REPORT ON SUBSURFACE EXPLORATIONS AND GEOTECHNICAL ENGINEERING RECOMMENDATIONS
THE WATERMARK
PORTLAND, MAINE

by

Haley & Aldrich, Inc.
Portland, Maine

for

Riverwalk, LLC
Portland, Maine

File No. 30322-000
16 May 2007

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Riverwalk, LLC
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Portland, Maine 04101

Attention: Mr. Drew Swenson
Manager

Subject: Report on Subsurface Explorations and
Geotechnical Engineering Recommendations
THE WATERMARK
Portland, Maine

Ladies and Gentlemen:

This report summarizes our evaluation of subsurface explorations and provides our recommendations for geotechnical design and construction of the subject project located at India and Fore Streets in Portland, Maine. Our study was undertaken in accordance with our proposal dated 23 September 2005 and your subsequent authorization.

We have coordinated our work with the following project team members:

▪ Intercontinental Real Estate Corp.	Owner
▪ Riverwalk, LLC	Owner
▪ Gilbane Building Co.	General Contractor
▪ The Architectural Team, Inc.	Architect
▪ McNamara/Salvia, Inc.	Structural Engineer
▪ Woodard & Curran	Civil Engineer
▪ AHA Consulting Engineers	MEP Engineer

SUMMARY

We recommend that the proposed structure be supported using a combination of precast prestressed concrete piles driven to bearing in/on the underlying bedrock and conventional spread footings bearing directly on a prepared and approved bedrock surface. We recommend that an underslab and perimeter foundation drainage system be installed beneath the below grade portion of the building and adjacent to the perimeter below grade foundation walls. We

also recommend that the lowest level floor slabs be designed as soil supported slabs-on-grade. A temporary excavation support system will be required to construct the below grade portion of the new structure.

To insure the recommendations stated herein are incorporated into the design as intended, we recommend that Haley & Aldrich be involved in preparing the geotechnical Contract Documents, reviewing geotechnical related submittals, and performing on-site monitoring of the geotechnical aspects of construction in the field on behalf of the Owner. Specific recommendations for foundation design and construction are presented below.

ELEVATION DATUM

The project elevation datum and elevations referenced herein are in feet and reference Portland City Datum (PCD). Portland City Datum relates to the National Geodetic Vertical Datum of 1929 (NGVD 29) as follows:

$$\text{Elevation in feet (PCD)} = \text{Elevation in ft (NGVD 29)} + 0.02 \text{ ft}$$

SITE LOCATION, EXISTING SITE CONDITIONS & PREVIOUS USE

The general location of the project site is shown on Figure 1, Project Locus. For the purposes of this report, we will refer to Fore Street as the west boundary, India Street as the south boundary, Hancock Street Extension the north boundary and Commercial Street Extension as the east boundary. A three-story brick building (Grand Trunk Building) is present in the southeast corner of the site, at the intersection of Commercial and India Streets. An existing wastewater pump station operated by the Portland Water District (PWD Pump Station) is located at the southwest corner of the site. The site is currently being used as a parking lot; the majority of which is gravel but with some exposed bituminous and concrete surfaces. Two single-story, prefabricated metal buildings were present in the southeast portion of the site, adjacent to the Grand Trunk Building, and were demolished in April 2007. The site is relatively flat with site grades ranging from El. 14 along Commercial Street Extension to El. 18 along India Street.

The parcel was previously occupied by the Canadian National Railways Grand Trunk Railway System and operated as such until the mid-1980s. During operation the parcel formerly housed a circular round house in the northwest portion of the site and a large passenger depot running parallel with the present Commercial Street Extension (circa 1886 through 1896) and later became a rail yard used for shipping (circa 1909 to 1980). Historical Sanborn Maps of the site are provided for informational purposes in Appendix E. Abandoned foundations from structures that formerly occupied the site will likely be encountered during construction. These potential obstructions may include, but are not limited to granite blocks, wood piles, concrete slabs, railroad rails, footings, pile caps and grade beams. Refer to subsequent sections of this report for additional information.

PROPOSED DEVELOPMENT

Based on the current site development plans and details provided by The Architectural Team, Inc. (TAT) we understand that the development will include a six-story, above-grade structure comprised of both residential and retail space. The proposed building will contain one level of below grade parking. The main portion of the structure measures approximately 240 ft by 215 ft in plan dimension with a townhouse wing extending roughly 120 ft to the south, between the Grand Trunk Building and the PWD Pump Station.

The finished floor elevation (FFE) of the below grade parking level is currently planned at approximately El. 4. Vehicular access to the below grade parking area is planned at the southwest corner of the site, off of Fore Street, between column lines 8 and 9. Based on discussions with TAT the entrance ramp will consist of a structural slab (i.e., the ramp will not be constructed as a slab-on-grade supported by earth fill).

The FFE for the ground floor residential/retail space varies across the building footprint and is currently planned as follows: El. 15 along Commercial Street Extension and El. 17 along Fore Street with the difference being made up along Hancock Street Extension (location not yet determined). The proposed elevation of the inner courtyard area will be approximately El. 16. The first floor level of the townhouse wing also varies; El. 17 for the unit furthest from India Street, El. 17.5 for the next three units to the south and El. 18.8 for the unit closest to India Street.

Bay spacing varies throughout the building footprint but is typically on the order of 25 ft by 30 ft in plan dimension. Design column loads (axial compression) were provided by McNamara/Salvia, Inc. (MacSal) and range from 500 to 700 kips (1,000 lbs = 1 kip) for the interior columns and 250 kips in the courtyard area. Maximum column loads (axial compression) for the townhouse wing will be approximately 400 kips. Based on discussions with MacSal, it is our understanding that axial uplift loads at specific column locations are negligible and will be resisted using the dead weight of the footings/pile caps, and that maximum lateral column loads will be 100 kips, with a maximum total lateral load for the building equal to 1,000 kips.

SUBSURFACE EXPLORATIONS

General

Multiple subsurface exploration programs were undertaken in and around the area proposed for the subject project. Test borings were drilled in the vicinity of the site for design of the PWD Pump Station by Northern New England Test Boring Company of Portland, Maine in 1975.

Test borings were drilled at the site by Maine Test Borings, Inc. (MTB) of Brewer, Maine in 2005 and 2007 for design the subject project. Haley & Aldrich personnel were present to monitor the drilling (2005 and 2007) and to document the soil, rock and groundwater conditions encountered at each test boring location. Test boring locations are shown on Figure 2 and test boring logs are provided in Appendices A (1975) and B (2005 and 2007), respectively.

In general, soil samples were obtained by driving a 24-in. long, 1-3/8-in. inside diameter (ID) split-spoon sampler with a 140-lb weight dropped 30 in. The number of hammer blows required to advance the sampler for each 6-in. interval was recorded and is provided on the test boring logs. The SPT N-value is the total number of the hammer blows required to advance the sampler through the middle 12-in. of the 24-in. sampling interval and is referred to herein.

1975 Subsurface Explorations

Previous explorations were conducted as part of the PWD Pump Station project. A total of five test borings, designated B5-19 through B5-23, were advanced to depths ranging from 16.5 to 47.9 ft below ground surface (BGS). Three test borings (B5-19, B5-20 and B5-21A) were advanced between 9.4 and 15 ft into bedrock.

2005 and 2007 Subsurface Explorations

A preliminary phase subsurface exploration program was undertaken in October 2005 to define general subsurface conditions to allow for a preliminary assessment of foundation alternatives for the proposed commercial/retail building. A total of thirteen test borings, designated HA05-11 through HA05-23A, were advanced to depths ranging from 5 to 79 ft BGS.

In-situ vane shear tests were conducted within the glaciomarine clay deposit in test borings HA05-12, HA05-14 and HA05-15. Vane shear tests were performed to provide information on the shear strength and compressibility characteristics of the glaciomarine clay deposit at the site. Strength and compressibility characteristics of the deposit are discussed later in this report. In addition, two, 2-½ in. ID undisturbed tube samples were obtained from test borings HA05-11 and HA05-13. Results of the vane shear tests are summarized in Table II and can be found on the test boring logs provided in Appendix B.

Three groundwater observation wells were installed in completed boreholes HA05-11, HA05-14 and HA05-17 in order to facilitate monitoring of the groundwater levels. The three wells were screened in the near surface soils to determine the static water levels at the site.

Due to the extreme variability in subsurface conditions encountered across the site during the preliminary phase boring program, it was necessary to conduct a design phase subsurface exploration program. The primary purpose of this program was to help determine the likely foundation conditions at specific column locations within the "transition zone" between shallow bedrock areas and thicker clay areas at the site. The program was undertaken in March 2007 and consisted of fourteen test borings, designated HA07-101 through HA07-113, that were drilled from depths ranging from 6.7 to 67.0 ft BGS. Test borings were advanced using steel casing, hollow stem augers or solid stem augers depending on the depth and purpose of each boring. Some test borings were advanced to refusal using a rod probe. This process consisted of driving a solid-stem, 2-in. diameter rod (with a 300-lb hammer dropping 18 in.), through the soil overburden to refusal at depth. Please note that only a few soil samples were collected (and SPT "N-values" recorded) during this program. Test boring HA07-101 was drilled approximately 8 ft into bedrock using a diamond tipped core barrel. The test borings were typically backfilled with the drill spoils upon completion.

In addition, one exploratory test pit was excavated at the northwest corner of the existing Grand Trunk Building to inspect the condition of the foundation wall and to determine the foundation type and bearing level. Detailed test pit sketches and photographs are provided in Appendix B.

SUBSURFACE CONDITIONS

Soil/Bedrock Conditions

The subsurface conditions encountered at the site consisted of the following geologic units presented in increasing depth below ground surface: bituminous concrete/portland cement concrete, miscellaneous fill, glaciomarine deposit (sand), organic deposit, glaciomarine deposit (clay), glaciomarine deposit (sand), glacial till and bedrock.

Bituminous Concrete/Portland Cement Concrete: A relatively thin layer of bituminous concrete and Portland cement concrete ranging from 1 to 5 in. thick was generally encountered at boring locations adjacent to Commercial Street Extension and in the central portion of the proposed building footprint. Additional concrete slabs are exposed at ground surface throughout the building footprint.

Miscellaneous Fill: The Portland waterfront has a long history of filling. Fill was encountered in each test boring at the site and generally ranged in thickness from 5 to 20 ft. The thickness of fill was typically less than 10 ft within the limits of the proposed townhouses (Column Lines 1 through 6) and within the limits of the residential/retail space (Column Lines 6 through 13) adjacent to the existing Grand Trunk Building. The thickness of fill generally increases to as much as about 20 ft to the west (Fore St.) and to the north (Hancock St. Ext.). Large obstructions were encountered within the fill at several test boring locations throughout the proposed building footprint (at locations shown on Figure 2; designated with "OB"). The material generally consisted of sand, gravel, brick and concrete fragments and miscellaneous construction rubble, and was medium dense to very dense with SPT N-values ranging from 2 to greater than 100 blows per foot (bpf). The majority of fill soils will be excavated during construction, assuming a finish floor elevation of El. 4 and a foundation bearing level of El. -1.5 in the below grade parking area.

Organic Deposit: Organic soil was encountered in test borings drilled in the central and northeastern portions of the site. The material was encountered at or below El. -4 and generally ranged in thickness from 4 to 6 ft. The deposit consisted of dark brown to gray, organic SILT (ML) and was typically soft to very stiff with SPT N-values ranging from 4 to 15 bpf. Where encountered, the deposit was typically overlain by a thin layer of glaciomarine sand. This material was most likely former harbor bottom sediment deposited prior to site filling.

Glaciomarine Deposit (clay): Glaciomarine clay was generally encountered in test borings drilled in the western and northern thirds of the building footprint. The deposit generally increases in thickness from south to north and east to west, and ranged between 23 and 44 ft thick. The upper 5 to 8 ft of the deposit consisted of olive gray lean CLAY (CL) and was typically medium stiff to very stiff with SPT N-values ranging from 9 to 23 bpf (referred to herein as the clay "crust"). The remainder of the deposit consisted of very soft to stiff, gray lean CLAY (CL) with SPT N-values ranging from weight of rods (WOR) to 9 bpf. The undrained shear strength of the clay typically ranged from 1,000 to 2,000 pounds per square foot (psf) in the clay crust, and from 400 to 1,000 psf in the clay below the crust.

