GEOTECHNICAL REPORT

Proposed Bay House II
Portland, ME

August 24, 2013

GSI Project No. 212234

Prepared for:

Mr. Demetri Dasco
Atlas Investment Group, LLC
35 Fay Street, Suite 107B
Boston, Massachusetts 02118

Prepared by:

Harry K. Wetherbee, P.E.
Geotechnical Services, Inc.
55 North Stark Highway
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STATE OF MAINE
Harry K. Wetherbee
6722
LICENSED
PROFESSIONAL ENGINEER

Geotechnical Services Inc.

Geotechnical Engineering ◣ Environmental Studies ◣ Materials Testing ◣ Construction Monitoring

GSI GEOTECHNICAL SERVICES INC.
Geotechnical Engineering · Environmental Studies · Materials Testing · Construction Monitoring

August 24, 2013

Mr. Demetri Dasco
Atlas Investment Group, LLC
35 Fay Street, Suite 107B
Boston, Massachusetts 02118

RE: Geotechnical Report
Proposed Bay House II
Portland, ME **GSI Project No. 212234**

Dear Mr. Dasco:

Geotechnical Services, Inc. (GSI) is pleased to submit this report in connection with a geotechnical investigation for the above-referenced project. This report comprises a review of preliminary studies made by Sebago Techniques, Inc. (SBI) augmented with subsequent effort by GSI which includes subsurface explorations involving the retrieval of undisturbed clay samples, laboratory strength and consolidation testing, and data synthesis and evaluation. The work described herein has been conducted in accordance with our proposal of July 18, 2013.

EXECUTIVE SUMMARY

Our principal findings reveal the site to be underlain with silty clay soils which will consolidate following the application of foundation loads. Unlike the Phase I project there are no significant cuts and resulting compensatory effect on subsurface stresses. It is estimated that as much as 3 inches of vertical soil compression due to consolidation will occur as a result of the applied foundation loads. This degree of settlement is considered excessive for the type of construction and we have identified two options as cost-effective for site development. As such, GSI recommends that the subgrade be improved with vibro-stone columns and load transfer platform by Subsurface Constructors' Inc. or other qualified contractor. Following subgrade improvement procedures, an allowable bearing pressure of 3,000 psf may be adopted for design of spread footings. The ground floor may be slab-on-grade

Purpose and Scope

This report presents the results of a geotechnical investigation completed by Geotechnical Services, Inc. (GSI) for the proposed Bay House II development at the corner of Newbury and Hancock Streets in Portland, ME. Included are the findings of our subsurface exploration program and an engineering evaluation of the subsurface conditions encountered. The evaluation concerns itself with foundation design and earthwork considerations for the proposed buildings. The contents of this report are subject to the **Limitations** included in Appendix A.

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30 Newbury Street, Boston, MA 02116 · 617/455/4248 · FAX 617/745/4308

Applicable Building Code

International Building Code, IBC (2009) is the Code, which the State of Maine requires for compliance for the proposed building, including geotechnical-foundation engineering applications.

Preliminary Findings

As discussed in the STI geotechnical report, the site subsurface profile was determined to contain significant compressible silty clay. This silty clay presented geotechnical issues related to settlement and bearing capacity. GSI recommended additional exploration and testing with the following objectives:

- Determine the technical feasibility of a spread footing foundation;
- Define the subsurface soil properties as they relate to bearing capacity and settlement;
- Determine the compatibility of the subsurface conditions for vibro-stone columns;
- Consider other foundation options such as timber piles.

Site Description and Project Description

The project site comprises approximately 0.7 acres and is rectangular in shape running parallel and to the north of Newbury Street. The topography ascends towards the north with the lowermost elevation around 39 feet and upper at 51 feet. The general vicinity is developed with commercial and residential properties. A retaining wall runs across the northwest of the site and continues southerly through the site in the area of the proposed Unit 3. The retaining wall accommodates from 8 to 3 feet of topographic relief.

The structure will be a multi-story structure with ground-level parking on the north side. A mechanical room/storage area at the northeast corner will require excavation of 4 to 6 feet. Construction will be wood framed residential and the area over the parking may be supported with steel columns.

SUBSURFACE EXPLORATION PROGRAM

Subsurface Explorations

The subsurface exploration program for this project included the advancement of 2 test borings within the footprint of the proposed building. The explorations were advanced by wash and drive methods utilizing 4-inch casing to depths of 28 feet and 32 feet within the building areas. Soil samples were obtained at 5 ft intervals or strata changes. Standard Penetration Tests (SPTs) were performed at the sampling intervals in general accordance with ASTM-D1586. The soils encountered during the preliminary exploration program were classified in the field by a representative from GSI. The samples obtained were furthered viewed in the laboratory and classified by a professional engineer. The soil classifications generally follow after the Burmister System. These soil descriptions, the observed depth to groundwater, and other pertinent data are contained in the test boring logs included in Appendix B.

During the exploration program four, 2.8 inch diameter undisturbed shelby tube samples were retrieved using a Gregory Undisturbed Sampler (GUS) thin-walled sampler in accordance with ASTM-D 1587. The GUS thin-walled sampler contains an actuating piston which is pushed with applied hydraulic pressure until disengaged at the prescribed stroke. Sample retrieval is facilitated with a check valve which sustains a suction after air is evacuated during piston advancement. The samples were retrieved within the cohesive silty clay deposit at varying depths.

The retrieved shelby tube samples were sealed within the tube with wax upon removal from the ground to protect against moisture loss.
All samples were transported in an upright position such that minimal disturbance was imparted to the tube.

The exploration locations were determined in the field by taping from existing site features. The test boring locations are illustrated on Figure 2.

Soil Laboratory Testing

The soil laboratory tests were performed to estimate the engineering properties of the existing soils and to evaluate the suitability of the surface soils for use as structural fill and the impact the underlying silty clay layer would impart on foundation recommendations. The laboratory testing program for the supplemental exploration program included the completion of the following tests:

Four Atterberg limit tests per ASTM-D 4318 were performed in order to determine the liquid limit (LL), plastic limit (PL) and natural moisture content (Wn) of the sample tested. From these values the plasticity index (PI) can be determined and this value is used to infer soil properties, particularly as they relate to published values for the Presumptscott Formation. In addition, moisture content determinations were performed to compare the insitu conditions to the Atterberg Limits particularly with respect to the liquid limit.

Three unconfined compressive strength tests per ASTM-D 2166 were performed in order to determine the compressive strength of the material. This value is used to determine the undrained-unconfined shear strength of the clay. The shear strength of the clay is used in determining bearing capacity and to make an assessment of seismic parameters in accordance with IBC 2009.

Four consolidation tests per ASTM-D 2435 were performed in order to define the stress history of the silty clay soils and develop a stress-strain hysteresis. These properties are used in calculations to estimate settlement and the time required for settlement to occur based on theory developed by Terzarghi and others.

The soil samples chosen for testing were from varied representative depths. Our aim was to evaluate the degree of uniformity of the compressible strata and to determine which portions of the soil were the most susceptible to consolidation due to loading from either building foundation loads or earth fill.

The laboratory results are included in Appendix C.

STRATIGRAPHIC DEVELOPMENT

The subsurface explorations performed for this investigation are described in descending order as follows:

Urban Fill

A black to dark brown, very dense, SAND and Gravel with trace to little Silt was encountered beneath the surficial pavement. The thickness of this unit was fairly uniform at 4 to 5 feet. The material also contained traces of red brick.

Silty Sand

Boring GSI-1 encountered a 7 foot layer of loose, Silty SAND. SPT values were on the order of 7 bpf. The soil appears to be native material and its gray color suggests that it resides in a reducing environment indicative of saturated conditions or in the capillary fringe.

Silty Clay

The next unit the borings encountered was a very soft to soft grey silty clay. The SPT procedure indicated a soft consistency as sampling resistance was on the order of 2 blows per foot to weight-of-hammer per foot. The blow counts are based on a 140 lb. hammer dropping 30 inch to drive the split spoon sampler. In the case of "weight-of-hammer" resistance, the dead-load of the drill rods and hammer were the only forces needed to advance the split spoon sampler.

Sand and Gravel

Sand and Gravel soil was encountered underlying the silty clay materials previously discussed. It is believed that this soil may originate as an ablation till. Glacial till is a non-sorted, non-stratified natural deposit of sand, silt, gravel, and boulders, mixed in various proportions and deposited directly by the glaciers in a non-aqueous depositional environment. SPT procedures indicated very dense conditions as sampling resistance was on the order of 30 to 50+ blows per foot.

Groundwater

Groundwater was encountered at depths varying from 5 to 6 feet below existing surface elevation. The groundwater depths were measured immediately upon completion of the borings. The drilling was accomplished by wash-casing methods and water was introduced into the borehole. Groundwater readings at these locations would be expected to be shallower than at borings advanced by hollow-stem augers. All the groundwater levels should be anticipated to fluctuate from those measured during drilling operations in response to differences in equilibration time, rainfall, snowmelt, and seasonal fluctuations.

FOUNDATION DESIGN CONSIDERATIONS

The silty clay soils encountered beneath the footprint of the proposed building present geotechnical issues related to settlement and bearing capacity. The soft clays are prone to compression when subject to loading. The degree of compression is related to the stress history of the clay, the consolidation properties of the soil, and the magnitude and manner of the surface loading. The rate of consolidation is a function of the drainage characteristics of the soil structure.

Primary consolidation settlement is the process by which the clay is stressed and excess pore water is expelled from the soil structure. The vertical component of the volume changes resulting in settlement of the building structure. Primary consolidation is a function of the soils stress history and stratigraphy, and the magnitude of the applied loads. Primary consolidation occurs until such time that effective stress within the cohesive strata becomes equal to the applied load. GSI has identified primary consolidation settlement as a primary concern in an evaluation of post construction settlements for this site.

Secondary compression is a continuation of the volume change that was initiated during primary consolidation, except that it occurs at a much slower rate than primary consolidation. Secondary compression takes place at a constant effective stress after essentially all excess pore water pressure has dissipated.

This component of settlement appears from studies to result from compression of the bonds between individual clay particles and domains, as well as other effects on the micro scale which have yet to be clearly defined.

The behavior of the clay was mimicked in laboratory consolidation tests performed on undisturbed shelby tube samples obtained during the supplemental boring operation using a loading frame and precision measurement devices. From this testing, compressibility characteristics were derived for various portions of the underlying silty clay. Those characteristics of primary interest are overconsolidation ratio (OCR), compression index (C_c') and rebound coefficient (C_r').

OCR is the ratio of the preconsolidation stress to the existing vertical effective overburden stress. Soils become overconsolidated due to the following: a change in the total stress (removal of overburden), change in pore water pressure or desiccation of the upper layers due to surface drying. The rebound coefficient, C_r', also known as recompression index, is the slope of either the recompression curve or the unload rebound curve. This value is used during calculation of the primary consolidation settlement that occurs until such time that the applied load exceeds the past preconsolidation pressure. The compression index, C_c', is the slope of the virgin curve. This value is used during calculation of the primary consolidation settlement that occurs after the past preconsolidation pressure has been exceeded.

SPREAD FOOTING FOUNDATION SETTLEMENTS

GSI modeled and analyzed the anticipated foundation settlements based on loading from the foundation loads based on an 8 foot square footing with an allowable bearing pressure of 3000 pounds per square foot. The calculated primary consolidation settlement for the model is estimated to be 3 inches. Secondary compression is based on a 100 year design life and the resulting settlement from secondary compression is calculated to be 0.2 inches. This estimate was determined by obtaining the coefficient of secondary compression from the time versus deformation graphs created during the consolidation tests. The coefficient of secondary compression appeared comparable to published values for coefficient of secondary compression versus natural water content (Mesri, 1971).

The time rate of settlement for the cohesive stratum is based on the coefficient of consolidation which is obtained from the time to achieve 50 percent consolidation (t_{50}) and the height of the cohesive stratum. The time to t_{50} is taken from the time versus deformation curves established during the consolidation tests. These various calculated coefficients of consolidation create a range of time to complete the previously described settlement. Based on these values the time to achieve total primary consolidation settlement is calculated to be 1 to 2 years following construction.

Spread Footings on Improved Subgrade

It is GSI's recommendation that the proposed structure be supported on an improved subgrade consisting of a load transfer platform and vibro-stone columns (VSC) which would be constructed through the FILL and soft clay soils. The foundation contractor that installed subgrade improvements for the Bay House I project, Subsurface Constructors Inc. (SCI) has been provided with the results of the geotechnical investigation and concurs that VSCs are appropriate for the project.

It is expected that individual VSC element can be constructed to support up to about 70 kips which may be adequate for the project requirements. Depending on the magnitude of the column load one or more VSC may be provided at a column location. Upon establishing the column lay out and the column loads (wall loads) for the proposed structure, SCI technical personnel will be able to prepare a suitable arrangement and provide a cost estimate for the complete foundation.

We anticipate that the VSC, as proposed, would be a cost saver as compared to using longer end-bearing or friction piles. It is expected that for a foundation system supported on VSC, the maximum post-construction settlement at a column location would not exceed one inch, and the maximum differential settlement between adjacent columns (assumed at a nominal distance of 30 ft) would not exceed ¾ inch.

The VSCs should terminate in a 2 foot thick layer of structural fill which acts as a "load transfer platform". The structural fill should be placed in compacted lifts as specified hereinbelow.

Seismic Design Parameters

In accordance with IBC2009, we have evaluated susceptibility of the project site to earthquake-induced liquefaction and have determined that the site would is susceptible to earthquake induced liquefaction. According to the criteria set in IBC2009, and based on the results of unconfined compressive strength testing by GSI, the project site has been evaluated to belong to Site Class E. However, with the subgrade improvement procedures imparted by the VSC, the site stiffness will be upgraded to Site Class D.

