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MEMORANDUM

2 July 2015
File No. 38354-020

TO:	THA Architects Tom House
C:	The Federated Companies: Jonathan Cox PC Construction: Tim Sommers Becker Structural Engineers, Inc.: Todd Neal, P.E., Matt Paladino, P.E. Fay, Spofford & Thorndike, Inc..: Bo Kennedy, P.E.
FROM:	Haley & Aldrich, Inc. Bryan C. Steinert, P.E., Wayne A. Chadbourne, P.E.
SUBJECT:	Geotechnical Design Memorandum midtownTwo Building Portland, Maine

STATE OF MAINE
WAYNE A.
CHADBOURNE
No. 10620
LICENSED
PROFESSIONAL ENGINEER

This memorandum presents geotechnical design recommendations for the midtownTwo Building in Portland, Maine. Please note that some of the information summarized herein was previously sent via a series of electronic mail (email) messages to members of the project team. This memorandum serves as the official documentation of the geotechnical design recommendations for the midtownTwo Building only. Geotechnical recommendations for site civil features will be included in the Contract Documents (CDs) once the details of the civil features have been developed. Engineering design recommendations for the midtownOne, midtownThree and midtownFour Buildings have been or will be provided under separate cover.

Please recall that a geotechnical data report (GDR) summarizing the subsurface conditions encountered in previous and recent subsurface explorations, and the results of in-situ and laboratory testing conducted at the site was issued on 8 May 2015. We recommend that the GDR be included as an attachment to the "Existing Subsurface Conditions" specification to be included in the CDs and provided to prospective Contractors during bidding.

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

• The Federated Companies (TFC)	Owner
• PC Construction, Inc. (PC)	Construction Manager
• THA Architects, LLC. (THA)	Architect
• Becker Structural Engineers, Inc. (BSE)	Structural Engineer
• Fay, Spofford & Thorndike (FST)	Civil Engineer

The recommendations included herein are superseded by the information contained in the CDs and the information contained in the CDs takes precedence over the information and recommendations provided in this memorandum.

Elevation Datum

Elevations referenced herein are in feet and reference Portland City Datum (PCD). PCD relates to National Geodetic Vertical Datum of 1929 (NGVD29) as follows:

Elevation in feet (PCD) = Elevation in feet (NGVD29) + 0.02 feet

Proposed Site Development

GENERAL

Based on our recent discussions and our review of the conceptual site drawings provided by THA and TFC, dated 19 February 2015, we understand that TFC is proposing to construct an urban infill mixed-use development (midtown) on an approximate 3.25-acre parcel of land located in the Bayside Area of Portland. The development will occur in one phase and consists of the following key elements:

North of Chestnut Street

- midtownOne: 6-story, ground floor retail/residential (apartment) building (21,163 sf)
- midtownTwo: 8-story, ground floor retail/minimum 800 space parking garage (45,360 sf)

South of Chestnut Street

- midtownThree: 6-story, ground floor retail/residential (apartment) building (63,811 sf)
- midtownFour: 6-story, ground floor retail/ residential (apartment) building (19,992 sf)

Each of the proposed structures will have lowest level floor slabs constructed at El. 12.0. Below grade space is not currently planned as part of the new development.

In addition to the proposed buildings summarized above, site improvements will include stormwater management, site roadways, surface parking and supporting utility infrastructure needed to service the planned development. The details of these site improvements, including site grading, are currently not known.

Based on discussions with TFC and THA, it is our understanding that design of all four buildings may not occur simultaneously and that preparation of multiple sets of CDs (plans and specifications) may be required for each building/site. We also understand that the current priority for design and CD development is, in order of highest priority to lowest, midtownTwo, midtownThree, midtownOne and midtownFour.

PRELIMINARY STRUCTURAL LOADS

Preliminary design column (interior and exterior) and continuous wall footing loads (axial compression only) were provided to Haley & Aldrich by BSE on 14 May 2015 and were used as the basis for the foundation type alternatives evaluation summarized in subsequent sections of this memorandum. The preliminary loads provided by BSE are summarized below.

