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GEODESIGN INC

REPORT OF GEOTECHNICAL ENGINEERING SERVICES

Block 8L
NW 1st Avenue and NW Davis Street
Portland, Oregon

For
Gerding Edlen Development Company
July 16, 2014

GeoDesign Project: Gerding-184-01

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GEODESIGN INC

July 16, 2014

Gerding Edlen Development Company
1477 NW Everett Street
Portland, OR 97209

Attention: Ms. Jill Sherman

Report of Geotechnical Engineering Services
Block 8L
NW 1st Avenue and NW Davis Street
Portland, Oregon
GeoDesign Project: Gerding-184-01

GeoDesign, Inc. is pleased to submit our report for the proposed mixed-use development at Block 8L in Portland, Oregon. Our services for this project were conducted in accordance with our revised proposal dated May 22, 2014.

We appreciate the opportunity to be of continued service to you. Please contact us if you have questions regarding this report.

Sincerely,

GeoDesign, Inc.

Brett A. Shipton, P.E., G.E.
Principal Engineer

cc: Ms. Jennifer Jenkins, Ankrom Moisan Architects (via email only)
Ms. Shirley Chalupa, DCI Engineers (via email only)
Mr. Dave Humber, MGH Associates (via email only)

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TABLE OF CONTENTS		PAGE NO.

TABLE OF CONTENTS	PAGE NO.

1.0 INTRODUCTION

This report presents the results of GeoDesign's geotechnical engineering evaluation for the proposed mixed-use development at Block 8L in Portland, Oregon. Block 8L is located southeast of the intersection of NW Davis Street and NW 1st Avenue. The site location is shown relative to surrounding features on Figure 1. For your reference, definitions of acronyms used herein are defined at the end of this document.

2.0 PROJECT UNDERSTANDING

The site is currently occupied by an asphalt-paved parking lot on the northern half and vacant grass-covered area on the southeast quarter. It is our understanding that the proposed development will consist of demolishing the existing parking lot and constructing an approximately six-level structure. As currently planned the structure will be five levels of wood-frame construction over one level of concrete construction. We further understand that the building will be constructed at grade; no basement is planned. Shallow soil on portions of the site is impacted with low levels of arsenic and lead and will be capped as part of the proposed development. Based on information provided, by DCI Engineers, we understand that maximum dead plus live column loads are 315 kips. Maximum seismic loads of 1,530 kips are expected in compression and tension. Wall loads will range from 8 to 11 kips per foot. Floor loads are expected to be less than 175 psf.

3.0 PURPOSE AND SCOPE

The purpose of our services was to explore subsurface conditions and provide geotechnical engineering recommendations for use in design and construction of the proposed structure. The specific scope of our services completed is summarized as follows:

- Reviewed readily available published geologic data and our in-house files for existing information on subsurface conditions in the site vicinity.
- Researched COP archives for pertinent geotechnical site information, including existing nearby facilities
- Reviewed the geotechnical report for the site provided by Gerding Edlen for the proposed development.
- Coordinated and managed the field investigation, including utility checks, site access authorizations, access preparation, and scheduling of subcontractors and GeoDesign field staff.
- Explored subsurface conditions by drilling two borings to depths of up to 75.5 feet BGS. The borings were drilled using a combination of hollow-stem auger and mud-rotary drilling equipment.
- Installed a vibrating wire piezometer in one of the borings to measure groundwater levels.
- Completed infiltration testing in both of the borings. Infiltration testing was conducted for shallow infiltration facilities at a depth of 5.0 feet BGS and deeper infiltration facilities at a depth of 15.0 feet BGS as specified by MGH Associates.
- Classified the materials encountered in the explorations and maintained a detailed log of each exploration.

- Completed laboratory analyses on disturbed and undisturbed soil samples obtained from the explorations as follows:
  - Nineteen moisture content determinations on selected soil samples in general accordance with ASTM D 2216
  - Two grain-size analysis on selected soil samples in general accordance with ASTM D 1140
  - Three Atterberg limits tests on selected soil samples in general accordance with ASTM D 4318
- Provided recommendations for site preparation, grading and drainage, fill type for imported material, compaction criteria, trench excavation and backfill, use of on-site soil, and wet weather earthwork.
- Provided geotechnical engineering recommendations for design and construction of foundations for support of the building.
- Provided recommendations for preparation of the subgrade for floor slabs.
- Recommended design criteria for retaining walls, including lateral earth pressures, backfill, compaction, and drainage.
- Provided recommendations for the management of identified groundwater conditions that may affect the performance of structures or pavement.
- Provided seismic design recommendations in accordance with the procedures outlined in the 2012 IBC.
- Prepared this report of our explorations, findings, conclusions, and recommendations.

4.0 SITE CONDITIONS

4.1 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.

4.2 SURFACE CONDITIONS

The site consists of an L-shaped parcel located southeast of the intersection of NW Davis Street and NW 1st Avenue. The site is currently developed with an asphalt-paved parking lot on the northern half and vacant grass lot on the southeast quarter. Surrounding site development primarily consists of retail and office buildings. The topography of the site is relatively level. Site elevations range from 32.2 feet on the northwest corner downward to 30.2 feet relative to the COP datum on the southeastern corner.

4.3 SUBSURFACE CONDITIONS

4.3.1 General

Our understanding of subsurface conditions is based on our review of previous geotechnical engineering studies conducted at the site and by drilling two borings (B-1 and B-2) to depths of up to 75.5 feet BGS (elevation -44.1 to -45.1 feet, relative to COP datum) on the southeastern portion of the site at the approximate locations shown on Figure 2. Descriptions of the field explorations and laboratory testing and logs of the explorations are provided in Appendix A of this report.

The borings drilled for this study were drilled on the southeast portion of the site to supplement previous 2003 explorations drilled by others on the north half of the site. A site plan showing boring locations and the boring logs and laboratory data from the previous explorations on the northern portion are provided in Appendix B of this report.

The geologic units encountered during our subsurface explorations generally consist of fill overlying alluvial silt, sand, and gravel.

4.3.2 Fill

Fill was encountered in our recent borings drilled on the southeast quadrant of the city block and borings drilled in 2003 on the north half of the block. The fill is variable and generally extends to depths of approximately 7.5 to 12 feet BGS in the borings, with the exception of boring B-5 drilled by Shannon & Wilson on the northeast corner of the site. This boring encountered utility trench backfill to depths of up to 21.5 feet BGS and was terminated due to the presence of a sewer pipe. The fill generally consists of loose to medium dense gravel and sand with debris and varying amounts of silt and medium stiff to stiff silt with gravel, clay, and sand. We expect the fill to be highly variable across the site. Laboratory tests of samples collected from our recent borings of the fill indicate the moisture content ranged from 4 to 15 percent at the time of our explorations.

4.3.3 Alluvial Silt and Sand

Alluvial silt and sand was encountered below the fill in most borings across the site to depths of approximately 22 to 38 feet BGS and to a depth of 61 feet BGS in 2003 boring B-5 on the northeast corner of the site. The silt has varying amounts of fine sand and the sand has varying a fraction of silt. Soil consistency of the silt ranges from soft to stiff based on SPTs. Relative density of the sand ranges between loose and medium dense based on SPTs. This unit represents alluvium deposited along the Willamette River and may include fine-grained Missoula Flood deposits overlain by younger alluvium.

Laboratory tests of samples of the alluvial silt collected during our recent investigation indicate the moisture content ranged from 15 to 40 percent at the time of our explorations. Atterberg limits testing indicate the silt generally has a plasticity index of 8 percent.

4.3.4 Alluvial Gravel

Brown to gray alluvial gravel was encountered underlying the alluvial silt and sand in borings B-1 and B-2 drilled for this study. The gravel is encountered to depth of 50.5 and 45.0 feet BGS in borings B-1 and B-2, respectively. The borings drilled in 2003 encountered the alluvial gravel to a maximum depth of 82 feet BGS in boring B-5. SPTs show that this gravel is generally very dense. Laboratory tests of samples of the gravel collected during our recent investigation indicate a moisture content between 9 and 41 percent at the time of our exploration.

4.3.5 Older Alluvial Silt and Clay

Alluvial silt and clay was encountered underlying the alluvial gravel in boring B-1 and B-2 drilled for this study. This unit was encountered to depths of 62.5 and 62.0 feet BGS in borings B-1 and B-2, respectively. SPTs show that this unit is generally very stiff in consistency in the borings drilled for this study. Laboratory tests of samples of this unit collected during our recent investigation indicate a moisture content of between 19 and 43 percent at the time of our exploration. This unit was also encountered in boring B-3 during the 2003 study.

4.3.6 Very Dense Gravel

Very dense gravel was encountered in borings B-1 and B-2 conducted for this study underlying the alluvial silt and clay. We interpret this unit as the Troutdale Formation. This unit was also encountered at depths of 62.5 and 82.0 feet BGS in borings B-2 and B-5, respectively, during the 2003 study. SPTs show that gravel is very dense. Laboratory tests show that the moisture content of this unit varied between 10 and 15 percent at the time of our exploration program.

4.3.7 Groundwater

A piezometer was installed in boring B-2 located in near the central portion of the site and depth to groundwater was measured at 20.1 feet BGS on June 19, 2014, approximately one week after installation. Perched water may be present at shallower depths. We anticipate that groundwater levels could rise substantially above the observed levels during extended wet periods and the water level in the Willamette River.

4.4 INFILTRATION TESTING

Infiltration tests were conducted in borings B-1 and B-2 at depths of 5.0 and 15.0 feet BGS. Infiltration testing was conducted in general accordance with the COP Bureau of Environmental Services Stormwater Management Manual.

Soil samples were collected directly below the depth of the infiltration test for particle-size analysis. Table 1 presents a summary of infiltration test results and fines content of the sample. The exploration logs and particle-size analyses are presented in Appendix A.

Table 1. Infiltration Test Results

Exploration	Depth (feet BGS)	Observed Infiltration Rate (inches/hour)	Fines Content[1] (percent)
B-1	5.0	60	5
	15.0	negligible	Not measured
B-2	5.0	6.4	4
	15.0	negligible	Not measured

1. Fines content: material passing the U.S. Standard No. 200 Sieve as determined by ASTM D 1140

5.0 CONCLUSIONS

Based on our review of available information and the results of our explorations, laboratory and field testing, and analyses, the site can be developed as proposed. The following items will have an impact on design and construction:

- The structure can be supported on a driven or drilled deep foundation system, or foundations underlain by ground a ground improvement system.
- The site is currently developed with an asphalt-paved parking lot on the northern half and vacant grass lot on the southeastern portion. Remnant foundation elements from previous structures, if any, will require removal and backfill with structural fill.
- The site soil is sensitive to moisture and is easily disturbed when at a moisture content that is above optimum. The subgrade should be protected from construction traffic.
- The proposed structure and excavations for foundation elements will be directly adjacent to existing structures along the west side. Foundation types and depths of any existing structures adjacent to the proposed development should be determined prior to any excavation. Shoring may be required to protect existing improvements.
- The fill soil will be prone to raveling and caving, and shoring will be necessary if vertical cuts will be excavated near existing improvements.
- Groundwater was measured at a depth of approximately 20 feet BGS. Excavations for drilled foundations will require casing or drilling fluid to maintain an open hole.

The following sections present general recommendations based on evaluation of results from the previous geotechnical investigation and our understanding of the proposed project.

6.0 SITE DEVELOPMENT RECOMMENDATIONS

6.1 SITE PREPARATION

Demolition includes removal of the existing asphalt-paved parking lot, concrete curbs, and abandoned utilities. Demolished material should be transported off site for disposal. Excavations remaining from removing the floor slab, foundations, utilities, environmental excavations, and other subsurface elements should be backfilled with structural fill where below planned site grades. Excavation bases should expose firm subgrade before backfilling. The sides of the excavations should be cut into firm material and sloped a minimum of 1.5H:1V. Utility lines abandoned under new structural components should be completely removed and backfilled with structural fill. Soft or loose soil encountered during site preparation should be replaced with structural fill.

6.2 CONSTRUCTION CONSIDERATIONS

Conventional earthmoving equipment in proper working condition should be capable of making necessary excavations for site cuts and utilities. The sandy soil is prone to raveling, and shoring will be required to maintain vertical excavation walls and protect adjacent facilities. Recommendations for use in shoring design are provided in subsequent sections of this report.

Groundwater was measured at a depth of approximately 20.1 feet BGS in a vibrating wire piezometer completed at the site. Dewatering might be required to control perched groundwater conditions encountered during excavation for below-grade structures, if constructed. We anticipate that perched groundwater flow, if encountered, will diminish over time and can be addressed using sumps and pumps internal to the excavation.

We recommend that the contractor protect the subgrade by placing a layer of stabilization material over it. The contractor has control of the construction schedule and equipment and, therefore, should be responsible for selecting the appropriate working blanket and thickness. However, in our opinion, a 12-inch thickness should be sufficient for light staging areas and an 18-inch-thick blanket for areas subject to heavy construction traffic. Stabilization material should consist of well-graded gravel, crushed gravel, or crushed rock with a minimum particle size of 3 inches and less than 5 percent by dry weight passing the U.S. Standard No. 4 Sieve. Stabilization material should be placed in one lift.

Excavations should be made in accordance with applicable OSHA and state regulations. While this report describes certain approaches to excavation and dewatering, the contractor should be responsible for selecting excavation and dewatering methods, monitoring the excavations for safety, and providing shoring, as required to protect personnel and adjacent utilities and structures.

6.3 EXCAVATION

Excavation may be required for foundation elements, elevator pits, utilities, and stormwater tanks. The excavations may be completed using open-cut methods in the absence of adjacent structures, pavements, and sidewalks. Due to the presence of sandy soil, even shallow excavation side walls may not stand vertical. Open excavation techniques may be used with depths up to 10 feet with temporary 1.5H:1V slopes, provided groundwater seepage does not occur and with the understanding that some sloughing may occur. Excavation sides should be flattened if excessive sloughing occurs.

Shoring will be required for temporary excavations adjacent to settlement-sensitive elements such as buildings, pavements, sidewalks, and utilities. Excavations where the sides cannot be sloped back as described above will also require temporary shoring. Recommendations for design of temporary shoring are provided in the following section.

Excavations will likely terminate in the alluvium that is sensitive to disturbance and easily damaged by standard construction equipment. We recommend that excavation subgrades be protected from disturbance as described in "Construction Considerations" section of this report.

6.4 TEMPORARY SLOPES

The use of temporary slopes during construction will likely not be possible because of site constraints. Where construction slopes are possible, excavation side slopes less than 10 feet high should be no steeper than 1.5H:1V, provided groundwater is not present. If slopes greater than 10 feet high are required, GeoDesign should be contacted to make additional recommendations. We recommend a minimum horizontal distance of 5 feet from the edge of the existing improvements to the top of the temporary slope. All cut slopes should be protected from erosion by covering them during wet weather. If sloughing or instability is observed, the slope should be flattened or the cut supported by shoring.

6.5 STRUCTURAL FILL

Structural fill includes fill beneath foundations, slabs, pavements, any other areas intended to support structures, or within the influence zones of structures. Structural fill should be free of organic matter and other deleterious material and, in general, should consist of particles no larger than 3 inches in diameter. Recommendations for suitable fill material are provided in the following sections.

6.5.1 On-Site Soil

The on-site native soil will be suitable for use as structural fill only if it can be moisture conditioned. Based on our experience, the on-site silty soil is sensitive to small changes in moisture content and may be difficult to compact adequately during wet weather or when its moisture content is more than a few percentage points above optimum.

If the aggregate base beneath the pavement can effectively be separated from the underlying silt, it will be suitable for use as structural fill.

6.5.2 Select Granular Fill

Granular material for use as structural fill should be pit- or quarry-run rock, crushed rock, or crushed gravel and sand that is fairly well graded between coarse and fine and has less than 5 percent by dry weight passing the U.S. Standard No. 200 Sieve. Granular fill used during periods of prolonged dry weather may have up to 10 percent by dry weight passing the U.S. Standard No. 200 Sieve provided it is properly moisture conditioned.

6.5.3 Pipe Bedding

Utility trench backfill for bedding and in the pipe zone should consist of well-graded granular material with a maximum particle size of ¾ inch and less than 5 percent by dry weight passing the U.S. Standard No. 200 Sieve or as required by the pipe manufacturer.

6.5.4 Crushed Rock

Crushed rock will be required as base material for floor slabs and pavements as specified. Crushed rock fill should consist of imported clean, durable, crushed, angular rock that meets the requirements of the pertinent sections of this report.

6.6 FILL PLACEMENT AND COMPACTION

Fill soil should be compacted at a moisture content that is near optimum. The maximum allowable moisture content varies with the soil gradation and should be evaluated during construction.

Fill and backfill material should be placed in uniform, horizontal lifts and compacted with appropriate equipment. The maximum lift thickness will vary depending on the material and compaction equipment used but should generally not exceed the loose thicknesses provided in Table 2. Fill material should be compacted in accordance with the compaction criteria provided in Table 3.

Table 2. Recommended Uncompacted Lift Thickness

Compaction Equipment	Recommended Uncompacted Lift Thickness (inches)		
	Silty Soil	Granular and Crushed Rock Maximum Particle Size ≤ 1½ Inches	Crushed Rock Maximum Particle Size > 1½ Inches
Hand Tools: Plate Compactor and Jumping Jack	4 to 8	4 to 8	Not Recommended
Rubber-Tired Equipment	6 to 8	10 to 12	6 to 8
Light Roller	8 to 10	10 to 12	8 to 10
Heavy Roller	10 to 12	12 to 18	12 to 16
Hoe Pack Equipment	12 to 16	18 to 24	12 to 16

Table 2 is based on our experience and is intended to serve only as a guideline. The information provided in this table should not be included in the project specifications.

Table 3. Compaction Criteria

Fill Type	Compaction Requirements in Structural Zones — Percent Maximum Dry Density Determined by ASTM D 1557		
	0 to 2 Feet Below Subgrade (percent)	> 2 Feet Below Subgrade (percent)	Pipe Zone (percent)
Area Fill	95[1]	92	-----
Aggregate Bases	95	95	-----
Trench Backfill	95[1]	92	90[2]
Retaining Wall Backfill	95[1,3]	92[3]	-----

1. May be reduced to 92 percent if native soils are used.
2. Or as recommended by the pipe manufacturer.
3. Should be reduced to 90 percent within a horizontal distance of 3 feet from the retaining wall.

6.6.1 Area Fills

Imported fill placed to raise site grades should be placed on a prepared subgrade that consists of firm, inorganic site soil or compacted fill. The fill material should be placed in uniform horizontal lifts and compacted to the recommended minimum density provided in Table 3.

6.6.2 Aggregate Bases

Aggregate base materials under foundations and floor slabs should be placed on a prepared subgrade that consists of firm, inorganic native soil or compacted fill. Aggregate base material should be placed in uniform horizontal lifts and compacted to the recommended minimum density provided in Table 3.

6.6.3 Trench Backfill

Trench backfill in structural areas should consist of select granular fill or crushed rock as described in the "Structural Fill" section of this report and be compacted to the minimum density provided in Table 3. Pipe bedding and fill in the pipe zone should be compacted to the minimum density presented in Table 3 or as recommended by the pipe manufacturer.