Slab-On-Grade

GSI recommends that a concrete slab-on-grade be constructed following subgrade improvement of the underlying clay soils. Structural fill should be placed in 8 inch lifts and compacted to at least 95 percent relative compaction as determined by the Modified Proctor Test (ASTM-D1557) until floor slab base course subgrade is achieved. The concrete floor slabs should rest on a minimum 8-inch layer of floor slab base course soil meeting the gradation requirements for Structural Fill. The floor slab base course material should be compacted to at least 95 percent relative compaction as determined by the Modified Proctor Test (ASTM-D1557). Based upon the foregoing slab base preparation a modulus of subgrade reaction (Ks) of 250 pci may be used for design.

Protection of Foundation Subgrades

The contractor should maintain stable-dewatered subgrades for foundations, pavement areas, utility trenches and other concerned areas during construction. Subgrade disturbance may be influenced by excavation methods, moisture, precipitation, groundwater control, and construction activities. The silty soils overlying the clay are inherently vulnerable to disturbance when exposed to wet conditions. The moisture sensitivity of these soils is related to their high composition of fine-grained constituent (silt-clay) which acts to retain water.

The contractor should be aware of the sensitivity of the silty soils and take precautions to reduce subgrade disturbance. Such precautions may include diverting storm run-off away from construction areas, reducing traffic in sensitive areas, and maintaining an effective de-watering program.

Soils exhibiting weaving or instability should be over-excavated to more competent bearing soil and replaced with structural fill. It may be desirable for the contractor to place a lean concrete mud mat or a lift of free-draining gravel atop the prepared silty soil subgrade for protection against weakening/softening as construction progresses. A qualified engineer should inspect bearing subgrades throughout construction.

Temporary Excavations

For slope layback design, the on-site soils should be considered Type C soils in accordance with Occupational Safety and Health Administration (OSHA) regulations (29 CFR Part 1926). The maximum temporary slope for Soil Type C soils is 1.5H:1V provided the groundwater is lowered below the bottom of the excavation. The foregoing slope geometry precludes surcharge loads at the crest of the slope. It should be noted that these slope requirements are minimums required by OSHA regulations.

CONSTRUCTION SPECIFICATIONS

Structural Fill (Compacted Granular Fill) - Structural Fill should consist of clean sand and gravel free of organic material, snow, ice, or other objectionable materials and should be well-graded within the following limits:

Sieve Size	Percent Finer by Weight
6 in.	100
No. 4	30-90
No. 40	10-50
No. 200	0-10

Structural Fill should be placed in lift thickness not exceeding 12 in. loose measure. Cobbles and boulders having a size exceeding 2/3 of the loose lift thickness should be removed prior to compaction. Compaction in open areas should consist of self-propelled vibratory rollers such as a BoMag BW-60S or equivalent. In confined areas, hand guided equipment such a s a large vibratory plate compactor, should be used and the loose lift thickness should not exceed 6 in. A minimum of four systematic passes of the compaction equipment should be used to compact each lift. Compaction effort should be verified by field density testing.

Common Fill - Common fill may be used to raise grades in paved and landscaped areas, subject to pavement design criteria and landscape planting or drainage requirements. Common fill should be granular mineral soil free from organic materials, loam, wood, trash, snow, ice, frozen soil, and other compressible materials. Common fill should not contain stones larger than 2/3 of the placement lift thickness, and have a maximum 80 percent passing the No. 40 sieve, and a maximum of 30 percent passing the No. 200 sieve. These soils typically would require moisture control during placement and compaction. The on-site FILL soils are anticipated to meet the Common Fill requirements.

Backfilling - We recommend that Structural Fill be used as backfill around and beneath the caps to receive the and beneath the slab (pavement)-on-grade. Backfill outside the building footprint may generally consist of Common Fill with the exception of special filling requirements for pavements or other site structures. Recommended compaction requirements are as follows:

Location	Minimum Compaction Recommendation
Beneath and around caps, grade beams, under slabs	95 %
Parking, roadways, and sidewalks	92 % up to 3 ft below finished grade; 95% in the upper 3 ft
Landscaped areas	90 %

Minimum compaction requirements refer to percentages of the maximum dry density determined in accordance with ASTM D1557.

Construction Dewatering - It is anticipated that groundwater control during foundation excavation will not be a serious concern as long as the surface runoff is controlled in an effective way.

Construction Monitoring - It is recommended that a geotechnical engineer or experienced technical personnel be present during foundation construction to:

1. Monitor the foundation excavation and removal of the existing foundations, and preparation of subgrade to receive the VSC elements;
2. Monitor the construction of the VSC elements;
3. Confirm that backfill materials meet the requirements of the project plans and specifications, and make judgments regarding the suitability of excavated soils for reuse as Structural Fill or Common Fill;
4. Observe placement and test Structural Fill (as required by the Building Code) to meet as-placed density requirements.

It is recommended that GSI be retained to provide the recommended monitoring services. This will enable us to observe compliance with the project specific design requirements.

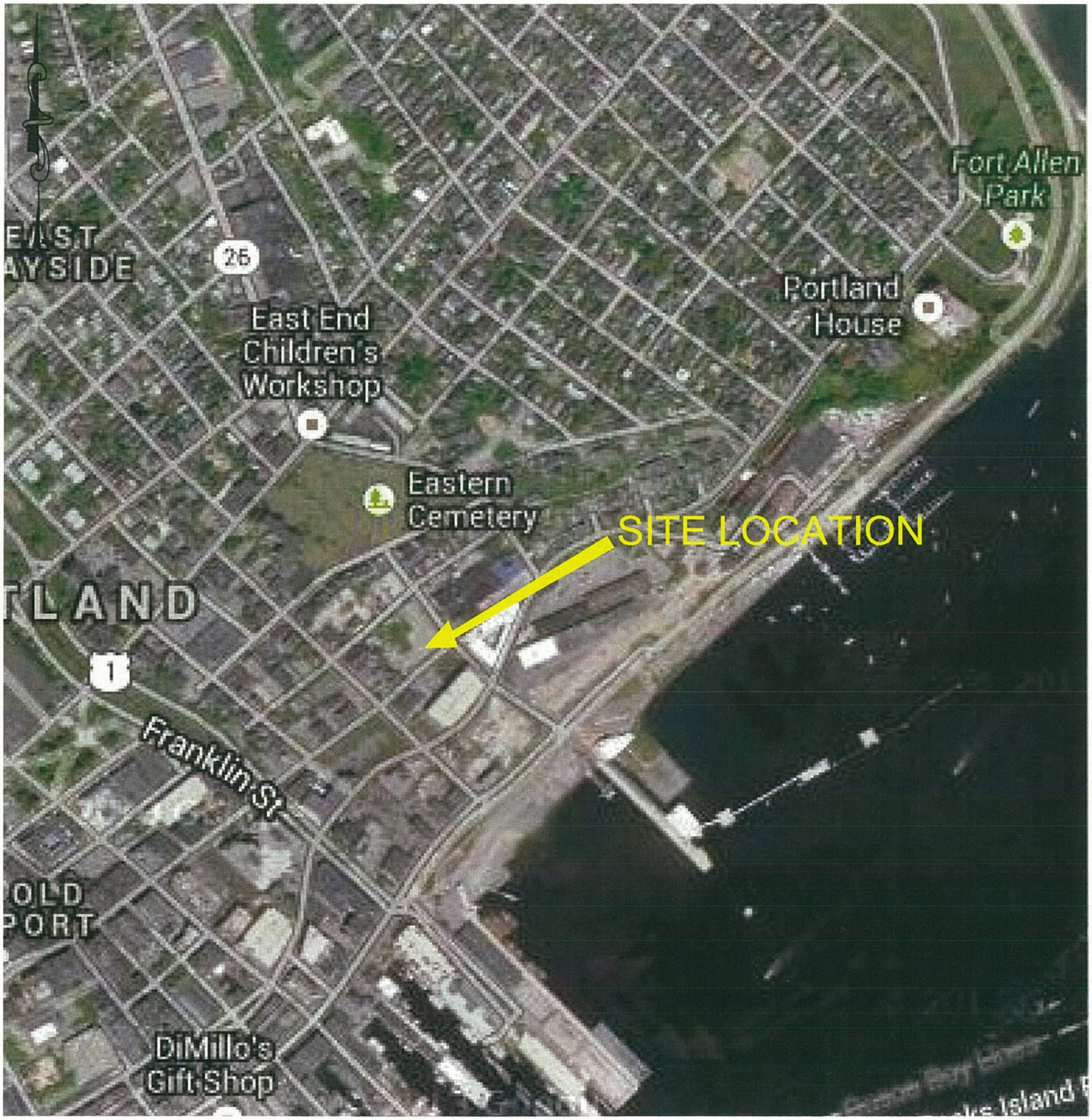
It has been a pleasure to serve you during the design phase of this project, and we look forward to its successful completion. If you have any questions on the content of this report, please do not hesitate to contact us.

GEOTECHNICAL SERVICES INC.

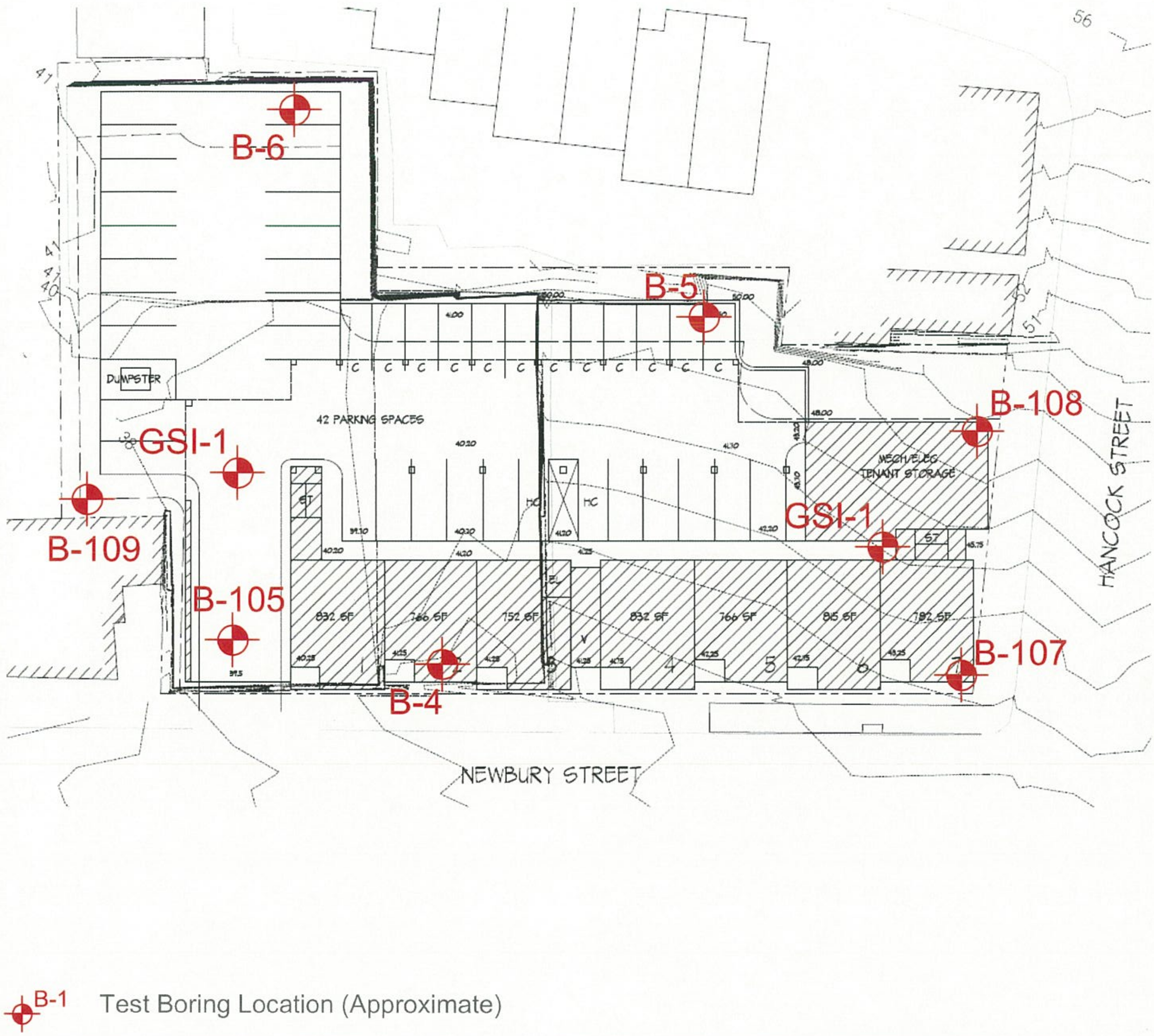
Harry K. Wetherbee, P.E.
Principal Engineer

Figures
Appendix A - Limitations
Appendix B – Exploration Logs
Appendix C – Laboratory Test Results
Appendix D – Stone Column Specifications

GSI



LOCUS MAP	GSI GEOTECHNICAL SERVICES INC. 55 NORTH STARK HIGHWAY, WEARE, NH 03281 TEL. (603) 529-7766 FAX. (603) 529-7780		
The Bay House Portland, Maine	DRAWN BY: KJM	DATE: August 2013	FIGURE NO. 1
	CHECKED BY: HKW	SCALE: 1" = @600'	
	FILE NAME: 212234 - Bay House.dwg	PROJECT NO.: 213193	



BORING LOCATION PLAN	GEOTECHNICAL SERVICES INC. 55 NORTH STARK HIGHWAY, WEARE, NH 03281 TEL. (603) 529-7766 FAX. (603) 529-7780		
The Bay House Portland, Maine	DRAWN BY: KJM	DATE: August 2013	FIGURE NO. 2
	CHECKED BY: HKW	SCALE: NTS	
	FILE NAME: 212234 - Bay House.dwg	PROJECT NO.: 213193	

APPENDIX A

LIMITATIONS

LIMITATIONS

Explorations

1. The analyzes, recommendations and designs submitted in this report are based in part upon the data obtained from preliminary subsurface explorations. The nature and extent of variations between these explorations may not become evident until construction. If variations then appear evident, it will be necessary to re-evaluate the recommendations of this report.