Column Location	Preliminary Axial Compressive Colum Load(kips)
End Condition	1,760
Corners	993
Typical Exterior	1,810
Wall Line B	120 k/lf

Axial uplift (tension) and lateral loads had not been developed by BSE at the time the technical evaluations and this memorandum were completed. An evaluation of lateral capacity for a select pile section was provided to BSE for reference, the results of which are summarized herein.

Geotechnical Evaluations and Design Recommendations

This section is primarily intended for members of the design team responsible for design of the structures and preparation of CDs and provides geotechnical recommendations for foundation design of the midtownTwo Building only.

The recommendations outlined herein are based on the lowest level floor slab elevation, preliminary foundation loads provided by BSE and assumed floor slab loads as summarized herein.

In general, design and construction of the midtownTwo Building should be completed in accordance with the requirements of the 2009 International Building Code (IBC Code).

SETTLEMENT EVALUATIONS

The soft, compressible nature of the harbor bottom and marine clay soils that are present at the site will affect the planning and design of the midtownTwo Building. As summarized above, the planned finish floor elevation is El. 12.0. Considering existing site grades within the proposed midtownTwo Building footprint range between El. 9 and 10, approximately 2 to 3 ft of fill will be required to meet proposed grades.

Placement of normal weight earthfill will cause consolidation settlement of the underlying harbor bottom and marine clay soils. As a result, settlement evaluations were completed to aid in assessing foundation and ground floor slab types as well as impacts to below grade utilities.

Stress History and Compressibility Characteristics of In-Situ Soils

The compressibility (stress-strain) characteristics of cohesive soil (harbor bottom and marine clay) deposits are highly dependent upon their stress history. Overconsolidation is a condition that results from the soil deposits having been exposed, at some time in the geologic past, to stresses greater than the present in-place stresses. If the soil deposits are stressed within the limits of the maximum previous stress (i.e., maximum past pressure), the magnitude of settlement will be a function of the re-compression ratio (RR) of the soil. If the applied stress exceeds the maximum previous stress, the magnitude of settlement will be a function of the virgin compression ratio (CR) of the soil. In general, measured values of CR for the type of harbor bottom and marine clay soils present at the site are typically 10 to 25 times greater than RR. Considering that the magnitude of consolidation settlement is directly related to CR and RR, the estimated settlement for normally consolidated soil (applied stress exceeds the maximum previous stress) would be 10 to 25 times greater than that of overconsolidated soil (applied stress within the limits of the maximum previous stress) for the same stress increase. Based on laboratory test results and our experience, the CR and RR values for the harbor bottom and marine clay deposits present at the site are as follows:

Cohesive Deposit	CR	RR
Harbor Bottom	0.25	0.022
Marine Clay	0.28	0.020

The stress history of the harbor bottom and marine clay deposits were estimated by comparing measured undrained shear strength values and estimated values of maximum past pressure (determined based on the standard oedometer test results) to estimate the overconsolidation ratio (OCR) profile through each of the deposits. Using the design shear strength profile, an empirical approach known as Stress History and Normalized Engineering Properties (SHANSEP) was used to establish a profile of maximum past pressure versus depth as a function of the shear strength profile.

The data indicates that the harbor bottom deposit and lower portion of the marine clay deposit are normally consolidated. The upper portion of the marine clay deposit is considered moderately to slightly overconsolidated. As a result, the harbor bottom deposit and lower portions of the marine clay deposit are considered to be highly compressible under even small stress increases.

Evaluation and Results

Detailed settlement evaluations were conducted modeling the raise in site grade (2 to 3 ft) only and a combination of the raise in site grade, as well as the presence of a soil-supported slab-on-grade and associated slab live load (LL). A floor slab LL equal to 125 psf was used in our evaluations. The stress history and compressibility characteristics of the harbor bottom and marine clay soils described above were input into the program to model the subsurface conditions present at the site. Settlement evaluations were completed using the computer program Settle 3D v 3.014 developed by Rocscience, Inc.

Estimates of settlement were computed at various locations adjacent to and within the limits of the midtownTwo Building assuming the use of normal weight earthfill (125 pcf) and are summarized below.

- Raise in grade only: ½ to 1¾ in.
- Raise in Grade, slab-on-grade and floor slab LL: 1 to 2¾ in.

The settlement estimates summarized above were provided to BSE so that decisions could be made regarding the type of ground floor slab (soil-supported slab-on-grade or structural slab) that will be needed. Discussion of the ground floor slab is included in a subsequent section of this memorandum.

