July 15, 2008

Portland State University
Facilities and Planning
617 SW Montgomery, Suite 202
Portland, OR 97201

Attention: Mr. Francis McBride

Report of Geotechnical Engineering Services
Seismic Upgrade
Lincoln Hall
1620 SW Park Avenue
Portland, Oregon
GeoDesign Project: PSU-7-01

INTRODUCTION

GeoDesign, Inc. is pleased to provide this report that presents our geotechnical recommendations for the remodel and seismic retrofit of Lincoln Hall located at 1620 SW Park Avenue in Portland, Oregon. We understand that the existing four-story building will be upgraded to conform with seismic design requirements outlined in the American Society of Civil Engineers (ASCE) 41-06 document. The location of the site is presented on Figure 1.

Based on information provided by Mr. Aaron Burkhardt of KPFF Consulting Engineers, it is our understanding that the existing building is a four-story structure supported on conventional shallow foundations. The proposed building remodel will consist of new concrete shotcrete walls for the four stairway cores and two new braced panels at the center of the building. Additional floors are not planned for the structure. The existing footings generally range from 4 to 6.5 feet square. The loads supported by the existing foundations and the increment of increase in loads are currently unknown. Underpinning using micropiles will be necessary for the stairway core foundations.

SCOPE OF SERVICES

The purpose of our services was to provide geotechnical engineering parameters for use in the proposed seismic upgrade of Lincoln Hall. Specifically, we performed the following scope of services:

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- Reviewed readily available reports of previous geotechnical studies completed in the site vicinity and published geologic information.
- Provided recommendations for use in evaluating existing footings and the design of new shallow foundations.
- Provided recommendations for micropile underpinning and uplift.
- Provided a discussion of seismic activity near the site, liquefaction potential and anticipated deformations, and recommendations for seismic design factors in accordance with the procedures outlined in the ASCE 41-06 document.
- Provided this written report summarizing our findings, conclusions, and recommendations.

SITE DESCRIPTION

GEOLOGIC SETTING

The site is located in the western portion of the Portland Basin physiographic province, which is bound by the Tualatin Mountains to the west and south, and the Cascade Range to the east and north. The Portland Basin is described as a fault-bounded, pull-apart basin that was formed by two northwest-trending fault zones (Pratt, et al., 2001). The Portland Hills Fault Zone trends along the west side of the basin and the Frontal Fault Zone trends along the east side of the basin near Lacamas Lake, east of Vancouver, Washington.

A review of published geologic literature, previous explorations in the area, and explorations conducted during our investigation indicates the site is underlain by Quaternary flood deposits (Gannet and Caldwell, 1998; Beeson, et al., 1991; and Madin, 1990), delineated as the fine-grained facies (Qff). The unit consists of unconsolidated coarse sand to silt with occasional clayey layers. The unit was deposited by multiple catastrophic glacial floods associated with the late Pleistocene (15,500 to 13,000 years before present) Missoula Floods. The thickness of the flood deposits in the site vicinity is approximately 30 to 60 feet.

Underlying the flood deposits is the Pliocene to Pleistocene age (5 million to 1.5 million years before present) Troutdale Formation (QTg), which consists of poorly to moderately consolidated, semi-cemented, subrounded to rounded sand and gravel. The thickness of the Troutdale Formation in the site vicinity is approximately 100 to 150 feet (Gannet and Caldwell, 1998; Beeson, et al., 1991; and Madin, 1990).

The Troutdale Formation is underlain by the Miocene age (20 million to 10 million years before present) Columbia River Basalt Group (Tcr), which is a series of basalt flows that originated from southeastern Washington and northeastern Oregon. The Columbia River Basalt Group is several hundred feet thick and considered the geologic basement unit for this report.

SUBSURFACE CONDITIONS

Our knowledge of site subsurface conditions is based on our review of geotechnical reports by Northwest Testing Laboratories (Northwest Testing Laboratories, 1980), directly southeast of Lincoln Hall at the southeast corner of SW Market Street and SW Broadway Street, and by L.R. Squier Associates, Inc. (L.R. Squier Associates, 1979; 1986), directly northeast of Lincoln Hall at the northeast corner of SW Park Avenue and SW Market Street.

The soils in the site vicinity generally consist of soft to stiff silt to depths ranging from 5 to 13.5 feet below the ground surface (BGS) underlain by loose to medium dense sand with varying amounts of silt interbedded with seams and layers of sandy silt to a depth of approximately 38 to 54 feet BGS. The loose to medium dense sand is underlain by medium dense to dense sand to a depth of 81 to 83 feet BGS. The sand is underlain by very dense gravel with sand to the maximum depths explored. Static groundwater was not encountered in the explorations; however, perched water was encountered at depths between 30 to 50 feet BGS. Based on our experience in the area, typical static groundwater levels range from approximately 90 to 100 feet BGS and may fluctuate more than 10 feet during extreme wet and dry seasons.