2. The generalized soil profile described in the text is intended to convey trends in subsurface conditions. The boundaries between strata are approximate and idealized and have been developed by interpretation of widely spaced explorations and samples; actual soil transitions are probably more gradual. For specific information, refer to the individual test pit and/or boring logs.

3. Water level readings have been made in the test pits and/or test borings under conditions stated on the logs. These data have been reviewed and interpretations have been made in the text of this report. However, it must be noted that fluctuations in the level of the groundwater may occur due to variations in rainfall, temperature, and other factors differing from the time the measurements were made.

Review

4. It is recommended that this firm be given the opportunity to review final design drawings and specifications associated with development of this site to evaluate the appropriate implementation of the recommendations provided herein.

5. In the event that any changes in the nature, design, or location of the proposed areas are planned, the conclusions and recommendations contained in this report shall not be considered valid unless the changes are reviewed and conclusions of the report modified or verified in writing by Geotechnical Services, Inc.

Construction

6. It is recommended that this firm be retained to provide geotechnical engineering services during the earthwork phases of the work. This is to observe compliance with the design concepts, specifications, and recommendations and to allow design changes in the event that subsurface conditions differ from those anticipated prior to the start of construction.

Use of Report

7. This report has been prepared for the exclusive use of Atlas Development and the design team in accordance with generally accepted soil and foundation engineering practices. No other warranty, expressed or implied, is made.

8. This report has been prepared for this project by Geotechnical Services, Inc. This report was completed for preliminary design purposes and may be limited in its scope to complete an accurate bid. Contractors wishing a copy of the report may secure it with the understanding that its scope is limited to evaluation considerations only.

APPENDIX B

EXPLORATION LOGS

Geotechnical Services, Inc. 55 North Stark Highway, Weare, NH 03281 Ph. 603/529/7766 Fax: 603/529/7080 30 Newbury Street, 3rd Floor, Boston, MA 02116 Ph. 857/238/9843 Fax: 857/239/9844

GSI

TEST BORING LOG

Boring No. **GSI-1**

Project	Bay House II	GSI Project No.	212234	Elevation	
Location	Portland, ME	Project Mgr.	HKW	Datum	
Client		Inspector	John Roth	Date Started	8/8/2013
Contractor	NHB	Checked By		Date Finished	8/8/2013
Driller	Rich Leonard	Rig Make & Model	Mobile Drill	Rig Model	53

Item:	Auger	Casing	Sampler	Core Barrel
Type		BW	SS	
Inside Diameter (in.)		4"	ST	
Hammer Weight (lb)			140	
Hammer Fall (in.)			30"	

- ☐ Truck ☐ Skid
- ☑ Track ☐ ATV
- ☐ Bomb. ☐ Geoprobe
- ☐ Tripod ☐ Other
- ☐ Winch ☑ Cat Head

Hammer Type:
- ☑ Safety Hammer
- ☐ Doughnut
- ☐ Automatic

☑ Roller Bit ☐ Cutting Head

Depth (ft)	Casing (Blows/ft)	No.	Depth (ft)	Rec (in.)	SPT (Bl./6-in.)	Rock RQD (%)	PID Rdg. (ppm)	Stratum Change (ft)	Soil-Rock Visual Classification and Description (Soils - Burmister System) (Rock - U.S. Corps of Engineers System)
0								ASPHALT	6 Inches of Asphalt
		S-1	0-2	24/17	16 13 14 8			URBAN FILL	Dry, medium dense, brown-black, fine to coarse, SAND and GRAVEL, trace to little silt. Some red brick mixed in.
5		S-2	5-7	24/8	3 5 1 2			SAND AND SILT	Wet, loose, gray, fine SAND and SILT.
10		UT-1	10-12						Shelby Tube Taken
12		S-3	12-14	24/24	2 1 1 1			CLAY	Wet, very soft, gray, CLAY, little fine sand.
15									
20		UT-2	20-22						Shelby Tube Taken
22		S-4	22-24	24/24	WH 1 1 2				Wet, very soft, gray, CLAY, trace fine sand.

Water Level Data

Date	Time	Depth (ft) to: Bott. of Casing	Bott. of Hole	Water
8/8	EOD		32'	6'

Sample Identification	Cohesive Soils N-Value	Granular Soils N- Value
O = Open Ended Rod	0 to 2: Very Soft	0 to 4: Very Loose
U = Undisturbed	2 to 4: Soft	4 to 10: Loose
S = Split Spoon	4 to 8: Medium Stiff	11 to 30: Medium Dense
C = Rock Core	8 to 15: Stiff	31 to 50: Dense
G = Geoprobe	15 to 30 Very Stiff	Over 50: Very Dense
	Over 30: Hard	

Trace (0 to 5%), Little (10 to 20%), Some (20 to 35%), And (35 to 50%)

Notes:

GSI-1

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TEST BORING LOG

Boring No. **GSI-1**

Project	Bay House II	GSI Project No.	212234	Elevation	
Location	Portland, ME	Project Mgr.	HKW	Datum	
Client		Inspector	John Roth	Date Started	8/8/2013
Contractor	NHB	Checked By		Date Finished	8/8/2013
Driller	Rich Leonard	Rig Make & Model	Mobile Drill	Rig Model	53

Item:	Auger	Casing	Sampler	Core Barrel
Type		BW	SS	
Inside Diameter (in.)		4"	ST	
Hammer Weight (lb)			140	
Hammer Fall (in.)			30"	

☐ Truck ☐ Skid
☑ Track ☐ ATV
☐ Bomb. ☐ Geoprobe
☐ Tripod ☐ Other
☐ Winch ☑ Cat Head ☑ Roller Bit ☐ Cutting Head

Hammer Type:
☑ Safety Hammer
☐ Doughnut
☐ Automatic

Depth (ft)	Casing (Blows/ft)	Sample Data: No.	Depth (ft)	Rec (in.)	SPT (Bl./ 6-in.)	Rock RQD (%)	PID Rdg. (ppm)	Stratum Change (ft)	Soil-Rock Visual Classification and Description (Soils - Burmister System) (Rock - U.S. Corps of Engineers System)
25									
30		UT-3	30-32					CLAY	Shelby Tube Taken
32		S-5	32-34	24/24	1 2 2 2				Wet, soft, gray, CLAY, little fine sand.
									Boring terminated at 32 feet and backfilled with cuttings.

Water Level Data

Date	Time	Depth (ft) to: Bott. of Casing	Bott. of Hole	Water
8/8	EOD		32'	6'

Sample Identification	Cohesive Soils N-Value	Granular Soils N- Value
O = Open Ended Rod	0 to 2: Very Soft	0 to 4: Very Loose
U = Undisturbed	2 to 4: Soft	4 to 10: Loose
S = Split Spoon	4 to 8: Medium Stiff	11 to 30: Medium Dense
C = Rock Core	8 to 15: Stiff	31 to 50: Dense
G = Geoprobe	15 to 30 Very Stiff	Over 50: Very Dense
	Over 30: Hard	

Trace (0 to 5%), Little (10 to 20%), Some (20 to 35%), And (35 to 50%)

Notes:

GSI-1

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GSI

TEST BORING LOG

Boring No. **GSI-2**

Project	Bay House II	GSI Project No.	212234	Elevation	
Location	Portland, ME	Project Mgr.	HKW	Datum	
Client		Inspector	John Roth	Date Started	8/8/2013
Contractor	NHB	Checked By		Date Finished	8/8/2013
Driller	Rich Leonard	Rig Make & Model	Mobile Drill	Rig Model	53

Item:	Auger	Casing	Sampler	Core Barrel
Type		BW	SS	
Inside Diameter (in.)		4"	ST	
Hammer Weight (lb)			140	
Hammer Fall (in.)			30"	

☐ Truck ☐ Skid
☑ Track ☐ ATV
☐ Bomb. ☐ Geoprobe
☐ Tripod ☐ Other
☐ Winch ☑ Cat Head ☑ Roller Bit ☐ Cutting Head

Hammer Type:
☑ Safety Hammer
☑ Doughnut
☐ Automatic

Depth (ft)	Casing (Blows/ft)	No.	Depth (ft)	Rec (in.)	SPT (Bl./ 6-in.)	Rock RQD (%)	PID Rdg. (ppm)	Stratum Change (ft)	Soil-Rock Visual Classification and Description (Soils - Burmister System) (Rock - U.S. Corps of Engineers System)
0		S-1	0-2	7/8	5 50/1			URBAN FILL	WET, VERY dense, brown-black, fine to coarse, SAND and GRAVEL, trace to little silt. Some red brick mixed in.
5		S-2	5-7	24/19	1 1 1 1			CLAY	Wet, very soft, gray, CLAY, trace fine sand.
10		UT-1	10-12						Shelby Tube Taken
12		S-3	12-14	24/24	1 1 1 1				Wet, very soft, gray, CLAY, trace fine sand.
15									
20		UT-2	20-22						Shelby Tube Taken
22		S-4	22-24	24/22	15 32 18 12			SAND	Wet, dense, gray, SAND, trace to little silt.

Water Level Data

Date	Time	Depth (ft) to: Bott. of Casing	Bott. of Hole	Water
8/8	EOD		28.3'	5'

Sample Identification
O = Open Ended Rod
U = Undisturbed
S = Split Spoon
C = Rock Core
G = Geoprobe

Cohesive Soils N-Value
0 to 2: Very Soft
2 to 4: Soft
4 to 8: Medium Stiff
8 to 15: Stiff
15 to 30 Very Stiff
Over 30: Hard

Granular Soils N- Value
0 to 4: Very Loose
4 to 10: Loose
11 to 30: Medium Dense
31 to 50: Dense
Over 50: Very Dense

Trace (0 to 5%), Little (10 to 20%), Some (20 to 35%), And (35 to 50%)

Notes:

GSI-2

TEST BORING LOG

Boring No.	GSI-2
Page	1 of 2

Project	Bay House II	GSI Project No.	212234	Elevation	
Location	Portland, ME	Project Mgr.	HKW	Datum	
Client		Inspector	John Roth	Date Started	8/8/2013
Contractor	NHB	Checked By		Date Finished	8/8/2013
Driller	Rich Leonard	Rig Make & Model	Mobile Drill	Rig Model	53

Item:	Auger	Casing	Sampler	Core Barrel
Type		BW	SS	
Inside Diameter (in.)		4"	ST	
Hammer Weight (lb)			140	
Hammer Fall (in.)			30"	

☐ Truck ☐ Skid
☑ Track ☐ ATV
☐ Bomb. ☐ Geoprobe
☐ Tripod ☐ Other
☐ Winch ☑ Cat Head ☑ Roller Bit ☐ Cutting Head

Hammer Type:
☑ Safety Hammer
☐ Doughnut
☐ Automatic

Depth (ft)	Casing (Blows/ft)	Sample Data: No.	Depth (ft)	Rec (in.)	SPT (Bl./ 6-in.)	Rock RQD (%)	PID Rdg. (ppm)	Stratum Change (ft)	Soil-Rock Visual Classification and Description (Soils - Burmister System) (Rock - U.S. Corps of Engineers System)
25								SAND	See note 1.
									Auger refusal at 28 feet.
									Boring terminated at 28 feet and backfilled with cuttings.

Water Level Data				
Date	Time	Depth (ft) to: Bott. of Casing	Bott. of Hole	Water
8/8	EOD		28.3'	5'

Sample Identification
O = Open Ended Rod
U = Undisturbed
S = Split Spoon
C = Rock Core
G = Geoprobe

Cohesive Soils N-Value
0 to 2: Very Soft
2 to 4: Soft
4 to 8: Medium Stiff
8 to 15: Stiff
15 to 30 Very Stiff
Over 30: Hard

Granular Soils N- Value
0 to 4: Very Loose
4 to 10: Loose
11 to 30: Medium Dense
31 to 50: Dense
Over 50: Very Dense

Trace (0 to 5%), Little (10 to 20%), Some (20 to 35%), And (35 to 50%)

Notes: 1. Let roller bit grind on refusal material for 5 minutes with no movement. Probable bedrock.

GSI-2

Geotechnical Services, Inc. 55 North Stark Highway, Weare, NH 03281 Ph. 603/529/7766 Fax: 603/529/7080 30 Newbury Street, 3rd Floor, Boston, MA 02116 Ph. 857/238/9843 Fax: 857/239/9844

APPENDIX C

LABORATORY RESULTS

GEOTECHNICAL SERVICES, INC
55 NORTH STARK HIGHWAY
WEARE, NH 03281

SHELBY TUBE EXTRACTION LOG

PROJECT:	Bay House				
PROJECT No.:	212234	**TESTED BY:**	KM/SH	**DATE TESTED:**	8/9/2013
SAMPLE No.:	L-287-13	**SAMPLED BY:**	NHB	**DATE SAMPLED:**	8/8/2013
ELEVATION:	10'-12'	**PLOTTED BY:**	KM	**DATE PLOTTED:**	8/19/2013
LOCATION:	Portland, ME	**CHECKED BY:**	HW		
SOURCE:	B1				

SOIL DESCRIPTION: Gray fine to medium SAND, little to some Silt
REMARKS:

Top (0") – 24"	24" – 30" Bottom
Gray fine to medium SAND, little to some Silt	Gray SILT and Clay, some fine Sand

0" 6" 12" 18" 24" 30"

PP1a		PP2a		PP3a		PP4a	0.25
PP1b		PP2b		PP3b		PP4b	0.25
PP1c		PP2c		PP3c		PP4c	0.5
Tv1		Tv2		Tv3		Tv4	N/A
MC1		MC2		MC3		MC4	
Tare		Tare		Tare		Tare	
Wet Wt		Wet Wt		Wet Wt		Wet Wt	
Dry Wt		Dry Wt		Dry Wt		Dry Wt	
% Moisture		% Moisture		% Moisture		% Moisture	

PP - Pocket Penetrometer
TV- Torevane
MC- Moisture Content

Att- Sample area used for Attenberg Limits
UC- Sample area used for Unconfined Compression

GEOTECHNICAL SERVICES, INC.

