
0	1	20	2-2-2-2	T	0.33	Topsoil	Boring is located 15 ft from the wall
	2	16	4-4-4-5	F		FILL: Brown to Black SILT, little M/F Sand, trace subangular Gravel, loose to medium density damp	First 4 in of S-1 contained organics
5	3	16	WOH/12"-1-WOH/6"		5.00		
				1		Gray to Black SILT, little Clay, trace M/F Sand, loose density, damp to moist	
	4	6	WOH/12"-2-3				S-4 is wet
10	5	24	1-2-3-3				S-5 is wet
15	6	22	1-1-WOH/12"				Top 2 in of S-6 is wet, trace organcis throughout
20	7	14	WOH/24"			Gray to Brown SILT, little Clay, little M/F Sand, loose density, damp	
					23.00		
25	8	10	8-50/4"	2		Gray M/C SAND, little rock fragments (1/4 in - 1/2 in), little Fine Sand, medium density, damp	S-8 is wet
					26.50		Auger refusal at 26.5 ft
	R-1	20 (11)	REC=81% RQD=44%	3		Gray to Black MICA SCHIST, medium to fine grained muscovite and biotite, quartz; slightly to moderately weathered; close fracture spacing; shallow to moderate dipping medium to strong hardness	Coring began at 26.5 ft
30	R-2	55 (44)	REC=92% RQD=73%				
35	R-3	31 (26)	REC=86% RQD=71%		36.50		

Borehole terminated at 36.50 feet.

NOTES:

Pennoni

TEST BORING LOG

CLIENT City of Philadelphia Department of Public Property
PROJECT NAME Renovation of the East Schuylkill River Wall
PROJECT NUMBER PHILP 18003-03
PROJECT LOCATION Reviewing Stands, Kelly Drive, Philadelphia, PA 19130
DATE STARTED 11/5/19 **COMPLETED** 11/5/19
GROUND ELEVATION ---
DRILLING CONTRACTOR CGC Geoservices, LLC
WATER ENCOUNTERED:
DRILLING METHOD Hollow Stem Auger
∇ **DURING DRILLING** 8.00'
DRILLER / HELPER Chris/ Joey
AT END OF DRILLING ---
LOGGED BY E. Iannetti **CHECKED BY** D. Marano
AFTER DRILLING ---

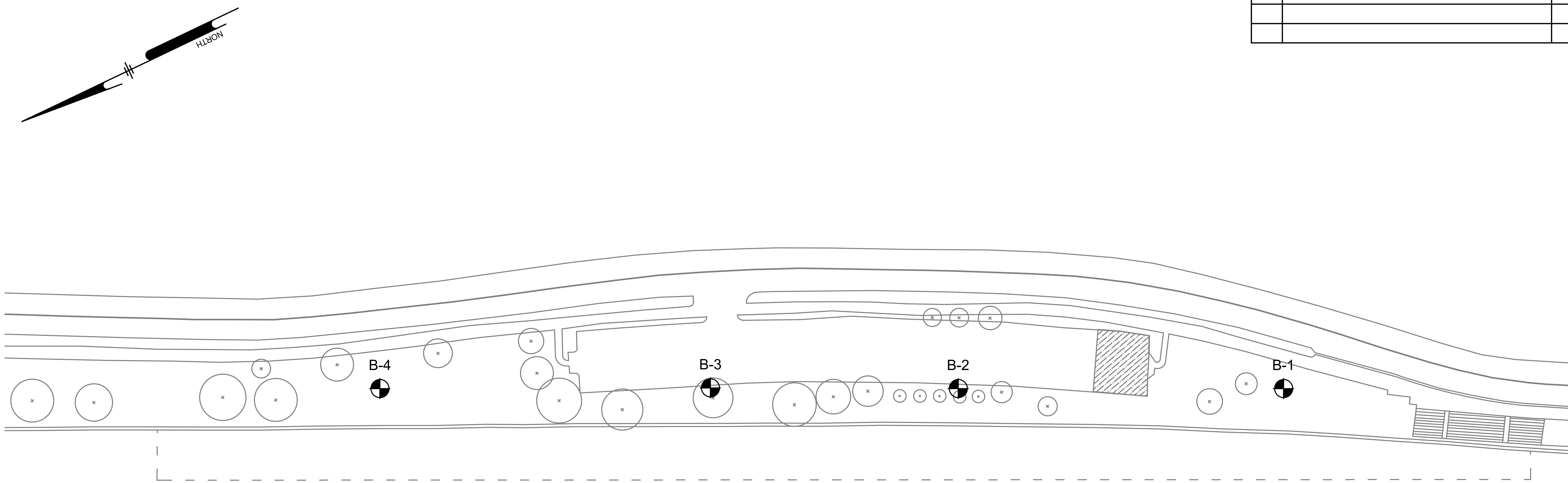
DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY (in.)	BLOW COUNTS	GRAPHIC LOG	STRATA	DESCRIPTION (Depth / Elev.)	REMARKS
0	1	12	2-3-3-4		T	0.33 Topsoil	
	2	17	2-2-3-4		F	FILL: Dark Brown SILT, little M/F Sand, trace Fine Gravel-sized rock fragments and mica	Moist
5	3	12	1-WOH/18"		1	4.00 Dark Brown to Gray SILT, some M/F Sand, trace Clay and mica	
∇	4	24	WOH/24"				
10	5	24	WOH/24"			Dark Brown to Gray C/M/F SAND, trace Fine Gravel, trace Silt and mica	
15	6	24	WOH/24"			Dark Brown to dark Grey SILT, some M/F Sand, trace mica and clay	
	7	1	50/1"		2	17.75 / 17.90 Dark Gray M/F SAND, trace silt (decomposed mica)	Auger refusal at 17.8 ft Pushed spoon to confirm refusal

