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MEMORANDUM

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TO: University of Southern Maine
Facilities Management
Ms. Carol M. Potter

C: Weidlinger Associates, Inc.; Wayne Siladi, P.E.
Koetter Kim & Associates, Inc.; Bill Fitzpatrick

FROM: Haley & Aldrich, Inc.
Bryan C. Steinert, Wayne A. Chadbourne, P.E.
BCS

SUBJECT: Geotechnical Design Recommendations
Proposed Osher Map & Glickman Family Library Addition
University of Southern Maine
Portland, Maine

This memorandum presents geotechnical design recommendations and construction considerations for the proposed Osher Map & Glickman Family Library (OML) Addition at the University of Southern Maine (USM) campus in Portland, Maine. This memorandum has been revised based on conversations between Koetter Kim & Associates (KKA), Weidlinger Associates (WAI) and USM regarding acceptable magnitudes of foundation and ground floor slab settlement.

A geotechnical data report summarizing the subsurface conditions encountered during our exploration program was issued on 23 October 2006. In addition, a summary of the conditions encountered in the exploratory test pit excavated to determine the condition/type of foundation system supporting the existing library building is provided in our memorandum dated 27 July 2007.

ELEVATION DATUM

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:

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

PROPOSED SITE DEVELOPMENT

The proposed OML Addition is planned to be approximately 7,800 square feet (sf) in plan area. The OML Addition will directly abut the western and southern exterior walls of the existing library. The portion of the addition south of the existing library is proposed to be a one-story entrance way with finish floor elevation (FFE) at El. 24.83, matching the ground floor level of the existing library. The portion of the addition west of the existing building is proposed to be a three-story structure with FFE of El. 24.83. Site grades within the footprint of the addition will be raised between 2 and 4 ft to reach the proposed FFE. Areas adjacent to the addition will require a maximum raise in grade of 2 ft to reach design grades.

Structural loading information and the proposed column layout were provided by Weidlinger Associates, Inc. (WAI). Bay spacing throughout the addition varies from 11 ft by 16 ft to 15 ft by 20 ft but is typically 20 ft by 20 ft. Maximum design column loads (axial compression) in the one-story portion of the addition ranged from 80 to 175 kips (1 kip = 1,000 lb) and ranged from 100 to 760 kips in the three-story portion. The design live load for the one-story addition is 100 psf. Each floor of the three-story portion of the addition will be designed to support a design live load of 250 pounds per square foot (psf) to account for compact storage. Because of the structural framing of the addition, there will be no design axial uplift loads imposed on the foundation system. The total lateral load on the building is 250 kips.

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 Design Recommendations

A shallow foundation system, bearing in the marine clay stratum was investigated for the one-story entrance portion of the addition. Estimates of settlement were made based on the design column loads (80-175 kips) provided by WAI. We estimate that the settlement of individual footings in this portion of the proposed addition would not exceed about 1-¼ in., with differential settlements between adjacent columns (spaced 16 to 20 ft apart) not exceeding about ½ in. It was our understanding, based on conversations with WAI on 14 August 2007 that these magnitudes of total and differential settlement were acceptable. However, based on subsequent conversations between WAI, KKA and USM it is our understanding that this magnitude of settlement is no longer considered to be acceptable. As a result, we recommend that this portion of the addition be supported by piles driven to practicable refusal in/on the underlying bedrock.

In addition, based on the magnitude of the "heavily" loaded (100-760 kips) columns, we recommend that the three-story portion of the proposed addition also be supported by piles driven through the overburden soils to end bearing in/on the underlying bedrock.

It is our opinion, based on the subsurface conditions and the range in design loads, that a variety of pile types (e.g., steel pipe piles, steel H-piles and precast, prestressed concrete (PPC) piles) are technically feasible to support this portion of the addition. Based on the condition of the bedrock and the range/magnitude of the design axial compressive loads, we recommend steel H-piles be used to support this portion of the addition. However, based on recent contractor bids for projects in the Portland area, we recommend that PPC and closed-ended steel pipe piles also be considered for use on this project based on current market conditions. Recognizing that the cost of installation of the various pile types fluctuates, the final pile selection should be determined based on pile availability and economics at the time the project is bid.