6.6.4 Retaining Wall Backfill

Retaining wall backfill should be compacted to the recommended minimum density provided in Table 3, except that fill within 3 horizontal feet of the wall should be placed in uniform horizontal lifts and compacted to a lesser density of 90 percent of the maximum density, as determined by ASTM D 1557, to reduce the effect of compaction-induced stresses against the retaining wall.

Settlement of up to 1 percent of the wall height commonly occurs immediately adjacent to retaining walls as the walls rotate and develop lateral active earth pressures. Consequently, we recommend that flatwork (slabs, sidewalks, or pavement) placed adjacent to retaining walls be postponed at least four weeks following wall construction, unless survey data indicates that settlement is complete prior to that time.

6.7 PERMANENT CUT AND FILL SLOPES

Permanent cut and fill slopes in the site soil should be inclined no steeper than 2H:1V. Buildings, access roads, and pavements should be set back a minimum of 5 feet from the crest of any such slopes.

6.8 EROSION CONTROL

The on-site soil is moderately susceptible to erosion. Consequently, we recommend that slopes be covered with an appropriate erosion control product if construction occurs during periods of wet weather. We recommend that all slope surfaces be planted as soon as practical to minimize erosion. Surface water runoff should be collected and directed away from slopes to prevent water from running down the slope face. Erosion control measures such as straw bales, sediment fences, and temporary detention and settling basins should be used in accordance with local and state ordinances.

7.0 FOUNDATION SUPPORT RECOMMENDATIONS

The proposed structure can be satisfactorily supported on deep foundations. Deep foundation alternatives considered include driven grout piles, closed-ended steel pipe piles, drilled shafts that extend into the very dense gravel or a mat foundation underlain by controlled modulus piers or stone columns extending to the very dense gravel. The depth and continuity of the upper alluvial gravel and Troutdale Formation varies across the site. The selected piling installation techniques should be adaptable in the field to avoid delays in construction. Our recommendations for use in foundation design and construction are provided in the following sections.

7.1 DRIVEN PILING

7.1.1 Axial Capacity

We have developed design recommendations for 12-inch-diameter driven steel pipe piles and 16-inch-diameter driven grout piles. The piles will achieve the majority of their compressive capacity from end bearing in the underlying very dense gravel at a depth of 65 feet BGS. Penetrating the upper alluvial and very dense gravel may be problematic with driven pile systems. Therefore, achieving the required uplift capacity may be problematic.

Figure 3 presents a capacity profile for 12-inch-diameter steel pipe piles. Figure 4 presents a capacity profile for 16-inch-diameter driven grout piles. We recommend a minimum pile spacing of 3 pile diameters on center. Pipe piles should be driven closed-ended with a steel plate.

Pile lengths may vary depending on the consistency of the bearing stratum at pile locations across the site. It is reasonable to assume that some piles may terminate at different depths than shown on Figures 3 and 4 for a given pile capacity.

Settlement of driven piling installed as recommended is anticipated to be negligible beyond the elastic pile deflection.

7.1.2 Other Considerations

Terminal blow counts will depend on the pile type and driving equipment chosen. If driven grout piles are selected, it should be verified that the mandrel has sufficient structural capacity to withstand the stresses induced by pile driving. GeoDesign should be consulted to select the appropriate hammer energy to develop the required capacity while avoiding excessive driving stresses. GeoDesign will develop the terminal blow criteria based on WEAP analysis considering the pile type, required capacity, and the selected driving equipment. If our analysis is verified in the field using a PDA, it will allow a reduction in the safety factor to 2.0 in compression 1.5 in tension.

Steel pipe piles should be driven to refusal in the underlying very dense gravel formation using a hammer having adequate energy to penetrate through the overlying soil and properly found the piles in the designated ground formation. Terminal blow counts will depend on the pile type and driving equipment chosen. It should be verified that the selected piles have sufficient structural capacity to withstand the stresses induced by pile driving. GeoDesign should be consulted to select the appropriate hammer energy to develop the required capacity while avoiding excessive driving stresses. GeoDesign will develop the terminal blow criteria based on WEAP analysis considering the pile type, required capacity, and the selected driving equipment.

The piling should be installed with suitable alignment tolerances. Vertical alignment should be within 3 percent of plumb or as determined by the structural engineer. Lateral alignment should be within tolerances determined by the structural engineer, considering the pile cap design. Settlement of piles driven to refusal in the very dense gravel will be negligible beyond the elastic compression of the pile.

If buried obstructions are encountered during driving, the pile should be extracted and the obstruction removed. If the buried obstruction cannot be removed, the structural engineer should be consulted to select a new pile location. Each pile should be carefully inspected for damage caused by impacting buried obstructions during driving.

We recommend full-time monitoring of pile installation to confirm that the piles are driven in accordance with the recommendations in this report and with the project specifications.

7.1.3 Lateral Pile Response

Group action should be considered when the pile spacing in the direction of loading is less than 8 pile diameters on center. We can perform a lateral pile analysis when pile type, size, and reinforcing has been selected.

In addition to the lateral resistance provided by the piles, resistance to lateral loads also can be developed by passive pressure on the face of pile caps, grade beams, tie beams, and other buried foundation elements. Sliding friction on the base of pile-supported foundation elements should be ignored. A maximum equivalent earth pressure of 350 pcf should be used to compute the pile caps' resistance to lateral forces. This value should be adjusted based on the design lateral deflection using the values in Table 4.

Table 4. Lateral Earth Pressure Factors

Deflection/Pile Cap Height	Passive Resistance Reduction Factor
0	0
0.01	0.70
0.02	0.85
0.04	1.0

7.2 DRILLED SHAFT RECOMMENDATIONS

7.2.1 Axial Capacity

Drilled shafts will achieve the majority of their capacity in bearing in the very dense gravel encountered at a depth of 62.5 feet BGS in borings B-1 and B-2 drilled for this study. Figure 5 shows an allowable capacity profile for a 30-inch-diameter drilled shaft. We recommend a minimum embedment of 10 feet into the underlying dense gravel encountered at a depth of 62.5 feet BGS in borings B-1 and B-2 drilled in 2014. Group effects have not been included as it is assumed that drilled concrete piles will be spaced at least 3 diameters on center.

Settlement of drilled shafts installed as recommended is anticipated to be negligible beyond the elastic pile deflection.

7.2.2 Quality Control

Quality of shaft construction will be critical to provide acceptable settlement behavior. At a minimum, we recommend the following quality control measures:

- The base of the hole should be thoroughly cleaned and observed by a qualified geotechnical engineer.
- A qualified technician should be present during construction to verify subsurface conditions are as interpreted and the general design intentions are met during construction.
- Casing or drilling fluid or a combination thereof will likely be required to maintain an open hole in the soil that overlies the gravel.

Subsurface conditions consist of uncontrolled fill underlain by silt, in turn underlain by alluvial gravel. We anticipated that casing will be required for installation of drilled concrete piers through the upper uncontrolled fill and alluvial gravel.

Cobbles and boulders might also result in difficult drilled shaft excavations to “roll” around the auger and cause belling or caving of the shaft sidewalls. A core barrel, mud bucket, or other enclosed auger has proved successful on other jobs for removing cobbles and boulders from shaft excavations. Because of the presence of unengineered fill, soft silt, granular and very dense gravel deposits, and excavations that might extend below the groundwater table, the use of full-depth casing might be required to reduce caving of the drilled shaft excavations. Cobbles and boulders might cause casings to hang up during installation, also resulting in more difficult drilled shaft installation.

7.2.3 Lateral Pile Response

Group action should be considered when the pile spacing in the direction of loading is less than 8 pile diameters on center. We can perform a lateral pile analysis when pile type, size, and reinforcing has been selected. Lateral resistance of pile caps, grade beams, tie beams, and other buried foundation elements can be computed as outlined in the "Driven Piling" section of this report.

7.3 MAT FOUNDATION UNDERLAIN BY CONTROLLED MODULUS PIERS OR STONE COLUMNS

A mat foundation underlain by controlled modulus piers or stone columns is another foundation alternative to support the structure. Mat foundations have the advantage of reducing the contact pressures of a structure by dispersing load over the entire footprint of the building. However, we anticipate that consolidation of the fill and soft soil overlying the gravel will cause excessive settlement. Consequently, if a mat foundation is planned, we recommend that the underlying compressive soil be improved in order to increase the subgrade reaction modulus to an acceptable level. Soil improvement is typically designed and constructed by the specialty ground improvement contractor. We have provided guidelines that can be used for cost estimating purposes in the following sections.

7.3.1 Controlled Modulus Piers

A combined modulus of the soil and the CLSM elements is typically used to design the mat. A soil subgrade reaction modulus of equal to 100 pci should be used for the soil. The modulus of the CLSM will be based on the strength of the CLSM; we recommend a minimum strength of 800 psi. The combined modulus will depend on the spacing and dimension of the CLSM elements. A maximum spacing of 8 feet and a diameter equal to 30 inches is typical.

The CLSM elements should extend to the underlying gravel formation. Also, the CLSM element will be more rigid than the surrounding soil. Due to this variation between the two materials, additional stresses will be imposed on the mat. To reduce this effect, at least 2 feet of crushed rock is typically placed between the top of the CLSM elements and bottom of the mat. In addition, reinforcing may still be required in the mat to accommodate the additional stress.

7.3.2 Stone Columns

Stone columns will improve the compressible silt layers by densifying the soil in place. The stone columns should be installed beneath the area of the mat foundation. The stone columns should be placed in a triangular pattern for maximum efficiency of soil improvement. We anticipate that the tip of the stone columns extend into alluvial gravel. Stone columns typically have a nominal diameter of between 36 and 42 inches with a center-to-center spacing of 8 feet or less.

A subgrade modulus (k_s) of 300 pci may be achievable using stone column ground improvement. A dynamic modulus of 600 pci is likely to be achieved for transitory loads, such as wind and seismic forces.

We anticipate the total foundation settlement to be on the order of 1 inch to 2 inches for a mat foundation designed in accordance with the recommendations provided above. We anticipate that half of the settlement will occur as the building is constructed. We anticipate the stiffness of the mat will limit differential settlement to less than ¾ inch.

7.4 SLABS ON GRADE

Undocumented fill is present at the site and there is a potential for floor slab settlement as the fill consolidates. Provided a small risk of floor slab distress is acceptable, improvement can be limited to the upper 18 inches of the subgrade by compacting it in place or removing it and replacing it with structural fill.

A modulus of subgrade reaction of 100 pci can be used for design of the floor slabs provided the subgrade is prepared in accordance with the recommendations presented in this section. The native soil is non-expansive, so heave is not anticipated beneath the floor slab.

We recommend that the floor slab be supported on at least 6 inches of imported granular material to aid as a capillary break and to provide uniform support. The imported granular material should be placed and compacted as previously recommended for aggregate bases.

Vapor barriers beneath floor slabs are typically required by flooring manufactures to maintain the warranty on their products. In our experience, adequate performance of floor adhesives can be achieved by using a clean base rock (less than 5 percent fines) beneath the floor slab with no vapor barrier. In fact, vapor barriers can frequently cause moisture problems by trapping water beneath the floor slab that is introduced during construction. If a vapor barrier is used, water should not be applied to the base rock prior to pouring the slab and the work should be completed during extended dry weather so that rainfall is not trapped on top of the vapor barrier. Selection and design of an appropriate vapor barrier, if needed, should be based on discussions among members of the design team. We can provide additional information to assist you with your decision.

8.0 PERMANENT RETAINING STRUCTURES

Permanent retaining structures free to rotate slightly around the base should be designed for active earth pressures using an equivalent fluid unit weight of 35 pcf. This value should be increased to 55 pcf if the wall is restrained against rotation. These values are based on the assumption that (1) the retained soil is level, (2) drainage is provided behind the wall to prevent hydrostatic pressured from developing, and (3) the wall is less than 10 feet in height.

Lateral pressures induced by surcharge loads can be computed using the methods presented on Figure 6. Seismic lateral forces can be calculated using a dynamic force equal to $6.5H^2$ pounds per linear foot of wall, where H is the wall height. The seismic force should be applied as a distributed load with the centroid located at 0.6H from the wall base. Footings for retaining wall should be designed as recommended for shallow foundations.

Drains consisting of a perforated drainpipe wrapped in a geotextile filter should be installed behind exterior walls. The pipe should be embedded in a zone of coarse sand or gravel containing less than 2 percent by dry weight passing the U.S. Standard No. 200 Sieve and should outlet to a suitable discharge.

9.0 DRAINAGE

9.1 GENERAL

We recommend that roof drains be connected to a tightline leading to a suitable discharge. Pavement surfaces and open space areas should be sloped such that surface water runoff is collected and routed to suitable discharge points. We also recommend that ground surfaces adjacent to buildings be sloped to facilitate positive drainage away from the buildings.

9.2 INFILTRATION SYSTEM

Measured infiltration rates are provided in the "Infiltration Testing" section of this report. The values have not been factored to account for potential site variability and the limited number of tests performed but has not been factored for design. This value should be factored by the civil designer during design to account for the system size, the degree of long-term maintenance and influent/pre-treatment control, as well as the potential for long-term clogging due to siltation and bio-buildup, depending on the proposed length, location, and type of use of infiltration facility.

10.0 SEISMIC DESIGN CRITERIA

10.1 SEISMIC DESIGN PARAMETERS

Seismic design is prescribed by 2014 SOSSC and the 2012 IBC. Table 5 presents the site design parameters prescribed by the 2012 IBC for the site.

Table 5. Seismic Design Parameters

Parameter	Short Period (T_s = 0.2 second)	1-Second Period (T_1 = 1.0 second)
MCE Spectral Acceleration, S	S_s = 0.981 g	S_1 = 0.421 g
Site Class	D	
Site Coefficient, F	F_a = 1.108	F_v = 1.579
Adjusted Spectral Acceleration, S_M	S_{MS} = 1.086 g	S_{M1} = 0.665 g
Design Spectral Response Acceleration Parameters, S_D	S_{DS} = 0.724 g	S_{D1} = 0.443 g

10.2 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 soil, which relies on interparticle friction for strength, is susceptible to liquefaction until the excess pore pressures can dissipate. In general, loose, saturated sand soil with low silt and clay content is the most susceptible to liquefaction. Silty soil with low plasticity is moderately susceptible to liquefaction under relatively higher levels of ground shaking. Liquefaction is not significant design concern under design levels of ground shaking.

11.0 OBSERVATION OF CONSTRUCTION

Satisfactory earthwork and foundation performance depends to a large degree on the quality of construction. Subsurface conditions observed during construction should be compared with those encountered during the subsurface explorations. 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. In addition, 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.

12.0 LIMITATIONS

We have prepared this report for use by Gerding Edlen Development and members of the design and construction teams for the proposed development. The data and report can be used for estimating purposes, but our report, conclusions, and interpretations should not be construed as a warranty of the subsurface conditions and are not applicable to other sites.

Soil explorations indicate soil conditions only at specific locations and only to the depths penetrated. They do not necessarily reflect soil strata or water level variations that may exist between exploration locations. If subsurface conditions differing from those described are noted during the course of excavation and construction, re-evaluation will be necessary.
The site development plans and design details were not finalized at the time this report was prepared. When the design has been finalized and if there are changes in the site grades or location, configuration, design loads, or type of construction for the buildings, the conclusions and recommendations presented may not be applicable. If design changes are made, we should be retained to review our conclusions and recommendations and to provide a written evaluation or modification.

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, express or implied, should be understood.

♦ ♦ ♦

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.

Tacia C. Miller, P.E., G.E.
Senior Associate Engineer

Brett A. Shipton, P.E., G.E.
Principal Engineer

REFERENCES

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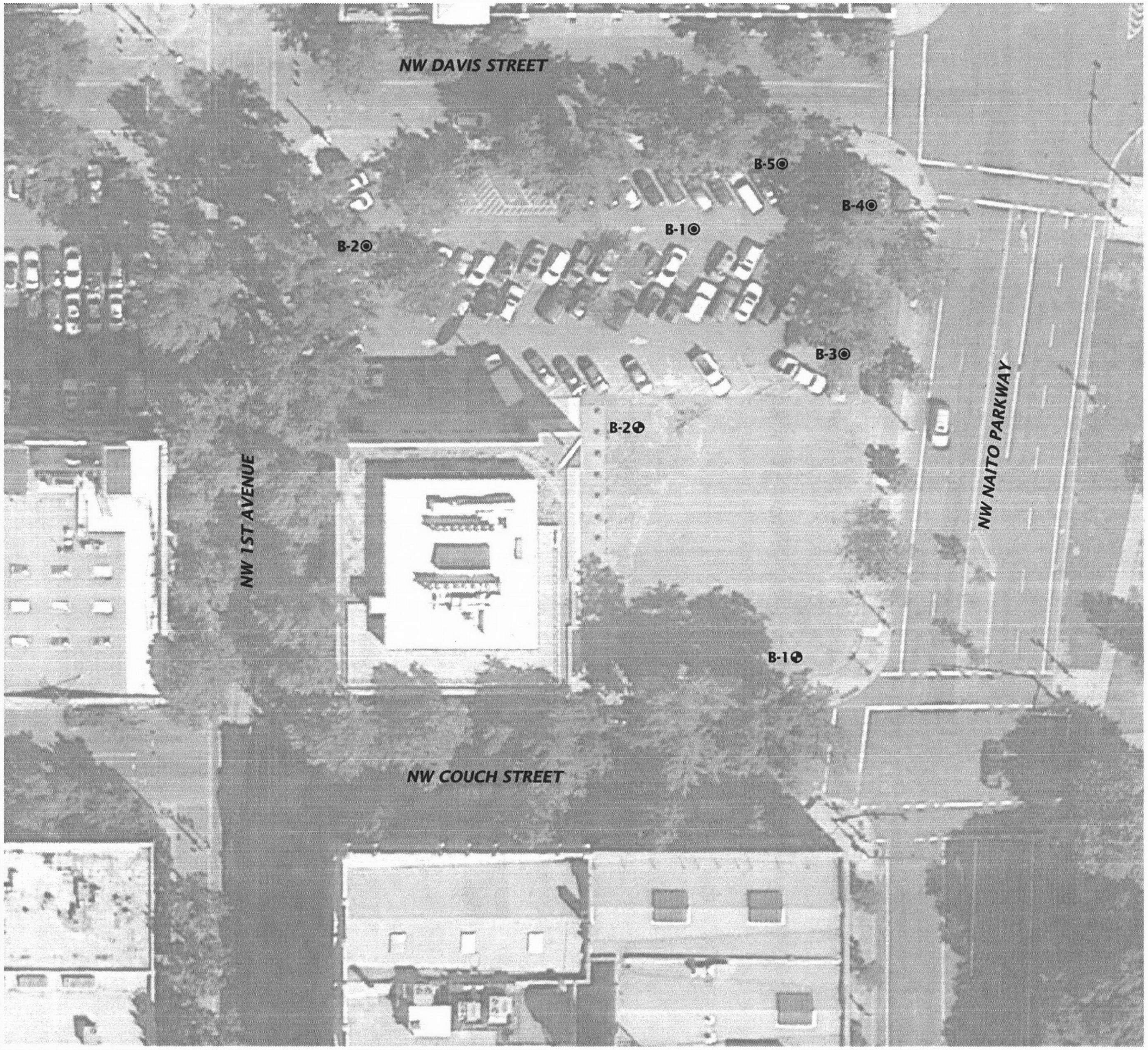
VICINITY MAP BASED ON AERIAL PHOTOGRAPH OBTAINED FROM GOOGLE EARTH PRO®