Glaciomarine Deposit (sand):

Shallow deposits of glaciomarine sand were encountered in test borings generally in the central and northeast portions of the site. The deposit was typically encountered between El. 4.5 to El. -7 and ranged in thickness from 2 to 5 ft. Where encountered, the sand was underlain by the organic deposit. Deeper deposits of glaciomarine sand were encountered beneath glaciomarine clay sporadically across the site (in test borings HA05-12, HA05-13 and HA07-106). The deposit was encountered from El. -34 to El. -44 and ranged in thickness from 4 to 7 ft. The material consisted of either gray, silty SAND (SM) or poorly graded SAND (SP). The shallow deposit was typically medium dense to dense with SPT N-values ranging from 11 to in excess of 50 bpf, while the deeper deposit was generally loose to medium dense with SPT N-values ranging from 7 to 16 bpf.

Glacial Till: Glacial till was encountered beneath the glaciomarine clay and/or glaciomarine sand in test borings located in the north and northeastern portions fo the building footprint. Where encountered, the elevation of the top of the deposit varied greatly from El. 1 to El. -38.5 and ranged in thickness from 1 ft to greater than 17 ft (HA05-13). The deposit was typically medium dense to very dense with SPT N-values ranging from 11 to greater than 50 bpf.

Bedrock: Bedrock was encountered and sampled at multiple test boring locations throughout the proposed building footprint. Within the limits of the townhouse wing the bedrock surface was generally encountered between El. 8.5 and El. 10.5, approximately 6 to 9 ft BGS. Within the limits of the proposed residential/retail space the bedrock surface varies significantly (see Figure 2 for top of rock elevations in each boring at the site). For example, along S-line the bedrock surface slopes down from south to north from El. 8 at the Grand Trunk Building to El. -59 at Hancock St. Extension (see Figure 4). In general, the bedrock surface drops steeply to the north between column lines 20 and 23. Along 8-line, the bedrock surface drops from east to west from El. 6.4 at G-line to El. -18 at C-line (see Figure 5).

The encountered bedrock is described as hard to moderately hard, fresh to slightly weathered SCHIST. Rock quality designation (RQD) is a common parameter that is used to aid in assessing the competency of sampled bedrock. RQD is defined as the sum of the lengths of pieces of recovered rock core greater than 4 in. in length, divided by the total length of the recovered rock core. RQD values for the bedrock encountered at the site typically ranged from between 50 and 75 percent and are shown on the test boring logs in Appendix B.

Subsurface profiles illustrating interpreted geologic conditions, determined from test boring data, were developed at several locations throughout the building footprint as follows:

- Figure 3 – Subsurface Profile A-A, between the PWD Pump Station and the Grand Trunk Building
- Figure 4 – Subsurface Profile B-B, along R-Line
- Figure 5 – Subsurface Profile C-C, along Column Line 8 between B-Line and G-Line

Groundwater Conditions

As previously mentioned, three groundwater monitoring wells were installed within the footprint of the proposed structure; one adjacent to Fore Street, one adjacent to India Street and one near the future intersection between Hancock and Commercial Streets. A summary of measured water levels is provided below.

Well Location (Test Boring No.)	**Approximate Groundwater Levels**
Adjacent to Fore Street (HA05-11)	El. 6.5 to El. 7.2
Intersection of Commercial & Hancock Street Extensions (HA05-14)	El. 4.7 to El. 6.3
Adjacent to India Street (HA05-17)	El. 4.3 to El. 5.3

Water levels were measured at times corresponding to local high and low tides to determine the tidal influence on the static groundwater levels at the site. Based on the measurements, it is our opinion that the water levels at the site are not substantially influenced by tidal fluctuations in Casco Bay. Please note that the elevations shown in the table above are approximate and were determined based on interpolating between ground surface contours provided by Woodard & Curran.

Groundwater levels can be expected to fluctuate, subject to seasonal variation, precipitation, local soil conditions, topography, leakage into and out of sewers, storm drains and other below-grade structures, and other factors. Groundwater levels encountered during construction may differ from those observed in the test borings or observation wells. Observation well installation and groundwater monitoring reports are included in Appendix C.

LABORATORY TESTING

A laboratory testing program was undertaken to classify the in-situ fill soils in order to help assess its reuse potential during site development. The laboratory testing program consisted of five grain size analyses, as summarized below.

Test Boring (Sample No.)	Sample Depth	Percent Gravel	Percent Sand (course/med/fine)	Percent Fines[1]	USCS Classification
HA05-11 (S1)	0.0-2.0	15.0	64.0 (12.0/27.0/25.0)	21.0	SM
HA05-13 (S2 & S3)	2.0-6.0	29.0	62.0 (8.0/27.0/27.0)	9.0	SP-SM
HA05-14 (S2)	2.0-4.0	13.0	75.0 (8.0/39.0/28.0)	12.0	SP-SM
HA05-19 (S1 & S2)	0.5-4.5	15.0	76.0 (8.0/37.0/31.0)	9.0	SW-SM
HA05-21 (S2 & S3)	2.0-6.0	16.0	72.0 (11.0/35.0/26.0)	12.0	SP-SM

Note: [1] refers to the percentage of soil particles finer than the No. 200 (0.075 mm) sieve

The results of the laboratory testing program are included in Appendix D. The potential for reusing these soils as common and/or compacted granular fill at the site is discussed in the Construction Considerations section of this report.

GEOTECHNICAL ENGINEERING RECOMMENDATIONS

This section, intended primarily for members of the design team responsible for design of the structures and preparation of contract documents, provides geotechnical recommendations for foundation design of the proposed structure. In general, design and construction of the proposed development should be completed in accordance with the requirements of the 2003 International Building Code (IBC). Recommendations provided herein refer to provisions in the IBC and relate to the subject project only.

Foundation Systems

Based on the proposed site development (basement FFE of El. 4, foundation bearing level of approximately El.-1.5 and design column loads provided by MacSal) foundation units will bear on bedrock or will extend through overburden soils, developing the required support in/on the underlying bedrock (depending on column location). Initial analyses were performed to assess the feasibility of supporting a portion of the building (area bound by 6-line and 20-line between A-line and E-line) with spread footings bearing on naturally deposited glaciomarine clay. Although technically feasible, Intercontinental preferred to support this portion of the building on piles to minimize the impact of potential differential settlement in the structural design of the building superstructure. As a result, we recommend that a combination of spread footings bearing on bedrock and end bearing piles be used to support the proposed structure. Both are discussed separately, below. A summary of recommended foundation support on a column by column basis is provided in Table III. We anticipate that approximately 45 percent of the columns will be supported using spread footing foundations. The anticipated transition between spread footings and pile foundations is shown graphically on Figure 2.

Footings

The townhouse wing and a portion of the residential/retail space that lies within "Foundation Design Zone A" (see Figure 2) will be supported by reinforced concrete footings bearing directly on competent bedrock. We recommend that the in-situ fill soils present within the townhouse wing, generally between G-line and N-line from 1-line to 6-line be over-excavated in order for the foundations to bear directly on bedrock. Based on the level of the bedrock encountered in this area, we estimate that the foundations for the townhouse wing will bear between El. 3 and El. 10.

Based on our discussions with MacSal and a proposed lowest level floor slab at El. 4 within the residential/retail portion of the building, it is our understanding that the footings would ideally bear at approximately El.-1.5. As a result, it will be necessary to remove up to 12 ft of bedrock to construct the footing foundations in the southeast portion of the basement footprint (generally bound by F-line and T-line between 6-line and 13-line).

Based on the condition of the bedrock encountered within "Foundation Design Zone A", we recommend that spread footing foundations be designed using an allowable bearing pressure equal to 30 tons per square foot (tsf) with a minimum footing width of 3 ft. We anticipate resulting elastic settlements will be less than ¼-in. Based on discussions with MacSal, we understand that this amount of settlement is acceptable.

Pile Foundations

It is our opinion, based on the subsurface conditions and the range in design loads, that a variety of pile types (e.g., steel H-piles and precast, prestressed concrete (PPC) piles) are technically feasible. However, based on recent contractor bids for the adjacent Ocean Gateway Parking Garage project, we recommend that PPC piles be used for this project based on the current market economics. Recognizing that the cost of installation of the various pile types fluctuates, the final pile selection may change based on pile availability and economics at the time the project goes out to bid.

We recommend that columns within "Foundation Design Zone B" be supported on 100-ton capacity, 12-inch square, PPC piles driven to practicable refusal in/on the underlying bedrock. PPC piles should be designed in accordance with the IBC and current standards of the Joint Committee of AASHTO and the Precast/Prestressed Concrete Institute (PCI) using a minimum 5,000 psi compressive strength concrete. In addition, we recommend the piles be equipped with a 1½-in. thick steel bottom plate and appropriate spiral steel reinforcement in the upper portion of the pile for seismic connection at the pile cap.

We anticipate that piles may advance up to 1 ft into the bedrock prior to achieving end bearing. Based on this, a proposed basement finish floor elevation of El. 4 and an average, assumed pile cut-off level equal to El. 0, pile lengths should vary between 20 and 70 ft. We anticipate that piles at columns J-19 and L-19 will be slightly shorter, on the order of 10 to 15 ft in length. Based on these pile lengths, we anticipate that some pile splicing will be needed for the piles installed in the northern portion of the building footprint, generally between column lines 20 and 23.

The piles should be installed to a minimum ultimate geotechnical capacity equal to the design capacity multiplied by 2.25 (225 tons). The installation/driving criterion for the piles is a function of pile hammer selected by the Contractor to install the piles. This criterion should the determined by the Contractor's engineer (using wave equation analysis; WEAP) and reviewed/approved by Haley & Aldrich prior to construction. The requirements of this analysis will be outlined in the pile specification. The installation/driving criterion provided by the Contractor will determine the number of hammer blows required to drive the pile over the final 6 in. of driving, which will result in the pile achieving the required minimum ultimate geotechnical capacity (2.25 x pile design capacity). If abrupt refusal is encountered, driving should be terminated when the pile penetration is less than ½-in. for 10 consecutive hammer blows.

It is our opinion that dynamic pile testing could be used in lieu of a static pile load test. Dynamic testing is more cost effective than static load testing, provides reliable pile capacity information and is accepted by the IBC. The dynamic testing will: 1.) verify that the pile ultimate capacity is achieved; 2.) confirm the bearing capacity value for rock used in the pile design; and 3.) confirm that the stresses in the pile do not exceed allowable limits during driving as specified by the IBC. We recommend that the Contractor monitor the installation of approximately 3 to 5 percent of the production piles (i.e., indicator piles) using the Case-Goble

Pile Driving Analyzer (PDA) equipment. In addition, CAPWAP analysis should be performed on a select number of the indicator piles installed during the PDA testing program. Use of dynamic testing alone will likely require approval from the City of Portland building official. Please note that installation of driven piles is a vibration and noise producing activity. If the potential vibration and noise caused by driving piles is not acceptable to City of Portland officials, then the use of alternative foundation units may become a more feasible option.

Frost Protection

Bottoms of exterior footings bearing on rock should be founded a minimum of 3 ft below the lowest adjacent ground surface exposed to freezing. Bottoms of interior footings in heated areas should be founded a minimum of 2 ft below the top of the adjacent floor slab. Based on the proposed site development, we anticipate that all exterior footings and pile caps will bear at depths greater than 3 ft below finished grade (10 to 20 ft typical).

Ground Floor Slab

We recommend that the ground floor slab for the townhouse wing and the below grade parking level in the residential/retail portion of the structure both be designed as a soil-supported, concrete slab-on-grade. The ground floor slab for the townhouse wing should bear on a minimum of 12 in. of compacted granular fill (CGF). We recommend that the slab for the below grade parking level bear directly on crushed stone placed as part of the underslab drainage system (see below). All previous construction debris (e.g., foundation walls, slabs, footings and underground utilities) should be removed from within the building limits prior to construction.

Resistance of Lateral Building Loads

We recommend that structure lateral loads (maximum 1,000 kips) be resisted by passive earth pressures acting against foundation walls, footings, pile caps and grade beams. The net passive resistance (passive minus active) provided by the fill surrounding foundation walls, grade beams and pile caps can be calculated using an equivalent fluid weight (triangular distribution) of 300 pounds per cubic foot (pcf). This value assumes that granular backfill is free-draining and is placed and compacted in lifts. If the backfill is not systematically compacted, an equivalent fluid unit weight of 250 pcf should be used. The top of the assumed passive zone should be 1 ft below the ground surface unless it is confined by a slab or bituminous concrete.

As discussed with MacSal, we anticipate that passive earth pressures acting on the below grade portions of foundation walls will be adequate to provide resistance for the design maximum building lateral loading condition (1,000 kips). A minimum factor of safety for sliding equal to 2.0 should be achieved for resistance of permanent lateral loads.