We conducted static pile capacity analyses to determine the axial design capacity of several different sizes of pipe, PPC and H-piles. We recommend that the following pile sections be considered for support of the proposed OML Addition:

Pile Section[1, 2]	**Allowable Axial Design Capacity – Compression[3]**
HP10x42 H-pile	140 kips
HP12x53 H-pile	170 kips
12-in. PPC concrete pile	170 kips
10.75 in. O.D. pipe pile, open ended with 0.344" wall	190 kips

[1] – capacities shown are based on minimum 50 ksi steel pile sections
[2] – capacities shown are based on 5,000 psi concrete infill for the pipe pile
[3] – capacities shown include a downdrag reduction to account for site filling (see below)

The capacity values presented above are gross values and do not take into account a reduction in pile cross sectional area for steel degradation as we do not consider the soils and groundwater at the site to be corrosive/saline. In addition, the design pile capacity values include an allowance of 25 to 35 kips for downdrag loads that will occur due to the raise in site grades within the addition footprint. Downdrag loads are downward forces acting on the piles that are caused when the soil around the pile moves downward relative to the pile itself. This commonly occurs in areas were a compressible stratum is present (i.e., glaciomarine clay) and when site fills are placed (i.e., to raise the site grades in order to match FFEs) or when groundwater levels are lowered.

Steel pipe and H-piles should be fabricated from Grade 50 (50 ksi) steel. Steel pipe and PPC piles should be outfitted with a minimum 1-½ in. thick steel bottom plate, and steel H-piles should be outfitted with steel driving shoes/points in order to protect the pile tips from damage during driving in the glacial till and rock. If pipe piles are selected, the piles should be infilled with minimum 5,000 psi strength concrete using tremie methods. Regardless of pile type, the piles should be installed to a minimum <u>ultimate</u> geotechnical capacity equal to the design capacity multiplied by 2.25. Per the requirements of IBC, three or more piles should be installed at discrete pile cap locations to provide lateral stability in all directions.

We anticipate that piles will advance through the glacial till and up to several feet into the bedrock prior to achieving end bearing. Based on this, a proposed finish floor elevation of El. 24.83 and an average, assumed pile cut-off level equal to El. 21, pile lengths should vary between 40 and 45 ft. Based on these pile lengths, we do not anticipate that pile splicing will be required.

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.

Prior to installation, one of the piles could be statically load tested to twice the pile design capacity. However, we recommend that dynamic pile testing 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 Code. Additional details on dynamic testing are provided in the construction considerations portion of this report.

Ground Floor Slab

We investigated the possibility of designing the slab in both the one-story and 3-story portions of the proposed addition as a soil supported slab-on-grade. Based on the proposed site grading provided by Woodard & Curran we understand the ground floor slab will be constructed between 2 and 4 ft above existing site grades. Estimates of settlement were made based on the proposed site grading and the live load acting on the ground floor slab (100 psf for the one-story and 250 psf for the 3-story portion) provided by WAI.

Due to increased floor loading, we estimate that 2 to 3 in. of settlement could occur (not acceptable for compact storage tolerances). We therefore recommend that the ground floor slab for the three-story portion of the proposed OML Addition will be designed as a structural slab.

We anticipate that the settlement of the floor slab in the one-story portion of the addition, under static loading conditions, would not exceed about ½-in. It was our understanding, based on conversations with WAI that this magnitude of settlement was acceptable. However, based on subsequent conversations between WAI, KKA and USM it is our understanding that this magnitude of settlement is no longer considered to be acceptable. As a result, we recommend that the ground floor slab in this portion of the addition be designed as a reinforced concrete structural slab.

Frost Protection

Bottoms of exterior footings should be founded a minimum of 4.5 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.

Foundation Drainage

Based on the proposed FFE of the lowest level floor slab, we do not consider installation of a foundation drain system necessary for the OML Addition.

Resistance of Lateral Design Building Loads

We recommend that lateral loads on the addition (maximum of 250 kips) be resisted by passive earth pressures acting against foundation units. The net passive resistance (passive minus active) provided by the fill surrounding foundation walls, footings, pile caps and grade beams 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 within a minimum 5 ft of the outside edge of the below-grade portions of foundation walls. 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.

As discussed with WAI, we anticipate that passive earth pressures acting on the below grade portions of the addition will be adequate to provide resistance for the design maximum building lateral loading condition. 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).

