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Geotechnical Report - Addendum

Proposed Wex Building

Corner of Hancock & Thames Street, Portland, Maine

Client
Archetype, PA
Union Wharf
Portland, Maine 04101

SUMMIT
GEOENGINEERING SERVICES

Project #: 17181
Date: 9/5/17

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September 5, 2017
Summit #17181

Attn: Bill Hopkins
Archetype, PA
Union Wharf
Portland, Maine 04101

Reference: Geotechnical Report Addendum – Proposed Wex Building
Corner of Hancock & Thames Street, Portland, Maine

Dear Bill;

A geotechnical report for the above referenced project was prepared by Summit Geoengineering Services, Inc. (SGS) dated July 20, 2017. Subsequent to the completion of our original geotechnical report, the structural loads for the building were revised by Veitas & Veitas (V&V). The load revisions were provided to SGS on August 8, 2017. In general, the loads for all columns increased. The maximum point load (dead + live load) increased from a value of 422 kips to a value of 720 kips. We understand that uplift loads are uncertain at this time.

This addendum also includes the results of test pits completed at the site on August 23, 2017. The purpose of these test pits was to determine the quality of the existing fill and to evaluate its potential for reuse during construction.

Based on the updated compressive loads, our previously recommended "floating stone column" ground improvement method will result in excessive foundations settlements due to applied stress to the underlying clay layer.

We recommend that the proposed building foundation be supported on end-bearing elements consisting of either driven piles or rigid inclusion ground improvement. An evaluation and recommendations are provided herein for both foundation types.

It should be noted that Section 1.0 (Project and Site Description), Section 2.0 (Subsurface Exploration and Laboratory Testing), Section 3.0 (Subsurface Conditions), Section 5.3 (Frost Protection), Section 5.4 (Seismic Design), Section 5.5 (Groundwater Control), Section 5.6 (Slab-on-Grade), Section 6.0 (Pavement Recommendations), and Section 7.0 (Earthwork Considerations) from the original geotechnical report are still valid and applicable.

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1.0 Geotechnical Evaluation and Recommendations

Based on the revised column loads provided to us by V&V, we anticipate that shallow spread footings supported by the existing fill or footings supported on ground improvement (stone columns) terminating in the fill will result in maximum total settlement exceeding 3 to 4 inches. We understand that this exceeds the tolerable settlement for the proposed structure.

To limit settlements of the proposed foundation to within a tolerable magnitude, we recommend that the proposed foundation be supported using end bearing piles or end bearing rigid inclusion elements. The slab for the new building can be constructed on-grade and does not need to be supported with piles or rigid inclusions.

1.1 Rigid Inclusion Ground Improvement:

Rigid inclusions (RI) typically consist of concrete or grouted stone elements extending to a dense stratum or refusal to support foundation loads. The foundation loads are transferred to the stiff RI elements using a Load Transfer Platform (LTP) constructed between the top of the RI elements and the bottom of footings. LTPs may consist of engineered fill (sand and/or gravel) with possible layers of geogrid within the fill to ensure complete load transfer to the RI elements.

One limitation of the RI ground improvement technique is the maximum available installation length of the elements. Based on discussions with a specialty contractor who design and installs RI, we understand that the maximum length of RI elements is 60 to 65 feet due to equipment limitations. Refusal was encountered in our explorations between 59.0 to 65.5 feet. Refusal was encountered at the adjacent hotel site, directly west across Hancock Street, at a depth of 72 feet. Since the elements must be founded on dense soil or refusal, there is a possibility that the required installation length will exceed 65 feet.

Uplift capacity for the proposed foundation should also be considered when selecting a foundation support type. Upon discussion with the RI specialty contractor, we understand that RI elements are not capable of supporting large uplift loads without a structural connection to the footing.

Additionally, we understand that any existing fill at the site that is exported will require special treatment and disposal. Therefore, to reduce construction costs, it is advantageous to limit the volume of exported material. The associated disposal cost of the soil displaced by the LTP and any over-excavation required to deepen the footing to add uplift capacity should be considered when evaluating the RI ground improvement option.