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PROJECT:	Bay House				
PROJECT No.:	212234	**TESTED BY:**	KJM	**DATE TESTED:**	8/12/2013
SAMPLE No.:	L-288-13	**SAMPLED BY:**	NHB	**DATE SAMPLED:**	8/8/2013
DEPTH:	10'-12'	**PLOTTED BY:**	KJM	**DATE PLOTTED:**	8/13/2013
LOCATION:	B1	**CHECKED BY:**			
SOURCE:					
SOIL DESCRIPTION:	Gray Clay				
REMARKS:					

ATTERBERG LIMITS

ASTM 4318

Plastic Limit

Trial Number	1	2
Thickness of thread (in)	1/8	1/8
WT. Can & wet Soil (g)	20.61	25.74
WT. Can & dry Soil (g)	19.5	24.71
WT. Can (g)	14.31	19.97
% Moisture	21.39	21.73

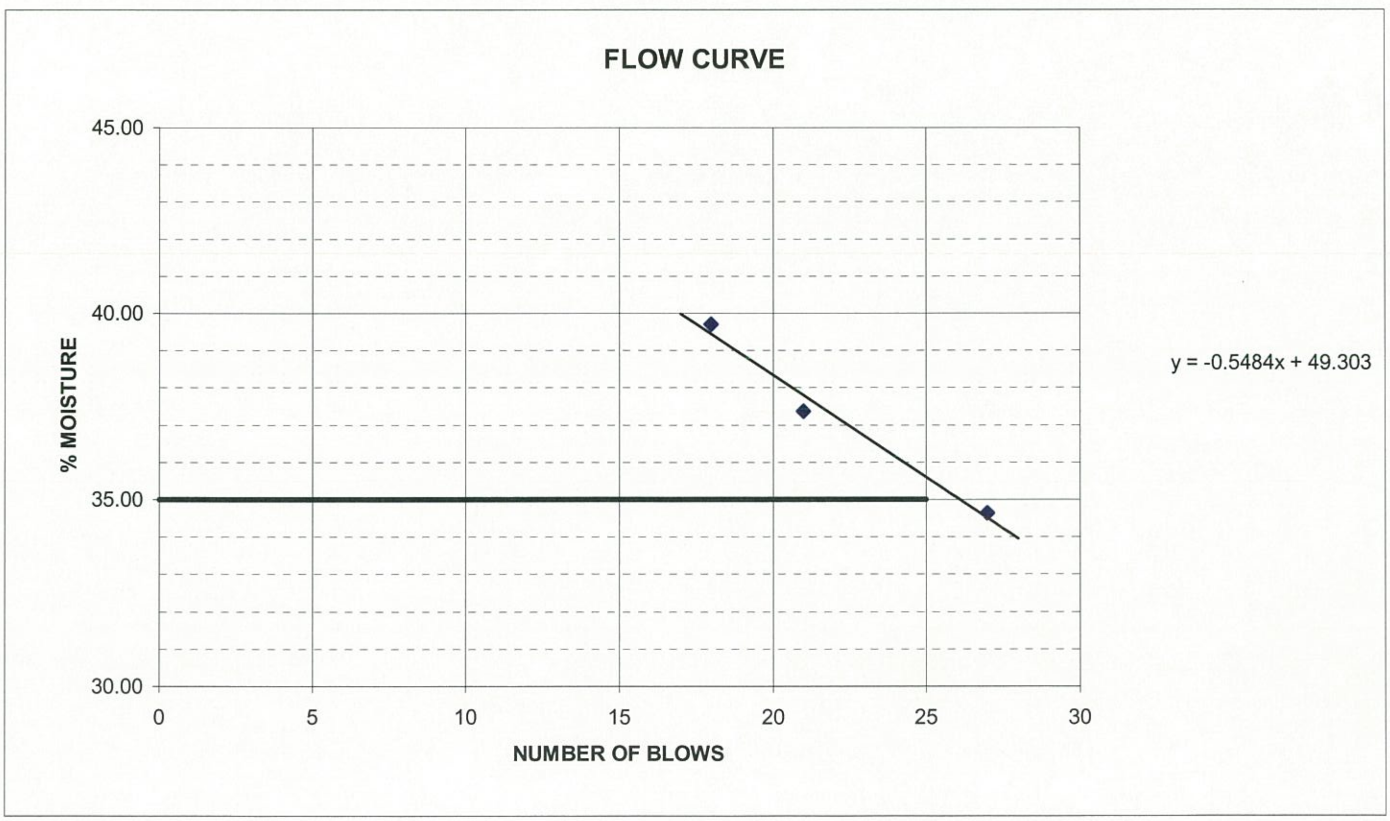
Liquid Limit

Trial Number	1	2	3
No. of Blows	18	27	21
WT. Can & wet Soil (g)	23.19	21.55	24.51
WT. Can & dry Soil (g)	21.26	19.60	21.73
WT. Can (g)	16.40	13.97	14.29
% Moisture	39.71	34.64	37.37

Plastic Limit = 22

Liquid Limit = 35

PLASTICITY INDEX (PI) = 13



GEOTECHNICAL SERVICES, INC
55 North Stark Highway
Weare, NH 03281

Moisture Content

ASTM (D2216)

Project: Bay House
Lab #: L-288-13
Source: B1: 10'-12'
Soil Description: Gray Clay

Project #: 212234
Submitted by: KM

Moisture #1	
Tare#	KK
Tare Wt.	14.19
Wet Wt.	48.92
Dry Wt.	39.17
% Moisture	39.0%

Moisture #2	
Tare#	Z
Tare Wt.	14.19
Wet Wt.	42.55
Dry Wt.	33.60
% Moisture	46.1%

Average	42.6%

$$\frac{(\text{Wet Wt.} - \text{Dry Wt.})}{(\text{Dry Wt.} - \text{Tare Wt.})} \times 100 = \%\text{Moisture}$$

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UNCONFINED COMPRESSIVE STRENGTH

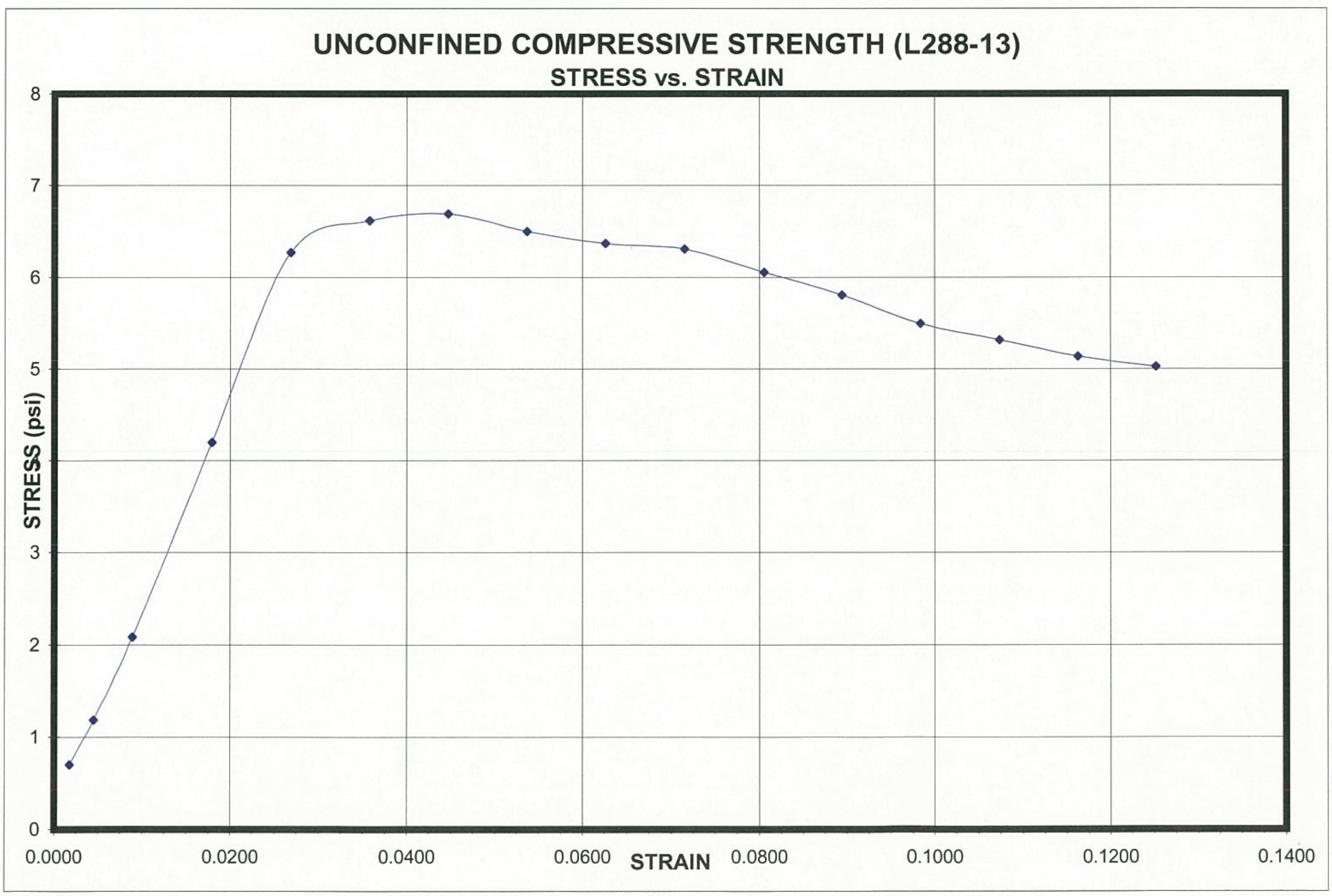
PROJECT:	Bay House	**SAMPLED BY:**	New Hampshire Boring
PROJECT No.:	212234	**DATE SAMPLED:**	8/8/2013
SAMPLE No.:	L-288-13	**TESTED BY:**	K. Maynard
ELEVATION:	10'-12'	**DATE TESTED:**	8/12/2013
LOCATION:	B1	**PLOTTED BY:**	K. Maynard
SOURCE:	In-Situ	**DATE PLOTTED:**	8/14/2013
DESCRIPTION:	*Gray Clay*		
REMARKS:			

ASTM D 2166

SAMPLE DATA

LENGTH (in)=	5.59	DIAMETER (in)=	2.86
WEIGHT (g)=	1092.8	MOISTURE (%)=	42.6%
WET UNIT WEIGHT (pcf)=	115.7	AREA, A_0 (in^2)=	6.42

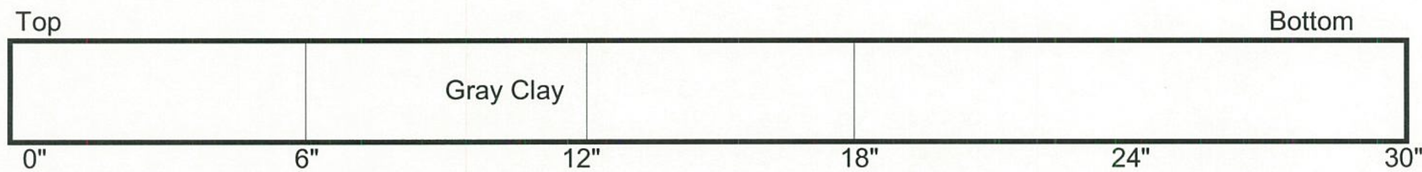
Maximum Unconfined Compressive Strength (psi)= 6.8



GEOTECHNICAL SERVICES, INC
55 NORTH STARK HIGHWAY
WEARE, NH 03281

SHELBY TUBE EXTRACTION LOG

PROJECT:	Bay House				
PROJECT No.:	212234	**TESTED BY:**	KM/SH	**DATE TESTED:**	8/9/2013
SAMPLE No.:	L-288-13	**SAMPLED BY:**	NHB	**DATE SAMPLED:**	8/8/2013
ELEVATION:	20'-22'	**PLOTTED BY:**	KM	**DATE PLOTTED:**	8/19/2013
LOCATION:	Portland, ME	**CHECKED BY:**	HW		
SOURCE:	B1				
SOIL DESCRIPTION:	Gray Clay				
REMARKS:					