Seismic Design Considerations

We recommend that the building be designed in accordance with the seismic requirements of the latest edition of the IBC Code as outlined below. The seismic design coefficient determination for this site is controlled primarily by the depth to bedrock and the presence of very soft to very stiff glaciomarine clay beneath the building footprint. Based on the shear strength information obtained during our test boring program, we recommend that the site be classified as "Site Class D". We recommend the following values be used by the project structural engineer 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: $S_S = 0.37$ g
- Mapped Spectral Response Accelerations for 1-second Periods: $S_1 = 0.10$ g
- Site Coefficient for Short Periods: $F_a = 1.5$
- Site Coefficient for 1-second Periods: $F_V = 2.4$

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

CONSTRUCTION CONSIDERATIONS

The primary purpose of this section is to comment on items related to excavation, earthwork, pile driving, dewatering and related geotechnical aspects of proposed construction. It is written primarily for the geotechnical engineer having responsibility for preparation of geotechnical related plans and specifications. Since it identifies potential construction problems related to foundations and earthwork, it will also aid personnel who monitor the construction activity. Prospective contractors for this project must evaluate the 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 proposed OML Addition only. A geotechnical data report summarizing the subsurface conditions encountered in the subsurface exploration program was issued previously.

Pile Load Testing Program

A static pile load test would normally be performed for piles with the design capacities required for this project. However we have pile dynamic test results for piles driven at the adjacent USM Parking Garage site. We also monitored the installation and have complete pile installation records for all the piles driven on that project. We therefore believe that dynamic testing can be used for this project in lieu of a static load test.

We recommend that the Contractor monitor the installation of a minimum <u>three</u> pre-selected production piles (i.e., indicator piles) using the Case-Goble Pile Driving Analyzer (PDA) equipment. The dynamic testing will: 1.) verify that the required minimum ultimate geotechnical 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 (e.g., $0.90f_y$, or 45 ksi for grade 50 steel piles). All <u>three</u> of the pre-selected indicator 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 load bearing stratum (likely bedrock). CAPWAP analysis should be performed on at least <u>two</u> of the indicator piles installed during the dynamic testing program. 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 for 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. If the results indicate the factor of safety is below 2.25, the PDA/CAPWAP results should be re-evaluated to provide driving criteria that are appropriate to achieve the minimum required factor of safety.

The indicator piles should be installed at production pile locations prior to the production driving in order to assist with establishing pile lengths. We recommend that the indicator piles be clearly identified on the structural foundation drawings. Additional construction considerations relative to pile installation, including driving criteria will be included in the pile specification.

Pile Installation

Due to the past site usage and the existing structures present at the site, it is likely that obstructions (i.e., concrete foundation walls, footings, slabs) may be encountered during pile installation (see Sanborn Maps in Appendix C of our 23 October 2006 data report). If encountered, obstructions should be removed by the Contractor prior to pile installation at no additional cost to the Owner.

We recommend that the site be graded to a level corresponding to a few feet below the design pile cut off elevation prior to mobilizing the pile driving equipment to the site. In addition, the exposed subgrade should be stabilized to establish an adequate working surface for pile installation (e.g., placing a lift of crushed stone). Full-time monitoring of pile installation should be performed by a geotechnical engineer in accordance with the requirements of the IBC code.

Excavation

Excavation will be required for general site grading, construction of pile caps/grade beams and underground utilities. We anticipate that excavation of 1-2 feet will be required to construct the pile caps/grade beams

We expect that excavation of the in-situ soils (fill) can be accomplished using normal earth-moving equipment. We do not anticipate that bedrock will be encountered during excavation. Obstructions, such as an old railroad platform foundation wall may be encountered during excavation for the pile caps/grade beams in the one-story portion of the addition (see Sanborn Maps in Appendix C of our 23 October 2006 data report). Old foundation elements may also be present northwest of the existing library within the limits of the Eastern Electric parking lot area. Temporary cut earth slopes should, typically, be stable if constructed no steeper than about 2H:1V. Some sloughing and raveling should be anticipated in temporary earth slopes. We recommend that the contractor be responsible for the design, stability and safety of all temporary excavations. All excavation support systems and temporary earth slopes shall comply with OSHA and all other applicable safety regulations.

Construction Dewatering

Based on the groundwater levels measured in the observation well at the site (between El. 9 and El. 14) and observed in the exploratory test pit (at approx. El. 12), it is likely that groundwater will not be encountered during excavation. However, water may accumulate in excavations due to rainwater, snowmelt etc. Dewatering can be accomplished by pumping from open sumps and temporary ditches located within and around the excavations. Sumps should be provided with filters suitable to prevent pumping of fine grained soil particles.

The contractor should be responsible for the design, installation, and removal of an appropriate excavation dewatering system. Dewatering should be performed as required to maintain the undisturbed nature of the soil bearing surfaces and enable all final excavations, foundation construction and backfilling to be done in the dry. Water entering any temporary excavation should be controlled and promptly removed to avoid subgrade disturbance. Surface water runoff should be directed away from exposed soil bearing surfaces.