Borehole terminated at 17.90 feet.

NOTES:

FOR REFERENCE ONLY

REVISIONS			
NO.	DESCRIPTION	DATE	BY
#	#	#	#



B-

INDICATES APPROXIMATE TEST BORING LOCATION AND IDENTIFYING NUMBER.

NOTES

WATER LEVELS, WHERE SHOWN, ARE THOSE OBSERVED AT THE TIME NOTED AND MAY NOT REFLECT DAILY OR SEASONAL VARIATIONS IN THE GROUND WATER LEVEL.

THE SUBSURFACE CONDITIONS REVEALED BY THIS STUDY REPRESENT CURRENT CONDITIONS AT THE SPECIFIED TEST LOCATIONS ONLY AND MAY NOT BE INDICATIVE OF CONDITIONS AT OTHER LOCATIONS.

0 100' 200'

Pennoni

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RENOVATION OF THE EAST SCHUYLKILL RIVER WALL
REVIEWING STANDS, KELLY DRIVE
PHILADELPHIA, PA

BORING LOCATION PLAN

CITY OF PHILADELPHIA
PHILADELPHIA PARKS & RECREATION
1515 ARCH STREET, 11TH FLOOR, PHILADELPHIA, PA 19102

PROJECT	PHILP18003
DATE	2020-01-07
DRAWING SCALE	1"=100'
DRAWN BY	DK
APPROVED BY	EI

LP-1

SHEET 1 OF 1

TEST BORING/TEST PIT/AUGER PROBE LOG KEY SHEET

COLUMN	DESCRIPTION
Depth	Depth in feet below ground surface
Description	Description of sample including color, texture, and classification of subsurface material as applicable. Estimated depths to bottom of strata as interpolated from the boring are also shown.
Stratum	Strata numbers as assigned by the geotechnical engineer
Sample No.	Split barrel sample and sample number (S-x) Undisturbed Tube sample and sample number (U-x) Rock core run and core number (R-x) NR indicates no recovery
Blow Counts	For soils sample (ASTM D 1586): indicates number of blows obtained for each 6 inches penetration of the standard split-barrel sampler. For rock coring (ASTM D 2113): indicates percent recovery (REC) per run and rock quality designation (RQD). RQD is the sum of rock pieces that are 4 inches or longer in length in one core run divided by the total core run.
Recovery	For soil samples indicates the length of recovery in the sample spoon
Remarks	Special conditions or test data as noted during drilling

Ground Water: Free water level as shown ( )*; * Free water level as noted may not be indicative of daily, seasonal, or long term fluctuations.