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If RI ground improvement is selected to support the proposed construction, we recommend that the RI elements and the LTP be designed and stamped by a qualified Maine Licensed Professional Engineer. The contractor submittal shall include detailed design computations and construction installation drawings. The submittal shall also include provisions for completion of the work if refusal depths exceed the equipment reach. SGS should be retained to review the contractor submittal on behalf of our client. The bearing capacity of the RI system should meet or exceed the 5,000 psf allowable bearing pressure provided in the original geotechnical report. Total settlement should not exceed 1.0" and differential settlement between two adjacent columns should not exceed 0.5".

Any rubble/debris encountered in the upper fill layer that restricts RI installation will need to be removed or the hole will need to be pre-augered. Based on our explorations and the historic maps of the site, we anticipate that old foundation elements and rubble will be encountered in various locations.

Soil parameters used in the design of the RI and LTP systems are at the discretion of the designer. We have provided some recommended geotechnical soil properties for the existing fill and the glacial marine (clay) in Section 2.0.

1.2 End Bearing Piles:

End bearing piles for this site would consist of H-Piles or steel pipe piles driven to refusal. We anticipate that the settlement of footings supported with end bearing piles will be negligible.

We recommend that piles consist of Grade 50 steel and that all piles be vibrated or driven to a dense stratum, either glacial till or bedrock, which is anticipated to range from 50 feet to 70 feet below the current ground surface.

To provide pile design recommendations, we have preliminarily assumed that the piles will have a minimum diameter of 10". SGS should be notified in order to provide updated recommendations if smaller piles are selected for design. We recommend that the pile design be performed and stamped by a qualified Maine Licensed Professional Engineer and the design be made available to Summit Geoengineering Services, Inc. for review.

We recommend that piles be designed and installed in accordance with the International Building Code 2015 (IBC 2015), Section 1810. The designed piles should be verified with a WEAP analysis to ensure that driving stresses do not exceed the allowable capacity of the piles. To ensure that the pile does not become damaged during driving through the upper fill layer, we recommend that a steel driving shoe (or steel conical tip, if a pipe pile is used), be welded to the end of the piles. Any rubble/debris encountered in the upper fill layer that restricts pile driving will need to be removed to prevent damage to the pile. Based on our explorations and the historic maps of the site, we anticipate that old foundation elements and rubble will be encountered in various locations.

The piles can be designed using the soil properties in Section 2.0. All non-load bearing elements such as grade beams can be proportioned using an allowable bearing pressure of 2,000 psf.

1.2.1 Lateral Support

We recommend that the allowable lateral capacity of the installed piles be taken as a maximum of 4 kips per pile in the direction of the major principle axis. If a higher lateral capacity is desired, the pile designer shall submit a lateral capacity computation for review by SGS. All soil within a 3 foot width beyond the edge of the pile in all directions should be proofrolled with a minimum of 4 passes in each of two perpendicular directions with a 5-ton (operating weight) vibratory roller. Any unsuitable soils exposed at the ground surface around the pile should be removed and replaced with SF or ¾" Crushed Stone. If fill is required to raise the grade around the pile, it should consist of SF placed in 12" lifts and compacted to 95% of the dry density in accordance with ASTM D1557. Lateral capacity of piles which are spaced closer than 8 pile diameters center-to-center in the direction of loading should be reduced using the following table:

Table 1: Lateral Capacity Reduction

LATERAL CAPACITY REDUCTION	
Pile Spacing (in direction of loading)	**Capacity Reduction**
8D	1.00
6D	0.70
4D	0.40
3D	0.25

1.2.2 Corrosion Protection

We recommend that corrosion resistance measures be taken to protect the long-term integrity of the piles. In the order of preference, these measures include:

- If pipe piles are used, filling the piles with concrete
- Increasing the size of the steel pile to account for area loss over time

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- Coating the pipe pile with a corrosion inhibitor

To increase the corrosion protection, more than one of the above mentioned methods can be used. The corrosion rate of an uncoated steel pile is estimated to be in the order of 0.001 in/year.

1.2.3 Uplift Resistance

We recommend that the ultimate uplift capacity of the H-piles or pipe piles be taken as the dead weight of the pile, pile cap, soil above the pile cap, friction of the mobilized soil, and soil friction resistance along the length of the pile. We recommend that factor of safety of 1.0 be used for the dead weight calculations, and a factor of safety of 2.5 be used for the mobilized soil and soil friction resistance along the pile. The ultimate soil friction resistance along the pile can be calculated using the coefficients provided in Table 2.

1.2.4 Pile Splices

We anticipate that pile splices will be required for some of the installed piles. The design of all pile splices should be in accordance with IBC 2015 Section 1810.3.6.