Dewatering should be performed in accordance with all applicable regulations. Dewatering discharge should be directed into on-site excavations and remain on-site if possible. Dewatering should be conducted in a manner that avoids disturbance or undermining of existing foundations, backfill, prepared foundation subgrades, and that limits pumping of fines.

Preparation and Protection of Bearing Surfaces

We recommend that excavations be conducted in a manner that minimizes disturbance to the in-situ fill soils when excavating to bearing level for pile caps and grade beams. It may be necessary to over-excavate and replace locally weak, disturbed or otherwise unacceptable foundation bearing soils using crushed stone. If encountered, all topsoil, debris and organic matter should be removed from within the limits of the excavation. Fill soils and/or naturally deposited marine clay are likely to be encountered at the subgrade level in excavations. Where encountered at subgrade level, we recommend that in-situ fill soils be proof-rolled with a minimum four passes of a self-propelled static roller or heavy hand-guided vibratory compactor until firm. Marine clay subgrade surfaces should not be proof-rolled.

Filling & Backfilling

Compacted granular fill should be used as backfill within 5 ft of interior and exterior frost walls to provide passive soil pressure to resist lateral building loads and adjacent to pile caps/grade beams.

Because the proposed floor slabs will be structurally supported, we recommend that common fill be used to raise grades beneath the floor slab (outside of the compacted granular fill placed adjacent to frost walls and pile caps/grade beams as outlined above). The primary purpose of the placed/compacted fill will be to provide an adequate working surface for construction of the structural slab. Common fill placed beneath the slab should be placed in maximum 12-in. (loose measure) lifts and compacted to at least 90 percent relative compaction.

Placement of compacted 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 prior to placement/compaction. Fill materials should not be placed on snow, ice or frozen soil. 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

CGF should be placed adjacent to pile caps/grade beams and within 5 ft of interior and exterior frost 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.

Compacted granular fill should be placed in lift thicknesses not exceeding 10 in. loose measure. Compaction equipment in open areas should consist of self-propelled vibratory rollers such as a BoMag BW-60S. In confined areas, hand-guided equipment such as a large vibratory plate compactor should be used and the loose lift thickness should not exceed 6 in.

A minimum of four systematic passes of the compaction equipment should be used to compact each lift. Cobbles or boulders having a size exceeding 2/3 of the loose lift thickness should be removed prior to compaction.

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.

Except for zones requiring special backfill (CGF within 5 ft of foundation walls) the exterior of foundation walls, beneath the structural slab and landscape areas may be backfilled with common fill. The existing fill soils are generally acceptable for reuse as common fill.

Reuse of Excavated On-Site Soils for Backfill

Based on visual inspection of the fill previously used to backfill the existing library building, it is likely that these soils may be suitable for reuse as CGF adjacent to pile caps and grade beams or adjacent to foundation walls. However, confirmation on the suitability of the excavated fill soils for reuse 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 should be free of oversize material, organic material, refuse and debris and be able to achieve the minimum compaction requirements outlined below.

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

Compaction Requirements

A summary of recommended compaction requirements is as follows:

Location	Minimum Compaction Requirements
Beneath footings, adjacent to pile caps/grade beams and adjacent to foundation walls	95 percent
Landscaped areas and beneath structural slabs	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. Therefore, we recommend that Haley & Aldrich be retained to prepare or assist in the preparation of the specifications and contract drawings related to the following topics:

- Demolition
- Earthwork
- Construction dewatering
- Pile installation and testing

The contract specifications will require the Contractor and the Contractor's engineer to perform analyses and submit results to the designers for review. We recommend that Haley & Aldrich 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.

Construction Monitoring

The recommendations contained herein are based on the predictable behavior of a properly engineered and constructed foundation. Monitoring of the foundation installation is required to enable the geotechnical engineer to verify that the procedures and techniques used during construction are in accordance with the recommendations contained herein and the contract documents. Therefore, it is recommended that a geotechnical engineer or experienced technician be present during construction to:

- Monitor the dynamic pile load test program and the installation of pile foundations per the requirements of the IBC Code.
- Observe and test placement and compaction of CGF and other compacted.
- Confirm that soils used as fill and backfill are in accordance with the project plans and specifications, and make judgments on the suitability of excavated soils for reuse as fill.

Closure

We trust this provides sufficient geotechnical design information to proceed with design development and estimation of project costs for geotechnical-related items. Please do not hesitate to contact us if you require additional information.

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