Lateral Earth Pressures on Below-Grade Foundation Walls

We recommend that exterior below-grade foundation walls retaining soil on one side and restrained at the top should be designed for static lateral earth pressures using an equivalent fluid unit weight of 60 pcf. Cantilever walls (i.e., walls that are free to rotate at the top) should be designed using an equivalent fluid unit weight of 40 pcf. These fluid weights assume a free-draining granular backfill is placed adjacent to the wall (with moist unit weight equal to 120 pcf) and that a perimeter foundation drain system is installed as recommended herein (i.e., no unbalanced hydrostatic pressures exist; "drained condition"). In particular, we anticipate that below grade portions of foundation walls will need to be designed to permanently resist lateral earth pressures up to approximately El. 18.

Seismic Design Considerations

We recommend that the structure be designed in accordance with the seismic requirements of the latest edition of the IBC as outlined below. Due to the nature and thickness of overburden soils and the depth to bedrock specifically in the northern and eastern portions of the site, we recommend the site be considered "Site Class D". In addition, we recommend the following values be used by MacSal to determine the design spectral response acceleration parameters (S_{DS} and S_{D1}) and to calculate the base shear for purposes of seismic design.

- Mapped Spectral Response Accelerations for Short Periods: $S_S = 0.368$ g
- Mapped Spectral Response Accelerations for 1-second Periods: $S_1 = 0.098$ g
- Site Coefficient for Short Periods: $F_a = 1.506$
- Site Coefficient for 1-second Periods: $F_V = 2.40$

Please note that "g" refers to acceleration due to gravity.

We do not consider the soils present at this site to be liquefaction susceptible.

Foundation Drainage System

Due the proximity of the static groundwater levels (El. 5 to El. 7) to the anticipated level of the basement floor slab (El. 4), we recommend that a foundation drainage system be installed to protect the slab from hydrostatic pressures and groundwater infiltration.

The system should include underslab drains installed below the level of the lowest level floor slab in the basement area. The system should consist of non-woven filter fabric placed on the prepared, approved rock/soil subgrade, a minimum 12 in. thick layer of ¾-in. crushed stone placed above the fabric, with a network of 4 in. diameter perforated PVC or corrugated HDPE drain pipe (laid flat) embedded mid-height in the crushed stone layer. We estimate that the invert of the pipes would be approximately 12 in. below the finish floor elevation (estimated

El. 3).

The system should also include perimeter drains installed along the backfilled (exterior) side of below-grade building foundation walls where the interior floor level is below the exterior finished grades (likely along Commercial Street Extension, Hancock Street Extension, Fore Street and along 6-line). The drain should consist of a 4-in. diameter continuous perforated PVC or HDPE drain pipe (laid flat), surrounded by a minimum of 6-in. of ¾-in. crushed stone and a non-woven, 4-oz. filter/separation fabric, placed outside of the foundation wall. Pipe perforations should be oriented downward. The invert of the drain pipe should be positioned above the bearing level of footings/pile caps/grade beams, and at least 12 in. below the adjacent floor slab surface. Per the requirements of the IBC, the perimeter drain (including the pipe, crushed stone and filter fabric) should extend a minimum of 12 in. beyond the outside edge of the footing/pile cap. We recommend that free draining granular backfill be placed within the space between the outside of the foundation walls and the temporary support of excavation system.

Ideally, perimeter and underslab drain pipes should be installed at roughly the same invert elevation. The underslab and perimeter drain pipes should be connected by constructing "wall-through" or "box-out" penetrations at discrete locations in the foundation wall. It will be necessary to install sump pit(s) with pumps to discharge the effluent from the system to the local storm drain system. Based on our groundwater seepage estimates, the pumps should be capable of pumping 20 gallons per minute (gpm). We have discussed this magnitude of seepage with AHA, and they will design their pump systems to accommodate this anticipated flow (likely using 50 gpm capacity pumps). The sump pit should be equipped with dual pumps with alternating cycles, and a back up power system. The sump pit could be constructed either inside the building, or outside of the building adjacent to the foundation wall.

Pipe cleanouts should be provided at system corners (for both perimeter and underslab drain piping) to allow for future maintenance.

As an additional measure, surface runoff should be directed away from the building. In general, the finished ground surface immediately around the building should be sloped downward away from the structure to divert surface runoff. To limit surface water infiltration into the drainage system, it is recommended that the upper 8 in. of backfill within 10 ft of the building, in unpaved areas, consist of topsoil or other soil having low permeability.

We will provide a foundation drainage plan along with the appropriate drain system details for inclusion in the contract documents once the location and elevations of the footings, pile caps, grade beams, and below slab utilities are finalized. The location and invert level of the drains, pipe cleanouts, wall through penetrations and connection to the storm drain system will be coordinated with AHA, MacSal and Woodard & Curran.

Dampproofing/Waterproofing

Waterproofing of walls and floor slabs for the below-grade portions of the building above the invert level of the foundation drain system is not needed.

In general, we recommend that dampproofing and insulation be placed on the outside face of foundation walls where the adjacent interior space is below the level of the exterior ground surface, in accordance with the IBC Code.

The base slab of the elevator pit(s) (top of slab at El. 0) should either be designed to resist hydrostatic uplift loads based on a groundwater level at El. 4, or should be permanently drained. If the slab is designed to resist uplift loads, we recommend that the walls and slab for the elevator pit(s) be waterproofed. If the slab is not designed to resist uplift loads, an underslab drainage system should be constructed beneath the pit slab(s). The system should consist of a minimum of 6 in. of crushed stone placed over a separation geotextile fabric (e.g., Mirafi 140N). The drain system should provide a discharge outlet for the water collected in the system (e.g., connection to the storm drain system or a sump inside/outside the building).

Based on the anticipated use of the below grade space, we do not consider the installation of vapor barriers necessary below the lowest level floor slab in the garage area.

Sidewalks

Brick sidewalks proposed around the exterior of the buildings should be supported on a minimum of 1.5 ft of CGF. The surficial fill soils at the site are considered to be moderately frost susceptible. The purpose of placing free-draining granular soil below the sidewalks is to help control the potential for frost induced post-construction differential heaving and cracking.

CONSTRUCTION CONSIDERATIONS

The primary purpose of this section is to comment on items related to excavation, earthwork, foundation installation, dewatering and related geotechnical aspects of proposed construction. This section is written primarily for the geotechnical engineer having responsibility for preparation of geotechnical related plans and specifications. Prospective contractors should evaluate the potential for construction problems on the basis of their own knowledge and experience in the Portland, Maine area, and on the basis of similar projects in other localities, taking into account their proposed construction methods, procedures, equipment and personnel. Please note that the construction considerations provided below relate to the subject project only.

Demolition

Two single story, prefabricated metal buildings were present in the southeast portion of the site, adjacent to the Grand Trunk Building. The structures were demolished as part of the Ocean Gateway Parking Garage construction (by MC Hall). Gravel, bituminous and concrete is exposed at ground surface on the remainder of the site. We recommend that this material is removed prior to construction. Large obstructions were encountered within the fill at several test boring locations throughout the proposed building footprint (at locations shown on Figure 2). Additional information will be provided in the Contract Documents.

Temporary Excavation Support System

Based on the anticipated elevation of the bottom of footings and pile caps/grade beams within the below grade portion of the building (approximately El. -1.5), existing site grades adjacent to the proposed basement excavation and the proximity of the property lines relative to the location of the proposed basement area, an excavation support system will be required to construct the below grade portion of the proposed building. Based on subsurface soil, rock and groundwater conditions at the site, we anticipate that the most cost effective excavation support system will consist of the following:

Excavation Support System Location	Approx. System Length (lf)	Approx. Max. Height of Retained Soil (ft)	Anticipated Excavation Support System
from N-2 to G-1 and from G-1 to G-5	160	15	permanent drilled-in soldier piles and lagging
from G-5 to G-6 and from G-6 to D-6	85	20	temporary drilled-in soldier piles and lagging
from D-6 to B-6 and from B-6 to A-23	265	20	temporary steel sheetpiling
from A-23 to T-23	250	20	temporary steel sheetpiling
from T-23 to U-20 and from U-20 and R-20	120	20	temporary steel sheetpiling
from R-20 to R-10 and from R-10 to T-10	140	NA	open cut; sloped excavation
from T-10 to T-7	65	10*	temporary drilled-in soldier piles and lagging
from T-7 to M-7, M-7 to M-6, and M-6 to G-6	110	NA	open cut; sloped excavation

Note: * approximately 10 to 15 ft of bedrock will need to be removed in this area below the bottom of the soldier pile and lagging wall.

Please note that the maximum height of retained soil shown above is based on footings, pile caps/grade beams bearing at El. -1.5. Also note that anticipated support of excavation system assumes no disturbance to the new sidewalk areas adjacent to Hancock and Commercial Street Extensions.

Based on discussions between the project team and PWD, permanent support of excavation will be required along G-line (east of the PWD property line) and along 1-line (adjacent to India Street) in order to prevent potential damage to the townhouse wing as a result of future maintenance/upgrade work by PWD on the 33-in. force main. We anticipate that support of excavation systems retaining greater than 15 ft of soil will require internal bracing. The steel sheeting system will aid in cutting off lateral groundwater flow into the excavation. The excavation support system will be designed by the Contractor's engineer as part of the submittal process based on the design requirements outlined in the project specifications.

Please note that Northeast Utilities will be performing vacuum excavation to determine the as-built location and invert elevations of the existing 4 in. diameter that runs along Fore Street adjacent to the western edge of the proposed building. Proper protection/support of this line during construction will be addressed once the vacuum excavation is completed and the results published. We anticipate that this field work will be completed by the end of May 2007.

Excavation

Soil

Excavation will be required for general site grading, and for construction of the building foundations, elevator pits, and underground utilities. We anticipate that excavation of as much as 20 ft BGS will be required to reach the proposed foundation bearing level in the below grade portion of the building.

Based on the proposed site development, we anticipate that between 7 and 14 ft of fill will need to be excavated within the townhouse footprint so that the footings can be supported directly on the underlying bedrock.

We expect that excavation of the in-situ soils (mostly fill and marine deposits) can be accomplished using normal earth-moving equipment. Obstructions will likely be encountered during excavation in the in-situ fill soils. The nature and extent of underground obstructions will likely not become apparent until excavation begins. We recommend that the contract documents require the contractor to include a contingency/line item for obstruction removal in their earthwork bid.

Prior to placing fill within the footprint of the new building, we recommend that all topsoil, debris and organic matter encountered at the subgrade level be removed.

Bedrock

Based on the anticipated bearing level of the spread footings (El. -1.5), we anticipate that up to 12 ft of bedrock will need to be removed in order to construct the foundations in the southeast portion of the basement area (see Table III). The area requiring bedrock removal is generally bound by F-line and T-line between 6-line and 12-line.

Based on our review of the bedrock encountered in the test borings and our experience with similar bedrock types in the Portland area, we anticipate that approximately 4,000 cubic yards of bedrock (in-place volume) will have to be removed using controlled drilling/blasting techniques in lieu of the conventional equipment (hoe-ramming).

We recommend that a pre-construction survey of the existing PWD Pump Station be completed prior to the start of construction. The purpose of the survey is to inspect the site area and existing adjacent buildings in order to identify potentially vibration sensitive structures, equipment and/or utilities. The survey will also aid in determining limiting vibration criteria that should be established to protect the existing structures, equipment and/or utilities. A geotechnical instrumentation program consisting of crack gages, vertical monitoring points and seismographs may be warranted to monitor the existing structures, equipment and/or adjacent utilities during construction.

Dewatering

Based on recently measured groundwater levels at the site, we anticipate that dewatering during construction of the basement area, including excavation for footings, pile caps, grade beams and elevator pits, will be required. We anticipate that partial use of a steel sheetpile support of excavation system will cutoff the majority of lateral groundwater flow into the excavation. As a result, we expect that dewatering could be performed using open sumps and temporary ditches within the excavation. Sumps should be provided with filters suitable to prevent pumping of fine grained soil particles. Rainwater or snowmelt should be directed away from exposed soil bearing surfaces.

Dewatering and discharge of dewatering effluent should be performed in accordance with all applicable local, state and federal regulations. Dewatering discharge should be recharged on site if possible. However, due to the size of the site and the relatively shallow depth to water, we anticipate that on-site recharge will not be feasible and that dewatering discharge will need to be directed to the local storm drain system. Sedimentation tanks and other treatment methods may be required for legal disposal of the effluent into the storm drain system.