DESCRIPTIVE TERMS

RELATIVE PROPORTIONS

Descriptive Term	Symbol	Estimated Percentages
Trace	tr	1 to 10
Little	l	10 to 20
Some	sm	20 to 35
And	and	35 to 50

GRADATION OF COARSE GRAINED COMPONENTS

Soil Component	Size Range	Particle Size Maximum	Particle Size Minimum
Boulders		-	12”
Cobbles		12”	3”
Gravel	Coarse	3”	¾”
	Fine	¾”	#4 Sieve
Sand	Coarse	#4 Sieve	#10 Sieve
	Medium	#10 Sieve	#40 Sieve
	Fine	#40 Sieve	#200 Sieve
Silt		#200 Sieve	.005 mm
Clay		.005 mm	-

COMPOSITION OF COARSE-GRAINED COMPONENTS

Gradation Designation	Symbol	Defining Proportions
Coarse to Fine	CF	All fractions greater than 10% of the component
Coarse to Medium	CM	Less than 10% Fine
Medium to Fine	MF	Less than 10% Coarse
Coarse	C	Less than 10% Fine and Medium
Medium	M	Less than 10% Coarse and Fine
Fine	F	Less than 10% Coarse and Medium

APPENDIX B – LABORATORY DATA

SUMMARY OF LABORATORY DATA

BORING NUMBER	SAMPLE NUMBER	DEPTH (ft)	SOIL GROUP SYMBOL	GRAIN SIZE DISTRIBUTION: GRAVEL %	GRAIN SIZE DISTRIBUTION: SAND %	GRAIN SIZE DISTRIBUTION: SILT/CLAY %	PLASTICITY: LIQUID LIMIT w_l	PLASTICITY: PLASTIC LIMIT w_p	PLASTICITY: PLASTICITY INDEX I_P	PLASTICITY: LIQUIDITY INDEX I_L	MOISTURE CONTENT w %	VOLUMETRIC: SPECIFIC GRAVITY (G) (*) ASSUMED	VOLUMETRIC: DRY UNIT WEIGHT (pcf)	VOLUMETRIC: VOID RATIO (e)	VOLUMETRIC: DEGREE OF SATURATION %	COMPACTION DATA: MAXIMUM DRY DENSITY (pcf)	COMPACTION DATA: OPTIMUM MOISTURE CONTENT %	COMPACTION DATA: STANDARD/MODIFIED	RESISTIVITY (KΩ-CM)	CHLORIDE (%)	SULFATE (%)	pH	SHEAR STRENGTH: UNCONFINED COMPRESSIVE STRENGTH (tsf)	SHEAR STRENGTH: COHESION (tsf)	SHEAR STRENGTH: AXIAL STRAIN (%)
B-1	S-3	4-6			21	79					26.7														
B-2	S-1-3	0-6		13	30	57					31.3								3.4	0.0085	0.0063	6.93	0.55	0.27	10.3
B-2	S-9	28-30	SP		97	3					19.6														
B-4	S-4	6-8					31	27	4	3.3	40.2														
B-1	R-2	33-38																					663.12		
B-3	R-2	28.5-33.5																					1769.04		

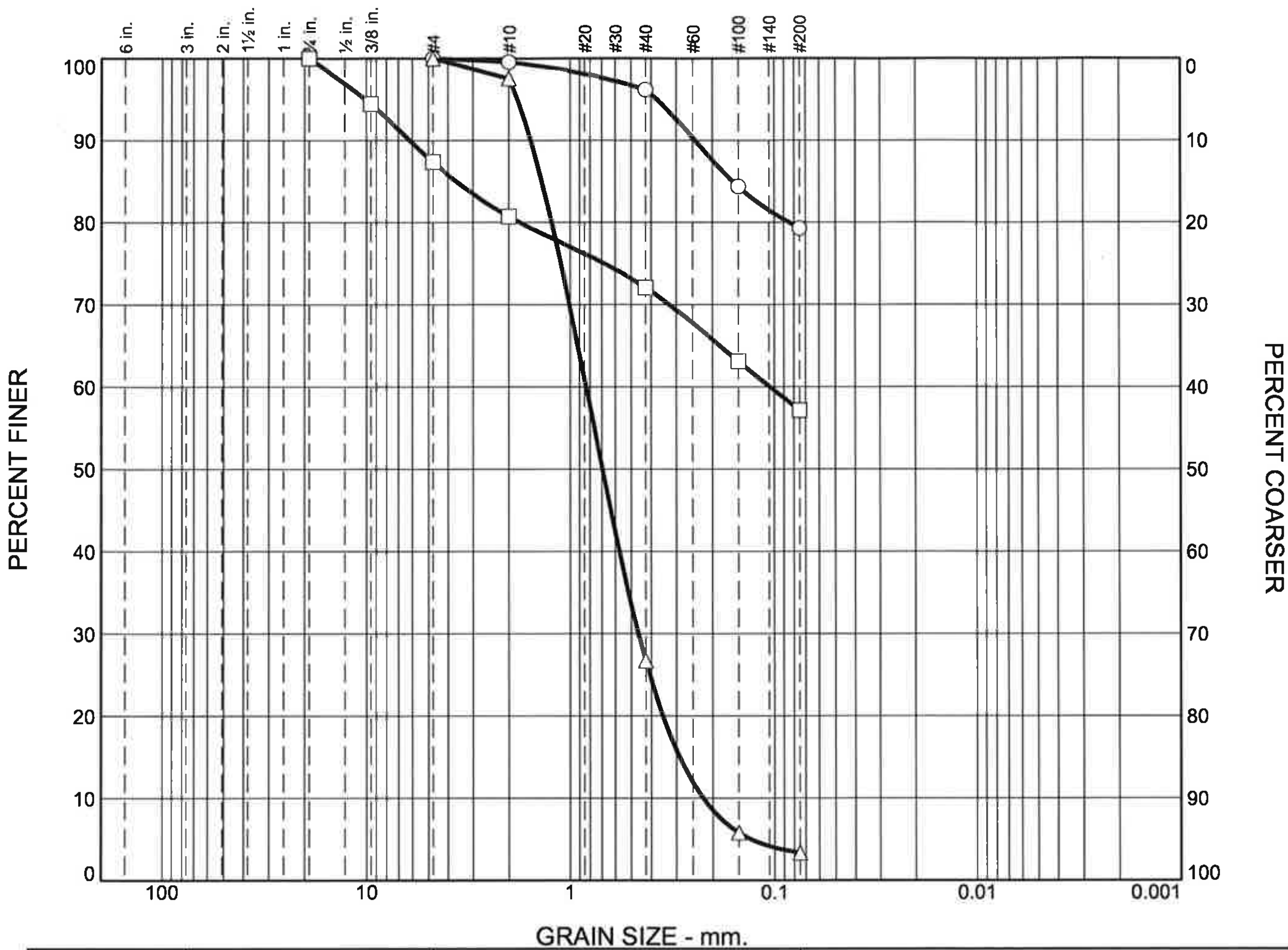
PENNONI ASSOCIATES INC.