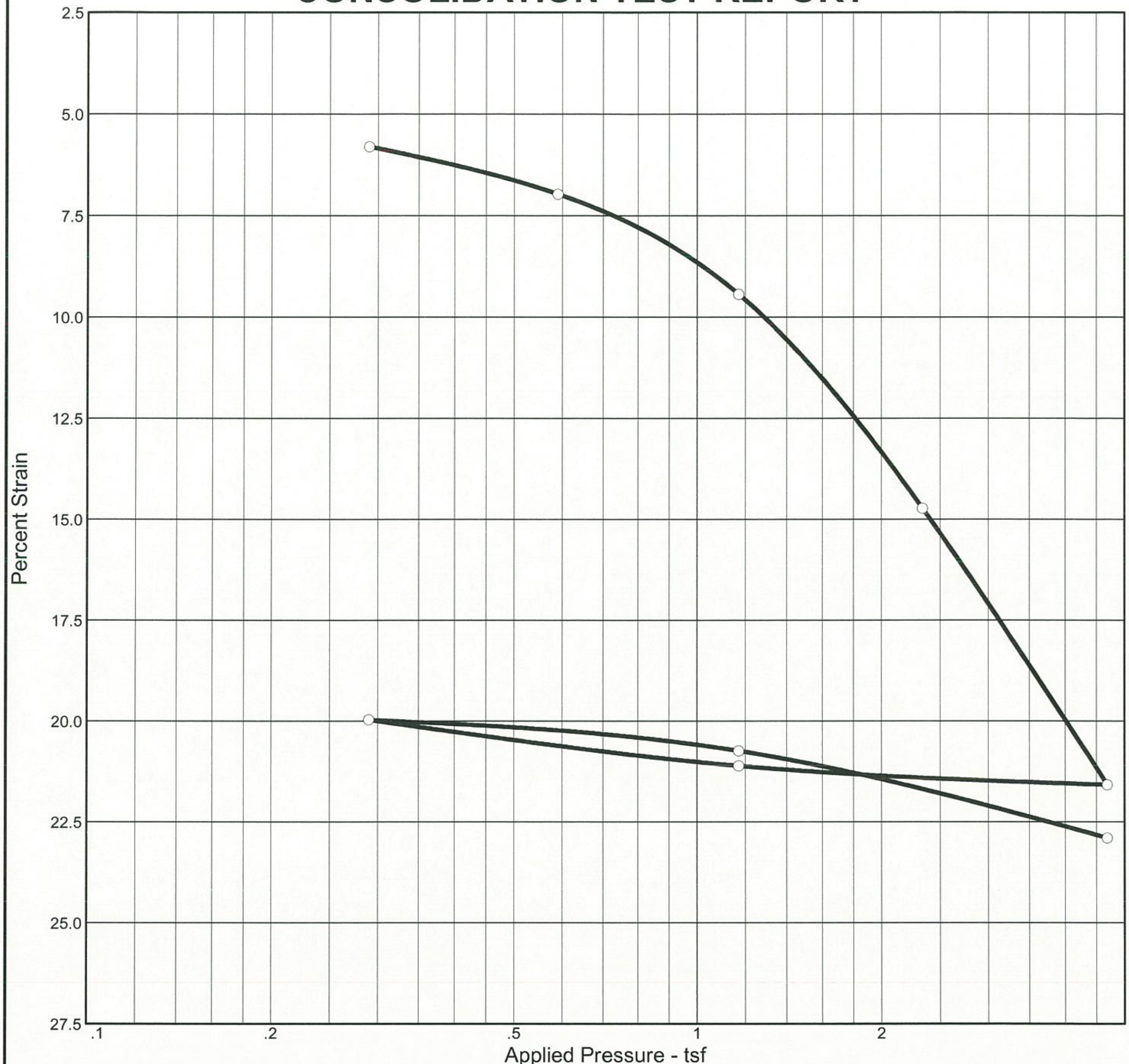


PP1a	0.25	PP2a		PP3a		PP4a	0.25
PP1b	0.25	PP2b		PP3b		PP4b	0.25
PP1c	0.25	PP2c		PP3c		PP4c	0.5
Tv1	0.2	Tv2		Tv3		Tv4	N/A
	0.25						
MC1		MC2		MC3		MC4	
Tare	14.19	Tare		Tare		Tare	
Wet Wt	48.92	Wet Wt		Wet Wt		Wet Wt	
Dry Wt	39.17	Dry Wt		Dry Wt		Dry Wt	
% Moisture	39%	% Moisture		% Moisture		% Moisture	

PP - Pocket Penetrometer
TV- Torevane
MC- Moisture Content

Att- Sample area used for Attenberg Limits
UC- Sample area used for Unconfined Compression

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (tsf)	P_c (tsf)	C_c	C_r	Initial Void Ratio
Saturation	Moisture									
245.2 %	39.2 %	85.0	35	13	1.65		1.47	0.29	0.02	0.264

MATERIAL DESCRIPTION	USCS	AASHTO
Gray Clay		

Project No. 212234 **Client:**

Project: Bay House
Portland, Maine

Location: Test Boring B-1 (20'-22')

GEOTECHNICAL SERVICES, INC.

Weare, New Hampshire

Remarks:

Plate

GEOTECHNICAL SERVICES, INC.

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UNCONFINED COMPRESSIVE STRENGTH

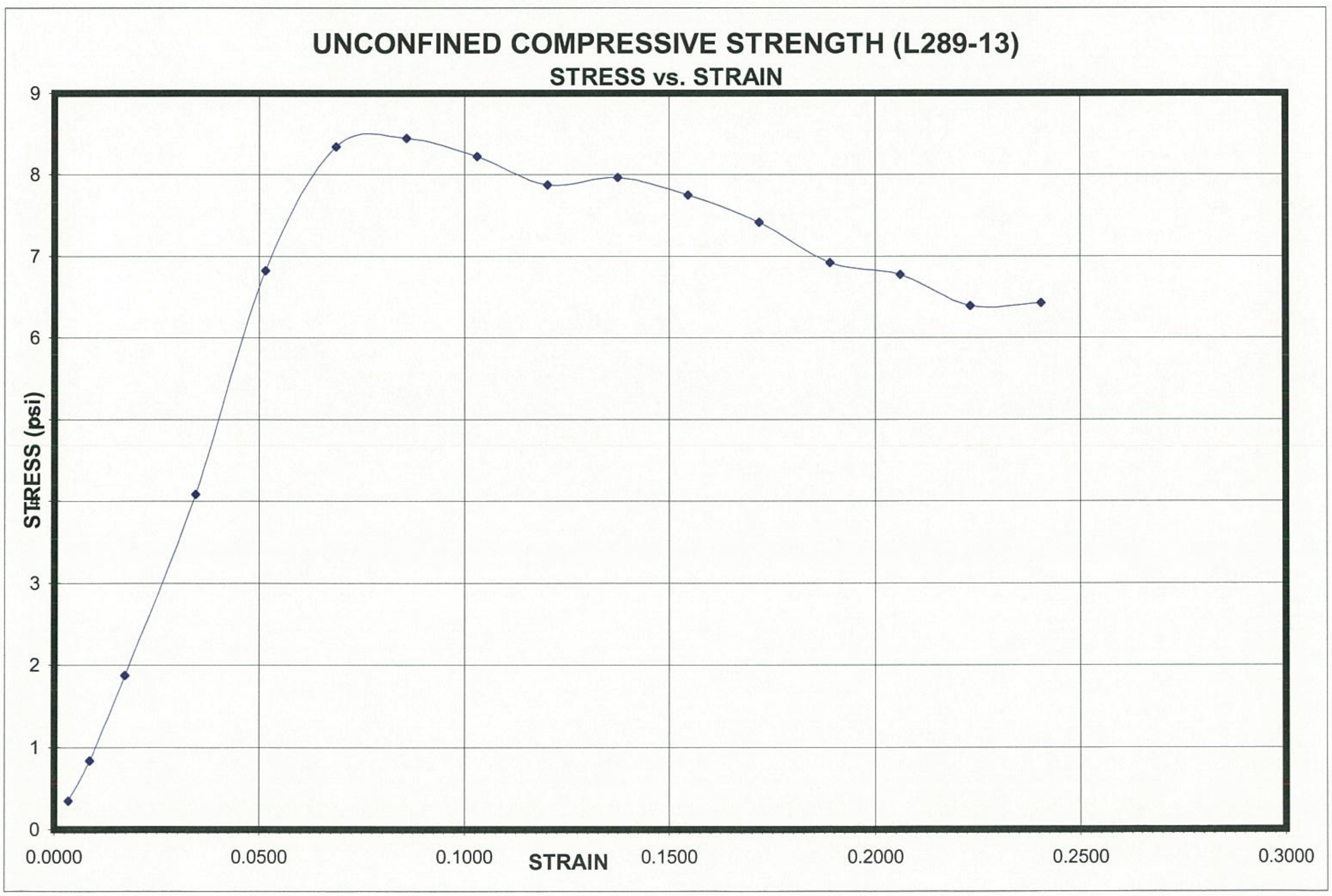
PROJECT:	Bay House	**SAMPLED BY:**	New Hampshire Boring
PROJECT No.:	212234	**DATE SAMPLED:**	8/8/2013
SAMPLE No.:	L-289-13	**TESTED BY:**	K. Maynard
ELEVATION:	20'-22'	**DATE TESTED:**	8/12/2013
LOCATION:	B1	**PLOTTED BY:**	K. Maynard
SOURCE:	In-Situ	**DATE PLOTTED:**	8/14/2013
DESCRIPTION:	*Gray Clay*		
REMARKS:			

ASTM D 2166

SAMPLE DATA

LENGTH (in)=	2.91	DIAMETER (in)=	2.85
WEIGHT (g)=	596.5	MOISTURE (%)=	34.5%
WET UNIT WEIGHT (pcf)=	122.2	AREA, A_0 (in^2)=	6.38

Maximum Unconfined Compressive Strength (psi)= 8.6



GEOTECHNICAL SERVICES, INC

55 NORTH STARK HIGHWAY
WEARE, NH 03281

SHELBY TUBE EXTRACTION LOG

PROJECT:	Bay House				
PROJECT No.:	212234	**TESTED BY:**	KM/SH	**DATE TESTED:**	8/12/2013
SAMPLE No.:	L-289-13	**SAMPLED BY:**	NHB	**DATE SAMPLED:**	8/8/2013
ELEVATION:	30'-32'	**PLOTTED BY:**	KM	**DATE PLOTTED:**	8/19/2013
LOCATION:	Portland, ME	**CHECKED BY:**	HW		
SOURCE:	B1				
SOIL DESCRIPTION:	Gray Silty Clay				
REMARKS:					

Top / Bottom

0" – 13"	13" – 24"	24" – 30"
Gray Silty Clay (0-13" (Gray Silt Varve 6"-7.1") (Laminations 8.2" and 9")	Gray Silt (13"-24")	Gray Silty Clay

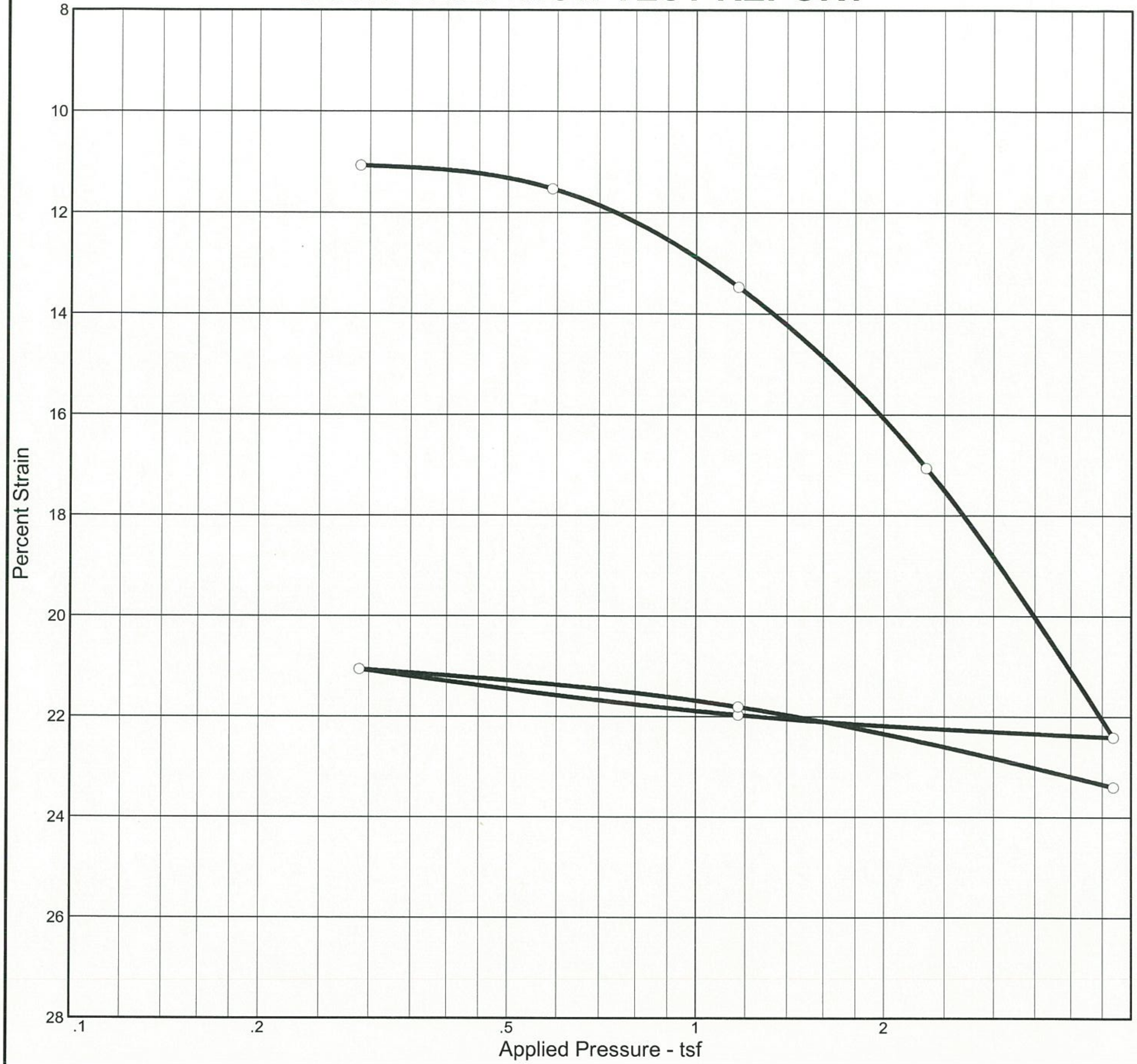
0" 6" 12" 18" 24" 30"

PP1a	0.25	PP2a	0.2	PP3a		PP4a	
PP1b	0.25	PP2b	0.25	PP3b		PP4b	
PP1c	0.25	PP2c	0.25	PP3c		PP4c	
Tv1	0.36	Tv2	0.32	Tv3		Tv4	
	0.31		0.35				
MC1		MC2		MC3		MC4	
Tare		Tare		Tare		Tare	
Wet Wt		Wet Wt		Wet Wt		Wet Wt	
Dry Wt		Dry Wt		Dry Wt		Dry Wt	
% Moisture		% Moisture		% Moisture		% Moisture	

PP - Pocket Penetrometer
TV- Torevane
MC- Moisture Content

Att- Sample area used for Attenberg Limits
UC- Sample area used for Unconfined Compression

CONSOLIDATION TEST REPORT



Natural Saturation	Natural Moisture	Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (tsf)	P_c (tsf)	C_c	C_r	Initial Void Ratio
454.3 %	28.3 %	93.9	36	14	1.65		1.28	0.20	0.02	0.103

MATERIAL DESCRIPTION	USCS	AASHTO
Gray Silty Clay		

Project No. 212234 **Client:**

Project: Bay House
Portland, Maine

Location: Test Boring B-1 (30'-32')

GEOTECHNICAL SERVICES, INC.