Pile Load Testing Program

We anticipate that the PCC piles will be driven to practicable refusal in/on bedrock. Therefore, we believe that dynamic testing can be used and implemented in lieu of a static load test. A minimum of 3 to 5 percent of the total number of piles should be pre-selected for monitoring during installation with a pile driving analyzer (PDA) to evaluate hammer system efficiencies, driving stresses in the pile and pile capacities. The selected piles should be allowed to stand a minimum of 24 hours after completion of initial driving and should then be re-driven (restrike) while being monitored with the PDA to assess the set-up/relaxation characteristics of the rock. If the results of a PDA/CAPWAP analysis show that the minimum safety factor of 2.25 has been achieved using the driving criteria established by the WEAP analysis, then this driving criteria would be used to install the remainder of the production piles without the use of PDA, and would be considered sufficient "evidence" that the piles have developed the required design capacity. The indicator piles should be driven at production pile locations prior to the production driving in order to assist with establishing pile lengths. Additional construction considerations relative to pile installation, including driving criteria will be included in the pile specification.

Pile Installation

We anticipate that the site will be initially cut down and that the majority of the piles will be installed from a prepared working surface approximately 15 to 20 ft BGS (approximately El. -2). The contractor will be responsible for stabilizing the soil subgrade and establishing a adequate working surface for pile installation (e.g., placing a lift of crushed stone).

It is possible that obstructions (i.e., boulders) could be encountered in the naturally deposited glacial till soils during pile installation.

Full-time monitoring of pile installation should be performed by a geotechnical engineer in accordance with the requirements of the IBC code.

As previously stated, pile driving is a noise and vibration inducing activity. We recommend that seismographs be used to monitor vibrations and noise levels during pile driving and other vibration inducing activities (e.g., controlled drilling and blasting, hoe-ramming etc.).

Preparation and Protection of Bearing Surfaces

In general, exposed subgrades should be examined in the field by a qualified geotechnical engineer to verify foundation bearing conditions. It may be necessary to over-excavate weak, disturbed or otherwise unacceptable soils (topsoil, debris and organic) using crushed stone, compacted granular fill (CGF) or concrete mudmats.

<u>Footings</u>

Footings supporting the townhouse wing and a portion of the residential/retail space will bear on bedrock (see Figure 2, "Foundation Design Zone A"). After final excavation to competent bedrock, the exposed rock surface should be cleaned to remove any loose fragments or any exposed weathered zones before concrete is poured for the footings. The bedrock surface should be observed in the field by a qualified geotechnical engineer to confirm the assumed foundation bearing conditions. Once the bearing surface has been properly cleaned and inspected, the foundation can be constructed.

The proposed foundation subgrade surfaces within the limits of residential/retail space should be relatively level. If the rock surface exposed within the limits of a foundation is steeper than 6 ft horizontal to 1 ft vertical (6H:1V), the rock surface should be benched or tapered to create a level bearing surface. Lean concrete may be used to backfill locally depressed areas if necessary. The lean concrete should have a minimum compressive strength of 2,000 psi.

Pile Caps/Grade Beams/Lowest Level Basement Slab

Assuming that the basement area will be excavated to approximately El. -2, we anticipate that in-situ fill and/or glaciomarine soils (clay or sand) will be present at subgrade level within about half the building footprint, specifically beneath the northern and western portions of the basement area (rock in the other areas). In general, we recommend that excavations be conducted in a manner that minimizes disturbance to the subgrade soils when excavating to bearing level. The following guidelines are recommended to protect subgrade soils:

- Make final excavations into natural bearing soils using smooth-bladed equipment to limit disturbance.
- Prevent water from accumulating on soil surfaces to reduce the possibility of soil disturbance. All filling and concreting of slabs, pile caps/grade beams and footings should be performed in-the-dry. Subgrades that become disturbed due to water infiltration should be re-excavated and stabilized. Subgrade stabilization is the responsibility of the Contractor; stabilization methods could include placement of a 2 to 3-in. thick lean concrete mud-mat or layer of crushed stone over approved subgrade.
- Do not permit temporary drainage trenches or other dewatering facilities to extend below the bearing level near pile caps/grade beams.
- Granular subgrade surfaces should be proofrolled with a self-propelled static roller or heavy hand-guided vibratory compactor until firm prior to placement of fill. To minimize disturbance, we recommend that glaciomarine soils (particularly clay) exposed at subgrade level not be proofrolled.
- To the extent possible, limit equipment traffic across the exposed soil bearing surfaces.

Filling and Backfilling

Placement of compaction of fills should not be conducted when air temperatures are low enough (approximately 30 degrees F., or below) to cause freezing of the moisture in the fill during or before placement. Fill materials should not be placed on snow, ice or uncompacted frozen soil. Compacted fill should not be placed on frozen soil. No fill should be allowed to freeze prior to compaction. At the end of each day's operations, the last lift of fill, after compaction, should be rolled by a smooth-wheeled roller to eliminate ridges of uncompacted soil.

Compacted Granular Fill

Compacted granular fill (CGF) should be placed after overexcavation down to top of rock beneath the townhouse wing, adjacent to pile caps and grade beams, beneath sidewalks and adjacent to foundation walls. We recommend this material consist of mineral bank-run sand and gravel, free of organic material, snow, ice, or other unsuitable materials. Additionally, the material should conform to the following gradation requirements:

Sieve Size	Percent Finer by Weight
6 in.[1]	100
No. 4	30-80
No. 40	10-50
No. 200	0-8

[1] – Cobbles or boulders having a size exceeding 2/3 of the loose lift Thickness should be removed prior to compaction.

Other materials could be acceptable for use as CGF. We recommend this be evaluated by the geotechnical engineer on a case-by-case basis.

In open areas, CGF should be placed in lift thicknesses not exceeding 12 in. loose measure (prior to compaction) and compacted using self-propelled vibratory rollers such as a BoMag BW-60S. In confined areas, CGF should be placed in lift thickness not exceeding 9 in. and compacted using as a large vibratory plate compactor or equivalent. A minimum of four systematic passes of the compaction equipment should be used to compact each lift.

Common Fill

Common fill should consist of mineral sandy soil, free from organic matter, plastic, metal, wood, ice, snow or other deleterious material and should have the characteristic that it can be readily placed and compacted. Common fill imported to the site should conform to the following gradation requirements:

Sieve Size	Percent Finer by Weight
No. 40	0-80
No. 200	0-30

The largest particle size for common fill should not exceed 6 in. Silty common fill soils may require moisture control during placement and compaction. Common fill should be placed in maximum 12 in. thick loose lifts using compaction equipment as described above for CGF.

Reuse of Excavated On-Site Soils for Backfill

In-Situ Fill Material

Based on visual inspection of the fill samples and the results of laboratory grain size tests (see Appendix D), we believe that the in-situ fill soils are suitable for reuse as common fill in landscaped areas and could be reused as CGF adjacent to pile caps and grade beams, beneath sidewalks or adjacent to foundation walls. The in-situ fill soils could also be used as CGF to raise grades beneath the townhouse wing (outside the ZOI of the footings). Confirmation on the suitability of the excavated fill soils for reuse as common fill or CGF should be made in the field based on the following information: 1.) visual inspection of the soils once they are excavated and stockpiled; and 2.) the results of additional laboratory testing on the stockpiled soil (grain size and compaction). In-situ fill soils will likely need to be processed using a mechanical screen to eliminate oversize material, organic material, refuse and debris. This material should be able to achieve the minimum compaction requirements outlined below. It is possible that some of the excavated in-situ fill material may not be acceptable for reuse as common fill.

Glaciomarine Soils

Glaciomarine sand and clay soils excavated during construction are not considered suitable for reuse as CGF. These materials may be used as common fill in landscaped areas if they can be placed and compacted adequately as stated herein.

Bedrock

Rock generated from the excavation for the basement level could be reused as crushed stone or CGF adjacent to pile caps and grade beams, beneath slabs or adjacent to foundation walls. However, the rock will need to be processed to meet gradation requirements for use as fill materials as stated herein.

Compaction Requirements

A summary of recommended compaction requirements is as follows:

Location	Minimum Compaction Requirements
Adjacent to pile caps & grade beams, beneath floor slab and adjacent to foundation walls	95 percent
Beneath sidewalks, parking areas and roadways	92 percent up to 3 ft below finished grade, 95 percent in the upper 3 ft
Landscaped areas	90 percent nominal compaction

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

Preparation of Contract Documents and Submittal Reviews

The contract drawings and specifications should be written so that the requirements of the documents are consistent with the design intent of the geotechnical recommendations outlined herein. Haley & Aldrich is planning on working with the design team to prepare the specifications and contract drawings related to the following topics:

- Demolition
- Earthwork
- Construction Dewatering
- Temporary Lateral Support of Excavation
- Pile Installation and Testing
- Foundation Drainage System Plan and Details

The contract specifications will require the Contractor and the Contractor's engineer to perform analyses and submit results to the designers for review. The design team should be allowed to review the geotechnical-related submittals to ensure that the Contractor's analyses/submittals are in accordance with the intent of the design. This will enable us to observe compliance with the design concepts, assumptions and specifications, and to facilitate design changes in the event that subsurface conditions differ from those anticipated prior to the start of construction.

Construction Monitoring

The foundation and earthwork recommendations contained herein are based on the known and predictable behavior of a properly engineered and constructed foundation. Monitoring of the foundation construction is required to enable the geotechnical engineer to keep in contact with procedures and techniques used in construction, and to comply with Section 1808.2.10 of the IBC Code. Therefore, it is recommended that an individual representing the Owner (Owner's Rep.), qualified by geotechnical training and experience be present at the site to provide full-time monitoring during the earthwork and foundation construction activities listed below.

- Installation of the excavation support system.
- Excavation to subgrade levels and subgrade inspection prior to construction of pile caps, grade beams and footings.
- Placement and compaction testing of crushed stone, CGF and site fills.
- Dynamic testing of the indicator piles and review of the PDA results.
- Installation of the production piles.

- Installation of the foundation drainage system.
- Backfilling adjacent to foundation walls and beneath the building floor slabs.
- Inspection of the slab subgrade prior to construction of floor slab.

LIMITATIONS OF RECOMMENDATIONS

This report is prepared for the exclusive use of Riverwalk, LLC relative to THE WATERMARK development in Portland, Maine. There are no intended beneficiaries other than Riverwalk, LLC., Haley & Aldrich shall owe no duty whatsoever to any other person or entity on account of the Agreement or the report. Use of this report by any person or entity other than Riverwalk, LLC for any purpose whatsoever is expressly forbidden unless such other person or entity obtains written authorization from Riverwalk LLC and from Haley & Aldrich. Use of this report by such other person or entity without the written authorization of Riverwalk LLC and Haley & Aldrich shall be at such other person's or entities sole risk, and shall be without legal exposure or liability to Haley & Aldrich.

Use of this Report by any person or entity, including by Riverwalk, LLC, for a purpose other than relative to THE WATERMARK project in Portland, Maine is expressly prohibited unless such person or entity obtains written authorization from Haley & Aldrich indicating that the Report is adequate for such other use. Use of this Report by any other person or entity for such other purpose without written authorization by Haley & Aldrich shall be at such person's or entities sole risk, and shall be without legal exposure or liability to Haley & Aldrich.
The analyses and recommendations are based, in part, upon the data obtained from the referenced subsurface explorations. The nature and extent of variations between explorations may not become evident until construction. If variations then appear, it may be necessary to reevaluate the recommendations of this report.

The planned construction will be supported on or in the soil at the site and below grade structures may be close to or penetrate the design groundwater level for the project. Recommendations for foundation and/or floor drainage, moisture protection, and/or waterproofing have been included herein, when appropriate. These recommendations address the conventional geotechnical engineering-related aspects of design and construction and are not intended to provide an environment that would prohibit infestation of mold or other biological pollutants. Our work scope did not include the development of criteria or procedures to minimize the risk of mold or other biological pollutant infestations in or near any structure.

We appreciate the opportunity to provide geotechnical engineering consulting services on this project. Please do not hesitate to call if you have any questions or comments.
Sincerely yours,

HALEY & ALDRICH, INC.