1.2.5 Downdrag

Assuming that the proposed fill height (including slab) is 2.5 feet or less, we anticipate that consolidation of the clay will be negligible and downdrag force along the length of the pile embedded in the clay can be ignored in the pile design.

1.2.6 Pile Testing and Field Monitoring

All piles should be installed to an ultimate capacity equal to the allowable axial capacity multiplied by a factor of safety of 2.5. To ensure that this capacity is developed, and to avoid over-stressing of the installed piles, we recommend dynamic pile testing (PDA) be performed on select piles in accordance with ASTM D4945. We further recommend that a specialty consultant be used to perform these tests.

In addition to the PDA testing, we also recommend that a detailed pile-driving log for each pile be performed and reviewed to evaluate pile installation and consistency. The contractor or a qualified technician can record the pile-driving logs. If the contractor is selected to record the pile driving logs, we recommend that SGS review the logs and verify that the piles are being installed within the design recommendations.

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We recommend that the skin friction values generated by the compressive load test (ASTM D4945) be evaluated to verify the field uplift capacity.

Field testing for lateral capacity is not required.

2.0 Soil Parameter Recommendations:

The following table presents soil parameters to be used in the structural design of the rigid inclusions or piles:

Table 2: Geotechnical Design Parameters

GEOTECHNICAL PARAMETERS					
PARAMETER	**[1]EXISTING FILL**	**GLACIAL MARINE**	**[2]STRUCTURAL FILL**	**[2]FOUNDATION BACKFILL**	**CRUSHED STONE**
Total Unit Weight (γ_t)	120 pcf	125 pcf	135 pcf	130 pcf	115
Submerged Unit Weight (γ_B)	58 pcf	63 pcf	73 pcf	68 pcf	53 pcf
Effective Friction Angle (ϕ')	32°	10°	34°	32°	40°
Cohesion (c)	0 psf	600 psf	0 psf	0 psf	0 psf
Interface Friction Angle (δ), C.I.P. Conc.	25°	0°	28°	26°	30°
Interface Friction Angle (δ), Steel	20°	0°	20°	20°	22°
Adhesion (c_a)	0 psf	500 psf	0 psf	0 psf	0 psf

[1]***Note:*** *Existing Fill refers to granular soil clear of trash, debris, and rubble.*
[2]***Note:*** *Soil Parameters for Structural Fill and Foundation Backfill assume that the fill is placed in 12" maximum lifts and compacted to 95% of the dry density in accordance with ASTM D1557.*

Lateral capacity can also be developed by the soil resistance against the pile caps, grade beams, footings, and walls. If lateral deflection of the foundation element is greater than 0.005 feet per foot of depth, the passive resistance of the soil will be mobilized. If lateral deflections are less than 0.005 feet per foot of depth, at-rest soil conditions will be present. Depending on the anticipated deflection, we recommend that either passive (K_p) or at-rest (K_o) coefficient be used to calculate the soil resistance against grade beams, pile caps, footings, and walls. These coefficients can be computed using the effective friction angles in Table 2.

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3.0 Earthwork Considerations – Reuse of Existing Soil

3.1 Subsurface Explorations

Summit Geoengineering Services (SGS) observed the subsurface conditions at the site with the excavation of 2 test pits on August 23, 2017, advanced to depths of 4.5 to 5 feet with no refusal. Test pits were excavated by Shaw Bros. using a Takeuchi TB285 Tracked Mini Excavator under direct supervision of SGS. The test pits were located on the day of the explorations by taping from existing site features. The locations of the test pits are shown on the exploration location plan in Appendix A. The test pit logs are included in Appendix B.

Existing rubble was encountered in TP-1 starting at a depth of 3.5 feet. The rubble consisted of granite pieces, which are likely remnants of an old foundation. Also intermixed in the fill was brick, glass, metal (railroad spikes), and small wood pieces. Only trace rubble (glass and metal) was encountered in TP-2.

Laboratory testing, consisting of Grain Size Analyses (*ASTM D422*), were performed on samples of the existing fill collected from each of the test pits. A summary of the results are presented below. Detailed results can be found in Appendix C.