Weare, New Hampshire

Remarks:

Plate

GEOTECHNICAL SERVICES, INC
55 NORTH STARK HIGHWAY
WEARE, NH 03281

SHELBY TUBE EXTRACTION LOG

PROJECT: Bay House
PROJECT No.: 212234
SAMPLE No.: L-293-13
ELEVATION: 10'-12'
LOCATION: Portland, ME
SOURCE: B2
SOIL DESCRIPTION: Gray Silty Clay
REMARKS:

TESTED BY: KM/SH
SAMPLED BY: NHB
PLOTTED BY: KM
CHECKED BY: HW

DATE TESTED: 8/13/2013
DATE SAMPLED: 8/8/2013
DATE PLOTTED: 8/19/2013

Top — Bottom

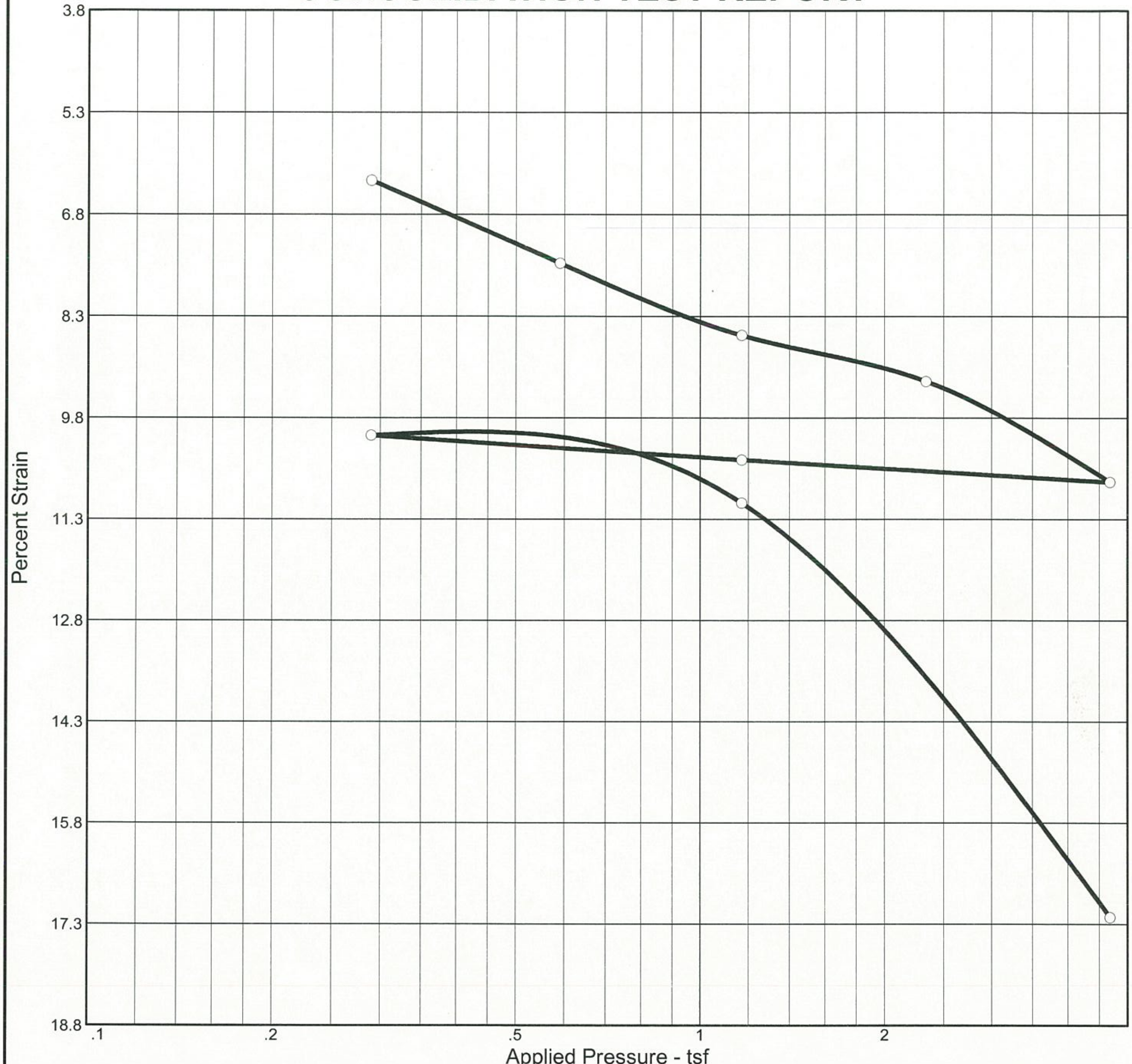
0"–6"	6"–12"	12"–18"	18"–24"	24"–30"
Gray Silty Clay (0-4.5")	Gray Clayey Silt (4.5-8.5")	Consolidation Test Gray Silty Clay (8.5-30") (Varves at 12.5", 18.5", and 21")	Atteberg Limits	Unconfined Comp. Strength

PP1a	0.25	PP2a	0.3	PP3a	0.25	PP4a	0.25
PP1b	0.5	PP2b	0.3	PP3b	0.25	PP4b	0.25
PP1c	0.3	PP2c	0.5	PP3c	0.2	PP4c	0.3
Tv1	0.2	Tv2	0.25	Tv3	0.25	Tv4	0.26
	0.2		0.25		0.2		0.2
MC1		MC2		MC3		MC4	
Tare	14.26	Tare	14.39	Tare	14.3	Tare	14.29
Wet Wt	40.48	Wet Wt	37.53	Wet Wt	51.31	Wet Wt	59.38
Dry Wt	34.19	Dry Wt	31.55	Dry Wt	43.61	Dry Wt	50.99
% Moisture	32%	% Moisture	34.85%	% Moisture	26.27%	% Moisture	22.86%

PP - Pocket Penetrometer
TV- Torevane
MC- Moisture Content

Att- Sample area used for Attenberg Limits
UC- Sample area used for Unconfined Compression

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (tsf)	P_c (tsf)	C_c	C_r	Initial Void Ratio
Saturation	Moisture									
224.6 %	33.0 %	87.2	33	11	1.65		2.54	0.06	0.04	0.243

MATERIAL DESCRIPTION	USCS	AASHTO
Gray Silty Clay		

Project No. 212234 **Client:**

Project: Bay House
Portland, Maine

Location: Test Boring B-2 (10'-12')

GEOTECHNICAL SERVICES, INC.

Weare, New Hampshire

Remarks:

Plate

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UNCONFINED COMPRESSIVE STRENGTH

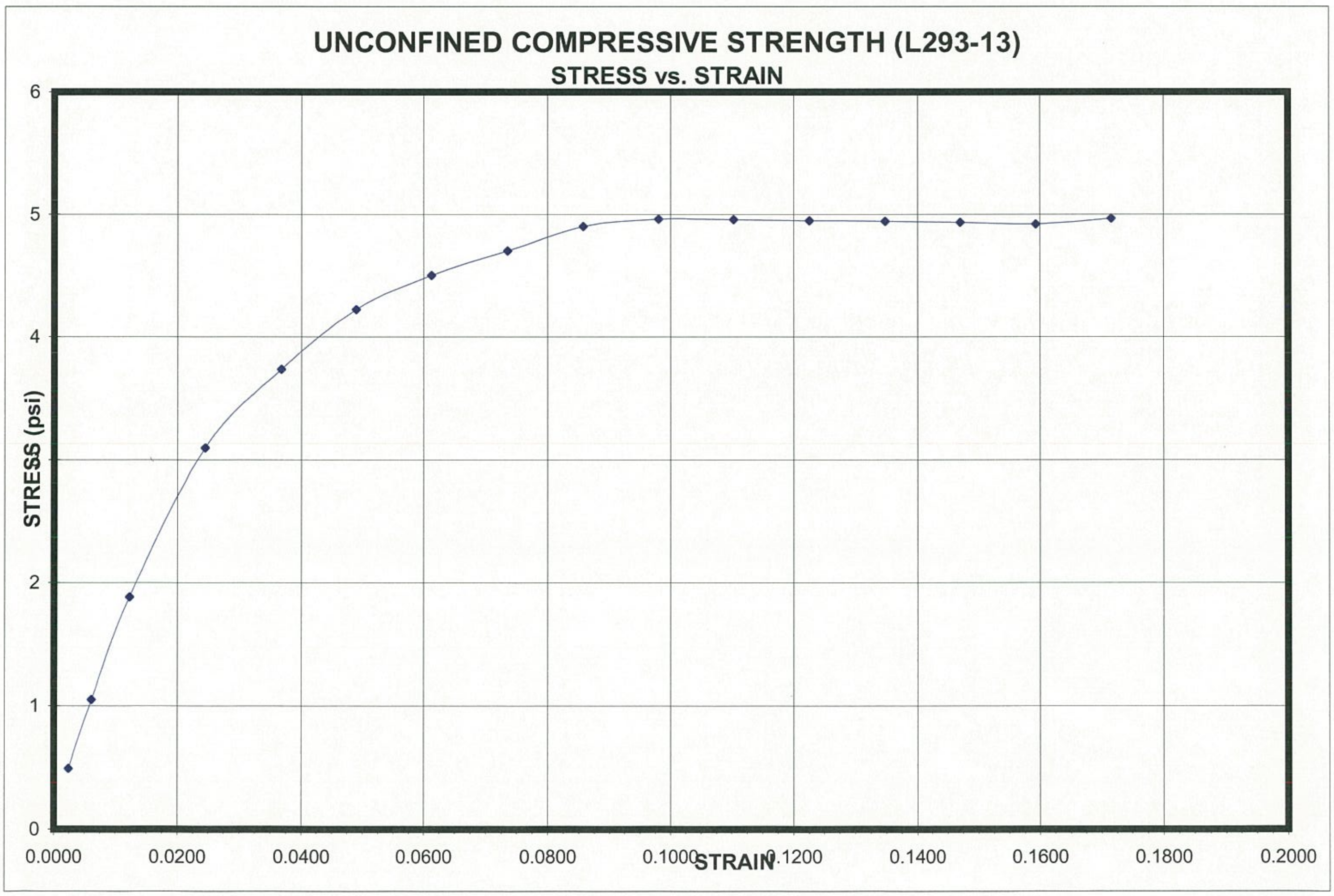
PROJECT:	Bay House	**SAMPLED BY:**	New Hampshire Boring
PROJECT No.:	212234	**DATE SAMPLED:**	8/8/2013
SAMPLE No.:	L-293-13	**TESTED BY:**	K. Maynard
ELEVATION:	10'-12'	**DATE TESTED:**	8/13/2013
LOCATION:	B2	**PLOTTED BY:**	K. Maynard
SOURCE:	In-Situ	**DATE PLOTTED:**	8/19/2013
DESCRIPTION:	*Gray Silty Clay*		
REMARKS:			

ASTM D 2166

SAMPLE DATA

LENGTH (in)=	4.08	DIAMETER (in)=	2.85
WEIGHT (g)=	776.6	MOISTURE (%)=	37.3%
WET UNIT WEIGHT (pcf)=	113.4	AREA, A_0 (in^2)=	6.38

Maximum Unconfined Compressive Strength (psi)= 5.0



PROJECT:	Bay House				
PROJECT No.:	212234	**TESTED BY:**	SH	**DATE TESTED:**	8/16/2013
SAMPLE No.:	L-293-13	**SAMPLED BY:**	NHB	**DATE SAMPLED:**	8/8/2013
DEPTH:	10'-12'	**PLOTTED BY:**	KJM	**DATE PLOTTED:**	8/19/2013
LOCATION:	B2	**CHECKED BY:**			
SOURCE:					
SOIL DESCRIPTION:	Gray Silty Clay				
REMARKS:					

ATTERBERG LIMITS
ASTM 4318

Plastic Limit

Trial Number	1	2
Thickness of thread (in)	1/8	1/8
WT. Can & wet Soil (g)	13.5	24.54
WT. Can & dry Soil (g)	13.27	23.81
WT. Can (g)	12.3	20.22
% Moisture	23.71	20.33