DRAWN BY: BL DATE: 11/21/2019
CHECKED BY: DATE:

PROJECT: RENOVATION OF THE EAST SCHUYKILL RIVER WALL
LOCATION: PHILADELPHIA, PA

JOB No.: PHILP 18003
TABLE No.: L-1

Particle Size Distribution Report



	% +3"	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○	0.0	0.0	0.0	0.5	3.3	16.9	79.3	
□	0.0	0.0	12.6	6.7	8.6	15.0	57.1	
△	0.0	0.0	0.0	2.5	70.8	23.4	3.3	

SOIL DATA

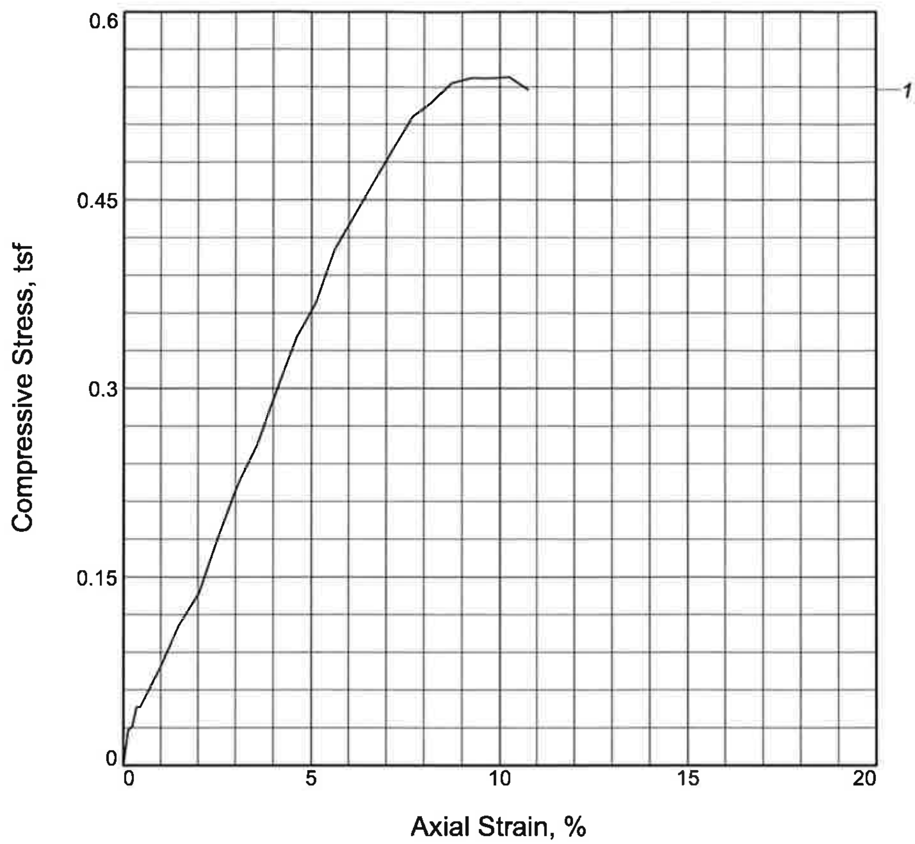
SYMBOL	SOURCE	SAMPLE NO.	DEPTH (ft.)	Material Description	USCS
○	B-1	S-3	4-6	SILT, SOME F/M SAND	
□	B-2	S-1-3	0-6	SILT, SOME F/M/C SAND, LITTLE FINE GRAVEL	
△	B-2	S-9	28-30	M/F SAND	SP