Table 3: Laboratory Test Results

GRAIN SIZE ANALYSIS RESULTS – FILL						
Test Pit	**Sample**	**Depth (ft.)**	**Composition**			**USCS**
			Gravel	Sand	Silt	
TP-1	S-1	3.5'	59%	38%	3%	GP
TP-2	S-1	1.0'	56%	41%	3%	GP
TP-2	S-2	4.0'	25%	69%	6%	SP-SM

USCS = Unified Soil Classification System, GP = Poorly Graded Gravel, SP-SM = Poorly Graded Sand with Silt

Based on the laboratory testing results, we anticipate that the clean portion (no rubble or debris) of the existing fill can be reused as Foundation Backfill (FB) or Structural Fill (SF). We recommend that when this soil is reused, all man-made materials and organics are removed from the soil stockpile. We further recommend that periodic grains size analyses be performed throughout the earthwork period to confirm that the reused fill is consistent with the gradation requirements of FB and SF.

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4.0 Closure

Our recommendations are based on professional judgment and generally accepted principles of geotechnical engineering and project information provided by others. Some changes in subsurface conditions from those presented in this report may occur. Should these conditions or the proposed development differ from those described in this report, SGS should be notified so that we can re-evaluate our recommendations. SGS should be provided an opportunity to review the Stone Column submittal package.

It is recommended that this report be made available in its entirety to contractors for informational purposes and be incorporated in the construction Contract Documents. We recommend that SGS be retained to review final construction documents relevant to the recommendations in this report.

We appreciate the opportunity to serve you during this phase of your project. If there are any questions or additional information is required, please do not hesitate to call.