Printed By: aday | Print Date: 7/16/2014 11:30:06 AM
File Name: J:\E-L\Gerding\Gerding-184\Gerding-184-01\Figures\CAD\Gerding-184-01-VM01.dwg | Layout: FIGURE 1

GEODESIGN INC 15575 SW Sequoia Parkway - Suite 100 Portland OR 97224 Off 503.968.8787 Fax 503.968.3068	GERDING-184-01	VICINITY MAP	
	JULY 2014	BLOCK 8L PORTLAND, OR	FIGURE 1



LEGEND:

B-1 ◉ BORING (SHANNON & WILSON, AUGUST 2003)

B-1 ⊕ BORING (GEODESIGN, JUNE 2014)

N

0 40 80

(SCALE IN FEET)

SITE PLAN BASED ON AERIAL PHOTOGRAPH OBTAINED FROM GOOGLE EARTH PRO®, JUNE 17, 2014

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JULY 2014

SITE PLAN

BLOCK 8L
PORTLAND, OR

FIGURE 2

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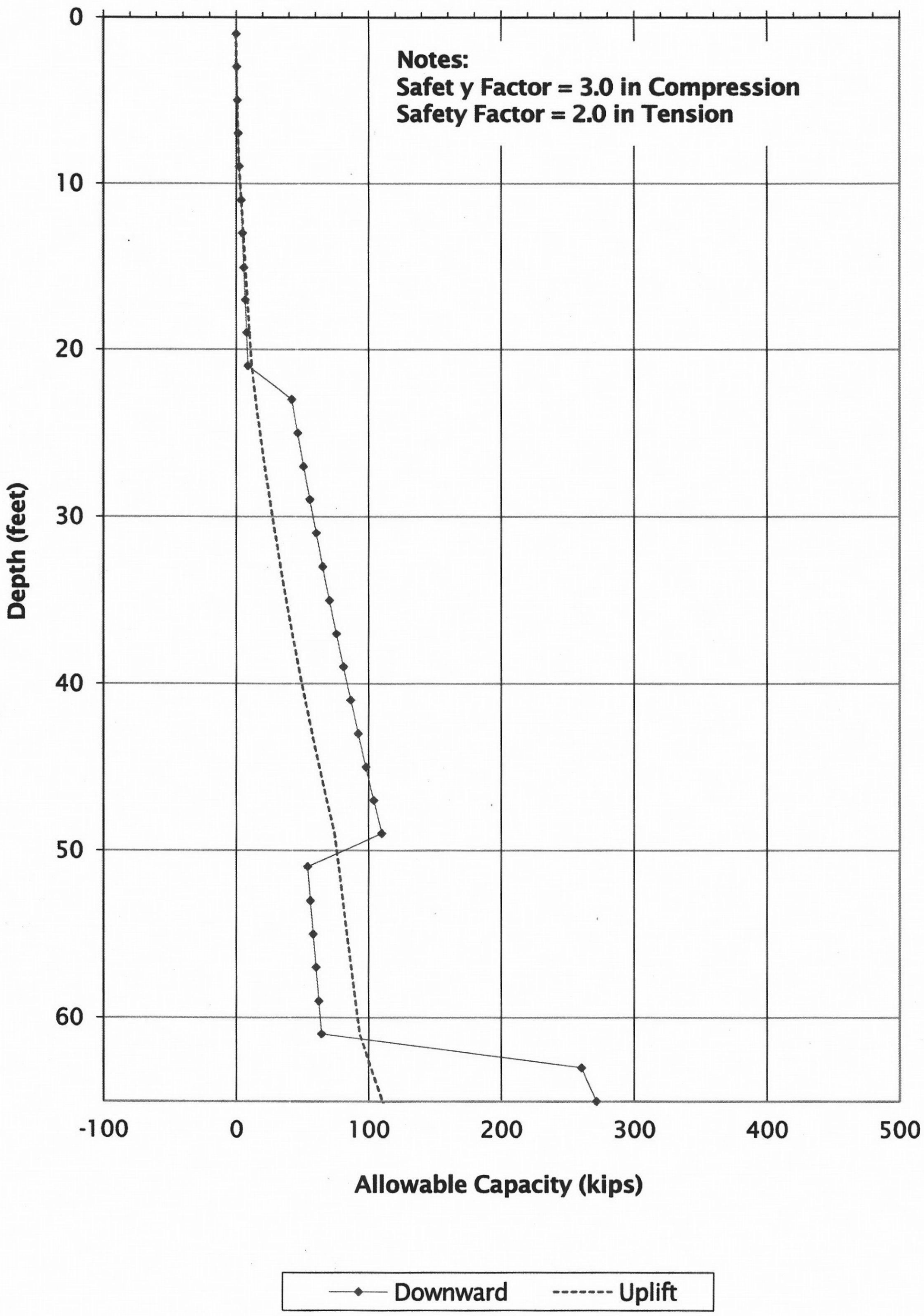

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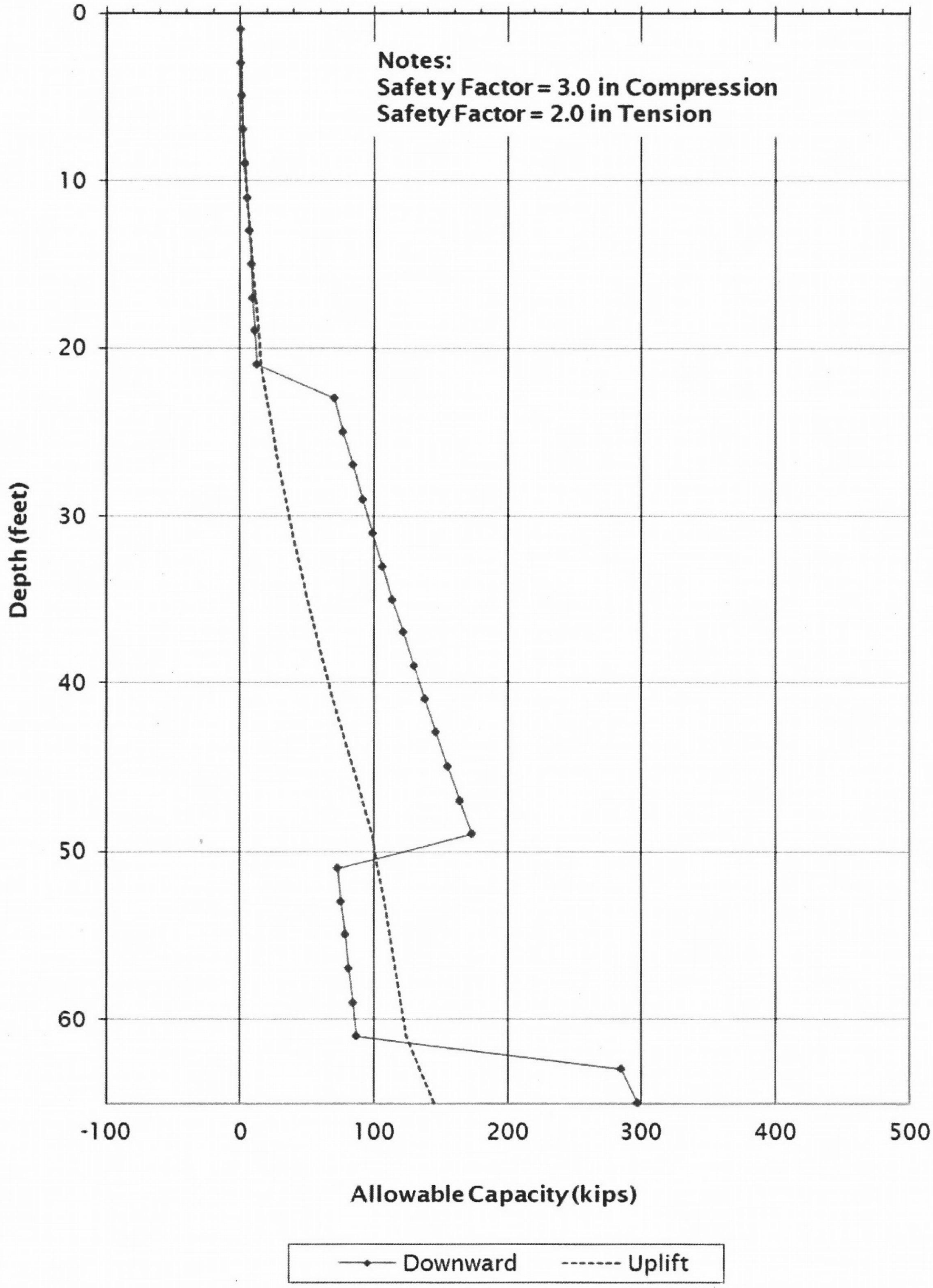
GEODESIGN INC 15575 SW Sequoia Parkway - Suite 100 Portland OR 97224 Off 503.968.8787 Fax 503.968.3068	GERDING-184-01	12-INCH-DIAMETER STEEL PIPE PILE CAPACITY PROFILE
	JULY 2014	BLOCK 8L PORTLAND, OR — FIGURE 3



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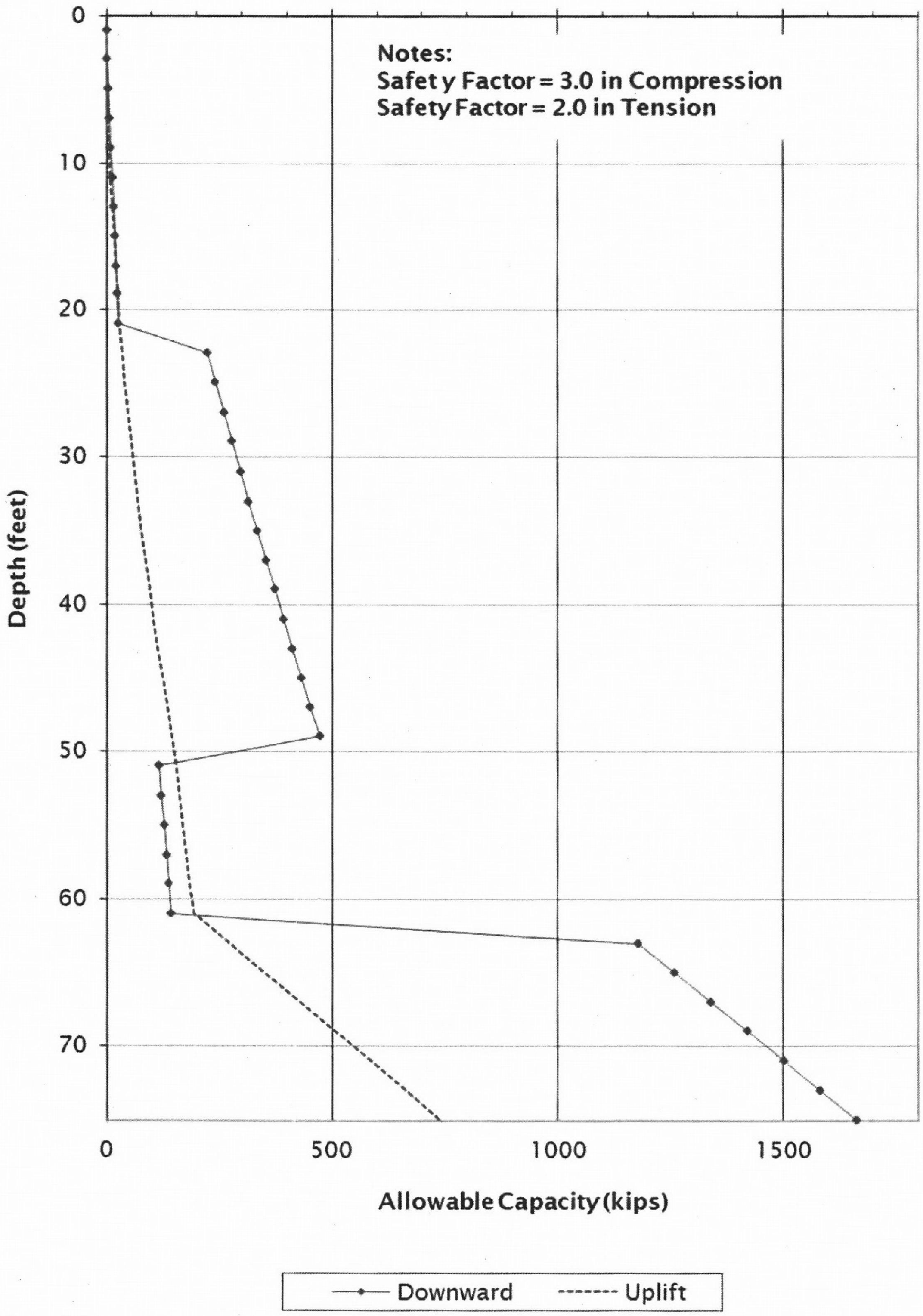
GEODESIGN INC 15575 SW Sequoia Parkway - Suite 100 Portland OR 97224 Off 503.968.8787 Fax 503.968.3068	GERDING-184-01	16-INCH-DIAMETER DRIVEN GROUT PILE CAPACITY PROFILE	
	JULY 2014	BLOCK 8L PORTLAND, OR	FIGURE 4



Gerding-184-01-F3_5.docx Print Date: 7/16/14

GEODESIGN INC 15575 SW Sequoia Parkway - Suite 100 Portland OR 97224 Off 503.968.8787 Fax 503.968.3068	GERDING-184-01	30-INCH-DIAMETER DRILLED SHAFT CAPACITY PROFILE	
	JULY 2014	BLOCK 8L PORTLAND, OR	FIGURE 5

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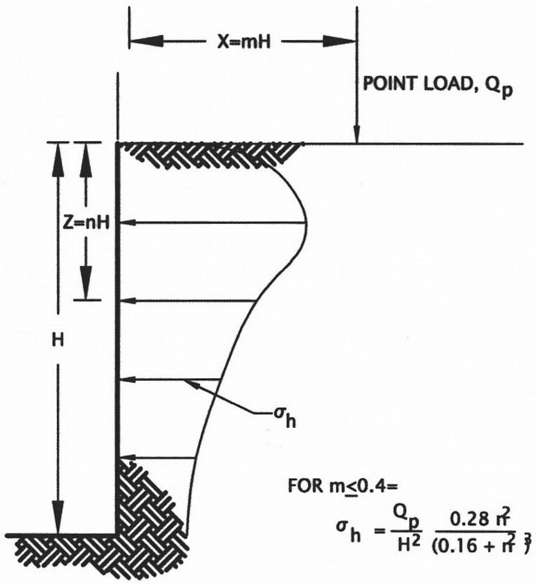
File Name: J:\E-L\Gerding\Gerding-184\Gerding-184-01\Figures\CAD\Gerding-184-01-DET01.dwg | Layout: FIGURE 6

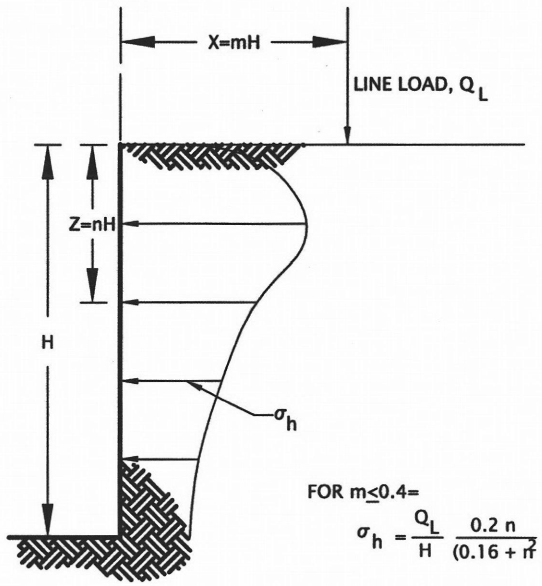


FOR $m \leq 0.4$=

$$\sigma_h = \frac{Q_p}{H^2} \frac{0.28\, n^2}{(0.16 + n^2)^3}$$

FOR $m > 0.4$=

$$\sigma_h = \frac{Q_p}{H^2} \frac{1.77 m^2 n^2}{(m^2 + n^2)^3}$$



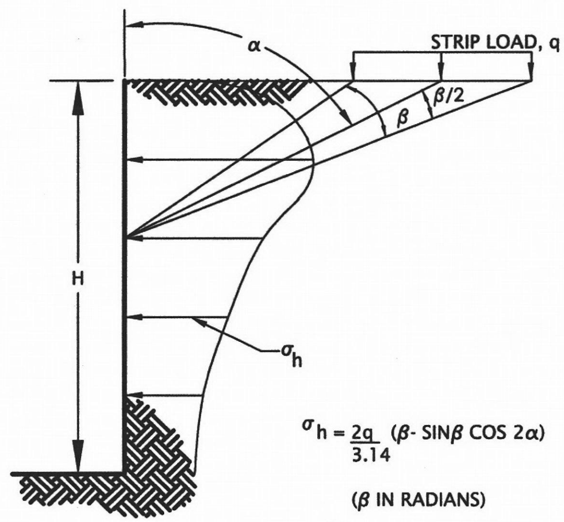
FOR $m \leq 0.4$=

$$\sigma_h = \frac{Q_L}{H} \frac{0.2\, n}{(0.16 + n^2)^2}$$

FOR $m > 0.4$=

$$\sigma_h = \frac{Q_L}{H} \frac{1.28 m^2 n}{(m^2 + n^2)^2}$$

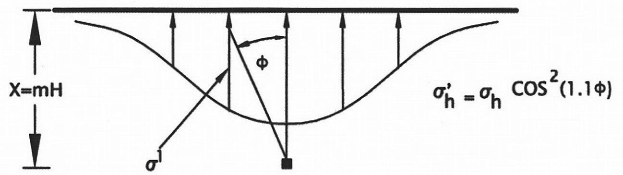
LINE LOAD PARALLEL TO WALL



$$\sigma_h = \frac{2q}{3.14} (\beta - \sin\beta \cos 2\alpha)$$

(β IN RADIANS)

STRIP LOAD PARALLEL TO WALL



$$\sigma'_h = \sigma_h \cos^2(1.1\phi)$$

DISTRIBUTION OF HORIZONTAL PRESSURES

VERTICAL POINT LOAD

NOTES:

1. THESE GUIDELINES APPLY TO RIGID WALLS WITH POISSON'S RATIO ASSUMED TO BE 0.5 FOR BACKFILL MATERIALS.
2. LATERAL PRESSURES FROM ANY COMBINATION OF ABOVE LOADS MAY BE DETERMINED BY THE PRINCIPLE OF SUPERPOSITION.
3. VALUES IN THIS FIGURE ARE UNFACTORED.

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GERDING-184-01	**SURCHARGE-INDUCED LATERAL EARTH PRESSURES**	
JULY 2014	BLOCK 8L PORTLAND, OR	**FIGURE 6**

APPENDIX A

FIELD EXPLORATIONS

GENERAL

We explored subsurface conditions at the site by drilling two borings (B-1 and B-2) to a maximum depth of 75.5 feet BGS. Figure 2 shows the approximate exploration locations. Borings were drilled on June 11 and 12, 2014 by Western States Soil Conservation, Inc. of Aurora, Oregon, using a combination of hollow-stem auger and mud-rotary drilling methods.