In addition, because the proposed utilities will likely be constructed within/or above the compressible soils at the site, the magnitudes of ground surface settlement summarized above also represent the magnitudes of vertical movement that the proposed utilities will likely experience. We recommend that the information summarized above be provided to FST and the Mechanical-Electrical-Plumbing (MEP) Consultant for their use in determining whether the anticipated magnitudes of settlement/movement are acceptable or if modifications to the utility design (such as hanging the utilities from the structural slab, providing flexible connections at penetration locations, etc.) need to be made so that the vertical movement can be tolerated without the utility becoming inoperable or damaged. We will work with FST and the MEP Consultant to ensure that the utilities are detailed on the CDs to take these magnitudes of settlement into account.

SEISMIC SITE CLASS AND DESIGN PARAMETERS

Seismic site class and seismic design parameters were determined in accordance with the requirements of Section 1613 of the IBC Code. Due to the nature and thickness of the overburden soil units and bedrock encountered in the previous and recent test borings, we recommend the site be considered "Site Class D". Based on the geographic site location and the recommended "Site Class D" designation, we recommend BSE use the following seismic design values to determine the design spectral response acceleration parameters (S_{DS} and S_{D1}) and to calculate base shear for seismic design.

Design Parameter	Recommended Design Value
Site Factor for short-period range of acceleration response spectrum (**F_a**)	1.548
Site Factor for long-period range of acceleration response spectrum (**F_v**)	2.400
Horizontal response spectral acceleration coeff. at 0.2-s period on rock (**S_S**)	0.315g
Horizontal response spectral acceleration coeff. at 1.0-s period on rock (**S_1**)	0.077g

Please note that "g" refers to acceleration due to gravity. We do not consider the soils encountered at this site to be liquefaction susceptible.

FOUNDATION TYPE ALTERNATIVES EVALUATION

In order to determine the most cost effective and technically feasible foundation type multiple foundation alternatives were evaluated. Factors including but not limited to technical feasibility, constructability, performance and cost were considered in determining the recommended foundation type alternative. Specifically, the following alternatives were evaluated and are discussed herein:

- Shallow Foundation Support (Spread Footings)
- Intermediate Foundation Support (Ground Improvement)
- Deep Foundation Support (Driven Piles)

Each foundation alternative is discussed separately in the following sections of this memorandum.

Shallow Foundation Support (Spread Footings)

Shallow foundations (spread footings) are considered to be the least expensive of the foundation support alternatives discussed herein. However, based on the presence, nature and extent of compressible marine silt/clay soil across the site and the magnitude of the preliminary axial compression column loads provided by BSE, it is our opinion that the technical feasibility of shallow (spread footing) foundation support is controlled by total and/or differential settlement.

Based on the results of settlement evaluations discussed in previous sections of this memorandum and comparison of the applied loads to the preliminary axial compressive column loads provided by BSE, it is our opinion that the magnitude of total and differential settlement for a spread footing foundation is likely to be excessive. Considering that consolidation settlements ranging from approximately 1 to 2¾ in. were predicted for an applied load equal to approximately 500 psf (raise in grade, slab-on-grade and floor slab LL) we anticipate that the predicted settlement for a spread footing foundation would be significantly greater since the spread footing foundation applied load would be significantly greater than 500 psf.

As a result, alternative foundation support types were evaluated, as discussed in subsequent sections of this memorandum, to determine the most technically feasible foundation support option.

Intermediate Foundation Support (Ground Improvement)

In order to mitigate the risk of shallow (spread footing) foundation settlement, techniques could be used to improve the ground and aid in transferring structure loads to more competent soil/rock strata while allowing the structure to be supported on spread footings. The following ground improvement technologies were considered:

- Aggregate Piers (APs)
- Grouted Aggregate Piers (GAPs)
- Geo-Concrete Columns (GCCs)

Due to the soft to medium stiff nature of the harbor bottom and marine clay soils present at the site as measured by in-situ vane shear testing, it is our opinion that the in-situ soils do not provide sufficient lateral confinement for APs to be technically feasible. In addition, based on our experience it is our opinion that the depth to a suitable competent bearing stratum at the site is at or in excess of the practical limit of GAP and GCC installation equipment. Finally, considering the magnitude of the preliminary axial compressive column loads provided by BSE, it is our opinion that neither APs, GAPs nor GCCs would provide sufficient capacity to support the proposed structure.