Sincerely yours,

Mathew Hardison, EI
Geotechnical Engineer

William M. Peterlein, PE
President & Principal Engineer

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APPENDIX A

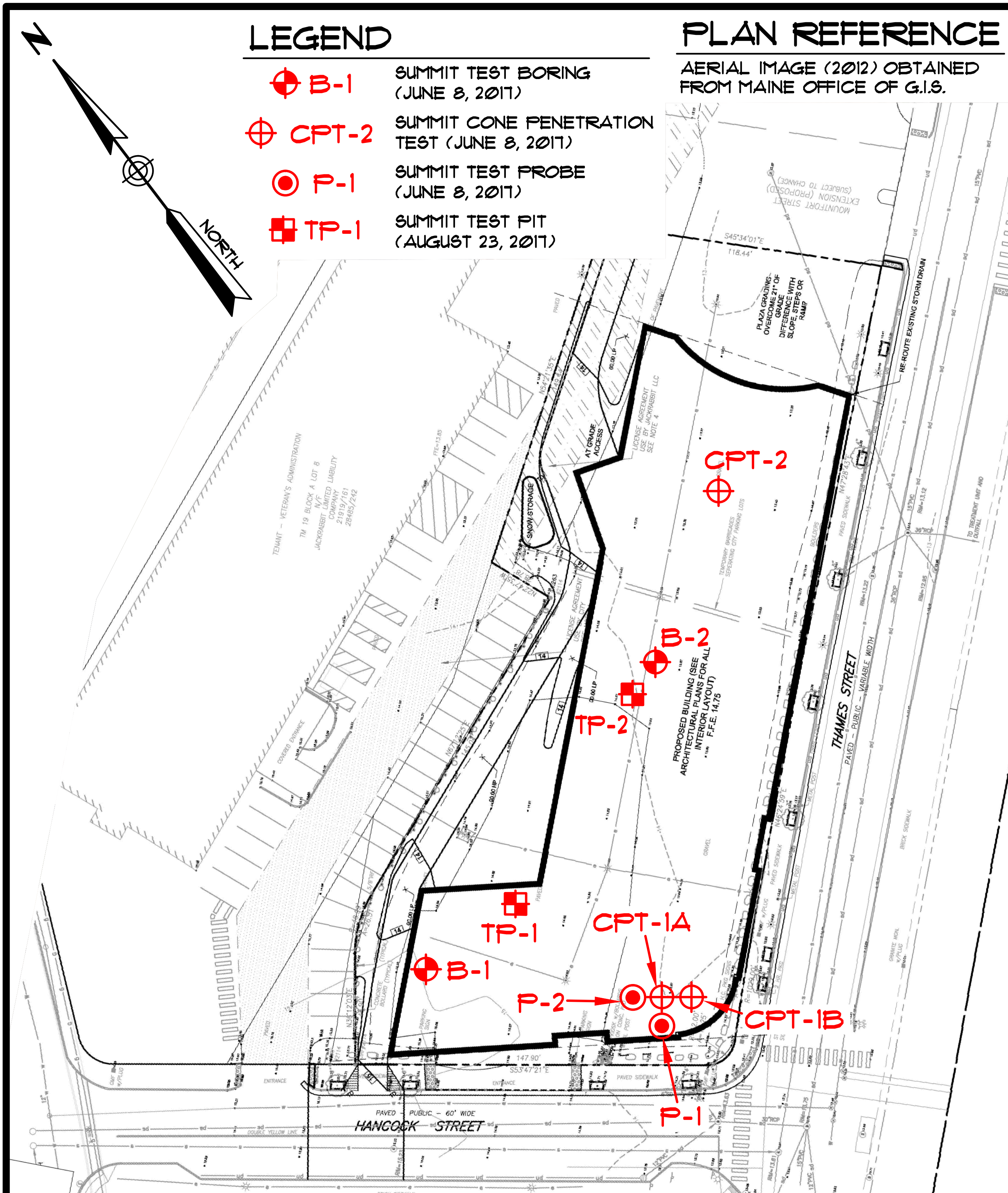
EXPLORATION LOCATION PLAN

LEGEND

B-1	SUMMIT TEST BORING (JUNE 8, 2017)
CPT-2	SUMMIT CONE PENETRATION TEST (JUNE 8, 2017)
P-1	SUMMIT TEST PROBE (JUNE 8, 2017)
TP-1	SUMMIT TEST PIT (AUGUST 23, 2017)

PLAN REFERENCE

AERIAL IMAGE (2012) OBTAINED FROM MAINE OFFICE OF G.I.S.



EXPLORATION LOCATION PLAN
WEX BUILDING

HANCOCK & THAMES STREETS - PORTLAND, ME
PREPARED FOR
ARCHETYPE, PA

DATE: 9-1-2017	DRAWN BY: KRF	CHECKED BY: WMP
JOB: 17181	SCALE: 1" = 60'	FILE: 17181 BOR

145 LISBON ST. - SUITE 601
LEWISTON, ME 04240
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APPENDIX B

TEST PIT LOGS

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SUMMIT GEOENGINEERING SERVICES	**TEST PIT LOG**		Test Pit # **TP-1**
	Project: Proposed Wex Building Corner of Hancock and Thames Portland, ME		Project #: 17181 Groundwater: None Encountered
Contractor: Shaw Brothers	Ground Surface Elevation: 14.5 ft. +/-		
Equipment: Takeuchi TB285 Mini Excavator	Reference: "Grading and Drainage Plan" prepared by Stantec 7/12/17		
Summit Staff: M. Hardison, E.I.	Date: 8/23/2017	Weather: Sunny, 65°	

Depth (ft)	DESCRIPTION: ENGINEERING	DESCRIPTION: GEOLOGIC/GENERAL
	3.5" Pavement, poor condition	PAVEMENT
1	Light Brown fine to coarse SAND, little to some Gravel, trace to little Silt, humid, SP-SM	FILL
2	Dark Brown Sandy GRAVEL, trace Silt, humid, GP	FILL with RUBBLE
3		Large Granite Block at 3.5' depth, probable remnant
4	Similar to above, intermixed brick, wood, glass, metal (railroad spikes), and granite pieces	of old foundation **Grain Size = 59% Gravel, 38% Sand, 3% Silt**
5	End of Test Pit at 4.5', no refusal	
6		
7		
8		
9		
10		
11		
12		
13		
14		
15		
16		
17		

TEST PIT LOG

SUMMIT GEOENGINEERING SERVICES	**TEST PIT LOG**	Test Pit #	**TP-2**
	Project: Proposed Wex Building, Corner of Hancock and Thames, Portland, ME	Project #:	17181
		Groundwater:	None Encountered
Contractor: Shaw Brothers	Ground Surface Elevation: 13.8 ft. +/-		
Equipment: Takeuchi TB285 Mini Excavator	Reference: "Grading and Drainage Plan" prepared by Stantec 7/12/17		
Summit Staff: M. Hardison, E.I.	Date: 8/23/2017	Weather:	Sunny, 65°