The exploration locations were located in the field by pacing from existing site features. This information should be considered accurate only to the degree implied by the methods used.

A member of our geology staff observed the explorations. We obtained representative samples of the various soil encountered in the explorations for geotechnical laboratory testing. Classifications and sampling intervals are shown on the exploration logs included in this appendix.

SOIL SAMPLING

Soil samples were obtained from the borings using the following methods:

- SPTs were performed in general conformance with ASTM D 1586. The sampler was driven with a 140-pound automatic trip hammer free-falling 30 inches. The number of blows required to drive the sampler 1 foot, or as otherwise indicated, into the soil is shown adjacent to the sample symbols on the exploration logs. Disturbed samples were obtained from the split barrel for subsequent classification and index testing.
- A Dames & Moore sampler was used to obtain samples at some locations. As with the SPTs the sampler was driven with a 140-pound automatic trip hammer free-falling 30 inches. The number of blows required to drive the sampler 1 foot, or as otherwise indicated, into the soil is shown adjacent to the sample symbols on the exploration logs.
- Relatively undisturbed samples were obtained at selected intervals by pushing a shelby tube sampler 24 inches ahead of the boring front. Shelby tube samples are preferred for consolidation and strength testing due to the lower level of disturbance.

SOIL CLASSIFICATION

The soil samples were classified in accordance with the "Exploration Key" (Table A-1) and "Soil Classification System" (Table A-2), which are included in this appendix. The exploration logs indicate the depths at which the soils or their characteristics change, although the change actually could be gradual. If the change occurred between sample locations, the depth was interpreted. Classifications and sampling intervals are shown on the exploration logs included in this appendix.

LABORATORY TESTING

CLASSIFICATION

The soil samples were classified in the laboratory to confirm field classifications. The laboratory classifications are included on the exploration logs if those classifications differed from the field classifications.

MOISTURE CONTENT

We determined the natural moisture content of selected samples obtained from the explorations in general accordance with ASTM D 2216. The natural moisture content is a ratio of the weight of the water to soil in a test sample and is expressed as a percentage. The moisture contents are included on the exploration logs presented in this appendix.

ATTERBERG LIMITS TEST

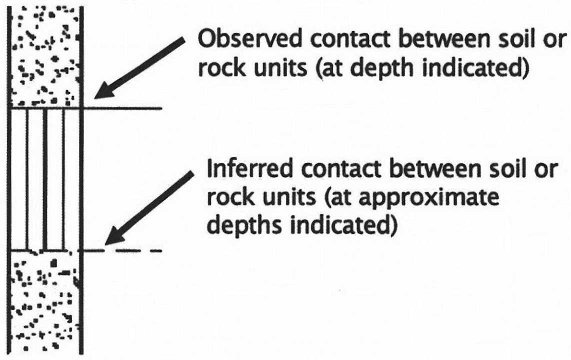
The plastic limit and liquid limit (Atterberg limits) of three selected soil samples were determined in accordance with ASTM D 4318. Atterberg limits include determining the liquid limit, plastic limit, and the plasticity index of soils. Results of the Atterberg limits testing are included in this appendix.

PARTICLE-SIZE TESTING

Particle-size analyses were performed on selected samples in general accordance with ASTM D 1140. This test determines of the amount of material finer than a 75-μm (No. 200) sieve expressed as a percentage of the dry weight of soil. The test results are included in this appendix.

SYMBOL	SAMPLING DESCRIPTION
	Location of sample obtained in general accordance with ASTM D 1586 Standard Penetration Test with recovery
	Location of sample obtained using thin-wall Shelby tube or Geoprobe® sampler in general accordance with ASTM D 1587 with recovery
	Location of sample obtained using Dames & Moore sampler and 300-pound hammer or pushed with recovery
	Location of sample obtained using Dames & Moore and 140-pound hammer or pushed with recovery
	Location of sample obtained using 3-inch-O.D. California split-spoon sampler and 140-pound hammer
	Location of grab sample
	Rock coring interval
	Water level during drilling
	Water level taken on date shown

Graphic Log of Soil and Rock Types



GEOTECHNICAL TESTING EXPLANATIONS

ATT	Atterberg Limits	PP	Pocket Penetrometer
CBR	California Bearing Ratio	P200	Percent Passing U.S. Standard No. 200 Sieve
CON	Consolidation		
DD	Dry Density	RES	Resilient Modulus
DS	Direct Shear	SIEV	Sieve Gradation
HYD	Hydrometer Gradation	TOR	Torvane
MC	Moisture Content	UC	Unconfined Compressive Strength
MD	Moisture-Density Relationship	VS	Vane Shear
OC	Organic Content	kPa	Kilopascal
P	Pushed Sample		

ENVIRONMENTAL TESTING EXPLANATIONS

CA	Sample Submitted for Chemical Analysis	ND	Not Detected
P	Pushed Sample	NS	No Visible Sheen
PID	Photoionization Detector Headspace Analysis	SS	Slight Sheen
		MS	Moderate Sheen
ppm	Parts per Million	HS	Heavy Sheen

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EXPLORATION KEY

TABLE A-1

RELATIVE DENSITY - COARSE-GRAINED SOILS

Relative Density	Standard Penetration Resistance	Dames & Moore Sampler (140-pound hammer)	Dames & Moore Sampler (300-pound hammer)
Very Loose	0 – 4	0 - 11	0 - 4
Loose	4 – 10	11 - 26	4 - 10
Medium Dense	10 – 30	26 - 74	10 - 30
Dense	30 – 50	74 - 120	30 - 47
Very Dense	More than 50	More than 120	More than 47

CONSISTENCY - FINE-GRAINED SOILS

Consistency	Standard Penetration Resistance	Dames & Moore Sampler (140-pound hammer)	Dames & Moore Sampler (300-pound hammer)	Unconfined Compressive Strength (tsf)
Very Soft	Less than 2	Less than 3	Less than 2	Less than 0.25
Soft	2 - 4	3 – 6	2 - 5	0.25 - 0.50
Medium Stiff	4 - 8	6 – 12	5 - 9	0.50 - 1.0
Stiff	8 - 15	12 – 25	9 - 19	1.0 - 2.0
Very Stiff	15 - 30	25 – 65	19 – 31	2.0 - 4.0
Hard	More than 30	More than 65	More than 31	More than 4.0

PRIMARY SOIL DIVISIONS			GROUP SYMBOL	GROUP NAME
COARSE-GRAINED SOILS (more than 50% retained on No. 200 sieve)	GRAVEL (more than 50% of coarse fraction retained on No. 4 sieve)	CLEAN GRAVELS (< 5% fines)	GW or GP	GRAVEL
		GRAVEL WITH FINES (≥ 5% and ≤ 12% fines)	GW-GM or GP-GM	GRAVEL with silt
			GW-GC or GP-GC	GRAVEL with clay
		GRAVELS WITH FINES (> 12% fines)	GM	silty GRAVEL
			GC	clayey GRAVEL
			GC-GM	silty, clayey GRAVEL
	SAND (50% or more of coarse fraction passing No. 4 sieve)	CLEAN SANDS (<5% fines)	SW or SP	SAND
		SANDS WITH FINES (≥ 5% and ≤ 12% fines)	SW-SM or SP-SM	SAND with silt
			SW-SC or SP-SC	SAND with clay
		SANDS WITH FINES (> 12% fines)	SM	silty SAND
			SC	clayey SAND
			SC-SM	silty, clayey SAND
FINE-GRAINED SOILS (50% or more passing No. 200 sieve)	SILT AND CLAY	Liquid limit less than 50	ML	SILT
			CL	CLAY
			CL-ML	silty CLAY
			OL	ORGANIC SILT or ORGANIC CLAY
		Liquid limit 50 or greater	MH	SILT
			CH	CLAY
			OH	ORGANIC SILT or ORGANIC CLAY
HIGHLY ORGANIC SOILS			PT	PEAT

MOISTURE CLASSIFICATION

Term	Field Test
dry	very low moisture, dry to touch
moist	damp, without visible moisture
wet	visible free water, usually saturated

ADDITIONAL CONSTITUENTS

Secondary granular components or other materials such as organics, man-made debris, etc.

Percent	Silt and Clay In: Fine-Grained Soils	Silt and Clay In: Coarse-Grained Soils	Percent	Sand and Gravel In: Fine-Grained Soils	Sand and Gravel In: Coarse-Grained Soils
< 5	trace	trace	< 5	trace	trace
5 – 12	minor	with	5 – 15	minor	minor
> 12	some	silty/clayey	15 – 30	with	with
			> 30	sandy/gravelly	Indicate %

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	BLOW COUNT	INSTALLATION AND COMMENTS
0.0	Medium dense, gray-brown GRAVEL with sand (GP), trace silt; moist, gravel is subrounded to subangular (0- to 1 1/2-inch-thick root zone) - **FILL.**	30.2			
2.5				24	
5.0	trace silt at 5.0 feet		P200	67	Infiltration rate: 60 inches per hour at 5.0 feet. P200 = 5%
7.5	Medium stiff, brown SILT (ML); moist.	22.7 / 7.5		7	
10.0	soft, light brown, trace sand and organics (rootlets); sand is fine at 10.0 feet		ATT	2	LL = NP PL = NP
12.5					
15.0	medium stiff, with sand, trace organics (isolated root); moist to wet, root is 1/4-inch diameter at 15.0 feet			7	Infiltration rate: ~0 inches per hour at 15.0 feet.
17.5				P	
20.0	soft, gray-brown, sandy; wet at 19.5 feet with gravel; stratified beds of silt at 21.0 feet		P200	1	P200 = 67%
22.0	Very dense, gray-brown GRAVEL with sand (GP), trace silt; wet.	8.2 / 22.0			
25.0	lens of silty sand from 26.0 to 28.0 feet			7	
27.5					
30.0					

▲ BLOW COUNT ● MOISTURE CONTENT % RQD% CORE REC% (scale 0 – 50 – 100)

DRILLED BY: Western States Soil Conservation, Inc. LOGGED BY: JGH COMPLETED: 06/11/14

BORING METHOD: mud rotary and hollow-stem auger (see report text) BORING BIT DIAMETER: 4 7/8-inch/10-inch

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GERDING-184-01

JULY 2014

BORING B-1

BLOCK 8L
PORTLAND, OR

FIGURE A-1

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	BLOW COUNT / MOISTURE CONTENT % / RQD% / CORE REC% (0 – 50 – 100)	INSTALLATION AND COMMENTS
30.0	(continued from previous page)		▲ 70	
32.5				
35.0	dense, with cobbles at 35.0 feet		▲ 119	
37.5				
40.0	Very dense, dark gray GRAVEL with sand (GW), trace silt; wet.	-9.8 40.0	▲ 30-50/5"	
42.5				
45.0	gravel is subrounded at 45.0 feet		▲ 50/3"	
47.5				
50.0				
50.5	Hard, light brown to gray CLAY (CL), minor gravel and sand; moist.	-20.3 50.5	▲ 92	
52.5				
55.0	Very stiff, gray SILT (ML), trace sand and clay; moist, sand is fine.	-24.8 55.0	▲ 55	
57.5				
60.0				

DRILLED BY: Western States Soil Conservation, Inc. LOGGED BY: JGH COMPLETED: 06/11/14

BORING METHOD: mud rotary and hollow-stem auger (see report text) BORING BIT DIAMETER: 4 7/8-inch/10-inch

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JULY 2014

BORING B-1
(continued)

BLOCK 8L
PORTLAND, OR

FIGURE A-1

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	SAMPLE	▲ BLOW COUNT ● MOISTURE CONTENT % RQD% CORE REC% (0 – 50 – 100)	INSTALLATION AND COMMENTS
60.0	(continued from previous page)				64	
62.5	Very dense, gray-light brown GRAVEL with sand and cobbles (GP), trace silt; moist to wet, tightly packed in light brown-yellow matrix.	-32.3 / 62.5				
65.0					50/3"	
67.5						
70.0					40-50/4"	
72.5						
75.0	Exploration completed at a depth of 75.3 feet.	-45.1 / 75.3			50/4"	
77.5						
80.0						
82.5						
85.0						
87.5						
90.0						

DRILLED BY: Western States Soil Conservation, Inc. LOGGED BY: JGH COMPLETED: 06/11/14

BORING METHOD: mud rotary and hollow-stem auger (see report text) BORING BIT DIAMETER: 4 7/8-inch/10-inch

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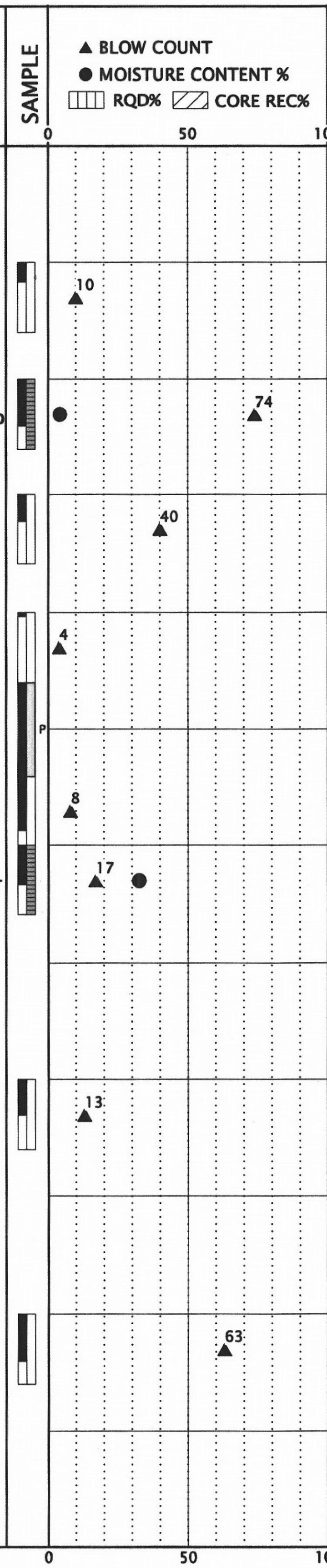
JULY 2014

BORING B-1
(continued)

BLOCK 8L
PORTLAND, OR

FIGURE A-1

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	INSTALLATION AND COMMENTS
0.0	Loose to medium dense, gray-brown SAND with gravel and debris (metal spikes and brick) (SP), trace silt; moist, medium to coarse - **FILL**.	31.4		Flush-mount monument with 1.5 feet of concrete backfill. Top of cut tremie pipe at 0.5 foot. Gravel backfill. Bentonite chips.
5.0	Medium dense to dense, gray-brown GRAVEL with sand (GP), trace silt; moist, sand is medium to coarse - **FILL**.	26.4 / 5.0	P200	Infiltration rate: 6.4 inches per hour at 5.0 feet. P200 = 4%
7.5	dense at 7.5 feet			Cement grout. 1 inch PVC pipe.
9.0	Soft to medium stiff, brown SILT (ML), trace sand; moist.	22.4 / 9.0		
13.5	medium stiff to stiff, light brown, minor sand; moist to wet, sand is fine at 13.5 feet stiff at 15.0 feet		ATT	Infiltration rate: ~0 inches per hour at 15.0 feet. LL = 34% PL = 26%
20.0	Medium dense, light brown SAND with silt (SP-SM); wet, fine.	11.4 / 20.0		
23.0	Very dense, gray-brown GRAVEL with sand (GW), trace silt; wet, stratified beds of brown silty sand.	8.4 / 23.0		
27.5	lens of silty sand from 27.5 to 29.0 feet			Vibrating wire piezometer #SN140084 set at 29.3 feet.
30.0				



DRILLED BY: Western States Soil Conservation, Inc. LOGGED BY: JGH COMPLETED: 06/12/14

BORING METHOD: mud rotary and hollow-stem auger (see report text) BORING BIT DIAMETER: 4 7/8-inch/10-inch

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GERDING-184-01

JULY 2014

BORING B-2

BLOCK 8L
PORTLAND, OR

FIGURE A-2

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	BLOW COUNT	INSTALLATION AND COMMENTS
30.0	(continued from previous page)			63	
35.0	with clay at 35.0 feet			76	
37.5					Bottom of tremie pipe at 37.3 feet.
					Bentonite chips.
40.0	gray at 40.0 feet			98	
45.0	Very dense, gray GRAVEL with silt, sand, and cobbles (GP-GM); wet, sand is fine to coarse.	-13.6 / 45.0		26-50/3"	
50.0				70	
52.0	Very stiff, light brown SILT (ML); moist.	-20.6 / 52.0			
55.0			ATT	19	LL = 32% PL = 26%
60.0					

▲ BLOW COUNT ● MOISTURE CONTENT % RQD% CORE REC%

DRILLED BY: Western States Soil Conservation, Inc. LOGGED BY: JGH COMPLETED: 06/12/14

BORING METHOD: mud rotary and hollow-stem auger (see report text) BORING BIT DIAMETER: 4 7/8-inch/10-inch

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GERDING-184-01

JULY 2014

BORING B-2
(continued)

BLOCK 8L
PORTLAND, OR

FIGURE A-2

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	SAMPLE	▲ BLOW COUNT ● MOISTURE CONTENT % RQD% CORE REC% (0 – 50 – 100)	INSTALLATION AND COMMENTS
60.0		(continued from previous page)				58	
62.5		Very dense, brown-gray GRAVEL with sand and cobbles (GW), trace silt; moist to wet (Troutdale Formation).	-30.6 / 62.0				
65.0						50/3"	
67.5							
70.0						30-120/6"	
72.5							
75.0		Exploration completed at a depth of 75.5 feet.	-44.1 / 75.5			50/6"	
77.5							
80.0							
82.5							
85.0							
87.5							
90.0							

DRILLED BY: Western States Soil Conservation, Inc. LOGGED BY: JGH COMPLETED: 06/12/14

BORING METHOD: mud rotary and hollow-stem auger (see report text) BORING BIT DIAMETER: 4 7/8-inch/10-inch