Bryan C. Steinert
Engineer

Wayne A. Chadbourne, P.E.
Vice President

Attachments:

Table I:	Subsurface Explorations (2 pages)
Table II:	In-Situ Vane Shear Test Results
Table III:	Proposed Foundation Support (2 pages)
Figure 1:	Project Locus
Figure 2:	Site and Subsurface Exploration Location Plan
Figure 3:	Subsurface Profile A-A
Figure 4:	Subsurface Profile B-B
Figure 5:	Subsurface Profile C-C
Figure 6:	Legend and Notes
Appendix A:	Logs of 1975 PWD India St. Pump Station Test Borings
Appendix B:	Logs of Recent Test Explorations
Appendix C:	Observation Well Installation & Groundwater Monitoring Reports
Appendix D:	Laboratory Test Results
Appendix E:	Historic Sanborn Maps
Appendix F:	PWD Pump Station Record Drawings

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STATE OF MAINE
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REFERENCES

1. Report entitled, "Proposed India Street Pump Station - Wastewater Facilities Contract No. 5 - Portland, Maine," prepared by Haley & Aldrich, Inc., dated 9 October 1975.

2. "Report on Evaluation of Wall Movement, BIW Containment Structure, Portland, Maine," prepared by Haley & Aldrich, Inc., dated 31 March 1983.

3. "Geotechnical Data Report on Proposed Ocean Gateway Project, Commercial Street, Portland, Maine," prepared by Haley & Aldrich, Inc., dated 12 September 2003.

4. "Report on Subsurface Explorations and Foundation Design Recommendations, Eastern Waterfront Development, Proposed Parking Garage and Office Building, Portland, Maine," prepared by Haley & Aldrich, Inc., dated 8 November 2005.

5. "Foundation Drainage System, Proposed Longfellow Residences and Retail, Longfellow at Ocean Gateway, Portland, Maine," memorandum prepared by Haley & Aldrich, Inc., dated 12 April 2006.

TABLE I
Subsurface Explorations
THE WATERMARK
Portland, Maine

Test Boring No.[1]	Estimated Ground Surface Elevation[2,3]	Thickness of Strata (ft)									Approx. Elevation of Top of Bedrock[3]	Elevation of Bottom of Exploration[3]
		Bituminous Concrete / Concrete	Topsoil	Fill	Glaciomarine Deposit (clay)	Glaciomarine Deposit (sand)	Organic Deposit	Glaciomarine Deposit (clay)	Glaciomarine Deposit (sand)	Glacial Till		
HA05-11 (OW)	16.3	NE	NE	10.0	NE	NE	NE	24.3	NE	NE	-18.0	-24.1
HA05-12	15.3	NE	NE	15.0	NE	NE	NE	44.0	6.6	8.5	-58.8	-63.8
HA05-13	16.5	NE	NE	19.0	NE	5.0	4.0	23.0	4.0	>17	-	-55.5
HA05-14 (OW)	14.5	NE	0.4	14.6	2.4	8.4	6.2	29.1	3.7	7.0	-57.3	-62.5
HA05-15	17.0	NE	NE	8.5	NE	NE	NE	26.0	NE	0.9	-18.4	-22.9
HA05-16	18.0	NE	NE	11.6	NE	NE	NE	NE	NE	NE	6.4	-1.5
HA05-17 (OW)	17.8	NE	NE	7.3	NE	NE	NE	NE	NE	NE	10.5	1.6
HA05-18	16.5	NE	NE	5.9	NE	NE	NE	NE	NE	NE	10.6	0.9
HA05-19A	15.4	0.1	NE	7.5	NE	NE	NE	NE	NE	NE	7.8	7.0
HA05-19B	15.4	0.1	NE	6.7	NE	NE	NE	NE	NE	NE	8.6	8.4
HA05-19C	15.4	0.1	NE	4.9	NE	NE	NE	NE	NE	NE	10.4	9.9
HA05-19D	15.4	0.1	NE	6.4	NE	NE	NE	NE	NE	NE	8.9	-18.5
HA05-20	17.0	0.3	NE	17.0	NE	4.3	4.2	0.6	NE	NE	-9.4	-16.2
HA05-21	16.5	NE	NE	15.5	NE	NE	NE	NE	NE	>2.1	-	-1.1
HA05-21A	16.5	NE	NE	16.4	NE	NE	NE	NE	NE	NE	0.1	-3.1
HA05-22	15.0	0.4	NE	9.9	NE	NE	NE	NE	NE	NE	NE	4.7
HA05-22A	15.0	0.4	NE	9.3	NE	NE	NE	NE	NE	NE	5.3	1.8
HA05-23	15.0	0.2	NE	>8.5	-	-	-	-	-	-	-	6.3
HA05-23A	15.0	0.2	NE	14.8	NE	NE	NE	NE	NE	>1.0	-	-1.0
HA07-101[5]	15.5	-	-	-	-	-	-	-	-	-	5.5	-3.3
HA07-102A	17.0	NE	NE	>3.0	-	-	-	-	-	-	-	14.0
HA07-102B	17.0	NE	NE	>5.0	-	-	-	-	-	-	-	12.0
HA07-102C	17.0	NE	NE	7.5	NE	NE	NE	NE	NE	NE	9.5	9.5
HA07-103	17.8	NE	NE	7.8	NE	NE	NE	NE	NE	NE	10.0	10.0
HA07-104[5]	17.8	NE	NE	>5.0	-	-	-	-	-	-	3.3	3.3
HA07-105	17.5	NE	NE	13.0	NE	NE	NE	NE	3.9	NE	0.6	0.2
HA07-105A[5]	17.3	-	-	-	-	-	-	-	-	-	-12.7	-13.2
HA07-106	16.5	NE	NE	13.5	NE	NE	NE	7.4	0.3	NE	-4.7	-4.7
HA07-107[6]	16.5	NE	NE	20.0	NE	NE	>0.4	-	-	-	-19.2	-19.2
HA07-108A	16.8	NE	NE	>11.0	-	-	-	-	-	-	-	5.8
HA07-108B	16.8	NE	NE	>10.6	-	-	-	-	-	-	-	6.2
HA07-108C	16.8	NE	NE	18.5	NE	2.4	>1.1	-	-	-	-38.2	-38.2
HA07-109[6]	16.0	NE	NE	18.5	NE	>3.5	-	-	-	-	-30.0	-30.0
HA07-110[6]	15.0	NE	NE	19.7	NE	>0.3	-	-	-	-	-16.5	-16.5
HA07-111[6]	16.0	NE	NE	13.5	>3.5	-	-	-	-	-	-51.9	-51.9
HA07-112[6]	15.8	NE	NE	18.5	NE	2.5	NE	>6.0	-	-	-64.0	-64.0
HA07-113[6]	14.0	NE	NE	21.0	NE	3.0	NE	>3.0	-	-	-58.9	-58.9

Notes:
1 Test boring locations are shown on Figure 2, Site and Subsurface Exploration Location Plan.
2 Ground surface elevations at test boring locations are approximate and were estimated by interpolating between elevation contour data provided by Woodard & Curran
3 Elevations are in feet and reference Portland City Datum.
4 "NE" indicates stratum was not encountered in test boring.
5 Test borings HA07-101, HA07-104 and HA07-105A were advanced with solid stem augers; no soil samples were collected.
6 Elevation of top of bedrock is approximate and was determined using rod probe drilling techniques.

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TABLE I
Subsurface Explorations
THE WATERMARK
Portland, Maine

Test Boring No.[1]	Estimated Ground Surface Elevation[2,3]	Thickness of Strata (ft)									Approx. Elevation of Top of Bedrock[3]	Elevation of Bottom of Exploration[3]
		Bituminous Concrete / Concrete	Topsoil	Fill	Glaciomarine Deposit (clay)	Glaciomarine Deposit (sand)	Organic Deposit	Glaciomarine Deposit (clay)	Glaciomarine Deposit (sand)	Glacial Till		
B5-19	18.0	NE	NE	4.0	-	-	NE	14.0	6.0	2.3	-8.3	-18.3
B5-20	18.3	NE	NE	5.5	-	-	NE	25.0	7.0	1.0	-20.2	-29.6
B5-21A	17.6	NE	NE	9.0	-	-	NE	NE	NE	NE	8.6	-6.4
B5-22A	16.9	NE	NE	5.5	-	-	NE	5.0	3.5	2.5	0.4	0.4
B5-23	18.3	NE	NE	4.0	-	-	NE	12.0	3.9	NE	-1.6	-1.6

1 Test boring locations are shown on Figure 2, Site and Subsurface Exploration Location Plan.
2 Ground surface elevations at test boring locations are approximate and were estimated by interpolating between elevation contour data provided by Woodard & Curran
3 Elevations are in feet and reference Portland City Datum.
4 "NE" indicates stratum was not encountered in test boring.
5 Test borings HA07-101, HA07-104 and HA07-105A were advanced with solid stem augers; no soil samples were collected.
6 Elevation of top of bedrock is approximate and was determined using rod probe drilling techniques.

TABLE II
In-Situ Vane Shear Test Results
THE WATERMARK
Portland, Maine

Test Boring No.[1]	Estimated Ground Surface Elevation[2,3]	Vane Size (in. x in.)	Test No.	Depth below ground surface (ft)	Approx. Elevation[3] (ft)	V_{max} [4] (ft-lbs)	$V_{remolded}$ [4] (ft-lbs)	S_u [5] (psf)	$S_{u(remolded)}$ [5] (psf)
HA05-12	15.3	2x7	V_1	30.0 - 30.6	-14.7 - -15.3	27	1	1,000	40
			V_2	40.0 - 40.6	-24.7 - -25.3	120	0	>1,860	0
			V_3	50.0 - 50.6	-34.7 - -35.3	120	0	>1,860	0
HA05-14(OW)	14.5	2x7	V_1	35.3 - 36.0	-20.8 - -21.5	89	30	3,302	1,110
			V_2	45.3 - 46.0	-30.8 - -31.5	22	5	820	190
HA05-15	17.0	3x5	V_1	20.0 - 20.6	-3.0 - -3.6	10	3	370	110
			V_2	30.0 - 30.6	-13.0 - -13.6	23	1	850	40

Notes:
1 Test boring locations are shown on Figure 2, Site and Subsurface Exploration Location Plan.
2 Ground surface elevations at test boring locations are approximate and were estimated by interpolating between elevation contour data provided by Woodard & Curran.
3 Elevations are in feet and reference Portland City Datum.
4 Vane test numbers are shown on the test boring reports presented in Appendices A and B, respectively.
5 V_{max} and $V_{remolded}$ represent direct peak and remolded vane torque values, respectively.
6 S_u and $S_{u(remolded)}$ represent corrected undrained peak and residual undrained shear strengths, respectively, rounded to the nearest 10 psf.
7 ft-lbs = foot-pounds of torque, psf = pounds per square foot.

TABLE III
Proposed Foundation Support
THE WATERMARK
Portland, Maine

Column Location	Foundation Support	Notes	Estimated Depth of Soil Removal (ft) [5]	Estimated Depth of Rock Removal (ft) [6]
M-1	footing	bearing on bedrock at El. 9	8	0
G-2	footing	bearing on bedrock at El. 8	10	0
K-2	footing	bearing on bedrock at El. 9	9	0
N-2	footing	bearing on bedrock at El. 9	9	0
G-3	footing	bearing on bedrock at El. 9	9	0
H-3	footing	bearing on bedrock at El. 10	8	0
K-3	footing	bearing on bedrock at El. 10	8	0
M-3	footing	bearing on bedrock at El. 9	8	0
H-4	footing	bearing on bedrock at El. 7	11	0
K-4	footing	bearing on bedrock at El. 9	9	0
M-4	footing	bearing on bedrock at El. 9	8	0
H-5	footing	bearing on bedrock at El. 3	14	0
K-5	footing	bearing on bedrock at El. 5	12	0
M-5	footing	bearing on bedrock at El. 9	7	0
B-6	pile	30 ft long pile, top at El. 0	-	-
C-6	pile	20 ft long pile, top at El. 0	-	-
D-6	pile	15 ft long pile, top at El. 0	-	-
F-6	footing	bearing on bedrock at El. -1.5	-	2
H-6	footing	bearing on bedrock at El. -1.5	-	5
K-6	footing	bearing on bedrock at El. -1.5	-	7
L-6	footing	bearing on bedrock at El. -1.5	-	9
M-6	footing	bearing on bedrock at El. -1.5	-	12
B-7	pile	30 ft long pile, top at El. 0	-	-
C-7	pile	20 ft long pile, top at El. 0	-	-
D-7	pile	15 ft long pile, top at El. 0	-	-
F-7	footing	bearing on bedrock at El. -1.5	-	2
J-7	footing	bearing on bedrock at El. -1.5	-	5
L-7	footing	bearing on bedrock at El. -1.5	-	9
P-7	footing	bearing on bedrock at El. -1.5	-	8
S-7	footing	bearing on bedrock at El. -1.5	-	9
T-7	footing	bearing on bedrock at El. -1.5	-	11
B-8	pile	30 ft long pile, top at El. 0	-	-
C-8	pile	20 ft long pile, top at El. 0	-	-
D-8	pile	15 ft long pile, top at El. 0	-	-
F-8	footing	bearing on bedrock at El. -1.5	-	2
J-8	footing	bearing on bedrock at El. -1.5	-	8
L-8	footing	bearing on bedrock at El. -1.5	-	10
P-8	footing	bearing on bedrock at El. -1.5	-	7
S-8	footing	bearing on bedrock at El. -1.5	-	9
T-8	footing	bearing on bedrock at El. -1.5	-	12
A-9	pile	30 ft long pile, top at El. 0	-	-
C-9	pile	20 ft long pile, top at El. 0	-	-
D-9	pile	15 ft long pile, top at El. 0	-	-
F-9	footing	bearing on bedrock at El. -1.5	-	1
J-9	footing	bearing on bedrock at El. -1.5	-	8
L-9	footing	bearing on bedrock at El. -1.5	-	9
P-9	footing	bearing on bedrock at El. -1.5	-	7
S-9	footing	bearing on bedrock at El. -1.5	-	8
T-9	footing	bearing on bedrock at El. -1.5	-	9
A-10	pile	30 ft long pile, top at El. 0	-	-
C-10	pile	20 ft long pile, top at El. 0	-	-
D-10	pile	15 ft long pile, top at El. 0	-	-
F-10	footing	bearing on bedrock at El. -1.5	-	0
J-10	footing	bearing on bedrock at El. -1.5	-	3
L-10	footing	bearing on bedrock at El. -1.5	-	7
P-10	footing	bearing on bedrock at El. -1.5	-	7
S-10	footing	bearing on bedrock at El. -1.5	-	8
T-10	footing	bearing on bedrock at El. -1.5	-	8