Depth (ft)	DESCRIPTION: ENGINEERING	DESCRIPTION: GEOLOGIC/GENERAL
1	Light Brown fine to coarse Sandy GRAVEL, trace Silt, humid, SP or SP-SM	FILL **Grain Size = 56% Gravel, 41% Sand, 3% Silt**
2		
3		
4	Similar to above, Gravelly SAND content, little Silt, moist, trace glass and metal pieces	**Grain Size = 25% Gravel, 69% Sand, 6% Silt**
5		
6	End of Test Pit at 5.0', no refusal	
7		
8		
9		
10		
11		
12		
13		
14		
15		
16		
17		

APPENDIX C

LABORATORY TEST RESULTS

SUMMIT GEOENGINEERING SERVICES

GRAIN SIZE ANALYSIS - ASTM D6913

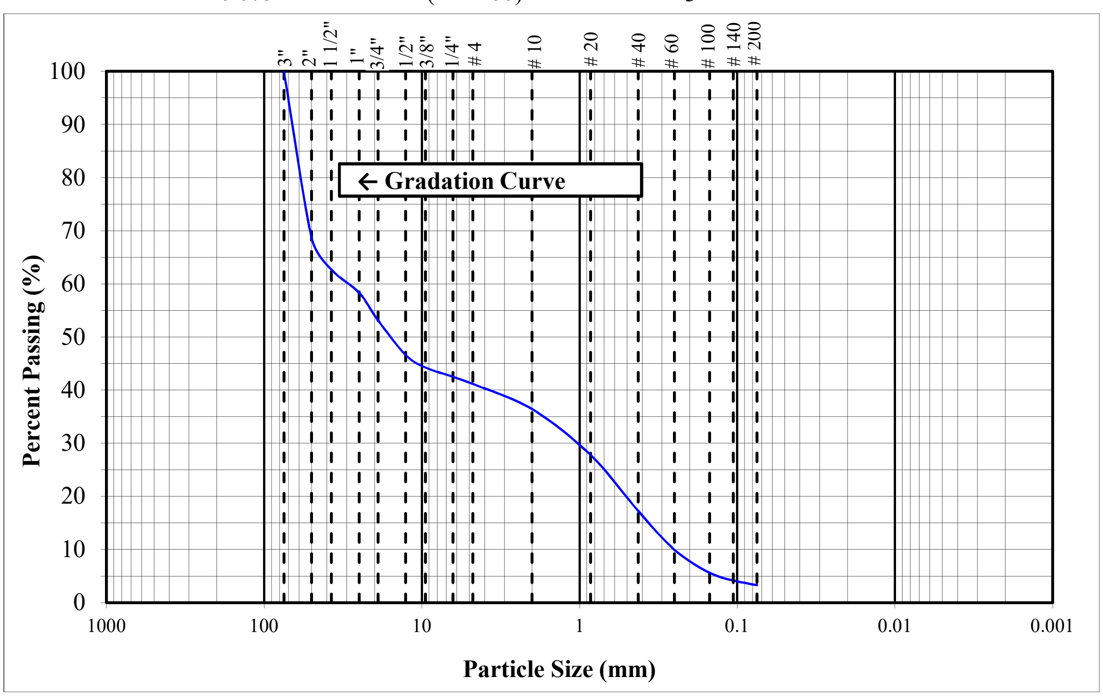
PROJECT NAME:	Proposed Wex Building	PROJECT #:	17181
PROJECT LOCATION:	Hancock & Thames St., Portland, ME	EXPLORATION #:	TP-1
CLIENT:	Archetype, P.A.	SAMPLE #:	S-1
TECHNICIAN:	Erika Stewart, P.E.	SAMPLE DEPTH:	3.5 ft
SOIL DESCRIPTION:	Sandy GRAVEL, trace Silt, GP	TEST DATE:	8/28/2017

TEST PROCEDURE

Sample Source: Test Pit	**Sieve Stack:** Composite	**Specimen Procedure:** Moist
Test Method: Method A	**Separating Sieve(s):** 3/8 Inch	**Dispersion Type:** Tap Water

DATA

STANDARD SIEVE DESIGNATION (mm)	ALTERNATIVE SIEVE DESIGNATION (in)	PERCENT PASSING (%)
75	(3 in)	100
50	(2 in)	69
37.5	(1-1/2 in)	63
25.0	(1 in)	58
19.0	(3/4 in)	53
12.7	(1/2 in)	47
9.5	(3/8 in)	44
6.35	(1/4 in)	42
4.75	(No. 4)	41
2.00	(No. 10)	36
0.850	(No. 20)	28
0.425	(No. 40)	17
0.250	(No. 60)	10
0.150	(No. 100)	6
0.106	(No. 140)	4
0.075	(No. 200)	3



REMARKS: Moisture Content = 5.5%. Sample contained gravel sized pieces of brick and tar conglomerates. Little to trace asphalt/tar pieces were observed in the wash water and finer sieve set. The sample contained one large gravel sized piece granite rock. The sample is undersized based the maximum partle size.