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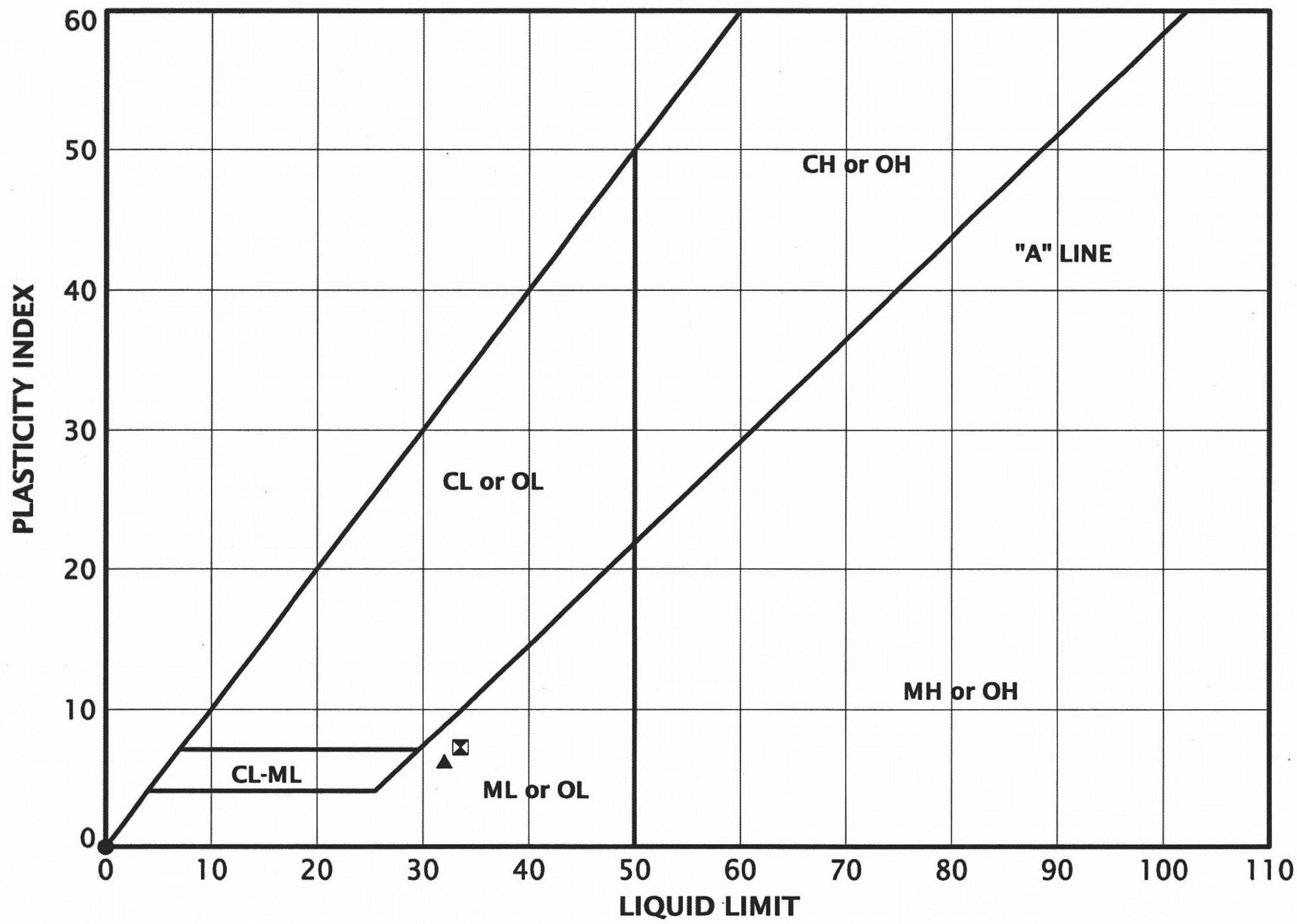
GERDING-184-01

JULY 2014

BORING B-2
(continued)

BLOCK 8L
PORTLAND, OR

FIGURE A-2



KEY	EXPLORATION NUMBER	SAMPLE DEPTH (FEET)	MOISTURE CONTENT (PERCENT)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
●	B-1	10.0	40	NP	NP	NP
⊠	B-2	15.0	33	34	26	8
▲	B-2	55.0	43	32	26	6

ATTERBERG_LIMITS 7 GERDING-184-01-B1_2.GPJ GEODESIGN.GDT PRINT DATE: 7/16/14:KT

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JULY 2014

ATTERBERG LIMITS TEST RESULTS

BLOCK 8L
PORTLAND, OR

FIGURE A-3

SAMPLE INFORMATION			MOISTURE CONTENT (PERCENT)	DRY DENSITY (PCF)	SIEVE			ATTERBERG LIMITS		
EXPLORATION NUMBER	SAMPLE DEPTH (FEET)	ELEVATION (FEET)			GRAVEL (PERCENT)	SAND (PERCENT)	P200 (PERCENT)	LIQUID LIMIT (PERCENT)	PLASTIC LIMIT (PERCENT)	PLASTICITY INDEX (PERCENT)
B-1	2.5	27.7	11							
B-1	5.0	25.2	9				5			
B-1	7.5	22.7	15							
B-1	10.0	20.2	40					NP	NP	NP
B-1	15.0	15.2	37							
B-1	20.0	10.2	39				67			
B-1	25.0	5.2	41							
B-1	35.0	-4.8	9							
B-1	45.0	-14.8	13							
B-1	60.0	-29.8	19							
B-1	75.0	-44.8	10							
B-2	5.0	26.4	4				4			
B-2	15.0	16.4	33					34	26	8
B-2	30.0	1.4	11							
B-2	40.0	-8.6	10							
B-2	50.0	-18.6	11							
B-2	55.0	-23.6	43					32	26	6
B-2	60.0	-28.6	19							
B-2	70.0	-38.6	15							

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JULY 2014

SUMMARY OF LABORATORY DATA

BLOCK 8L
PORTLAND, OR

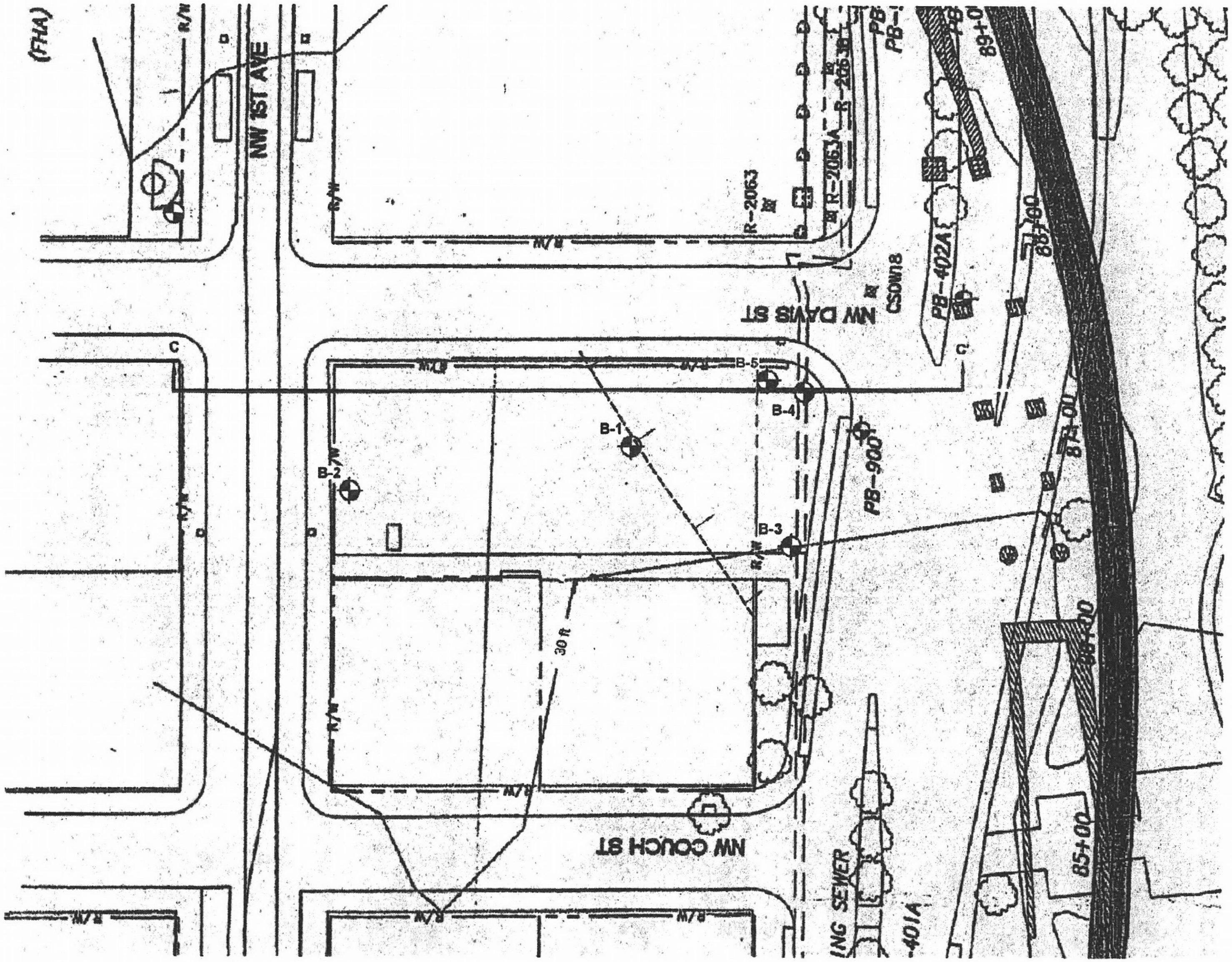
FIGURE A-4

APPENDIX B

PREVIOUS INVESTIGATION

We reviewed an outside report that was prepared for the northern parking lot area of the site and provided to us by Gerding Edlen Development. The previous study at the site was prepared by Shannon & Wilson. Explorations for geotechnical studies were completed in 2003 and included three borings that ranged in depth from 21.5 to 85.1 feet BGS.

The approximate locations of the explorations are shown on Figure 2 included in this appendix. Explorations logs and laboratory testing that were completed on the site are included in this appendix.



LEGEND

BORING LOCATION AND NUMBER

B-1 1979 S&W BORING LOCATION

HINDGE POINT AND DIRECTION OF SLOPING SUBSURFACE TOPOGRAPHY

0 50 100

Approximate Scale in Feet

Portland Fire Station No. 1
Preliminary Sites Assessment (Block 8)
Portland, Oregon

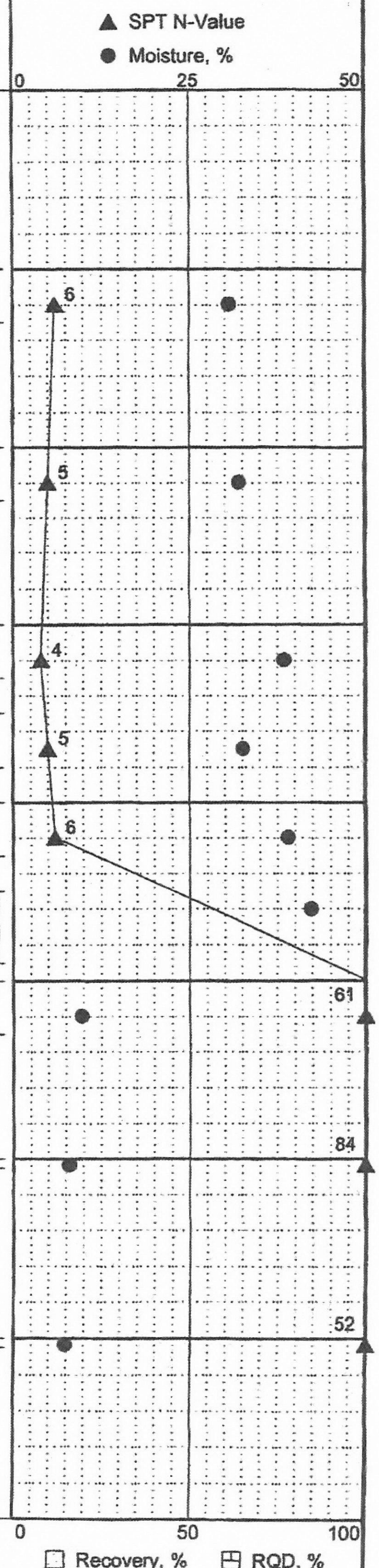
SITE PLAN

August 2003 24-1-03235-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. 1

Ground Water	Remarks	Elev. Depth Feet 30.0	CLASSIFICATION OF MATERIAL	Depth In Feet	Samples
		29.3 / 0.7	**PAVEMENT**, 4 inches of Asphalt over 4 inches of 3/4-inch minus base rock. (Pavement Section)	0	
			Gravelly Clayey Sandy SILT, medium stiff, gray and brown, moist, low plasticity, sand is fine-grained, gravel is fine-grained and angular. (Fill)	5	S-1
	%-200 = 62	20.0 / 10.0	**Sandy SILT**, slightly clayey, medium stiff, brown, moist, low-plasticity, micaceous, sand is fine-grained. (Recent Alluvium)	10	S-2
	%-200 = 76	17.0 / 13.0	**Clayey SILT**, trace of sand, soft to medium stiff becoming medium stiff, brown with gray and rust mottling, moist to wet, low to medium plasticity, micaceous, sand is fine-grained. (Recent Alluvium)	15	S-3
▼	%-200 = 88 7/28/2003 Approximate Water Level				S-4
	%-200 = 45 Dry Density = 77.5 pcf	10.5 / 19.5	**Stratified Clayey SILT and Silty SAND**, 2- to 4-inch lenses, medium stiff, brown, wet, low plasticity clayey silt, no to low plasticity silty sand, micaceous, sand is fine-grained. (Fine-grained Catastrophic Flood Deposits)	20	S-5
	Torvane = 0.1 tsf Pocket Pen. = 0.25 tsf	7.5 / 22.5	**Clayey Sandy SILT**, medium stiff, brown, wet, low plasticity, micaceous, sand is fine-grained. (Fine-grained Catastrophic Flood Deposits)		S-6
	%-200 = 78	6.0 / 24.0	Becomes slightly gravelly, fine-grained and rounded.		
			Silty Sandy GRAVEL, slightly clayey, dense to very dense, gray, wet, low plasticity matrix, micaceous, fine- to coarse-grained sand and gravel, gravel is subrounded to rounded. (Coarse-grained Catastrophic Flood Deposits)	25	S-7
		0.0 / 30.0	**Sandy GRAVEL**, with scattered to numerous cobbles, very dense, gray, wet, micaceous, fine- to coarse-grained sand and gravel, gravel is subrounded to rounded. (Coarse-grained Catastrophic Flood Deposits)	30	S-8
				35	S-9



LEGEND

- ⌶ = 2.0" O.D. Split Spoon Sample
- ⌶ = 3.0" O.D. Thin-Walled Sample
- * = Sample Not Recovered
- ⌶ = 3.0" O.D. Split Spoon Sample
- ▮ = Core Rock Sample

- Impervious Seal (Bentonite)
- Cement Grout
- Random Backfill
- Granular Backfill
- ▽ Ground Water Level on Date Shown
- Piezometer/Inclinometer Tubing
- Perforated Zone

ATTERBERG LIMITS
- Liquid Limit
- Natural Water Content
- Plastic Limit

NOTE:
Lines between soil/rock units are approximate and transition may be gradual.

Portland Fire Station No. 1
Preliminary Sites Assesment, Block 8

LOG OF BORING B-1

page 1 of 2

August 2003 *24-1-03235-001*

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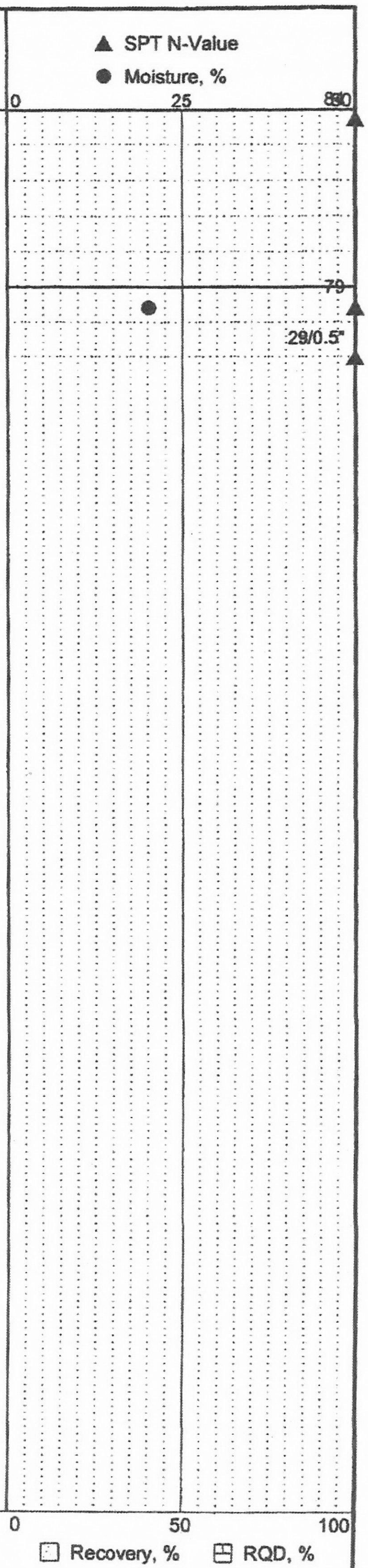
Portland, Oregon

FIG. 3

REV 4

WLG FS-1 BLOCK 8.GPJ 8/29/03 Log: DJH Typ: DJH Rev:

Ground Water	Remarks	Elev. Depth Feet	CLASSIFICATION OF MATERIAL	Log	Depth In Feet	Samples
			Sandy GRAVEL, continued.			*S-10
			Lost circulation at 41.5 feet, approximately 50 gallons of drill mud leaked into formation.			
					45	S-11
		-17.1	Drilling progress became very slow at 46 feet.			
		47.1	Bottom of Boring, Completed 7/28/03			S-12



LEGEND

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Impervious Seal (Bentonite)
Cement Grout
Random Backfill
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Ground Water Level on Date Shown
Piezometer/Inclinometer Tubing
Perforated Zone

ATTERBERG LIMITS
Liquid Limit
Natural Water Content
Plastic Limit

NOTE:
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Portland Fire Station No. 1
Preliminary Sites Assesment, Block 8

LOG OF BORING B-1
page 2 of 2

August 2003 *24-1-03235-001*

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Geotechnical Consultants
Portland, Oregon

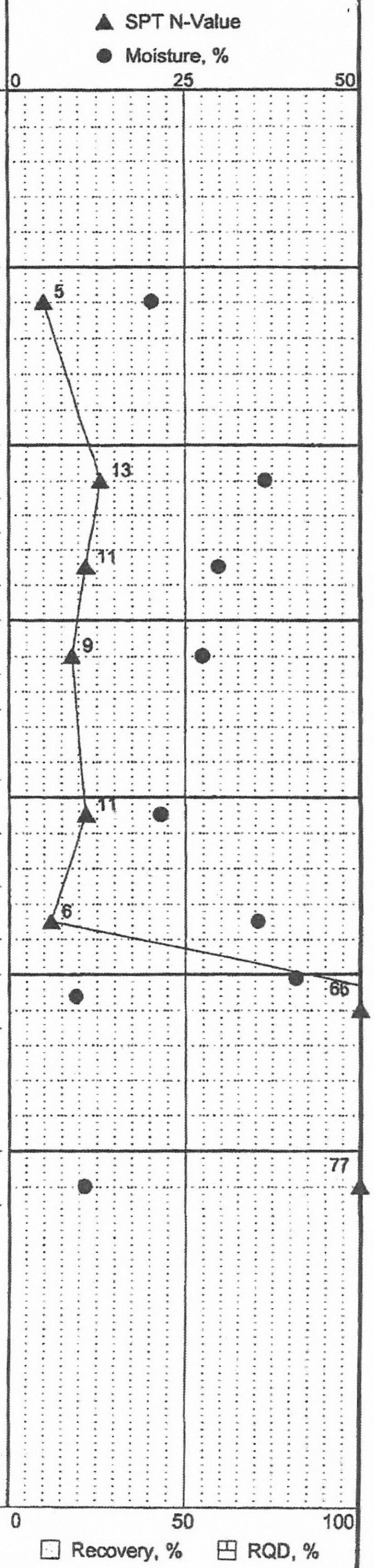
FIG. 3

REV 4

Log: DJH Typ: DJH Rev:

WLG FS-1 BLOCK 8.GPJ 8/29/03

Ground Water	Remarks	Elev. Depth Feet	CLASSIFICATION OF MATERIAL	Depth In Feet	Samples
		31.0	**Pavement**, 4 inches of Asphalt over 20 inches of 1-inch minus base rock. (Pavement Section)	0	
		29.0 2.0	**Clayey Sandy SILT**, medium stiff to stiff, gray to brown, moist, low plasticity, micaceous; sand is fine grained. (Fill)		
	%-200=64.9			5	S-1
	%-200=64.6			10	S-2
	%-200=60.3	19.0 12.0	**Sandy SILT** with trace clay, stiff, light brown, moist, low plasticity, micaceous; sand is fine-grained. (Fine-Grained Catastrophic Flood Deposits)		S-3
	%-200=94.3			15	S-4
		14.0 17.0	**Silty SAND** with trace clay, medium dense grading to loose, light brown, moist grading to wet, non-plastic, micaceous, fine-grained. (Fine-Grained Catastrophic Flood Deposits)		S-5
	%-200=38.3			20	S-6
▼	8/18/2003				
	%-200=39.3				S-7
		5.7	Clayey silt layer at contact.	25	
	%-200=87.8	25.3	**Sandy GRAVEL** with some cobbles and trace silt, very dense, gray, wet, non-plastic, micaceous; fine- to coarse-grained sand; fine to coarse sub-rounded to rounded gravel. (Coarse-Grained Catastrophic Flood Deposits)		S-8
				30	S-9
		-0.5 31.5	Bottom of Boring, Completed 8/18/03.		