Overall, due to the nature and extent of overburden soils present at the site, the depth to a suitable competent bearing stratum and the magnitude of preliminary axial compressive pile loads it is our opinion that ground improvement used to support spread footing foundations for midtownTwo is not technically feasible and is not recommended. It is possible that APs or GAPs may be feasible to support a soil-supported slab-on-grade. Further technical evaluations would be needed to evaluate the feasibility and cost of using these ground improvement methods in lieu of a structural slab.

Deep Foundation Support (Driven Piles)

Deep foundation support elements, such as driven piles, transfer structural loads through weak soil layers to underlying competent soil or bedrock bearing strata and are considered to be technically feasible. The following driven pile types were considered:

- Steel H-Piles (non-displacement pile)
- Precast, Prestressed Concrete (PPC) Piles (displacement pile)
- Closed-Ended Steel Pipe Piles (displacement pile)

Due to the soft and weathered/decomposed nature of the bedrock present at the site it is our opinion that displacement pile types are better suited for providing greater capacity at shallower depths as compared to non-displacement pile types. Furthermore, based on our discussions with PC we understand that current local market conditions suggest that closed-ended steel pipe piles are more economical as compared to PPC piles. Therefore, closed-ended steel pipe piles are judged to be the most appropriate option to limit pile lengths (cost effectiveness) while maximizing capacity available to support the structure.

Recommended Foundation Support Alternative

Based on the design information summarized herein and the foundation type alternatives that were evaluated, only driven pile foundation support is considered technically feasible. Moreover, for the reasons stated above it is our opinion that displacement pile types are more appropriate than non-displacement pile types and closed-ended steel pipe piles are more cost effective than PPC piles. Therefore, we recommend that closed-ended steel pipe piles, grade beams and pile caps be used to support the proposed structure.

Downdrag

The geotechnical engineering design of the piles included consideration of downdrag loading caused when the soil adjacent to an installed pile moves downward relative to the pile (in this case, caused by consolidation settlement of harbor bottom and marine clay soils under the weight of the proposed fill material).

It is our opinion that the magnitude of anticipated consolidation settlement summarized above is sufficient to mobilize downdrag forces. As a result, downdrag loads on the piles associated with the proposed fill placement and resulting consolidation settlement of the harbor bottom and marine clay soils were calculated for the pile sections recommended herein. A summary of the recommended downdrag allowances for each of the recommended pile sections are summarized below.

Pile Section	Downdrag Load (kips)		
	Column Line A	Column Line B	Column Line C
12.75-in. O.D. closed-ended pipe w/ 3/8-in. thick wall and concrete filled	26	30	22
14.0-in. O.D. closed-ended pipe w/ 1/2-in. thick wall and concrete filled	30	34	26
16.0-in. O.D. closed-ended pipe w/ 1/2-in. thick wall and concrete filled	34	40	30

Corrosion and Deterioration

The geotechnical engineering design of the recommended piles included consideration of corrosion. Based on our visual review of the soil samples for the presence of deleterious materials, including coal and ash, and our experience on similar projects with similar soil conditions, it is our opinion that the in-situ soils have low corrosive potential. Therefore, the allowable pile capacities summarized in the following section of this memorandum do not include a reduction in pile cross-sectional area for steel degradation.

Axial Compression Pile Capacity

Based on the range in magnitude of preliminary design axial compressive column loads provided by BSE, it is our opinion that the following three pile sections are the most appropriate for support of the midtownTwo Building loads. Allowable axial compressive capacities for each pile section are as follows:

Pile Section	Allowable Pile Capacity (kips)[1]		
	Column Line A	Column Line B	Column Line C
12.75-in. O.D. close-ended pipe w/ 3/8-in. thick wall and concrete filled	174	170	178
14.0-in. O.D. close-ended pipe w/ 1/2-in. thick wall and concrete filled	220	216	224
16.0-in. O.D. close-ended pipe w/ 1/2-in. thick wall and concrete filled	316	310	320

[1] - Please note that the allowable axial compressive capacities shown above have been reduced as a result of the downdrag loads summarized above.