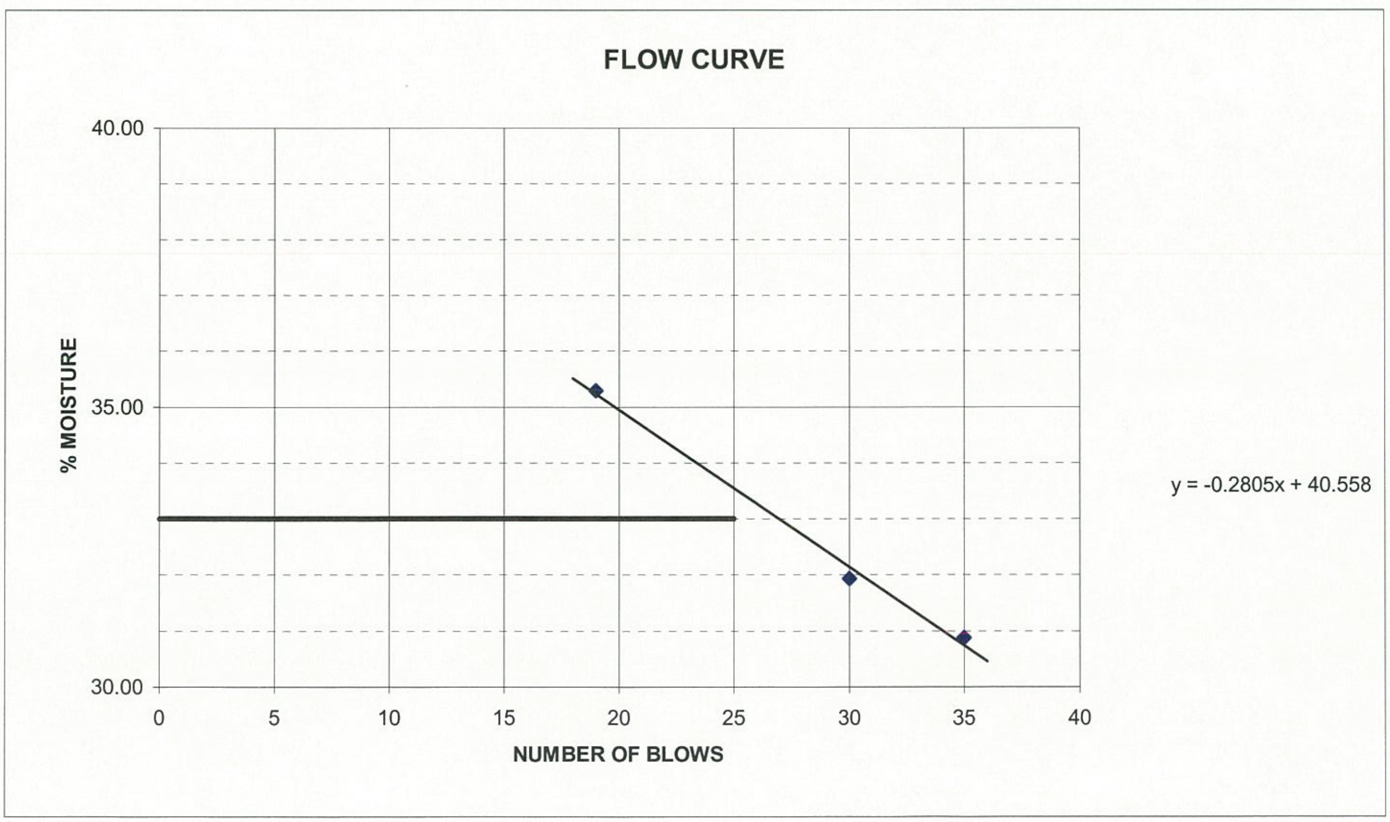
Liquid Limit

Trial Number	1	2	3
No. of Blows	19	30	35
WT. Can & wet Soil (g)	27.20	34.90	29.28
WT. Can & dry Soil (g)	25.04	31.55	27.22
WT. Can (g)	18.92	21.06	20.55
% Moisture	35.29	31.94	30.88

Plastic Limit = 22

Liquid Limit = 33

PLASTICITY INDEX (PI) = 11



GEOTECHNICAL SERVICES, INC
55 NORTH STARK HIGHWAY
WEARE, NH 03281

SHELBY TUBE EXTRACTION LOG

PROJECT:	Bay House				
PROJECT No.:	212234	**TESTED BY:**	KM/SH	**DATE TESTED:**	8/13/2013
SAMPLE No.:	L-294-13	**SAMPLED BY:**	NHB	**DATE SAMPLED:**	8/8/2013
ELEVATION:	20'-22'	**PLOTTED BY:**	KM	**DATE PLOTTED:**	8/19/2013
LOCATION:	Portland, ME	**CHECKED BY:**	HW		
SOURCE:	B2				
SOIL DESCRIPTION:	Gray Silty Clay				
REMARKS:					

Top					Bottom
			0-5.5" Gray Clay with 2 Large Stones	5.5" to 8.5" Gray Silty Clay	8.5"-11" Gray fine Sand/Silt
0"	6"	12"	18"	24"	

PP1a	0.2	PP2a	0.2	PP3a	0.1	PP4a	0.5
PP1b	0.2	PP2b	0.25	PP3b	0.15	PP4b	0.3
PP1c	0.2	PP2c	0.2	PP3c	0.1	PP4c	0.5
Tv1	0.15	Tv2	0.2	Tv3	0.15	Tv4	0.3
	0.2		0.2		0.1		0.25
MC1		MC2		MC3		MC4	
Tare	19.35	Tare	21	Tare	14.4	Tare	14.33
Wet Wt	54.01	Wet Wt	44.31	Wet Wt	46.28	Wet Wt	51.52
Dry Wt	46.86	Dry Wt	40.26	Dry Wt	39.78	Dry Wt	43.9
% Moisture	26%	% Moisture	21.03%	% Moisture	25.61%	% Moisture	25.77%

PP - Pocket Penetrometer
TV- Torevane
MC- Moisture Content

Att- Sample area used for Attenberg Limits
UC- Sample area used for Unconfined Compression

PROJECT:	Bay House				
PROJECT No.:	212234	**TESTED BY:**	SH	**DATE TESTED:**	8/16/2013
SAMPLE No.:	L-294-13	**SAMPLED BY:**	NHB	**DATE SAMPLED:**	8/8/2013
DEPTH:	20'-22'	**PLOTTED BY:**	KJM	**DATE PLOTTED:**	8/19/2013
LOCATION:	B2	**CHECKED BY:**			
SOURCE:					
SOIL DESCRIPTION:	Gray Silty Clay				
REMARKS:					

ATTERBERG LIMITS
ASTM 4318

Plastic Limit

Trial Number	1	2
Thickness of thread (in)	1/8	1/8
WT. Can & wet Soil (g)	18.46	20.55
WT. Can & dry Soil (g)	17.72	19.44
WT. Can (g)	13.96	13.57
% Moisture	19.68	18.91

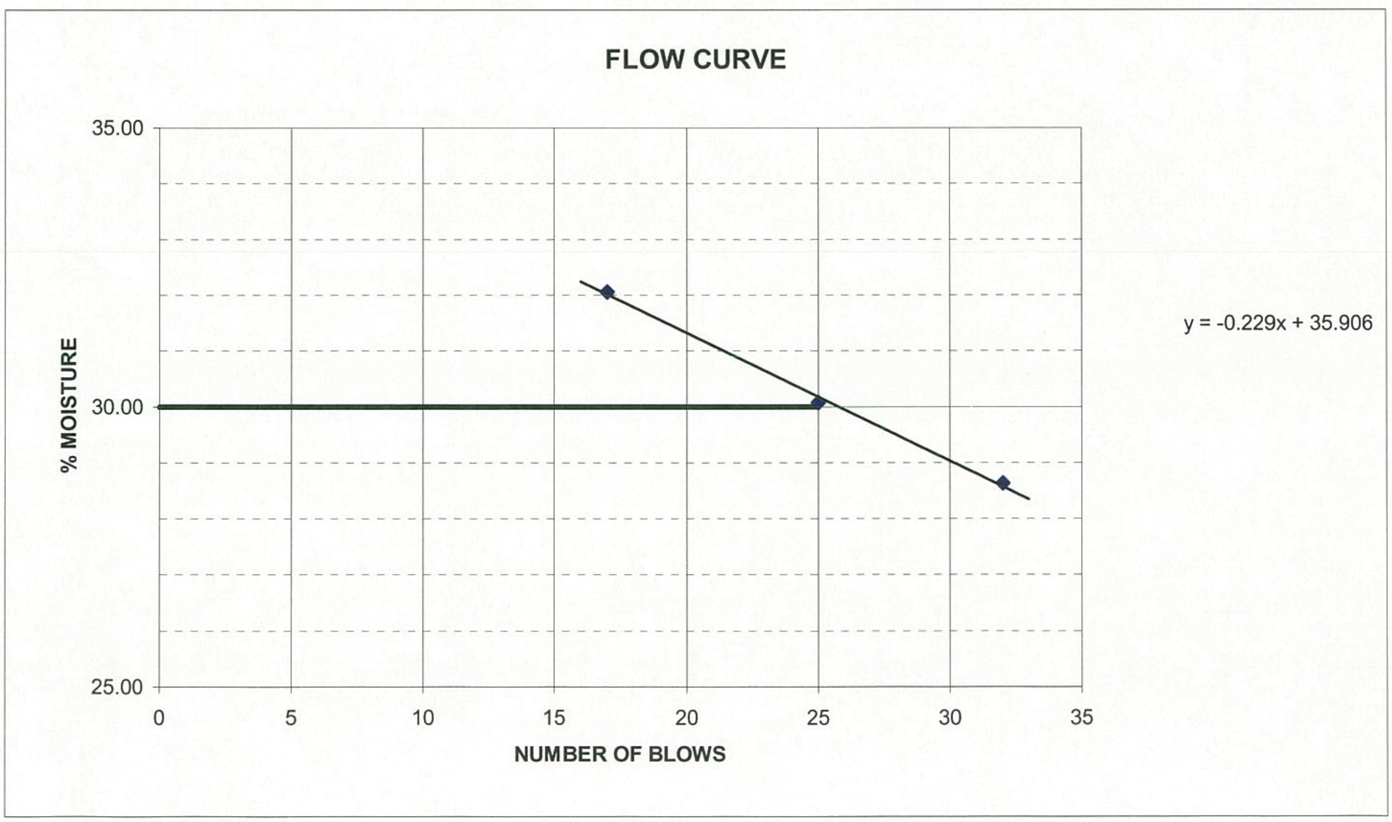
Liquid Limit

Trial Number	1	2	3
No. of Blows	17	25	32
WT. Can & wet Soil (g)	22.96	25.71	32.79
WT. Can & dry Soil (g)	20.86	23.06	30.13
WT. Can (g)	14.31	14.25	20.84
% Moisture	32.06	30.08	28.63

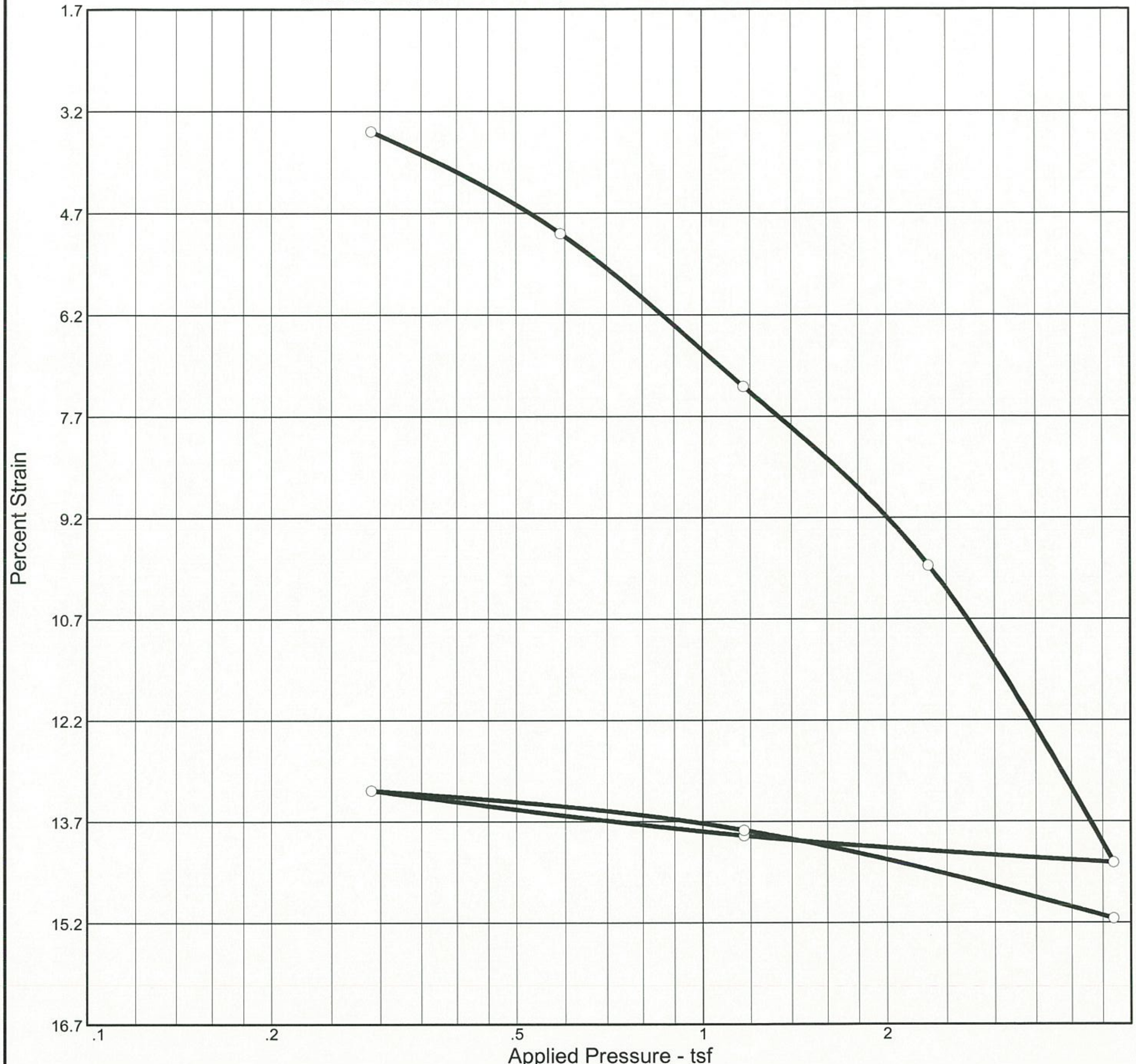
Plastic Limit = 19

Liquid Limit = 30

PLASTICITY INDEX (PI) = 11



CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (tsf)	P_c (tsf)	C_c	C_r	Initial Void Ratio
Saturation	Moisture									
363.6 %	28.7 %	94.8	30	11	1.65		2.48	0.17	0.01	0.130

MATERIAL DESCRIPTION	USCS	AASHTO
Gray Silty Clay		

Project No. 212234 **Client:**

Project: Bay House
Portland, Maine

Location: Test Boring B-2 (20'-22')

GEOTECHNICAL SERVICES, INC.