Log: DJH Typ: KEE Rev:

LEGEND

- ⌶ = 2.0" O.D. Split Spoon Sample
- ⌶ = 3.0" O.D. Thin-Walled Sample
- * = Sample Not Recovered
- ⌶ = 3.0" O.D. Split Spoon Sample
- ⌶ = Core Rock Sample

- Impervious Seal (Bentonite)
- Cement Grout
- Random Backfill
- Granular Backfill
- ∇ Ground Water Level on Date Shown
- Piezometer/Inclinometer Tubing
- Perforated Zone

ATTERBERG LIMITS
- Liquid Limit
- Natural Water Content
- Plastic Limit

NOTE:
Lines between soil/rock units are approximate and transition may be gradual.

WLG FS-1 BLOCK 8.GPJ 8/29/03

Portland Fire Station No. 1
Preliminary Sites Assesment, Block 8

LOG OF BORING B-2

page 1 of 1

August 2003 *24-1-03235-001*

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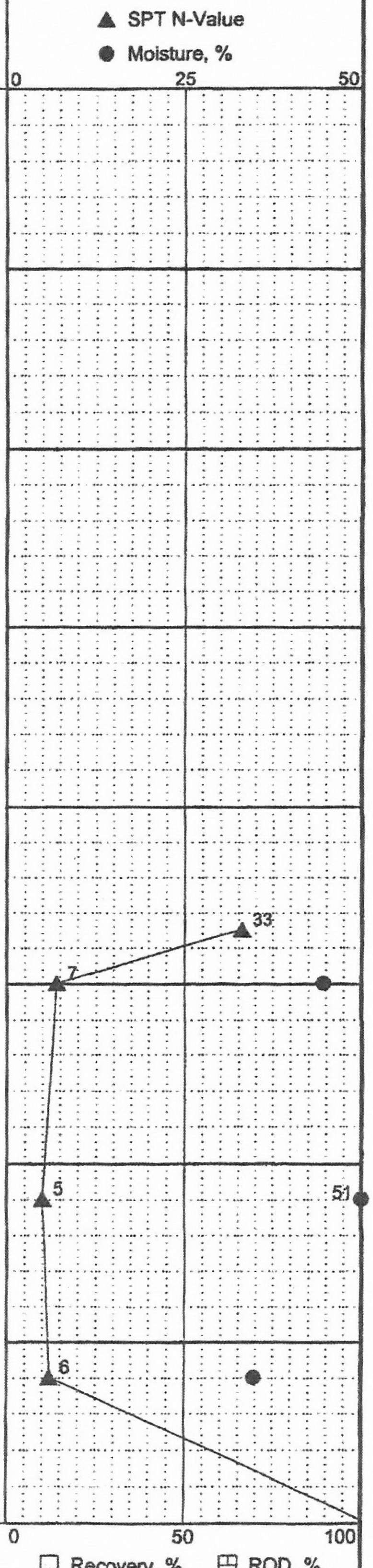
Portland, Oregon

FIG. 4

Ground Water	Remarks	Elev. Depth Feet 30.0	CLASSIFICATION OF MATERIAL	Log	Depth In Feet	Samples
		28.0	**Pavement**, 4 inches of asphalt over 20 inches of base rock. (Pavement Section)		0	
		2.0 / 11.0	**Gravel**, medium dense 0 to 5 feet; loose 5 to 19 ft; hole kept caving, cuttings consited of 1/2 to 1/4 inch subround to sub angular rock. (Sewer Trench Backfill)		5, 10, 15	
		19.0 / 2.0	**Silty SAND**, dark brown, wet, non-plastic. (Recent Alluvium) Wood encountered 20 to 25 feet. Poor sample recovery.		20, 25	*S-1, S-4, S-3
		28.0 / -3.0	**Sandy SILT**, medium stiff, dark gray, wet, low to medium plasticity; sand is fine-grained; contains some shredded wood. (Recent Alluvium)		30	S-5
		33.0 / -8.0	**Clayey Sandy SILT**, medium stiff, dark gray, wet, low to medium plasticity; trace wood. (Recent Alluvium)		35	S-6
		38.0	**Sandy GRAVEL** with trace cobbles, dense, gray, wet. (Coarse-Grained Catastrophic Flood Deposits)			



LEGEND

- = 2.0" O.D. Split Spoon Sample
- = 3.0" O.D. Thin-Walled Sample
- * = Sample Not Recovered
- = 3.0" O.D. Split Spoon Sample
- = Core Rock Sample

Impervious Seal (Bentonite)
Cement Grout
Random Backfill
Granular Backfill
Ground Water Level on Date Shown
Piezometer/Inclinometer Tubing
Perforated Zone

ATTERBERG LIMITS
Liquid Limit
Natural Water Content
Plastic Limit

NOTE:
Lines between soil/rock units are approximate and transition may be gradual.

Portland Fire Station No. 1
Preliminary Sites Assesment, Block 8

LOG OF BORING B-3

page 1 of 3

August 2003 | *24-1-03235-001*

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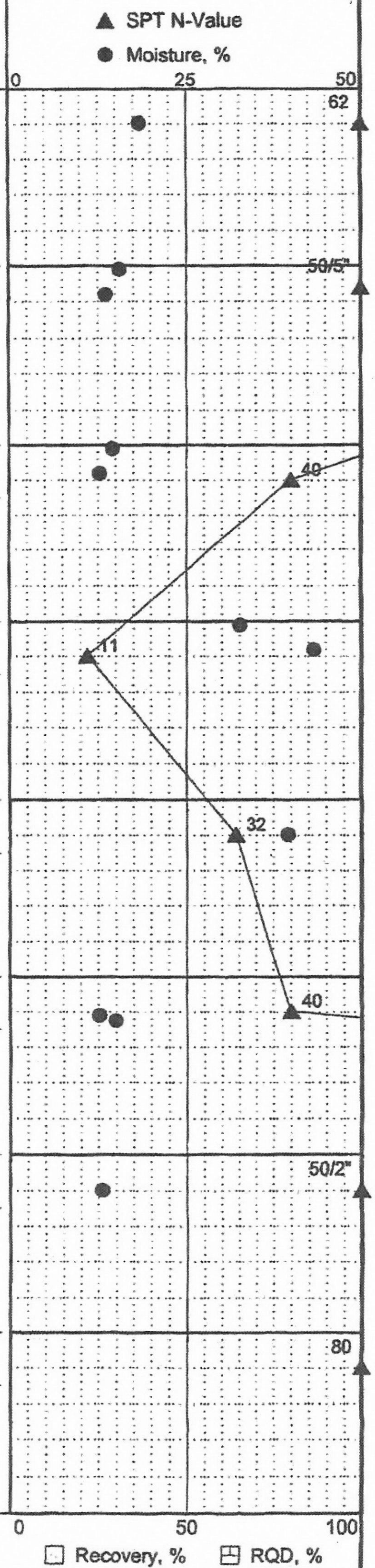
Portland, Oregon

FIG. 5

Log: djh/del/Typ: deh Rev:

WLG FS-1 BLOCK 8.GPJ 8/29/03

Ground Water	Remarks	Elev. Depth Feet	CLASSIFICATION OF MATERIAL	Log	Depth In Feet	Samples
			Sandy GRAVEL, continued.			S-7
		-12.0 42.0	**GRAVEL** with cobbles, very dense, gray, wet, non-plastic, sub angular. (Coarse-Grained Catastrophic Flood Deposits)		45	S-8
					50	S-9
		-23.0 53.0	**Silty SAND**, slightly sandy, medium dense, wet, gray, micaceous. (Fine-Grained Catastrophic Flood Deposits)			
	%-200=28.7 %-200=94.4	-25.1 55.1	Stratified **Clayey SILT**, stiff, gray to brown, moist, medium plasticity. (Older Alluvium)		55	S-10
	%-200=86.8				60	S-11
		-32.5 62.5	Stratified **Sandy GRAVEL to Gravely SAND** with some cobbles, dense to very dense, red-yellow-gray gravel clasts, sand layers are red-brown, moist; sand is medium-grained; gravel is fine and subrounded to rounded. (Coarse-Grained Catastrophic Flood Deposits)		65	S-12
					70	S-13
					75	S-14



LEGEND

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- = 3.0" O.D. Thin-Walled Sample
- * = Sample Not Recovered
- = 3.0" O.D. Split Spoon Sample
- = Core Rock Sample

NOTE:
Lines between soil/rock units are approximate and transition may be gradual.

- Impervious Seal (Bentonite)
- Cement Grout
- Random Backfill
- Granular Backfill
- Ground Water Level on Date Shown
- Piezometer/Inclinometer Tubing
- Perforated Zone

ATTERBERG LIMITS
- Liquid Limit
- Natural Water Content
- Plastic Limit

Portland Fire Station No. 1
Preliminary Sites Assesment, Block 8

LOG OF BORING B-3

page 2 of 3

August 2003 *24-1-03235-001*

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Portland, Oregon

FIG. 5

WLG FS-1 BLOCK 8.GPJ 8/29/03 Log: djh/def/Typ: deh Rev:

Ground Water	Remarks	Elev. Depth Feet	CLASSIFICATION OF MATERIAL	Log	Depth In Feet	Samples	▲ SPT N-Value ● Moisture, % (0 – 25 – 50)
			Stratified **Sandy GRAVEL to Gravelly SAND** continued. Drilling progress becomes very slow at 83.5 feet (Troutdale Formation?)				
		-55.0 85.0	Bottom of Boring, Completed 8/19/03.		85	*S-15	50/1st 3"

0 50 100

☐ Recovery, % ⊞ RQD, %

LEGEND

- = 2.0" O.D. Split Spoon Sample
- = 3.0" O.D. Thin-Walled Sample
- * = Sample Not Recovered
- = 3.0" O.D. Split Spoon Sample
- = Core Rock Sample

NOTE:
Lines between soil/rock units are approximate and transition may be gradual.

- Impervious Seal (Bentonite)
- Cement Grout
- Random Backfill
- Granular Backfill
- Ground Water Level on Date Shown
- Piezometer/Inclinometer Tubing
- Perforated Zone

ATTERBERG LIMITS
- Liquid Limit
- Natural Water Content
- Plastic Limit

Portland Fire Station No. 1
Preliminary Sites Assesment, Block 8

LOG OF BORING B-3

page 3 of 3

August 2003 *24-1-03235-001*

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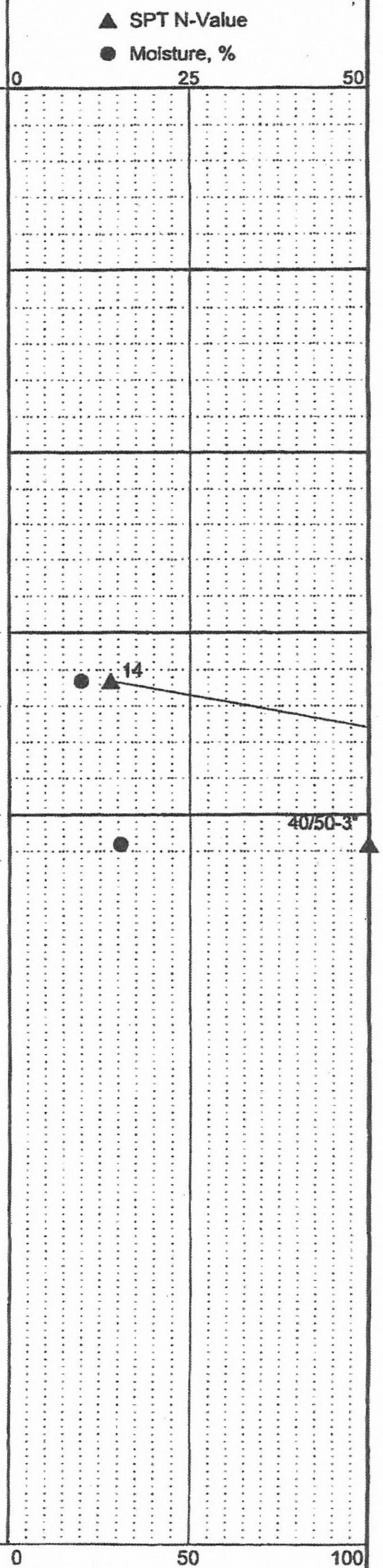
Portland, Oregon

FIG. 5

Ground Water	Remarks	Elev. Depth Feet 30.0	CLASSIFICATION OF MATERIAL	Log	Depth In Feet	Samples
			Pavement, 4 inches of asphalt over 20 inches of base rock. (Pavement Section)		0	
		28.0 2.0	**GRAVEL**, loose, cuttings consisted of 1/2 to 1/4 inch subround to sub angular rock. (Sewer Trench Backfill)			
		25.0 5.0	**GRAVEL**, pit run gravel with binder. (Sewer Trench Backfill)		5	
		24.0 6.0	**GRAVEL**, loose, cuttings consisted of 1/2 to 1/4 inch subround to sub angular rock. (Sewer Trench Backfill)			
					10	
					15	
		14.0 16.0	Stratified**SAND and GRAVEL**, very dense, brown to gray, moist; contains cement dust. (Sewer Trench Backfill)			S-1
					20	S-2
		8.5	Hit 48 by 56-inch concrete sewer pipe.			
		21.5	Bottom of Boring, Completed 8/20/03.			



LEGEND

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Impervious Seal (Bentonite)
Cement Grout
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Ground Water Level on Date Shown
Piezometer/Inclinometer Tubing
Perforated Zone

ATTERBERG LIMITS
Liquid Limit
Natural Water Content
Plastic Limit

NOTE:
Lines between soil/rock units are approximate and transition may be gradual.

Portland Fire Station No. 1
Preliminary Sites Assesment, Block 8

LOG OF BORING B-4
page 1 of 1

August 2003 *24-1-03235-001*

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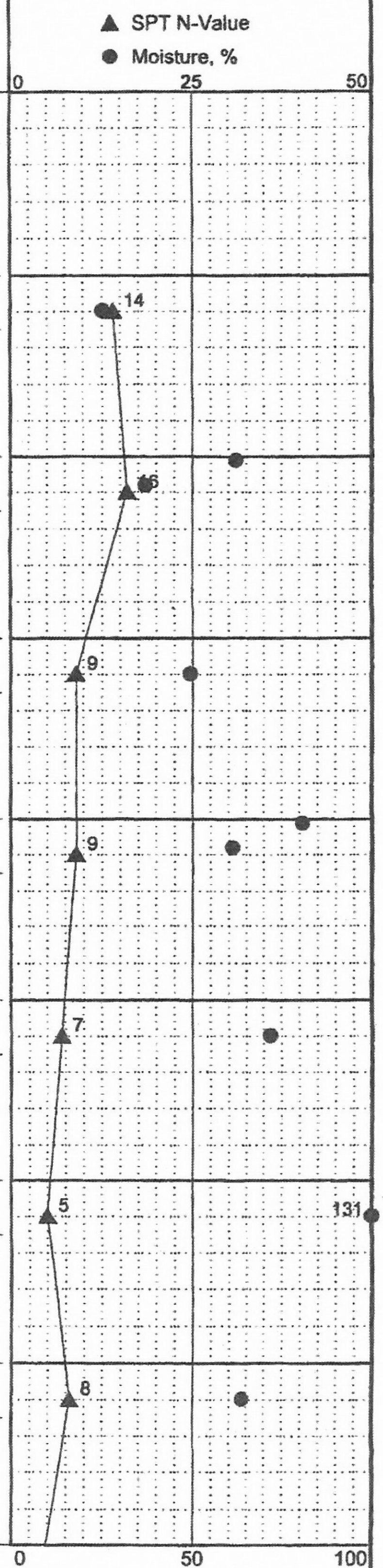
Portland, Oregon

FIG. 6

Log: rbb/deh/typ: deh Rev:

WLG FS-1 BLOCK 8.GPJ 8/29/03

Ground Water	Remarks	Elev. Depth Feet 30.0	CLASSIFICATION OF MATERIAL	Log	Depth In Feet	Samples
		28.0	**Pavement**, 4 inches of asphalt over 20 inches of base rock. (Pavement Section)		0	
		2.0	**Sandy Silty GRAVEL**, medium dense,brown, moist. (Fill)		5	S-1
	%-200=83.9	19.0			10	S-2
	%-200=14.2	11.0	Stratified **Silty SAND and Sandy Clayey SILT**, loose/soft to medium stiff, gray, moist to wet, non-plastic to medium plasticity; stratified in layers from 3 to 12 inches thick; contains occassional pieces of wood. (Recent Alluvium)			
	%-200=22.3				15	S-3
	%-200=42.1 %-200=61.3				20	S-4
	%-200=18.5				25	S-5
	%-200=86.9				30	S-6
	%-200=19.1				35	S-7



LEGEND

- = 2.0" O.D. Split Spoon Sample
- = 3.0" O.D. Thin-Walled Sample
- * = Sample Not Recovered
- = 3.0" O.D. Split Spoon Sample
- = Core Rock Sample

NOTE:
Lines between soil/rock units are approximate and transition may be gradual.