We recommend that minimum pile spacing, clearance and embedment (into the pile cap) meet the requirements of the IBC Code. Pipe pile material should be fabricated in accordance with ASTM A252, Grade 3 Modified, with a minimum yield strength of 50 ksi. We also recommend that the piles be outfitted with 1.5-in. thick flat steel bottom plates that are no larger in diameter than the outside diameter of the pile. After installation and inspection for damage, we recommend that the pipe piles be filled concrete having a minimum compressive strength of 5,000 psi. We also recommend that the concrete be placed from the bottom of pile to the top, using tremie methods.

In order to generate the minimum required capacities summarized above it is likely that the piles will need to be driven to practicable refusal in/on the underlying bedrock. We recommend the piles be installed to a minimum ultimate capacity equal to the allowable pile capacity plus the downdrag load multiplied by 2.0 (e.g., minimum ultimate capacity equal to 400 kips for a 12.75-in. O.D. pile located along column line A).

It is our opinion that dynamic pile testing should be conducted on a select number of piles prior to production pile installation to: 1.) determine the installation criteria required to develop the minimum ultimate capacity based on pile hammer selected by the Contractor and 2.) confirm that the stresses in the pile do not exceed allowable limits during driving as specified in the IBC Code. 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 analyses should be performed on a select number of the indicator piles installed during the dynamic pile testing program. Additional details on the dynamic pile testing program and the number of piles that should be monitored using PDA equipment and that undergo CAPWAP analysis will be provided in the specifications.

Lateral Pile Capacity

Per the request of BSE and in accordance with Section 1810.3.3.2 of the IBC Code we estimated the allowable lateral pile load for the 16.0-in. O.D. pipe pile alternative only. In accordance with the IBC Code, the allowable lateral pile load is equal to one-half of the lateral load that produces a gross lateral movement of 1 in. Lateral pile evaluations were completed modeling the subsurface conditions present at the site and the results are summarized below.

Pile Section	Ultimate Lateral Load Resulting in 1 in. Deflection (kips)		Allowable Lateral Load[1] (kips)		Pile Head Deflection @ Allowable Lateral Load (in.)	Maximum Bending Moment @ Allowable Lateral Load (ft-lbs.)	
	"Free-Head"	"Fixed-Head"	"Free-Head"	"Fixed-Head"		"Free-Head"	"Fixed-Head"
16.0-in. O.D. closed-ended pipe w/ 1/2-in. thick wall and concrete filled	18	44	9	22	~0.33	51,200	138,000

[1] – Based on a factor of safety equal to 2.0 in accordance with the requirements of the IBC Code.

As shown, allowable lateral pile loads were estimated for both "free-head" and "fixed-head" pile-to-pile cap connection conditions. Please note that with a "free-head" condition the top of the pile is allowed to rotate and with a "fixed-head" condition the top of the pile is not allowed to rotate. BSE should select the most appropriate allowable lateral load based on their design of the pile-to-pile cap connection.

In addition, the values summarized above are for an individual pile only and do not consider group effects. We recommend that BSE consider group effects once preliminary pile layout/spacing is established. We also recommend that BSE confirm that the pile section is adequate to resist stresses due to combined axial load, shear load and bending moments with the appropriate factor of safety.

GROUND FLOOR SLAB

Based on the predicted magnitude of ground surface settlement as a result of raising site grades, it is our opinion that a soil-supported slab-on-grade within the ground floor retail space is not desirable given the potential for damage (total and differential vertical movement). We understand that BSE has reviewed the settlement estimates and made a determination that the predicted total and differential settlements summarized herein are not acceptable and are moving forward with designing a structural slab.

RESISTANCE OF LATERAL BUILDING LOADS

We recommend that lateral loads be resisted by a combination of lateral pile capacity (recommendations provided in previous sections of this memorandum) and passive earth pressure acting against below-grade foundation units (pile caps, grade beams and foundation walls).

The net passive resistance (passive minus active) provided by the fill surrounding the foundation units can be calculated using an equivalent fluid weight (triangular distribution) of 150 pounds per cubic foot. The top of the assumed passive zone should be 1 ft below the ground surface unless it is confined by a slab. This value assumes that existing fill material present on site is free-draining and is placed and compacted in lifts during backfilling operations. Please note that the recommend passive resistance assumes that existing fill material present on site is re-used during backfilling operations and that imported granular fill is not required.