SITE PREPARATION

Demolition includes complete removal of the existing structures, floor slabs, pavements, concrete curbs, and abandoned utilities. Demolished material should be transported off site for disposal. Excavations remaining from subsurface elements should be backfilled with structural fill beneath planned site grades. The bottoms of the excavations should be excavated to expose firm subgrade before placing and compacting structural fill. The sides of the excavations should be cut into firm material and sloped a minimum of 1½ horizontal to 1 vertical. Utility lines abandoned under new structural components should be completely removed or grouted full if left in place. Soft soil encountered in utility line excavations should be removed and replaced with structural fill. The demolition contractor should take appropriate measures to avoid disturbing adjacent structures.

STRUCTURAL FILL

GENERAL

All material used as structural fill should be free of organic material or other unsuitable materials and particles larger than 4 inches in diameter.

RECYCLED MATERIALS

Concrete curbs, floor slabs, asphalt pavement, and base rock from the existing structure, as well as rubble fill observed under the site, can be used in structural fill provided it is broken into particles no greater than 4 inches and relatively well graded. We recommend that the moisture content of fill be within 3 percent of the optimum, as determined by American Society for Testing and Materials (ASTM) D 1557 and the fill compacted to at least 95 percent of the maximum dry density, as determined by ASTM D 1557, or to an unyielding condition.

IMPORTED GRANULAR MATERIAL

Imported granular material for structural fill should be pit- or quarry-run rock, crushed rock, or crushed gravel and sand. It should be fairly well graded between coarse and fine material, have less than 5 percent by dry weight passing a U.S. Standard No. 200 Sieve, and have a minimum of two fractured faces. The material should be placed in lifts with a maximum uncompacted thickness of 12 inches and compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D 1557. During the wet season or when wet subgrade conditions exist, the initial lift should be approximately 18 inches in uncompacted thickness and compacted by rolling with a smooth-drum roller without use of vibratory action.

FOUNDATION SUPPORT

We understand the structure is supported on spread footings established at depths of approximately 2.5 to 9.5 feet below street level. Based on our understanding of site subsurface conditions, we anticipate that the soils at this depth interval consist of interbedded layers of stiff silt and medium dense to dense sand.

BEARING CAPACITY

Bearing pressures under long-term loads are controlled by settlement. Given the age of the structure, settlement under dead and long-term live loads is essentially complete.

Bearing pressures under short-term transient loads, such as wind and seismic forces, are controlled by bearing capacity of the soil. Based on our project understanding, we recommend that foundations be evaluated using an allowable bearing capacity of 4,000 pounds per square foot. This value can be increased by one-third when considering short-term loads, such as wind and seismic forces. The weight of the footing and overlying backfill can be ignored in calculating footing loads.

LATERAL RESISTANCE

Lateral loads can be resisted by passive earth pressure on sides of the footings and by friction on the base of the footings. We recommend a friction coefficient of 0.45 for computing the friction capacity of building foundations that bear on compacted crushed rock pads and 0.35 for footings bearing on native soils. An equivalent fluid unit weight of 250 pounds per cubic foot (pcf) is recommended to compute the passive earth pressure acting on footings constructed in direct contact with compacted structural fill or native soils. This value is based on the assumptions that the adjacent confining structural fill or native soils is level and that groundwater remains below the base of the footing. The top 1 foot of soil should be neglected when calculating lateral earth pressures unless the foundation area is covered with pavement or is inside a building.

The passive and frictional resistance may be combined provided that the passive component does not exceed two-thirds of the total. These values do not include a factor of safety. We recommend a safety factor of 3 when designing for dead loads plus frequently applied live loads and a safety factor of 2 be applied when considering transitory loads, such as wind and seismic forces.

UNDERPINNING AND UPLIFT

Underpinning may be required to provide support for the additional loads on and provide additional uplift resistance for the existing spread footings in the stair cores. The uplift capacity of spread footings can be estimated using the dead weight of the soils overlying the foundation and within a plane inclined at 30 degrees from vertical out from the mat perimeter. We recommend that a unit weight of 110 pcf for the backfill above the footings. We have provided preliminary recommendations for use in selection of the appropriate foundation system.

Micropiles or uplift anchors can also be used to support uplift loads. Based on our experience in the site vicinity, an 8-inch-diameter micropile is capable of supporting working loads of 3 to 5 kips per foot of bonded pile length in the silt that underlies the site. Up to 10 kips per foot is possible in the gravel that underlies the silt.