- Impervious Seal (Bentonite)
- Cement Grout
- Random Backfill
- Granular Backfill
- Ground Water Level on Date Shown
- Piezometer/Inclinometer Tubing
- Perforated Zone

ATTERBERG LIMITS
- Liquid Limit
- Natural Water Content
- Plastic Limit

Portland Fire Station No. 1
Preliminary Sites Assesment, Block 8

LOG OF BORING B-5
page 1 of 3

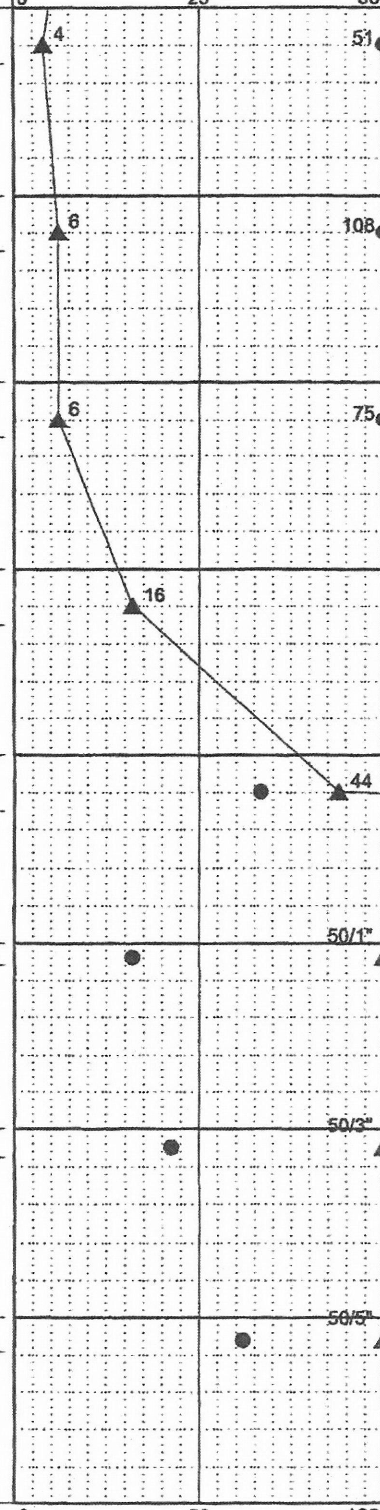
August 2003 *24-1-03235-001*

SHANNON & WILSON, INC.
Geotechnical Consultants
Portland, Oregon

FIG. 7

WLG FS-1 BLOCK 8.GPJ 8/29/03 Log: deh Typ: deh Rev:

Ground Water	Remarks	Elev. Depth Feet	CLASSIFICATION OF MATERIAL	Log	Depth In Feet	Samples
	%-200=91.8		Stratified **Silty SAND and Sandy Clayey SILT**, continued.			S-8
	%-200=77.7		Abundant wood fragments at 45 to 50 feet.		45	S-9
	%-200=39.2				50	S-10
		-24.5 54.5	Poor recovery...**Clayey SILT** with lenses of gravel; possibly contains wood. (Recent Alluvium)		55	S-11
		-31.0 61.0	Stratified **Sandy GRAVEL and Gravelly SAND**, very dense, dark gray, wet, non-plastic; sand is medium- to coarse-grained; subrounded to rounded gravel to 2 inches; contains layers of sandy gravel, sand and gravelly sand. (Older Alluvium)		60	S-12
					65	S-13
					70	S-14
					75	S-15



LEGEND

- = 2.0" O.D. Split Spoon Sample
- = 3.0" O.D. Thin-Walled Sample
- * = Sample Not Recovered
- = 3.0" O.D. Split Spoon Sample
- = Core Rock Sample

Impervious Seal (Bentonite)
Cement Grout
Random Backfill
Granular Backfill
Ground Water Level on Date Shown
Piezometer/Inclinometer Tubing
Perforated Zone

ATTERBERG LIMITS
Liquid Limit
Natural Water Content
Plastic Limit

NOTE:
Lines between soil/rock units are approximate and transition may be gradual.

Portland Fire Station No. 1
Preliminary Sites Assesment, Block 8

LOG OF BORING B-5

page 2 of 3

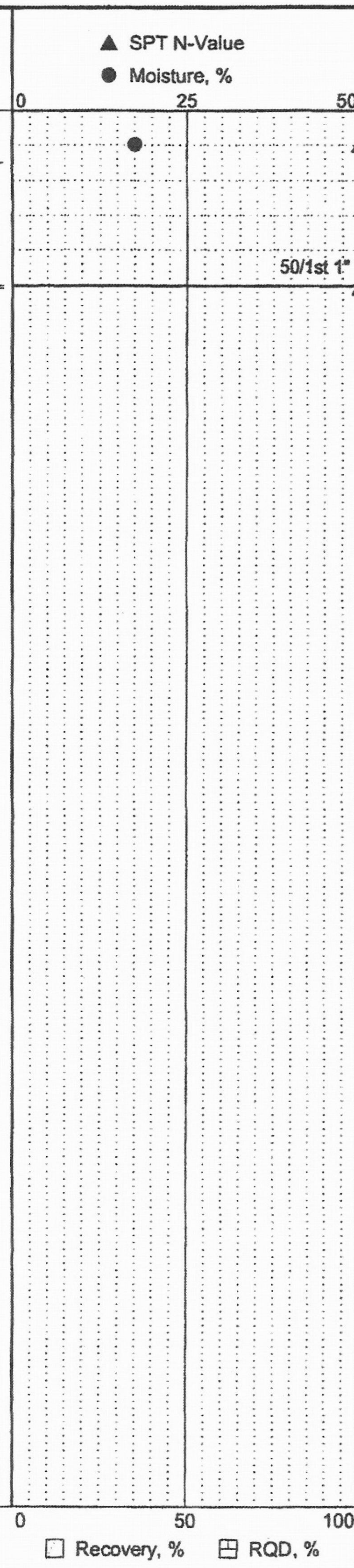
August 2003 *24-1-03235-001*

SHANNON & WILSON, INC.
Geotechnical Consultants
Portland, Oregon

FIG. 7

WLG FS-1 BLOCK 8.GPJ 8/29/03 Log: deh Typ: deh Rev:

Ground Water	Remarks	Elev. Depth Feet	CLASSIFICATION OF MATERIAL	Log	Depth In Feet	Samples
	%-200=3.4		Stratified **Sandy GRAVEL and Gravelly SAND**, continued.			S-16
		-52.0 82.0	**Sandy GRAVEL and COBBLES**, very dense; poor recovery. (Troutdale Formation?) Drilling progress became very slow at 46 feet.		85	*S-17
		-55.1 85.1	Bottom of Boring, Completed, 8/25/03.			



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ATTERBERG LIMITS
- Liquid Limit
- Natural Water Content
- Plastic Limit

Portland Fire Station No. 1
Preliminary Sites Assesment, Block 8

LOG OF BORING B-5

page 3 of 3

August 2003 *24-1-03235-001*

SHANNON & WILSON, INC.
Geotechnical Consultants
Portland, Oregon

FIG. 7

WLG FS-1 BLOCK 8.GPJ 8/29/03

Log: deh Typ: deh Rev:

ACRONYMS

ASTM	American Society for Testing and Materials
BGS	below ground surface
CLSM	controlled low strength material
COP	City of Portland
g	gravitational acceleration (32.2 feet/second^2)
H:V	horizontal to vertical
IBC	International Building Code
MCE	maximum considered earthquake
OSHA	Occupational Safety and Health Administration
pcf	pounds per cubic foot
pci	pounds per cubic inch
PDA	Pile Driving Analyzer®
psf	pounds per square foot
SOSSC	State of Oregon Structural Specialty Code
SPT	standard penetration test
µm	micrometer
WEAP	wave equation analysis program

Scan

GEODESIGN INC

December 11, 2014

Gerding Edlen Development Company
1477 NW Everett Street
Portland, OR 97209

Attention: Ms. Jill Sherman

Geotechnical Engineering Services
Response to City of Portland Checksheet
Application # 14-211482-STR-01-CO
Block 8L
NW 1st Avenue and NW Davis Street
Portland, Oregon
GeoDesign Project: Gerding-184-05

INTRODUCTION

This letter responds to the City of Portland's Checksheet for the proposed Block 8L development dated December 4, 2014 (Application # 14-211482-STR-01-CO). Block 8L is located southeast of the intersection of NW Davis Street and NW 1st Avenue in Portland, Oregon. We provided geotechnical engineering recommendations for use in design and construction of the facility in a report[1] dated July 16, 2014. Specifically, this letter addresses the items directed at GeoDesign.

ITEM #2

We conducted a liquefaction analysis assuming maximum considered earthquake (MCE) events on the Portland Hills Fault and Cascadia Subduction Zone. The peak ground accelerations (PGAs) used in our analysis are the following publications:

- Earthquake Scenario and Probabilistic Ground Shaking Maps for the Portland, Oregon, Metropolitan Area. Portland Hills fault M 6.8 earthquake, peak horizontal acceleration (g) at the ground surface, State Of Oregon, Department of Geology and Mineral Industries, 2000.
- Earthquake Scenario and Probabilistic Ground Shaking Maps for the Portland, Oregon, Metropolitan Area. Cascadia Subduction Zone M 9.0 earthquake, peak horizontal acceleration (g) at the ground surface, State Of Oregon, Department of Geology and Mineral Industries, 2000.

---

[1] *Report of Geotechnical Engineering Services; Block 8L; NW 1st Avenue and NW Davis Street; Portland, Oregon;* dated July 16, 2014. GeoDesign Project: Gerding-184-01

maa 2

The two seismic mechanisms reported in these maps (1) a magnitude 6.8 earthquake on the Portland Hill Fault and (2) a magnitude 9.0 on the Cascadia Subduction Zone are the sources that will control liquefaction at the site. These events also represent the MCE on these two faults.

Table 1. Computed Liquefaction-Induced Settlement

MCE Seismic Event	Magnitude, M_w	Reported PGA (g's)	PGA Used in Analysis (g's)
Portland Hills Fault	6.8	0.4 to 0.5	0.45
Cascadia Subduction Zone	9.0	0.2	0.20

Lateral spreading can occur where liquefiable soil is present in the subsurface near a riverbank. In our opinion, lateral spreading is not considered a design concern for the following reasons:

- The site is 250 feet from the riverbank.
- A log of a boring shows relatively competent soil is present between the subject site and Willamette River. A log of the boring and map showing the boring location is in the Attachment to this letter.
- The riverbank is protected with a large gravity bulkhead. A typical cross section of the bulkhead is presented in the Attachment.

◆ ◆ ◆

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

Sincerely,

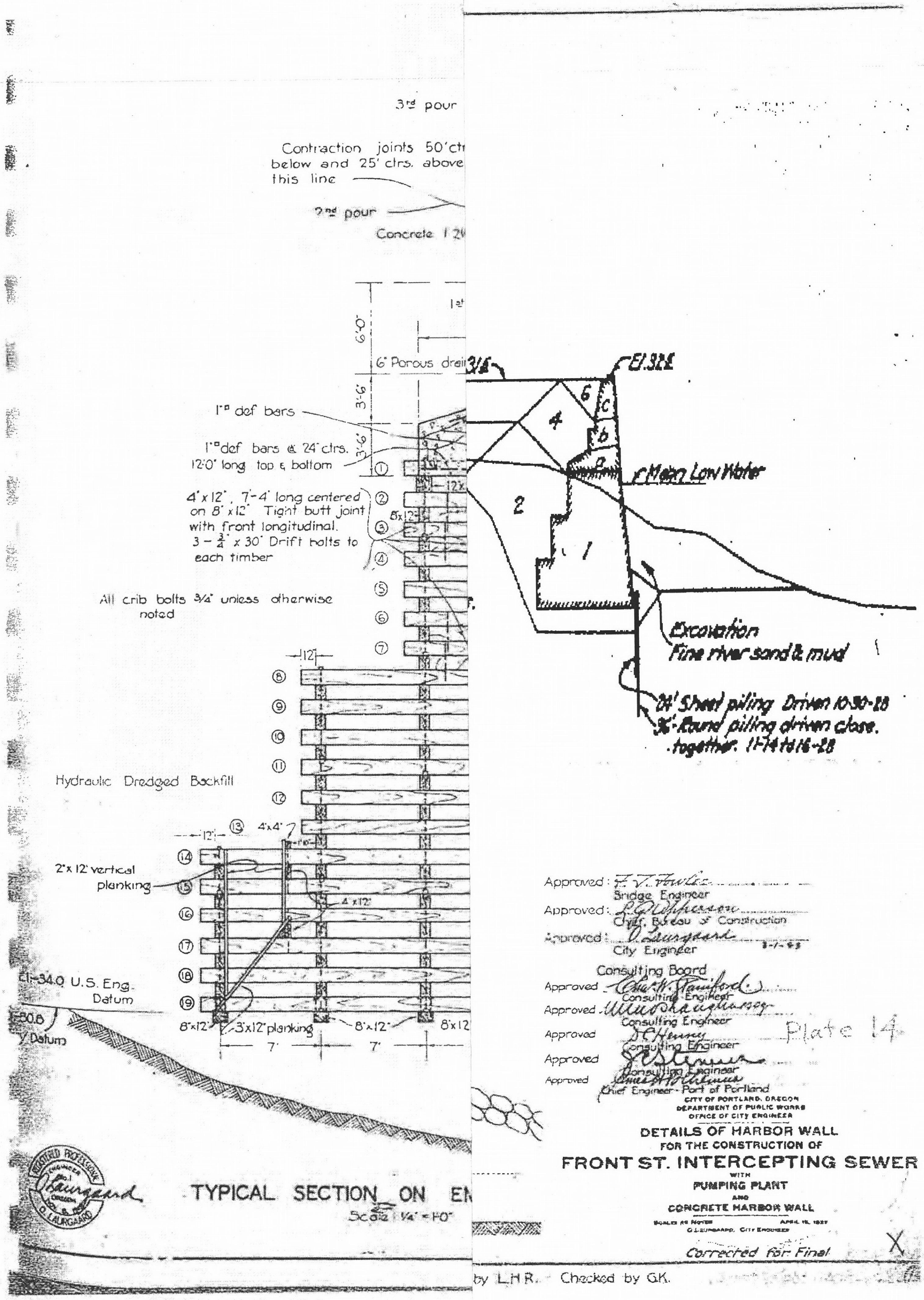
GeoDesign, Inc.

REGISTERED PROFESSIONAL ENGINEER
56207
OREGON
July 11, 2000
BRETT A. SHIPTON
EXPIRES: 6/30/16

Brett A. Shipton, P.E., G.E.
Principal Engineer

cc: Mr. Jeb Koerner, Gerding Edlen Development Company (via email only)
Ms. Jennifer Jenkins, Ankrom Moisan Architects (via email only)
Ms. Elisa Catanzarite, Ankrom Moisan Architects (via email only)

BAS:kt
Attachment
One copy submitted (via email only)
Document ID: Gerding-184-05-121114-geol-checksheet_response_rev.docx


3rd pour
Contraction joints 50' ctr[illegible] below and 25' ctrs. above this line
2nd pour
Concrete 1:2[illegible]
1st
6'-0"
6" Porous drai[illegible]
3'-6"
3'-6"
1" def bars
1" def bars @ 24" ctrs. 12'-0" long top & bottom
4" x 12", 7'-4" long centered on 8" x 12". Tight butt joint with front longitudinal. 3 - 3/4" x 30" Drift bolts to each timber
8x12
12x
All crib bolts 3/4" unless otherwise noted
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
18
19
12"
Hydraulic Dredged Backfill
12"
4x4
2" x 12" vertical planking
4"x12"
El-34.0 U.S. Eng. Datum
El-30.6
Datum
8"x12"
3"x12" planking
8"x12"
8"x12
7'
7'
El.32.E
6
c
4
b
a
3
Mean Low Water
2
1
Excavation
Fine river sand & mud
24' Sheet piling Driven 10-30-28
36' Round piling driven close together. 11-14 to 16-28
Approved: F. T. Fowler
Bridge Engineer
Approved: L. G. Whipperman
Chief, Bureau of Construction
Approved: O. Laurgaard
City Engineer
3-1-93
Consulting Board
Approved: Chas. H. Swigert
Consulting Engineer
Approved: Wm. O'Shaughnessy
Consulting Engineer
Approved: D. C. Henny
Consulting Engineer
Approved: J. Steinman
Consulting Engineer
Approved
Chief Engineer - Port of Portland
Plate 14
CITY OF PORTLAND, OREGON
DEPARTMENT OF PUBLIC WORKS
OFFICE OF CITY ENGINEER
DETAILS OF HARBOR WALL
FOR THE CONSTRUCTION OF
FRONT ST. INTERCEPTING SEWER
WITH
PUMPING PLANT
AND
CONCRETE HARBOR WALL
SCALES AS NOTED
APRIL 15, 1927
O. LAURGAARD, CITY ENGINEER
Corrected for Final
X
REGISTERED PROFESSIONAL ENGINEER
OREGON
O. LAURGAARD
O. Laurgaard
TYPICAL SECTION ON EN[illegible]
Scale 1/4" = 1'-0"
[illegible] by L.H.R. Checked by G.K.

PARSONS BRINCKERHOFF

BORING LOG
PB-402A

Foundation Engineering, Inc.

PROJECT: West Side CSO Project
CITY: Portland, Oregon
PROJECT NO. 2002013
DATE DRILLED 10/17/00 LOGGED BY AR

SHEET 1 OF 3
STATION NO. 87+72 (55 L)
SURFACE ELEV. 31.9 ft

INITIAL GWL@ Not Available
EQUIPMENT Mobil Drill B-59
DRILLING METHOD Mud Rotary
HAMMER SYS. Manual 140 lb. drop

SAMPLE TYPE: Ring (3.25" OD) | Standard Penetration Test (2" OD) | Shelby Tube

Elevation (ft)	DEPTH (ft)	SAMPLE TYPE	SAMPLE NO	FIELD BLOWS /6"
26.9	5	Standard Penetration Test	1	6, 4, 3
21.9	10	Standard Penetration Test	2	0, 1, 2
16.9	15	Standard Penetration Test	3	9, 8, 8
11.9	20	Standard Penetration Test	4	13, 9, 10
6.9	25	Standard Penetration Test	5	3, 4, 3
1.9	30	Standard Penetration Test	6	2, 4, 9
		Standard Penetration Test	7	11, 8, 9
-3.1	35	Ring	8*	16, 14, 11

Chart column headings: PERCENT FINES 20 40 60 80; PLASTIC M.C. LIQUID 20 40 60 80; LITHOLOGY; UNCORRECTED BLOW COUNTS (last 12") 20 40 60 80; WELL

SOIL DESCRIPTION

ASPHALTIC CONCRETE AND PORTLAND CEMENT CONCRETE (Road surface).

GRAVELLY SILT WITH SAND (ML); fine to coarse grained sand, angular gravel, low plasticity, grey-brown, moist, medium stiff, (Fill).

Scattered wood fragments encountered at 9 feet.

Fill becomes soft with few gravel below 10 feet.

Interbedded SANDY SILT and SILTY SAND (SM/ML); fine grained sand, low plasticity silt, grey and brown, wet, medium stiff/medium dense, stratified, (Sand/silt alluvium).

* 3 inch sampler/140 lb hammer

Supersedes By 12/11/2014

GEODESIGN INC

December 5, 2014

Gerding Edlen Development Company
1477 NW Everett Street
Portland, OR 97209

Attention: Ms. Jill Sherman

Geotechnical Engineering Services
Response to City of Portland Checksheet
Application # 14-211482-STR-01-CO
Block 8L
NW 1st Avenue and NW Davis Street
Portland, Oregon
GeoDesign Project: Gerding-184-05

INTRODUCTION

This letter responds to the City of Portland's Checksheet for the proposed Block 8L development dated December 4, 2014 (Application # 14-211482-STR-01-CO). Block 8L is located southeast of the intersection of NW Davis Street and NW 1st Avenue in Portland, Oregon. We provided geotechnical engineering recommendations for use in design and construction of the facility in a report[1] dated July 16, 2014. Specifically, this letter addresses the items directed at GeoDesign.