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).

FOUNDATION DRAINAGE, DAMPPROOFING AND WATERPROOFING

Groundwater levels were measured at approximately El. 6 in observation wells installed at the site during previous subsurface exploration programs, approximately 6 ft below the proposed elevation of the ground floor slab (FFE = El. 12.0). Accordingly, a foundation drainage system (perimeter and underslab drain) is not considered necessary. We recommend that surface water runoff be directed away from the garage by sloping the finish grade downward away from the structure. To minimize the potential for surface water infiltration adjacent to the garage we recommend that the upper portions of backfill adjacent to the garage in unpaved areas consist of topsoil or other soil having low permeability.

It is our opinion that dampproofing and waterproofing the outside face of foundation walls is not necessary. We do however, recommend that below-grade portions of foundation walls and slabs below El. 7 (e.g., elevator pits) either be designed to resist hydrostatic uplift loads (based on a design groundwater level at El. 7), or be permanently drained. If the slab is designed to resist uplift loads, we recommend that the walls and slab for the elevator pit be waterproofed below El. 7 and dampproofed above El. 7. If the slab is not designed to resist uplift loads, an underslab drainage system should be constructed beneath the pit slab. If determined necessary, we will coordinate with the project team on the requirements for and design details of the underslab drainage system.

The use of vapor barriers beneath the ground floor slab is recommended within the footprint of the ground floor retail space. We understand the vapor barrier will be specified and detailed by THA. If vapor barriers are used, the floor slab design should be coordinated with vapor barrier installation, as it may impact concrete curing and curling.

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 summarized herein. As discussed with you and other members of the design team, Haley & Aldrich is planning on working with the design team to prepare the specifications and contract drawings related to the following topics:

- Existing Subsurface Conditions
- Earthwork (in coordination with FST)
- Construction Dewatering (in coordination with FST)
- Temporary Lateral Earth Support
- Pile Installation and Testing
- Pavement Section and Drainage Details (in coordination with FST)
- Utility details specifically within the limits of the midtownTwo Building and at transition points from the Building to the site (in coordination with FST and the MEP Consultant)

Several of the specifications will require the Contractor and the Contractor's engineer to perform analyses and submit means and methods and results to the design team for review. The design team should be allowed to review the submittals to ensure that the Contractor's work meets the requirements of the CDs and is 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 recommendations contained herein are based on the known and predictable behavior of a properly engineered and constructed foundation. Monitoring of the foundation construction is essential to enable the geotechnical engineer to keep in contact with procedures and techniques used in construction, and is also required in order to comply with Section 1808.2.10 of the IBC Code. Therefore, it is recommended that a representative of Haley & Aldrich be present at the site to provide full-time monitoring during foundation construction (dynamic testing of indicator piles, review of dynamic testing data and observation and documentation of production pile installation).

Limitations and Closure

This memorandum is prepared for the exclusive use of TFC and THA relative to the midtownTwo Building in Portland, Maine. There are no intended beneficiaries other than TFC and THA. Haley & Aldrich shall owe no duty whatsoever to any other person or entity on account of the Agreement or the memorandum. Use of this memorandum by any person or entity other than of TFC and THA for any purpose whatsoever is expressly forbidden unless such other person or entity obtains written authorization from TFC and THA and from Haley & Aldrich. Use of this memorandum by such other person or entity without the written authorization of TFC and THA 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 memorandum by any person or entity, including by TFC and THA, for a purpose other than relative to the midtownTwo Building in Portland, Maine is expressly prohibited unless such person or entity obtains written authorization from Haley & Aldrich indicating that the memorandum is adequate for such other use. Use of this memorandum 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 information provided herein is 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 memorandum.

Please note that the recommendations included herein are superseded by the information contained in the CDs and that the information contained in the CDs takes precedence over the information provided in this memorandum.

We appreciate the opportunity to provide geotechnical engineering consulting services on this project and trust this provides sufficient geotechnical design information. We look forward to our continued association with you through final design, construction document preparation, and construction. Please do not hesitate to contact us if you require additional information or have questions regarding any of the information provided herein.