Weare, New Hampshire

Remarks:

Plate

APPENDIX D

STONE COLUMN SPECIFICATIONS

SECTION XXXXX – STONE COLUMNS

PART 1 - GENERAL

1.1 RELATED DOCUMENTS: Drawings and general provisions of the Contract, including General and Supplementary Conditions and other Division 00 and Division 01 Specification Sections, apply to this Section.

1.2 DESCRIPTION: Work shall consist of designing, furnishing and installing materials, and constructing a ground improvement system at the locations noted on the drawings and as specified herein. Ground improvement system shall be vibro stone columns.

1.3 WORK INCLUDED:

A. Provision of all equipment, material, labor, and supervision to design and install stone columns. Design shall rely on subsurface information presented in the project geotechnical report. Removal of spoils from the site (which result from stone column construction), removal of spoils off the working pad, footing excavation, and subgrade preparation following stone column installation is not included.

B. Drawings and General Provisions of the Contract, including General and Supplemental Conditions, and Division 1 Specifications, apply to the work in this specification.

1.4 APPROVED INSTALLERS:

A. Installers of stone column foundation systems shall have a minimum of 3 years of experience with the installation of stone columns and shall have completed at least 10 projects.

1.5 RELATED WORK:

A. Section 033000 – Cast in Place Concrete.

B. Section 312000 – Building Earthwork.

C. Geotechnical Report and Recommendations.

1.6 REFERENCE STANDARDS:

A. Design: The ground improvement installer shall be responsible for design of a vibro stone column ground improvement system that meets the allowable bearing capacity, and settlement requirements stated on the contract plans. Industry recognized standards or design methods specific to the installer's equipment and construction methods shall be used.

B. Modulus and Uplift Testing:

1. ASTM D-1143 – Pile Load Test Procedures.

2. ASTM D-1194 – Spread Footing Load Test.

3. ASTM-D-3689 – Uplift Load Test (if required).

C. Materials and Inspection:

1. ASTM D-1241 – Aggregate Quality.

2. ASTM STP 399 – Dynamic Penetrometer Testing (if applicable).

3. ASTM D-422 – Gradation Soils.

1.7 CONFLICTS IN SPECIFICATIONS/REFERENCES: Where specifications and reference documents conflict, the Architect/Engineer shall make the final determination of the applicable document.

1.8 CERTIFICATIONS AND SUBMITTALS:

A. The installer shall submit detailed design calculations and construction drawings to the Architect and to the Geotechnical Engineer of Record for approval at least three (3) weeks prior to the start of construction. All plans shall be sealed by a Professional Engineer in the State in which the project is constructed (referred in this specification as "the Designer").

B. The Stone Column engineer shall have Errors and Omissions design insurance for the work. The insurance policy should provide a minimum coverage of $2 million per occurrence.

C. Modulus and uplift test data - The Installer shall furnish the General Contractor a description of the installation equipment, installation records, complete test data, analysis of the test data and recommended design parameter values based on the modulus test results. The report shall be prepared under supervision of a registered professional engineer.

D. Daily Progress Reports – The Installer shall furnish a complete and accurate record of stone column installation to the General Contractor. The record shall indicate the pier location, length, average lift thickness and final elevations of the base and top of piers. The record shall also indicate the type and size of the densification equipment used. The Installer shall immediately report any unusual conditions encountered during installation to the General Contractor, to the Designer and to the Testing Agency.

1.9 BASIS OF PAYMENT:

A. This work will be paid for at the contract lump sum price for GROUND IMPROVEMENT.

PART 2 - PRODUCTS

2.1 MATERIALS:

A. Aggregate used for piers be selected by the Installer and successfully used in the modulus test.

B. Potable water or other suitable source shall be used to increase aggregate moisture content where required. Access to water on site shall be provided to the Installer.

C. Installer to coordinate adequate and suitable marshalling areas on the project site for the use of the Installer for the storage of aggregate and equipment.

PART 3 - DESIGN REQUIREMENTS

3.1 STONE COLUMN DESIGN:

A. The Stone column design stiffness modulus value shall be verified by the results of the modulus test, described in this specification.

B. Stone Columns shall be designed in accordance with generally-accepted engineering practice and the methods described in Section 1 of these Specifications. The design shall meet the following criteria.

1. Minimum Allowable Bearing Pressure for Stone column Reinforced Soils:X,XXX psf.

2. Minimum Stone column Area Coverage (for square Spread Footings): 20%.

3. Estimated Total Long-Term Settlement for Footings: XX-inch.

4. Estimated Long-Term Differential Settlement of Adjacent Footings: XX-inch.

C. The design submitted by the Installer shall consider the bearing capacity and settlement of all footings supported by stone columns, and shall be in accordance with acceptable engineering practice and these specifications. Total and differential settlement shall be considered. The design life of the structure shall be 50 years.

D. The Stone Column system shall be designed to preclude plastic bulging deformations at the top-of–pier design stress and to preclude significant tip stresses. The results of the modulus test shall be used to verify the design assumptions.

3.2 DESIGN SUBMITTAL: The Installer shall submit eight (8) sets of detailed design calculations, construction drawings, and shop drawings, (the Design Submittal), for approval at least three (3) weeks prior to the beginning of construction. A detailed explanation of the design parameters for settlement calculations shall be included in the Design Submittal. Additionally, the quality control test program for stone columns, meeting these design requirements, shall be submitted. All computer-generated calculations and drawings shall be prepared and sealed by a Professional Engineer, licensed in the State or Province where the piers are to be built.

PART 4 - CONSTRUCTION

4.1 STONE COLUMNS:

A. Install stone columns with a down-hole vibrator capable of densifying the aggregate by forcing it radially into the surrounding soil. The vibrator shall be of sufficient size and capacity to construct stone columns to the diameters and lengths shown on the installer's approved construction drawings.

B. The probe and follower tubes shall be of sufficient length to reach the elevations shown on the installer's approved construction drawings. The probe, used in combination with the available pressure to the tip jet, shall be capable of penetration to the required tip elevation. Preboring shall be permitted if it is specified in the installer's approved construction procedure submittal.

C. The probe and follower shall have visible markings at regular increments to enable measurement of penetration and repenetration depths.

D. Provide methods for supplying to the tip of the probe a sufficient quality of air or water to widen the probe hole to allow adequate space for stone backfill placement around the probe.

E. The probe shall penetrate into the foundation soil layer to the minimum depths required in the installer's construction plans.

F. Lift thickness shall not exceed 4 feet. After penetration to the treatment depth, slowly retrieve the vibrator in 12-inch to 18-inch increments to allow backfill placement.

G. Compact the backfill in each lift by repenetrating it at least twice with the vibrating probe to densify and force the stone into the surrounding soil.

H. Install stone columns so that each completed column is continuous throughout its length.

4.2 PLAN LOCATION AND ELEVATION OF STONE COLUMNS: The center of each stone column shall be within six inches of the plan locations indicated. The final measurement of the top of piers shall be the lowest point on the aggregate in the last compacted lift. Piers installed outside of the above tolerances and deemed not acceptable shall be rebuilt at no additional expense to the Owner.

4.3 REJECTED STONE COLUMNS: Stone columns improperly located or installed beyond the maximum allowable tolerances shall be abandoned and replaced with new piers, unless the Designer approves other remedial measures. All material and labor required to replace rejected piers shall be provided at no additional cost to the Owner.

PART 5 - QUALITY CONTROL

5.1 QUALITY CONTROL REPRESENTATIVE:

A. The Installer shall have a full-time Quality Control (QC) representative to verify and report all QC installation procedures. The Installer shall immediately report any unusual conditions encountered during installation to the Design Engineer, the General Contractor, and to the Testing Agency.

B. Stone Column installation shall be monitored by an on board computer monitoring system. Monitoring system shall log stone column number, time of installation, depth, hydraulic pressure applied during the boring process and during the compacting process. Recorded data for each stone column shall be plotted depth/pressure versus time. Installation records for each shall be made available upon request in electronic format within 24 hours of installation.

C. The QC procedures shall include the preparation of Stone Column Progress Reports completed during each day of installation and containing the following information:

1. Footing and stone column location.
2. Stone column length and drilled diameter (if pre-drilled).
3. Planned and actual stone column elevations at the top and bottom of the element.
4. Average lift thickness for each stone column.
5. Soil types encountered at the bottom of the stone column and along the length of the element.
6. Depth to groundwater, if encountered.
7. Documentation of any unusual conditions encountered.
8. Type and size of densification equipment used.

5.2 QUALITY CONTROL VERIFICATION PROGRAM:

A. The installer shall be responsible for design of a verification program to assure the quality of the construction. The program shall verify that the installed ground improvement system satisfies the performance requirements noted on the contract plans and the design requirements determined by the ground improvement system designer. As a minimum, the verification program shall include the following:

1. Proposed means and methods for verification that the installed stone columns meet the strength and/or stiffness criteria required by the design. This may include, but shall not be limited to, modulus or load tests on individual elements and/or groups, soil borings, and other methods as approved by the Engineer.
2. Quality control program to verify that the ground improvement system is installed in accordance with the designer's specifications and the requirements in this special provision. The quality control program shall include testing and observations by qualified personnel employed by the ground improvement installer or an independent testing laboratory.

PART 6 - QUALITY ASSURANCE

6.1 INDEPENDENT ENGINEERING TESTING AGENCY: The Owner or General Contractor is responsible for retaining an independent engineering testing firm to provide Quality Assurance services. The Testing Agency should be the Geotechnical Engineer of Record.

6.2 RESPONSIBILITIES OF GEOTECHNICAL ENGINEER & INDEPENDENT ENGINEERING TESTING AGENCY:

A. The Geotechnical Engineer of Record shall review and approve the Installer's Design Submittal.

B. The Testing Agency shall monitor the installation of stone columns to verify that all work is performed in accordance with the approved Design Submittal.

C. The Testing Agency & Geotechnical Engineer of Record shall observe footing excavations and densification of stone columns and provide written reports per section 7.3.D.

D. The Testing Agency shall report any discrepancies to the Installer and General Contractor immediately.

PART 7 - RESPONSIBILITIES OF GENERAL CONTRACTOR

7.1 PREPARATION:

A. The Installer shall locate and protect underground and aboveground utilities and other structures from damage during installation of the stone columns.

B. The General Contractor will provide the site to the Installer, after earthwork in the area has been completed.

C. Site subgrade shall be established by the General Contractor within 6 inches of final design subgrade, as approved by the Design Engineer.

7.2 UTILITY EXCAVATIONS:

A. The General Contractor shall coordinate all excavations made subsequent to Stone column installations so that at least five feet of horizontal distance remains between the edge of any installed Stone column and the excavation. In the event that utility excavations are required at horizontal distances of less than five feet from installed Stone columns, the General Contractor shall notify the Stone column Designer to develop construction solutions to minimize impacts on the installed Stone columns.

B. Recommended procedures may include:

1. Using cement-treated base to construct portions of the Stone columns subject to future excavations.

2. Replacing excavated soil with compacted crushed stone in the portions of excavations where the stone columns have been disturbed. The placement and compaction of the crushed stone shall meet the following requirements.

a. The crushed stone shall meet the gradation specified by the Designer.

b. The crushed stone shall be placed in a controlled manner using motorized impact compaction equipment.

c. The aggregate should be compacted to 95% of the maximum dry density as determined by the modified Proctor method (ASTM D-1557).

d. The Testing Agency shall be on site to observe placement, compaction, and provide density testing. The test results shall be submitted to the Designer and the General Contractor. The subcontractor shall provide notification to the Testing Agency and the Designer when excavation, placement, and compaction will occur and arrange for construction observation and testing.

7.3 FOOTING BOTTOMS:

A. Excavation and surface compaction of all footings shall be the responsibility of the General Contractor.

B. Foundation excavations to expose the tops of Stone columns shall be made in a workmanlike manner, and shall be protected until concrete placement, with procedures and equipment best suited to (1) prevent softening of the matrix soil between and around the Stone columns before pouring structural concrete, and (2) achieving direct and firm contact between the dense, undisturbed Stone columns and the concrete footing.

C. Recommended procedures for achieving these goals are to:

1. Limit over-excavation below the bottom of the footing to 3-inches (including disturbance from the teeth of the excavation equipment,

2. Compaction of surface soil and top of stone columns shall be prepared using a motorized impact compactor ("Wacker Packer," "Jumping Jack," or similar). Sled-type tamping devices shall not be used. Compaction shall be performed over the entire footing bottom to compact any loose surface soil and loose surface pier aggregate.

3. Place footing concrete immediately after footing excavation is made and approved, preferably the same day as the excavation. Footing concrete must be placed on the same day if the footing is bearing on expansive or sensitive soils.

4. If same day placement of footing concrete is not possible, place a minimum 3-inch thick lean concrete seal ('mud mat") immediately after the footing is excavated and approved.

D. The following criteria shall apply, and a written inspection report sealed by the project Geotechnical Engineer shall be furnished to the Installer to confirm:

1. That water (which may soften the unconfined matrix soil between and around the Stone columns, and may have detrimental effects on the supporting capability of the stone column reinforced subgrade) has not been allowed to pond in the footing excavation at any time.

2. That all stone columns designed for each footing have been exposed in the footing excavation.

3. That immediately before footing construction, the tops of all the Stone columns exposed in each footing excavation have been inspected and recompacted as necessary with mechanical compaction equipment, and that the tops of any Stone columns which may have been disturbed by footing excavation and related activity have been recompacted to a dry density equivalent to at least 95% of the maximum dry density obtainable by the modified Proctor method (ASTM D-1557).

4. That no excavations or drilled shafts have been made after installation of Stone columns within horizontal distance of five feet from the edge of any pier, without the written approval of the Installer or Designer.

END OF SECTION XX XX XX