Notes:
1. Column locations taken from plan entitled, "P-Basement-Parking Plan-Preliminary Geotechnical Layout Drawing," provided by The Architectural Team dated 24 January 2007.
2. Approximate footing bearing elevation in the basement area was provided by MacNamara-Salvia. Approximate pile lengths are based on linear interpolation between subsurface explorations and approximatley bottom of pile cap at El. -1.5 (also provided by MacNamara-Salvia). Subsurface conditions may vary at locations other than at specific exploration locations.
3. Elevations are in feet and reference Portland City Datum.
4. Foundation bearing elevations based on assumed basement FFE of El. 4.
5. Depth of soil removal values represent estimated amount of soil that will need to be removed to expose bedrock subgrade.
6. Depth of rock removal values represent estimated amount of rock that will need to be excavated/blasted to reach design footing bearing levels.

TABLE III
Proposed Foundation Support
THE WATERMARK
Portland, Maine

Column Location	Foundation Support	Notes	Estimated Depth of Soil Removal (ft) [5]	Estimated Depth of Rock Removal (ft) [6]
F-11	footing	bearing on bedrock at El. -5.0	3	0
J-11	footing	bearing on bedrock at El. -1.5	-	3
L-11	footing	bearing on bedrock at El. -1.5	-	3
P-12	footing	bearing on bedrock at El. -1.5	-	7
R-12	footing	bearing on bedrock at El. -1.5	-	7
D-13	pile	15 ft long pile, top at El. 0	-	-
E-13	pile	15 ft long pile, top at El. 0	-	-
A-14	pile	30 ft long pile, top at El. 0	-	-
C-14	pile	20 ft long pile, top at El. 0	-	-
B-16	pile	30 ft long pile, top at El. 0	-	-
C-16	pile	20 ft long pile, top at El. 0	-	-
D-17	pile	15 ft long pile, top at El. 0	-	-
E-17	pile	15 ft long pile, top at El. 0	-	-
P-18	footing	bearing on bedrock at El. -1.5	10	0
R-18	footing	bearing on bedrock at El. -1.5	10	0
J-19	pile	15 ft long pile, top at El. 0	-	-
L-19	pile	15 ft long pile, top at El. 0	-	-
A-20	pile	40 ft long pile, top at El. 0	-	-
C-20	pile	40 ft long pile, top at El. 0	-	-
D-20	pile	40 ft long pile, top at El. 0	-	-
F-20	pile	40 ft long pile, top at El. 0	-	-
J-20	pile	40 ft long pile, top at El. 0	-	-
L-20	pile	30 ft long pile, top at El. 0	-	-
P-20	pile	30 ft long pile, top at El. 0	-	-
S-20	pile	20 ft long pile, top at El. 0	-	-
U-20	pile	60 ft long pile, top at El. 0	-	-
A-21	pile	70 ft long pile, top at El. 0	-	-
C-21	pile	60 ft long pile, top at El. 0	-	-
D-21	pile	50 ft long pile, top at El. 0	-	-
F-21	pile	50 ft long pile, top at El. 0	-	-
J-21	pile	40 ft long pile, top at El. 0	-	-
L-21	pile	40 ft long pile, top at El. 0	-	-
P-21	pile	30 ft long pile, top at El. 0	-	-
S-21	pile	30 ft long pile, top at El. 0	-	-
U-21	pile	70 ft long pile, top at El. 0	-	-
A-22	pile	70 ft long pile, top at El. 0	-	-
C-22	pile	60 ft long pile, top at El. 0	-	-
D-22	pile	60 ft long pile, top at El. 0	-	-
F-22	pile	70 ft long pile, top at El. 0	-	-
J-22	pile	70 ft long pile, top at El. 0	-	-
L-22	pile	70 ft long pile, top at El. 0	-	-
P-22	pile	65 ft long pile, top at El. 0	-	-
S-22	pile	65 ft long pile, top at El. 0	-	-
U-22	pile	65 ft long pile, top at El. 0	-	-
B-23	pile	60 ft long pile, top at El. 0	-	-
C-23	pile	60 ft long pile, top at El. 0	-	-
D-23	pile	60 ft long pile, top at El. 0	-	-
F-23	pile	65 ft long pile, top at El. 0	-	-
J-23	pile	65 ft long pile, top at El. 0	-	-
L-23	pile	65 ft long pile, top at El. 0	-	-
P-23	pile	65 ft long pile, top at El. 0	-	-
S-23	pile	65 ft long pile, top at El. 0	-	-

Notes:

1. Column locations taken from plan entitled, "P-Basement-Parking Plan-Preliminary Geotechnical Layout Drawing," provided by The Architectural Team dated 24 January 2007.
2. Approximate footing bearing elevation in the basement area was provided by MacNamara-Salvia. Approximate pile lengths are based on linear interpolation between subsurface explorations and bottom of pile cap at El. -1.5 (also provided by MacNamara-Salvia Subsurface conditions may vary at locations other than at specific exploration locations.
3. Elevations are in feet and reference Portland City Datum.
4. Foundation bearing elevations based on assumed basement FFE of El. 4.
5. Depth of soil removal values represent estimated amount of soil that will need to be removed to expose bedrock subgrade.
6. Depth of rock removal represent estimated amount of rock that will need to be excavated/blasted to reach design footing bearing lev

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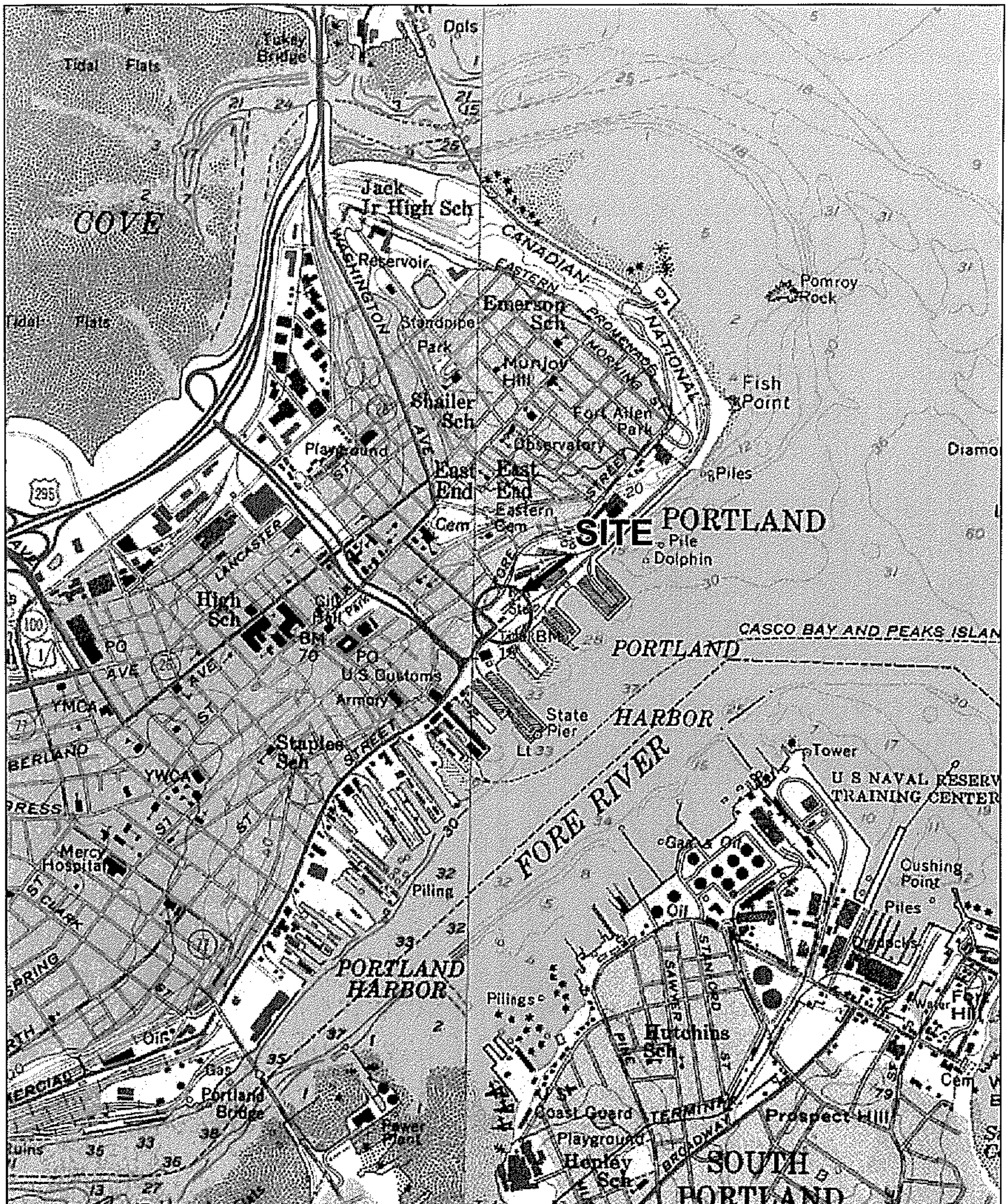
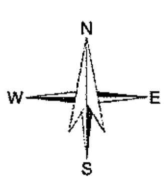