PENNONI ASSOCIATES INC.

Client: PHILADELPHIA PARKS & RECREATION
Project: RENOVATION OF THE EAST SCHUYKILL RIVER WALL

Project No.: PHILP 18003 **Figure** S-1

UNCONFINED COMPRESSION TEST



Sample No.	1			
Unconfined strength, tsf	0.548			
Undrained shear strength, tsf	0.274			
Failure strain, %	10.3			
Strain rate, in./min.	0.03			
Water content, %	40.2			
Wet density, pcf	112.9			
Dry density, pcf	80.5			
Saturation, %	102.9			
Void ratio	1.0171			
Specimen diameter, in.	1.38			
Specimen height, in.	2.76			
Height/diameter ratio	2.00			

Description:

LL = 31	**PL =** 27	**PI =** 4	**Assumed GS=** 2.60	**Type:** SPT

Project No.: PHILP 18003
Date Sampled:
Remarks:

Client: PHILADELPHIA PARKS & RECREATION

Project: RENOVATION OF THE EAST SCHUYKILL RIVER WALL

Source of Sample: B-4 **Depth:** 6-8
Sample Number: S-4

UNCONFINED COMPRESSION TEST
PENNONI ASSOCIATES INC.

Figure U-1

Tested By: BL

LABORATORY TESTING PROCEDURES

All testing is either done in accordance with the indicated ASTM Designation-latest edition, or with other standard or generally accepted engineering practice as described:

1. Consolidation Test of Soils
   Preparation of samples and testing procedures generally follow the methods described in Lambe, op. Cit. In addition, the time of loading may be selected on the basis of:

   a. Controlled rate of percent of consolidation
   b. Controlled pore pressure gradient
   c. Controlled strain

   The method of test is selected to suit the soil type in question and the test is conducted in accordance with generally accepted engineering practice.

2. Atterberg Limits – Plasticity Indices
   a. Liquid limit of soils, ASTM D 4318
   b. Plastic limit and plasticity index of soils, ASTM D 4318
   c. Shrinkage factors of soils, ASTM D 427

   (Moisture content is also determined with the Atterberg Limit test, and liquidity index is also computed)

3. Moisture Content of Soil
   ASTM D 2216

4. Particle Size Analysis of Soils
   ASTM D 421, Dry preparation of soil samples; ASTM D 422, Sieve and/or hydrometer analysis.

5. Triaxial Compression Test of Soils
   Sample preparation, apparatus, and testing generally follow the procedures outlined in Soil Testing for Engineers, T.W. Lambe, John Wiley & Sons, Inc., New York, 1951 and in The Measurement of Soil Properties in the Triaxial Test, Alan W. Bishop & D.J. Henkel, 2nd Edition, St. Martin's Press, New York, 1962

6. Unconfined Compression Strength of Cohesive Soil
   ASTM D 2166

7. Specific Gravity of Soils
   ASTM D 854

8. Unit Weight Determination of Soils
   See ASTM D 2166 for preparation of specimen except that sample size may differ. For moisture content see ASTM D 2216.

9. Visual Identification of Soil Samples
   All soil samples are visually identified and/or classified. The classification system used is shown in Table L-1.

10. Identification of Rock
    Rock core samples are identified by the character and appearance of newly fractured surfaces of unweathered pieces, by core conditions and characteristics, and by the determination of simple physical and chemical properties.