A wide variety of construction techniques are available for construction of these types of foundations. Consequently, we recommend that the foundation bid documents be performance based. The required allowable loads and deflection tolerances should be included in the project specifications, and the contactor should be responsible for selecting the appropriate system that meets the project requirements. The bid documents should also include load testing requirements to verify that the design loads have been achieved.

SEISMIC CONSIDERATIONS

Seismic upgrade of the facility will be performed in accordance with the guidelines in the ASCE 41-06 document. The following sections provide seismic considerations for use in evaluation of the facility.

SEISMIC DESIGN CRITERIA

We understand that the building retrofit will be designed using the ASCE/SEI 41-06 Standard. The seismic design parameters prescribed by this document are based on the 2003 National Earthquake Hazards Reduction Program Seismic Design Provisions. The parameters in Table 1 can be used to compute seismic base shear forces.

Table 1. Seismic Design Parameters

Parameter	0.2 Seconds	1 Second
Spectral Acceleration - 2 percent in 50 years[1]	$S_s = 0.986$ g	$S_1 = 0.385$ g
Spectral Acceleration - 10 percent in 50 years[1]	$S_s = 0.450$ g	$S_1 = 0.176$ g
Site Class	D	
Site Coefficient	$F_a = 1.106$	$F_v = 1.707$

1. Probability of exceedance

SURFACE FAULT RUPTURE

The Portland Hills Fault is mapped approximately 3,000 feet to the east of the site (Beeson, et al., 1991; Madin, 1990). The mapped location is based on limited deep borehole data and geophysical data showing offset of the Troutdale Formation and the Columbia River Basalt in the Portland Basin. The fault trace is not exposed in the vicinity of the site nor has fault surface rupture previously been documented. In addition, the evidence of fault offset of Holocene (less than 10,000 years) sediments is limited and not conclusive.

In our opinion, the location of the Portland Hills Fault has not been accurately constrained in the site vicinity and likely does not trend within the site boundary. The hazard for fault surface rupture at the site cannot be accurately assessed given the limited geologic evidence and documented seismic history of the Portland Hills Fault. In addition, it is our opinion that strong evidence for recent (less than 10,000 years) movement of the fault has not been established and the potential for surface rupture of the fault is not considered a seismic hazard at this site. We conclude that the probability of surface fault rupture beneath site is low.

LIQUEFACTION AND LATERAL SPREADING

Liquefaction is caused by a rapid increase in pore water pressure that reduces the effective stress between soil particles to near zero. Granular soils, which rely on interparticle friction for strength, are susceptible to liquefaction until the excess pore pressures can dissipate. In general, loose, saturated sand soils with low silt and clay contents are the most susceptible to liquefaction. Silty soils with low plasticity are moderately susceptible to liquefaction under relatively higher levels of ground shaking.

Based on our experience in the area, groundwater is anticipated to be deeper than 90 feet BGS. We conclude that that the risk of liquefaction and associated lateral spread at the site are considered low under design levels of ground shaking.

OBSERVATION OF CONSTRUCTION

Sufficient observation of the contractor's activities is a key part of determining that the work is completed in accordance with the construction drawings and specifications. Consequently, we recommend that GeoDesign be retained to observe all geotechnical construction.

Subsurface conditions observed during construction should be compared with those assumed in our analysis. Recognition of changed conditions often requires experience; therefore, qualified personnel should visit the site with sufficient frequency to detect whether subsurface conditions change significantly from those anticipated.

LIMITATIONS

We have prepared this report for use by Portland State University, and members of the design and construction team for the proposed building seismic retrofit project. The data and report may be used for bidding or estimating purposes, but our report, conclusions, and interpretations should not be construed as a warranty of the subsurface conditions.

We have made recommendation based on a subsurface exploration completed at the site and adjacent sites that indicates the soil conditions at only the specific location and only to the depths penetrated. These observations do not necessarily reflect soil types, strata thickness, or water level variations that may exist away from the exploration. If subsurface conditions differing from those described are observed during the course of excavation and construction, re-evaluation will be necessary.

When the design has been finalized, we recommend that the final design and specifications be reviewed by our firm to confirm that our recommendations have been interpreted and implemented as intended. If there are changes in the grades, location, configuration, or type of construction for the buildings, the conclusions and recommendations presented may not be applicable. If design changes are made, we request that we be retained to review our conclusions and recommendations and to provide a written modification or verification.

The scope of our services does not include services related to construction safety precautions, and our recommendations are not intended to direct the contractor's methods, techniques, sequences or procedures, except as specifically described in our report for consideration in design.

Within the limitations of scope, schedule, and budget, our services have been executed in accordance with the generally accepted practices in this area at the time this report was prepared. No warranty or other conditions, expressed or implied, should be understood.

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We appreciate the opportunity to be of continued service to you. Please call if you have questions concerning this report or if we can provide additional services.

Sincerely,

GeoDesign, Inc.

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One copy submitted
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