SITE COORDINATES:43°39'35"N 70°14'53"W



U.S.G.S. QUADRANGLE: PORTLAND EAST, ME

HALEY & ALDRICH

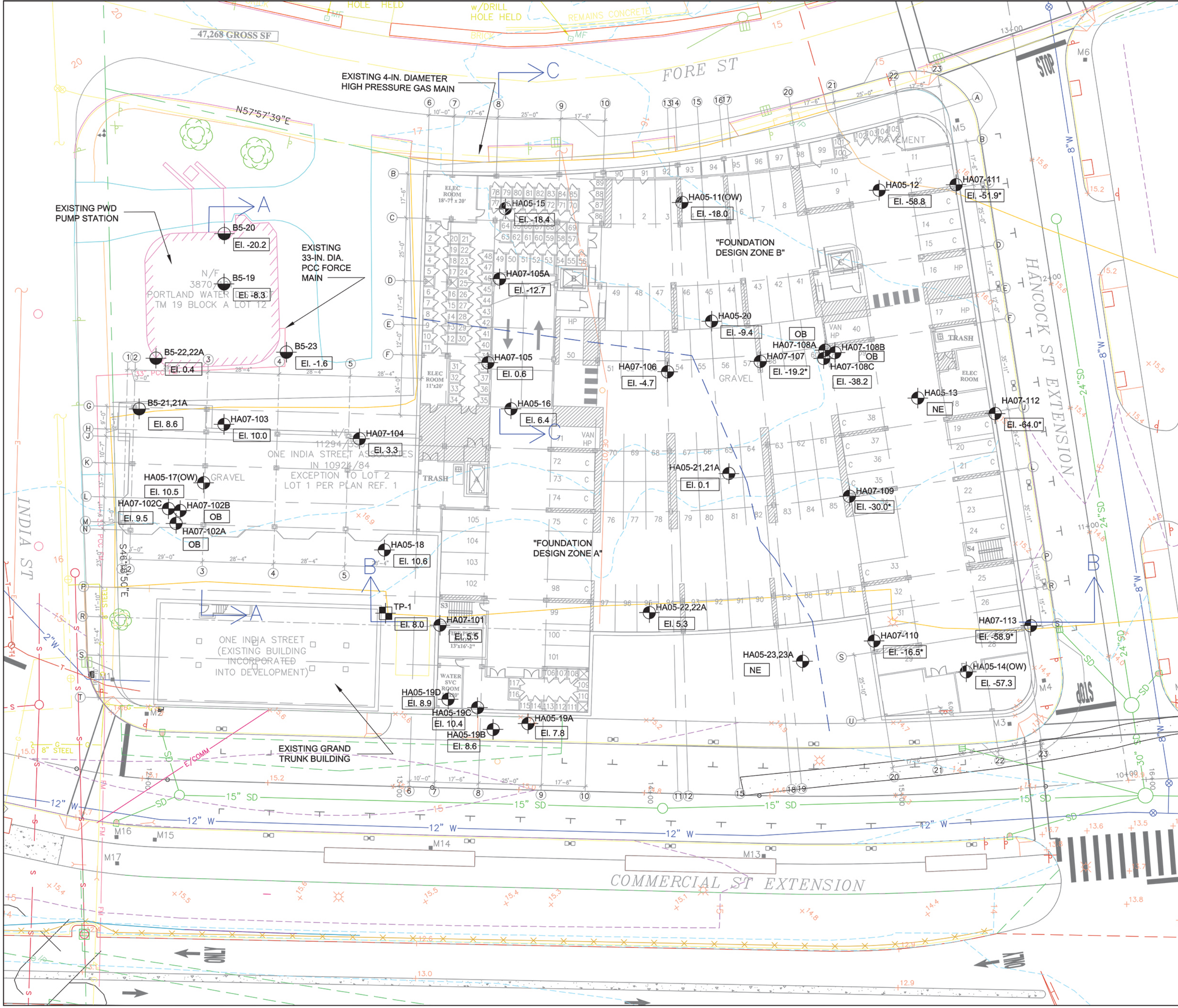
THE WATERMARK
PORTLAND, MAINE

PROJECT LOCUS

SCALE: 1:24,000
APRIL 2007

FIGURE 1

30322-000 1.PDF



NOTES:

1. EXISTING SITE CONDITIONS, CONTOURS OF EXISTING GROUND SURFACE ELEVATIONS AND LOCATION AND ORIENTATION OF EXISTING SITE FEATURES (EG. BUILDINGS, UTILITIES ETC) ARE TAKEN FROM THE ELECTRONIC AUTOCAD FILE "HA-203555-X00C.dwg", PROVIDED BY WOODARD & CURRAN, DATED 30 JANUARY 2007.
2. PROPOSED SITE CONDITIONS AND THE LOCATION AND ORIENTATION OF PROPOSED SITE FEATURES ARE TAKEN FROM THE ELECTRONIC AUTOCAD FILE "P - BASEMENT - PARKING PLAN - PRELIMINARY GEOTECHNICAL LAYOUT DRAWING.dwg", PROVIDED BY THE ARCHITECTURAL TEAM, DATED 24 JANUARY 2007.
3. LOCATIONS OF THE 1975 AND 2005 SERIES OF TEST BORINGS AND THE TEST PIT EXCAVATED IN 2007 ARE APPROXIMATE AND WERE DETERMINED IN THE FIELD BY TAPING/PACING DISTANCES FROM EXISTING SITE FEATURES. LOCATIONS OF THE 2007 SERIES OF TEST BORINGS WERE DETERMINED IN THE FIELD USING GPS SURVEY EQUIPMENT.
4. ELEVATIONS ARE IN FEET AND REFERENCE PORTLAND CITY DATUM.
5. SUBSURFACE EXPLORATIONS WERE MONITORED IN THE FIELD BY HALEY & ALDRICH PERSONNEL.
6. REFER TO APPENDIX A FOR LOGS OF 1975 SERIES TEST BORINGS, APPENDIX B FOR LOGS OF 2005 AND 2007 SERIES TEST BORINGS AND APPENDIX C FOR OBSERVATION WELL INSTALLATION AND GROUNDWATER MONITORING REPORTS.
7. REFER TO TEXT OF REPORT AND TABLE III FOR DISCUSSION REGARDING "FOUNDATION DESIGN ZONES A AND B".

LEGEND:

HA07-101 EL. 5.5	DESIGNATION, LOCATION AND APPROXIMATE ELEVATION OF TOP OF BEDROCK OF TEST BORING DRILLED BY MAINE TEST BORING, INC. OF BREWER, MAINE IN MARCH 2007
HA05-14 EL. -57.3	DESIGNATION, APPROXIMATE LOCATION AND APPROXIMATE ELEVATION OF TOP BEDROCK OF TEST BORING DRILLED BY MAINE TEST BORING, INC. OF BREWER, MAINE IN OCTOBER 2005
B5-19 EL. -8.3	DESIGNATION, APPROXIMATE LOCATION AND APPROXIMATE ELEVATION OF TOP OF BEDROCK OF TEST BORING DRILLED BY NORTHERN NEW ENGLAND TEST BORING COMPANY OF PORTLAND, MAINE IN MAY 1975
TP-1 EL. 8.0	DESIGNATION, APPROXIMATE LOCATION AND APPROXIMATE ELEVATION OF TOP OF BEDROCK OF TEST PIT EXCAVATED BY MC HALL DEMOLITION OF SOUTH PORTLAND, MAINE IN APRIL 2007
17	ELEVATION CONTOUR OF EXISTING GROUND SURFACE
(OW)	DENOTES OBSERVATION WELL INSTALLED IN COMPLETED BOREHOLE
NE	DENOTES BEDROCK NOT ENCOUNTERED IN BOREHOLE
OB	DENOTES OBSTRUCTION ENCOUNTERED IN BOREHOLE; BOREHOLE ABANDONED
A A	DESIGNATION, LOCATION AND ORIENTATION OF SUBSURFACE PROFILE
*	DENOTES ELEVATION OF TOP OF BEDROCK IS APPROXIMATE AND WAS DETERMINED USING ROD PROBE DRILLING TECHNIQUES
	ANTICIPATED TRANSITION BETWEEN "FOUNDATION DESIGN ZONE A" AND "FOUNDATION DESIGN ZONE B"

HALEY & ALDRICH

THE WATERMARK
PORTLAND, MAINE

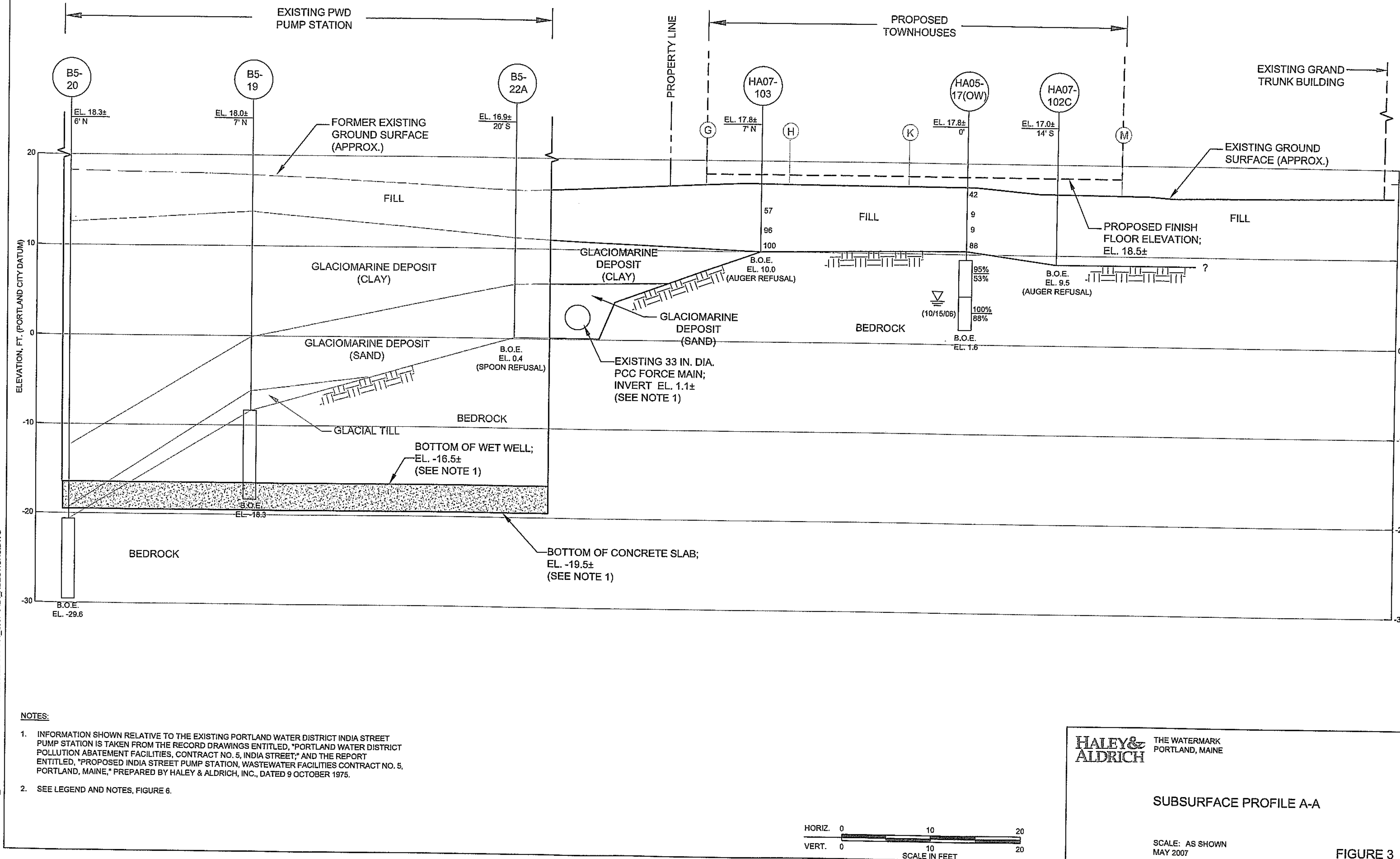
SITE AND SUBSURFACE EXPLORATION LOCATION PLAN

SCALE: AS SHOWN
MAY 2007

FIGURE 2

S:\30322\2007_0124 - CURRENT RES. FIG 2\HA-203555-X00C.DWG

S:\30322\2007_0124 - CURRENT RES. FIG 2\30322-000-001_2007-0424_XSECTIONS.DWG



NOTES:

1. INFORMATION SHOWN RELATIVE TO THE EXISTING PORTLAND WATER DISTRICT INDIA STREET PUMP STATION IS TAKEN FROM THE RECORD DRAWINGS ENTITLED, "PORTLAND WATER DISTRICT POLLUTION ABATEMENT FACILITIES, CONTRACT NO. 5, INDIA STREET;" AND THE REPORT ENTITLED, "PROPOSED INDIA STREET PUMP STATION, WASTEWATER FACILITIES CONTRACT NO. 5, PORTLAND, MAINE," PREPARED BY HALEY & ALDRICH, INC., DATED 9 OCTOBER 1975.
2. SEE LEGEND AND NOTES, FIGURE 6.