11. Compaction Test of Soils
    a. Moisture-density relations of soils using 5.5 lb. hammer and 12 in. drop, ASTM D 698
    b. Moisture-density relations of soils using 10 lb. hammer and 18 in. drop, ASTM D 1557

12. Maximum and Minimum Densities of Granular Soils
    Testing procedures follow D.M. Burmeister, "Suggested Method of Test for Maximum and Minimum Densities of Granular Soils" cited in Proceedings for Testing Soils, Fourth Edition, ASTM, Philadelphia. 1964, pp 175-177.

13. Bearing Ratio of Laboratory Compacted Soils
    ASTM D 1883 (Sometimes called California Bearing Ratio or CBR)

14. Organic Content
    A modified dichromate oxidation method using ferrous ammonium sulfate is employed in determining the percent of organic matter in soil.

APPENDIX C – STANDARD SYMBOLS

STANDARD SYMBOLS

B	Width of footing
c	cohesion
c_v	coefficient of consolidation
C_c	compression index
C	coefficient of secondary compression
C_3	swelling index
C_u	uniformity coefficient (D_{60}/D_{10})
CBR	California Bearing Ratio
D_f	depth of foundation
D_p	diameter of grain corresponding to percentage p on grain size curve
D_{10}	effective grain size
E	modulus of linear deformation
E_s	Young's Modulus
e	void ratio
F_s	factor of safety
G	specific gravity
h	hydraulic head
H	stratum thickness
i	hydraulic gradient
I_L	liquidity index
I_P	plasticity index
k	coefficient of permeability
k_h	coefficient of horizontal subgrade reaction
k_v	coefficient of vertical subgrade reaction
l	length of footing
n	porosity
P	deviator stress
P_c	estimated probable preconsolidation pressure
P_o	existing overburden pressure
q_a	allowable soil bearing pressure
Q	triaxial compression test unconsolidated and undrained
Q_c	triaxial compression test consolidated and undrained
S	triaxial compression test consolidated and drained
S_r	degree of saturation
υ	pore-water pressure
U	degree of consolidation
U_c	unconfined compression test
w_f	moisture content at end of test
w_l	liquid limit
w_n	natural moisture content
w_p	plastic limit
γ	unit weight
γ_d	dry unit weight
γ_b	submerged unit weight
ε	unit linear strain
ε_f	unit linear strain at failure
σ	normal stress
σ_1	major principal stress
σ_3	minor principal stress
τ	shear stress
ϕ	angle of internal friction
k_a	coefficient of active pressure
k_p	coefficient of passive pressure
δ	friction angle
$\tan \delta$	friction factor

APPENDIX D – IMPORTANT INFORMATION ABOUT THIS GEOTECHNICAL ENGINEERING REPORT (PUBLISHED BY THE GBA)

Important Information about This Geotechnical-Engineering Report

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

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

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

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical-engineering study conducted for a civil engineer may not fulfill the needs of a constructor — a construction contractor — or even another civil engineer. Because each geotechnical- engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client. No one except you should rely on this geotechnical-engineering report without first conferring with the geotechnical engineer who prepared it. *And no one — not even you* — should apply this report for any purpose or project except the one originally contemplated.

Read the Full Report

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

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

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

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

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

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

As a general rule, *always* inform your geotechnical engineer of project changes—even minor ones—and request an assessment of their impact. *Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.*

Subsurface Conditions Can Change

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

Most Geotechnical Findings Are Professional Opinions

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

A Report's Recommendations Are Not Final

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

A Geotechnical-Engineering Report Is Subject to Misinterpretation

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

Do Not Redraw the Engineer's Logs

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

Give Constructors a Complete Report and Guidance

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

Read Responsibility Provisions Closely

Some clients, design professionals, and constructors fail to recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that have led to disappointments, claims, and disputes. To help reduce the risk of such outcomes, geotechnical engineers commonly include a variety of explanatory provisions in their reports. Sometimes labeled "limitations," many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Environmental Concerns Are Not Covered

The equipment, techniques, and personnel used to perform an *environmental* study differ significantly from those used to perform a *geotechnical* study. For that reason, a geotechnical-engineering report does not usually relate any environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated environmental problems have led to numerous project failures.* If you have not yet obtained your own environmental information, ask your geotechnical consultant for risk-management guidance. *Do not rely on an environmental report prepared for someone else.*

Obtain Professional Assistance To Deal with Mold

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

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

Membership in the Geotechnical Business Council of the Geoprofessional Business Association exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project. Confer with you GBC-Member geotechnical engineer for more information.

GBC GEOTECHNICAL BUSINESS COUNCIL
of the Geoprofessional Business Association

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