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SUMMIT GEOENGINEERING SERVICES

GRAIN SIZE ANALYSIS - ASTM D6913

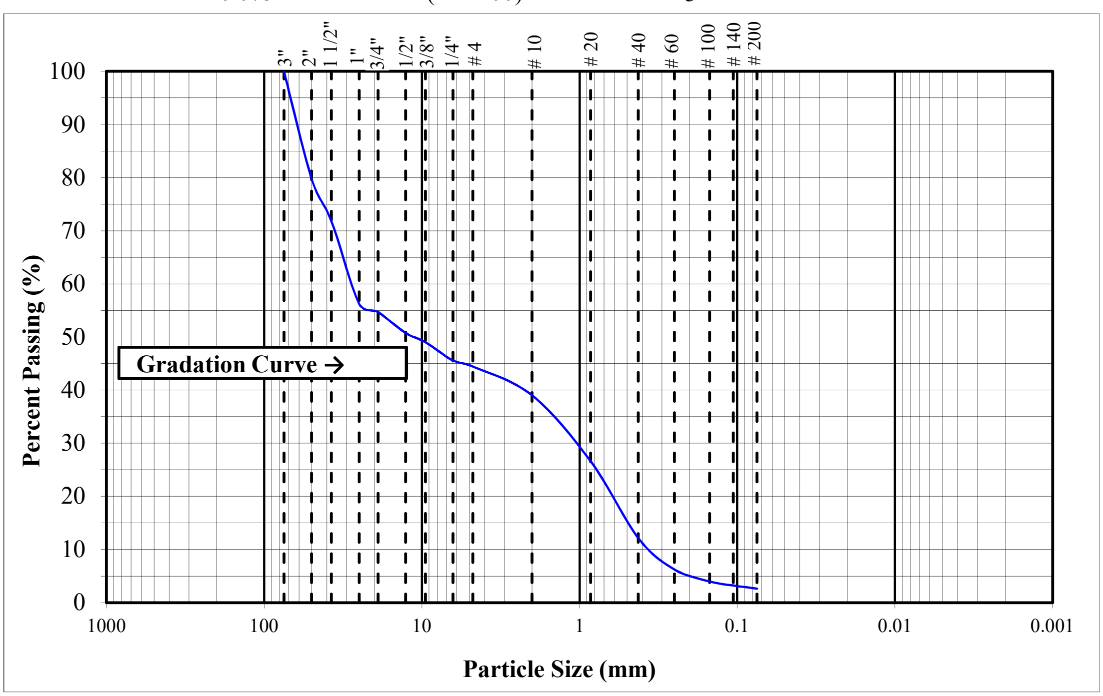
PROJECT NAME:	Proposed Wex Building	PROJECT #:	17181
PROJECT LOCATION:	Hancock & Thames St., Portland, ME	EXPLORATION #:	TP-2
CLIENT:	Archetype, P.A.	SAMPLE #:	S-1
TECHNICIAN:	Erika Stewart, P.E.	SAMPLE DEPTH:	1 ft
SOIL DESCRIPTION:	Sandy GRAVEL, trace Silt, GP	TEST DATE:	8/28/2017

TEST PROCEDURE

Sample Source: Test Pit	**Sieve Stack:** Composite	**Specimen Procedure:** Moist
Test Method: Method A	**Separating Sieve(s):** 3/8 Inch	**Dispersion Type:** Tap Water

DATA

STANDARD SIEVE DESIGNATION (mm)	ALTERNATIVE SIEVE DESIGNATION (in)	PERCENT PASSING (%)
75	(3 in)	100
50	(2 in)	80
37.5	(1-1/2 in)	72
25.0	(1 in)	56
19.0	(3/4 in)	55
12.7	(1/2 in)	51
9.5	(3/8 in)	49
6.35	(1/4 in)	46
4.75	(No. 4)	44
2.00	(No. 10)	39
0.850	(No. 20)	27
0.425	(No. 40)	12
0.250	(No. 60)	6
0.150	(No. 100)	4
0.106	(No. 140)	3
0.075	(No. 200)	3



REMARKS: Moisture Content = 2.3%. The gravel sized particles are rounded. The sample is undersized based the maximum partle size.