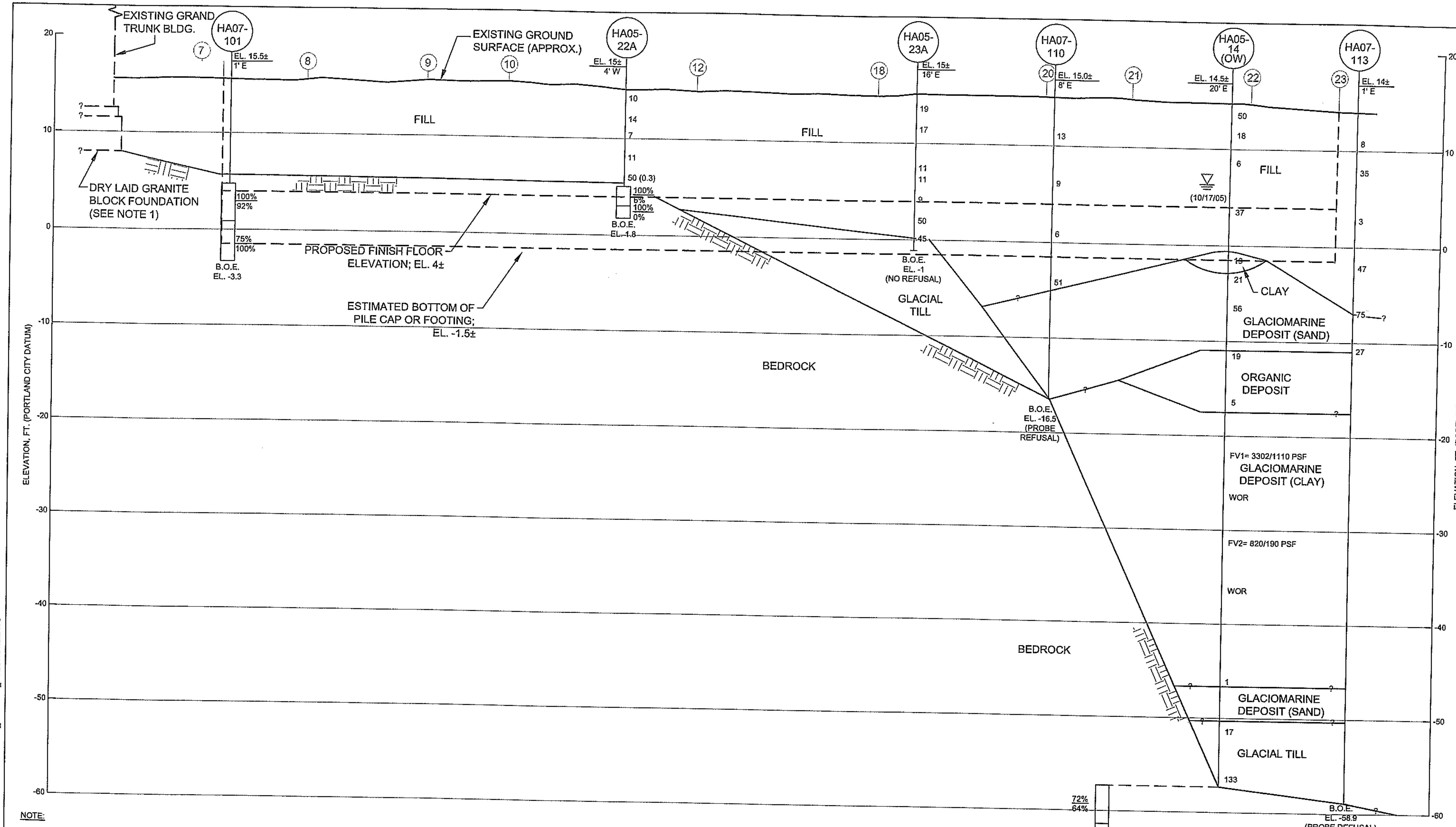
HALEY & ALDRICH

THE WATERMARK
PORTLAND, MAINE

SUBSURFACE PROFILE A-A

SCALE: AS SHOWN
MAY 2007

FIGURE 3



NOTE:

1. INFORMATION SHOWN RELATIVE TO THE EXISTING GRAND TRUNK BUILDING FOUNDATION IS TAKEN FROM AN EXPLORATORY TEST PIT EXCAVATED IN APRIL 2007. REFER TO APPENDIX B FOR A DETAILED SKETCH AND PHOTOGRAPHS.
2. SEE LEGEND AND NOTES, FIGURE 6.

HORIZ. 0 20 40
VERT. 0 10 20
SCALE IN FEET

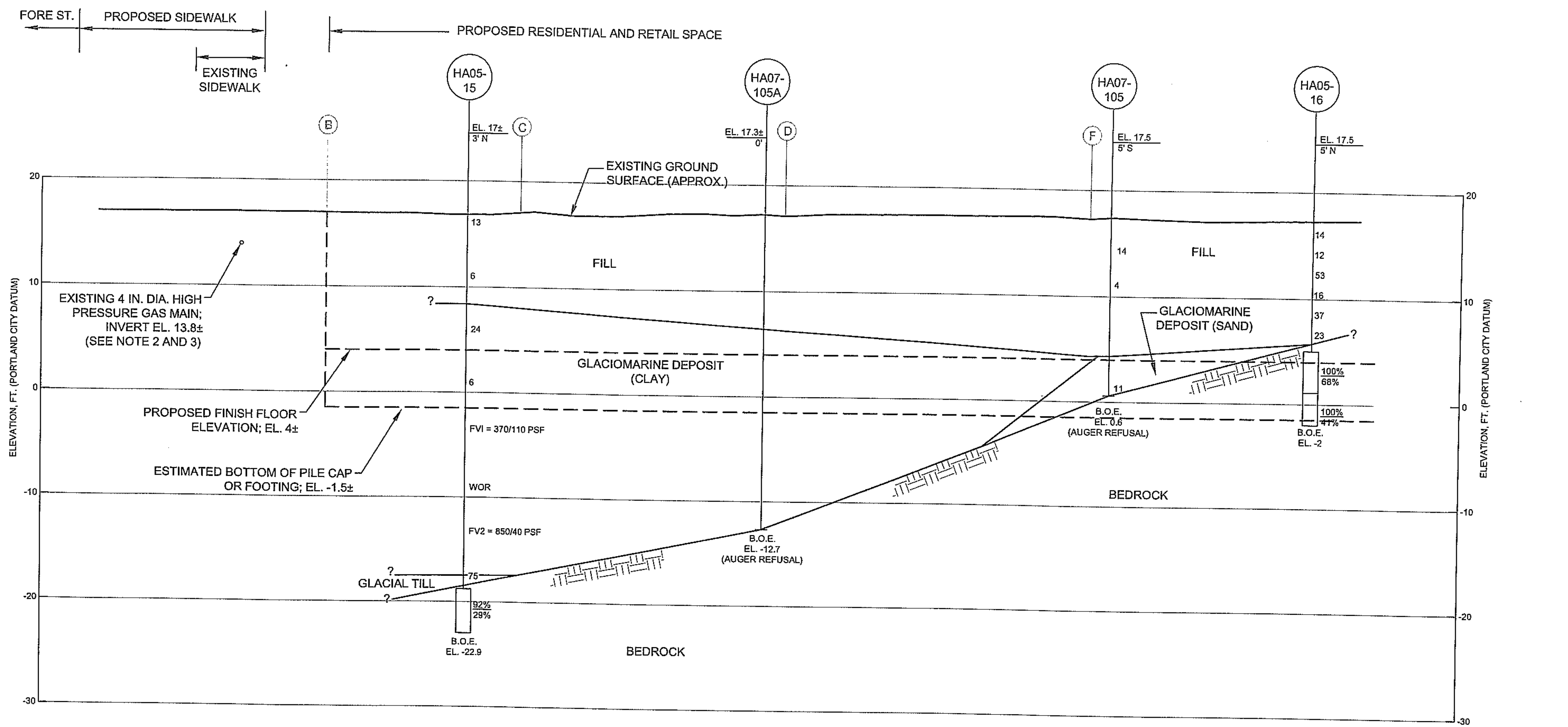
HALEY & ALDRICH

THE WATERMARK
PORTLAND, MAINE

SUBSURFACE PROFILE B-B

SCALE: AS SHOWN
MAY 2007

FIGURE 4



NOTE:

1. SEE LEGEND AND NOTES, FIGURE 6.
2. THE HORIZONTAL LOCATION OF THE EXISTING HIGH PRESSURE GAS MAIN IS TAKEN FROM THE ELECTRONIC AUTOCAD FILE "HA-203555-X00C.dwg", PROVIDED BY WOODARD & CURRAN, DATED 30 JANUARY 2007.
3. THE DIAMETER AND INVERT ELEVATION OF THE EXISTING HIGH PRESSURE GAS MAIN IS BASED ON VERBAL COMMUNICATION WITH WOODARD & CURRAN.

HORIZ. 0 10 20
VERT. 0 10 20
SCALE IN FEET

HALEY & ALDRICH

THE WATERMARK
PORTLAND, MAINE

SUBSURFACE PROFILE C-C

SCALE: AS SHOWN
MAY 2007

FIGURE 5

S:\30322\2007_0124 - CURRENT RES. FIG 2\30322-000-001_2007-0424_XSECTIONS.DWG

LEGEND:

HA07-101	DESIGNATION AND APPROXIMATE LOCATION OF TEST BORING DRILLED BY NORTHERN NEW ENGLAND TEST BORING COMPANY OF PORTLAND, MAINE IN 1975 AND MAINE TEST BORING, INC. OF BREWER, MAINE IN 2005 AND 2007
EL. 15.5±	APPROXIMATE GROUND SURFACE ELEVATION AT TEST BORING LOCATION DETERMINED BY INTERPOLATING BETWEEN ELEVATION CONTOURS
1' E	APPROXIMATE PERPENDICULAR OFFSET DISTANCE AND DIRECTION MEASURED FROM THE SUBSURFACE PROFILE LINE
42	STANDARD PENETRATION TEST (SPT) N-VALUE (BLOWS PER FOOT); WOR = WEIGHT OF RODS; WOH = WEIGHT OF HAMMER
FVI = 370/110 PSF	DENOTES IN-SITU VANE SHEAR TEST PERFORMED AT DEPTH WITH CORRECTED PEAK/RESIDUAL SHEAR STRENGTHS PROVIDED, SEE TABLE II FOR DETAILS
100% / 88%	PERCENT RECOVERY / PERCENT ROCK QUALITY DESIGNATION (RQD)
B.O.E. EL. 1.6	DENOTES BOTTOM OF EXPLORATION AT APPROXIMATE ELEVATION SHOWN
▽	GROUNDWATER LEVEL MEASURED IN COMPLETED OBSERVATION WELL ON DATE SPECIFIED
(OW)	DENOTES OBSERVATION WELL INSTALLED IN COMPLETED BOREHOLE
C	DESIGNATION AND APPROXIMATE LOCATION OF BUILDING COLUMN LINE

NOTES:

1. SEE FIGURE 2 FOR LOCATION AND ORIENTATION OF SUBSURFACE PROFILES.
2. EXISTING GROUND SURFACE ELEVATIONS AT TEST BORING LOCATIONS ARE APPROXIMATE AND WERE DETERMINED BY INTERPOLATION USING TOPOGRAPHIC CONTOUR INFORMATION PROVIDED BY WOODARD & CURRAN IN THE ELECTRONIC AUTOCAD FILE ENTITLED, "HA-203555-X00C.dwg", DATED 30 JANUARY 2007.
3. PROPOSED SITE FEATURES ARE TAKEN FROM THE ELECTRONIC AUTOCAD FILE ENTITLED, "P-BASEMENT - PARKING PLAN - PRELIMINARY GEOTECHNICAL LAYOUT DRAWING.dwg", PROVIDED BY THE ARCHITECTURAL TEAM, DATED 24 JANUARY 2007.
4. LINES REPRESENTING CHANGES IN STRATA SHOWN ON THE SUBSURFACE PROFILES ARE BASED ON LINEAR INTERPOLATION BETWEEN SUBSURFACE EXPLORATIONS. THESE INTERPRETED STRATA LINES DO NOT REPRESENT ACTUAL FIELD CONDITIONS OTHER THAN AT SPECIFIC EXPLORATION LOCATIONS.
5. LOCATIONS OF THE 1975 AND 2005 SERIES OF TEST BORINGS ARE APPROXIMATE AND WERE DETERMINED IN THE FIELD BY TAPING/PACING DISTANCES FROM EXISTING SITE FEATURES. LOCATIONS OF THE 2007 SERIES OF TEST BORINGS WERE DETERMINED IN THE FIELD USING GPS SURVEY EQUIPMENT.
6. ELEVATIONS ARE IN FEET AND REFERENCE PORTLAND CITY DATUM.
7. REFER TO APPENDIX A FOR LOGS OF THE 1975 SERIES TEST BORINGS, APPENDIX B FOR LOGS OF 2005 AND 2007 SERIES TEST BORINGS AND APPENDIX C FOR OBSERVATION WELL INSTALLATION AND GROUNDWATER MONITORING REPORTS.

HALEY & ALDRICH

THE WATERMARK
PORTLAND, MAINE

LEGEND AND NOTES

SCALE: AS SHOWN
MAY 2007

FIGURE 6

S:\30322\2007_0124 - CURRENT RES. FIG 2\30322-000-001_2007-0424_XSECTIONS.DWG