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GRAIN SIZE ANALYSIS - ASTM D6913

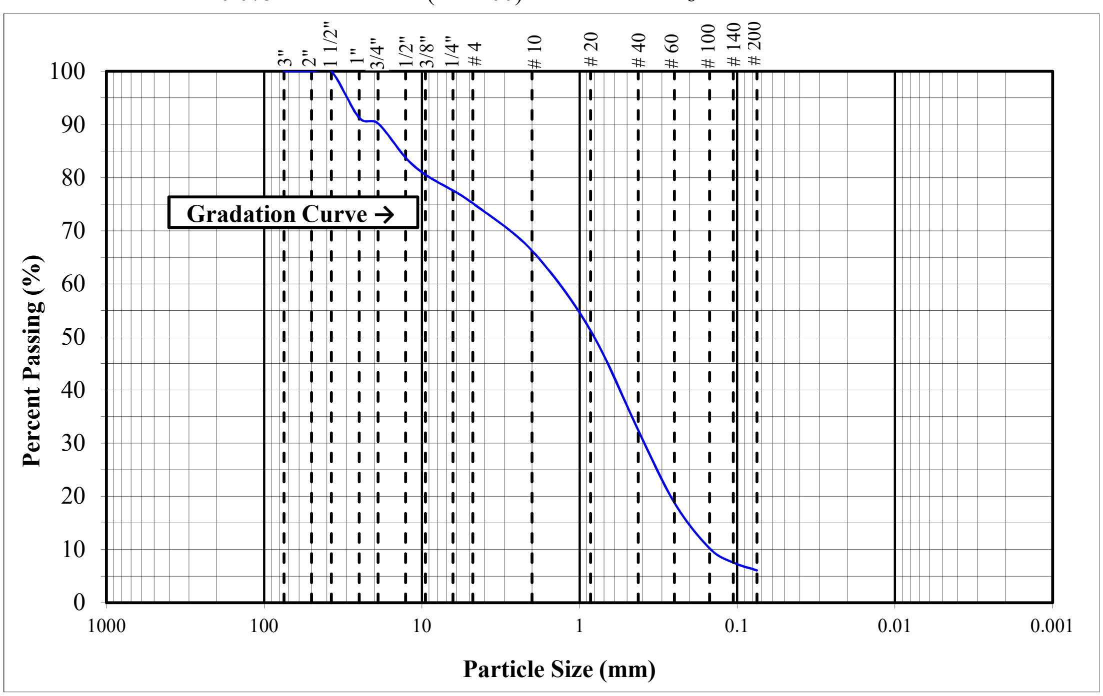
PROJECT NAME:	Proposed Wex Building	PROJECT #:	17181
PROJECT LOCATION:	Hancock & Thames St., Portland, ME	EXPLORATION #:	TP-2
CLIENT:	Archetype, P.A.	SAMPLE #:	S-2
TECHNICIAN:	Erika Stewart, P.E.	SAMPLE DEPTH:	4 ft
SOIL DESCRIPTION:	SAND, some Gravel, little Silt, SP-SM	TEST DATE:	8/28/2017

TEST PROCEDURE

Sample Source: Test Pit	**Sieve Stack:** Composite	**Specimen Procedure:** Moist
Test Method: Method A	**Separating Sieve(s):** 3/8 Inch	**Dispersion Type:** Tap Water

DATA

STANDARD SIEVE DESIGNATION (mm)	ALTERNATIVE SIEVE DESIGNATION (in)	PERCENT PASSING (%)
75	(3 in)	100
50	(2 in)	100
37.5	(1-1/2 in)	100
25.0	(1 in)	91
19.0	(3/4 in)	90
12.7	(1/2 in)	84
9.5	(3/8 in)	81
6.35	(1/4 in)	78
4.75	(No. 4)	75
2.00	(No. 10)	66
0.850	(No. 20)	51
0.425	(No. 40)	32
0.250	(No. 60)	19
0.150	(No. 100)	10
0.106	(No. 140)	8
0.075	(No. 200)	6



REMARKS: Moisture Content = 6.3%. The sample contains trace wood pieces.

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