ITEM #2

We conducted a liquefaction analysis assuming maximum considered earthquake (MCE) events on the Portland Hills Fault and Cascadia Subduction Zone. The peak ground accelerations (PGAs) used in our analysis are the following publications:

- Earthquake Scenario and Probabilistic Ground Shaking Maps for the Portland, Oregon, Metropolitan Area. Portland Hills fault M 6.8 earthquake, peak horizontal acceleration (g) at the ground surface, State Of Oregon, Department of Geology and Mineral Industries, 2000.
- Earthquake Scenario and Probabilistic Ground Shaking Maps for the Portland, Oregon, Metropolitan Area. Cascadia Subduction Zone M 9.0 earthquake, peak horizontal acceleration (g) at the ground surface, State Of Oregon, Department of Geology and Mineral Industries, 2000.

[1] *Report of Geotechnical Engineering Services; Block 8L; NW 1st Avenue and NW Davis Street; Portland, Oregon;* dated July 16, 2014. GeoDesign Project: Gerding-184-01

The two seismic mechanisms reported in these maps (1) a magnitude 6.8 earthquake on the Portland Hill Fault and (2) a magnitude 9.0 on the Cascadia Subduction Zone are the sources that will control liquefaction at the site. These events also represent the MCE on these two faults.

Table 1. Computed Liquefaction-Induced Settlement

MCE Seismic Event	Magnitude, M_w	Reported PGA (g's)	PGA Used in Analysis (g's)
Portland Hills Fault	6.8	0.4 to 0.5	0.45
Cascadia Subduction Zone	9.0	0.2	0.20

Lateral spreading can occur where liquefiable soil is present in the subsurface near a riverbank. In our opinion, lateral spreading is not considered a design concern for the following reasons:

- The site is 250 feet from the riverbank.
- The riverbank is protected with a bulkhead.
- Soil prone to liquefaction at the site is discontinuous.

We also estimated lateral displacement using the Seismic Landslide Movement Modeled using Earth Records (SLAMMER). We compute a horizontal displacement of less than 1 inch for the following events.

Table 2. Computed Liquefaction-Induced Settlement

MCE Seismic Event	Magnitude, M_w	Critical Acceleration (g's)	PGA (g's)
Portland Hills Fault	6.8	0.25[1]	0.45
Cascadia Subduction Zone	9.0	0.25[1]	0.20

1. Assumed values

We utilized the Rathje and Saygli (2009) rigid model to compute displacements. The above model does not consider the bulkhead at the riverbank and in our opinion over predicts lateral displacement. We conclude that lateral spreading is not considered a design issue at the site.

♦ ♦ ♦

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

Sincerely,

GeoDesign, Inc.

REGISTERED PROFESSIONAL ENGINEER
56207
OREGON
July 11, 2000
BRETT A. SHIPTON
EXPIRES: 6/30/16

Brett A. Shipton, P.E., G.E.
Principal Engineer

cc: Ms. Jennifer Jenkins, Ankrom Moisan Architects (via email only)
Ms. Elisa Catanzarite, Ankrom Moisan Architects (via email only)

BAS:kt
One copy submitted (via email only)
Document ID: Gerding-184-05-120514-geol-checksheet_response.docx

14-211482-STR-01-CO

GEODESIGN INC

November 21, 2014

Gerding Edlen Development Company
1477 NW Everett Street
Portland, OR 97209

Attention: Ms. Jill Sherman

Geotechnical Engineering Services
Response to City of Portland Checksheet
Application # 14-211482-STR-01-CO
Block 8L
NW 1st Avenue and NW Davis Street
Portland, Oregon
GeoDesign Project: Gerding-184-05

This letter responds to the City of Portland's Checksheet for the proposed Block 8L development dated November 13, 2014 (Application # 14-211482-STR-01-CO). Block 8L is located southeast of the intersection of NW Davis Street and NW 1st Avenue in Portland, Oregon. We provided geotechnical engineering recommendations for use in design and construction of the facility in a report[1] dated July 16, 2014. Specifically, this letter addresses the items directed at GeoDesign.

Item #2
We conducted a liquefaction analysis assuming maximum considered earthquake (MCE) events on the Portland Hills Fault and Cascadia Subduction Zone. Table 1 summarizes our maximum computed liquefaction induced settlement.

Table 1. Computed Liquefaction-Induced Settlement

MCE Seismic Event	Magnitude, M_w	Peak Ground Acceleration (g's)	Computed Settlement (inches)
Portland Hills Fault	6.8	0.45	3.28
Cascadia Subduction Zone	9.0	0.2	3.32

[1] *Report of Geotechnical Engineering Services; Block 8L; NW 1st Avenue and NW Davis Street; Portland, Oregon;* dated July 16, 2014. GeoDesign Project: Gerding-184-01

A copy of our analysis is attached. Differential settlement is anticipated across the site. The site plan in the Attachment shows the anticipated settlement at each boring location. We recommend using a differential settlement equal to the difference in the settlement magnitudes shown. Linear interpolation between boring locations can be assumed. The upper approximately 20 feet of the site in not liquefiable and will likely manifest in differential settlement. Our analysis did not consider this effect. A copy of our analysis at the location of boring B-5 (conducted by Shannon & Wilson) is attached.

Item #3

In our opinion, the majority of foundation settlement will occur as the foundation loads are applied, which is consistent with elastic theory.

Item #4

Uncontrolled fill is present at the site and there is a potential for floor slab settlement. Settlement of uncontrolled fill is difficult to predict. However, we anticipate that settlement could be between 1 inch and 1.5 inches. We believe that this is a serviceability issue and does not pose a threat to life safety.

♦ ♦ ♦

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

Sincerely,

GeoDesign, Inc.

Brett A. Shipton, P.E., G.E.
Principal Engineer

REGISTERED PROFESSIONAL ENGINEER
56207
OREGON
July 11, 2000
BRETT A. SHIPTON
EXPIRES: 6/30/16

cc: Ms. Jennifer Jenkins, Ankrom Moisan Architects (via email only)

BAS:kt
Attachments
One copy submitted (via email only)
Document ID: Gerding-184-05-112114-geol-checksheet_response.docx

SPT Liquefaction Spreadsheet

From Robertson and Fear, Liquefaction Workshop, 1996/1998

						M	ground surface a max
JOB NAME	Block 8L	Boring #	B-5			9	0.2
DATE	11/15/2014	unit wt	110	pcf	Magnitude Scaling Factor (MSF)	0.63	%
BY	BAS	grwater	21	ft			

CRR DATABASE (1998 updated Seed equation)

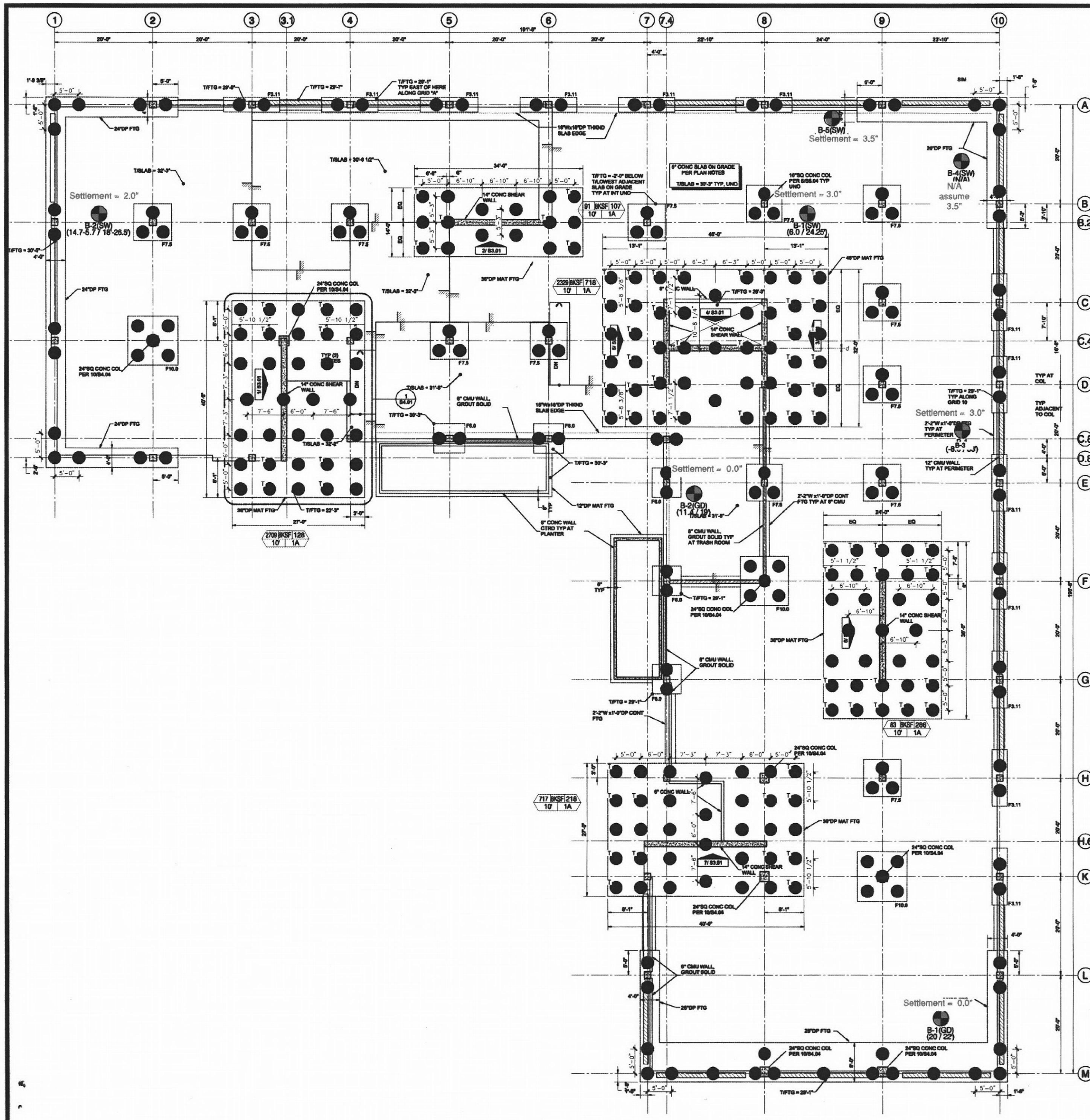
TOTAL SETTLEMENT = **3.32 INCHES**

m n o **p q**

RESISTING CSR

Equations — *Eqn. 4* — *Eqn. 3* — *Eqn. 2*

Depth (feet) *notes*	Soil Unit	Nraw *averages*	σo' (psf)	Ncorrections C_E *safety hammer?* energy	C_N stress	C_B *5.9"* diam	C_S *no liner* sampler	C_R rod	$(N_1)_{60}$ *interim Corr.*	Fines Correction %fines	Alpha	Beta	ΔN *(corrected for fines)*	$(N_1)_{60}$ FINAL	CRR *1998 eq.* *<N=30*	CSR	FOS	FOS Adjusted for Magnitude	SETTLEMENT (%)	(inches)
1		30.0	110	1	1.76	1.05	1.2	0.65		95	5.00	1.20				0.207	2.00	2.00		
2		30.0	220	1	1.69	1.05	1.2	0.66		95	5.00	1.20				0.206	2.00	2.00		
3	Not	30.0	330	1	1.62	1.05	1.2	0.67		95	5.00	1.20				0.205	2.00	2.00		
4	Liquefiable	30.0	440	1	1.56	1.05	1.2	0.68		95	5.00	1.20				0.205	2.00	2.00		
5	Above GW	30.0	550	1	1.51	1.05	1.2	0.69		95	5.00	1.20				0.204	2.00	2.00		
6		30.0	660	1	1.46	1.05	1.2	0.7		95	5.00	1.20				0.203	2.00	2.00		
7		30.0	770	1	1.41	1.05	1.2	0.71		95	5.00	1.20				0.202	2.00	2.00		
8		30.0	880	1	1.36	1.05	1.2	0.72		95	5.00	1.20				0.201	2.00	2.00		
9		30.0	990	1	1.32	1.05	1.2	0.73		95	5.00	1.20				0.200	2.00	2.00		
10		30.0	1100	1	1.28	1.05	1.2	0.74		95	5.00	1.20				0.199	2.00	2.00		
11		30.0	1210	1	1.24	1.05	1.2	0.75		95	5.00	1.20				0.198	2.00	2.00		
12		30.0	1320	1	1.21	1.05	1.2	0.77		95	5.00	1.20				0.197	2.00	2.00		
13		30.0	1430	1	1.17	1.05	1.2	0.79		95	5.00	1.20				0.196	2.00	2.00		
14		30.0	1540	1	1.14	1.05	1.2	0.81		95	5.00	1.20				0.195	2.00	2.00		
15		30.0	1650	1	1.11	1.05	1.2	0.83		95	5.00	1.20				0.194	2.00	2.00		
16		30.0	1760	1	1.08	1.05	1.2	0.85		95	5.00	1.20				0.193	2.00	2.00		
17		30.0	1870	1	1.06	1.05	1.2	0.861		95	5.00	1.20				0.192	2.00	2.00		
18		9.0	1980	1	1.03	1.05	1.2	0.872		61	5.00	1.20				0.191	2.00	2.00		
19		9.0	2090	1	1.01	1.05	1.2	0.883		61	5.00	1.20				0.190	2.00	2.00		
20	ML	9.0	2200	1	0.98	1.05	1.2	0.894		61	5.00	1.20				0.189	2.00	2.00		
21		9.0	2310	1	0.96	1.05	1.2	0.905	10	61	5.00	1.20	7.0	17	0.179	0.188	0.95	0.60	0.002	0.02
22		9.0	2358	1	0.95	1.05	1.2	0.916	10	61	5.00	1.20	7.0	17	0.179	0.192	0.93	0.58	0.002	0.02
23		9.0	2406	1	0.94	1.05	1.2	0.927	10	61	5.00	1.20	7.0	17	0.179	0.196	0.92	0.57	0.002	0.02
24		9.0	2454	1	0.93	1.05	1.2	0.938	10	61	5.00	1.20	7.0	17	0.180	0.200	0.90	0.56	0.002	0.02
25		7.0	2502	1	0.92	1.05	1.2	0.949	8	18	3.23	1.07	3.7	11	0.126	0.203	0.62	0.39	0.023	0.28
26	SM	7.0	2550	1	0.91	1.05	1.2	0.96	8	18	3.23	1.07	3.7	11	0.127	0.206	0.61	0.39	0.023	0.28
27		7.0	2598	1	0.91	1.05	1.2	0.971	8	18	3.23	1.07	3.7	12	0.127	0.209	0.61	0.38	0.023	0.28
28		7.0	2646	1	0.90	1.05	1.2	0.982	8	87	5.00	1.20	6.6	14	0.153	0.211	0.73	0.45	0.004	0.05
29		5.0	2694	1	0.89	1.05	1.2	0.993	6	87	5.00	1.20	6.1	12	0.128	0.214	0.60	0.38	0.004	0.05
30	ML	5.0	2742	1	0.88	1.05	1.2	1.004	6	87	5.00	1.20	6.1	12	0.128	0.216	0.59	0.37	0.004	0.05
31		5.0	2790	1	0.87	1.05	1.2	1.015	6	87	5.00	1.20	6.1	12	0.128	0.219	0.59	0.37	0.004	0.05
32		5.0	2838	1	0.87	1.05	1.2	1.026	6	87	5.00	1.20	6.1	12	0.129	0.221	0.58	0.37	0.004	0.05
33		8.0	2886	1	0.86	1.05	1.2	1.037	9	19	3.43	1.07	4.1	13	0.141	0.223	0.63	0.40	0.020	0.24
34		8.0	2934	1	0.85	1.05	1.2	1.048	9	19	3.43	1.07	4.1	13	0.141	0.224	0.63	0.39	0.020	0.24
35		8.0	2982	1	0.84	1.05	1.2	1.059	9	19	3.43	1.07	4.1	13	0.141	0.226	0.63	0.39	0.020	0.24
36		8.0	3030	1	0.84	1.05	1.2	1.07	9	19	3.43	1.07	4.1	13	0.142	0.227	0.62	0.39	0.020	0.24
37	SM	8.0	3078	1	0.83	1.05	1.2	1.081	9	19	3.43	1.07	4.1	13	0.142	0.229	0.62	0.39	0.020	0.24
38		8.0	3126	1	0.82	1.05	1.2	1.092	9	19	3.43	1.07	4.1	13	0.142	0.230	0.62	0.39	0.020	0.24
39		8.0	3174	1	0.81	1.05	1.2	1.103	9	19	3.43	1.07	4.1	13	0.142	0.231	0.61	0.38	0.020	0.24
40		8.0	3222	1	0.81	1.05	1.2	1.114	9	19	3.43	1.07	4.1	13	0.142	0.232	0.61	0.38	0.020	0.24
41		4.0	3270	1	0.80	1.05	1.2	1.125	5	85	5.00	1.20	5.9	10	0.117	0.233	0.50	0.31	0.004	0.05
42		4.0	3318	1	0.79	1.05	1.2	1.136	5	85	5.00	1.20	5.9	10	0.117	0.234	0.50	0.31	0.004	0.05
43	ML	4.0	3366	1	0.79	1.05	1.2	1.147	5	85	5.00	1.20	5.9	10	0.117	0.235	0.50	0.31	0.004	0.05
44		6.0	3414	1	0.78	1.05	1.2	1.158	7	85	5.00	1.20	6.4	13	0.143	0.236	0.60	0.38	0.004	0.05
45		6.0	3462	1	0.78	1.05	1.2	1.169	7	85	5.00	1.20	6.4	13	0.143	0.237	0.60	0.38	0.004	0.05



NOTES AND SYMBOL KEY

1. THIS GEORAM® ENGINEERED AGGREGATE PIER PLAN CONSTITUTES A PORTION OF THE PROPRIETARY DESIGN-BUILD SCOPE OF GEOTECH FOUNDATION COMPANY - WEST (GTFC-WEST). IT IS PREDICATED ON THE INSTALLATION EQUIPMENT, METHODS (INCLUDING HIGH ENERGY VERTICAL RAMMING), PROPRIETARY QUALITY CONTROL, AND ENGINEERS' OVERSIGHT AS PROVIDED BY GTFC-WEST OR ITS SPECIFICALLY DELEGATED REPRESENTATIVES. THEREFORE, THIS PLAN SHALL NOT BE USED BY ANY COMPANY OTHER THAN GTFC-WEST FOR ANY PURPOSE WHATSOEVER.
2. THE SOLE PURPOSE OF THIS DRAWING IS TO INDICATE THE LOCATIONS, ORIENTATION, AND OTHER INFORMATION PERTINENT TO THE INSTALLATION OF THIS ENGINEERED AGGREGATE PIER SYSTEM. ALL OTHER INFORMATION IS FROM SHEET S2.01, DATED 09.10.2014, BY DCI ENGINEERS, PORTLAND, OREGON.
3. THE DESIGNER OF THE ENGINEERED AGGREGATE PIER SYSTEM HAS RELIED ON GEOTECHNICAL DATA AND STRUCTURAL LOADING INFORMATION PROVIDED BY OTHERS. NOTWITHSTANDING ANY CONTRACTUAL OR OTHER PROVISIONS, GTFC-W HAS NO RESPONSIBILITY TO DETERMINE THE COMPLETENESS OR ACCURACY OF THE INFORMATION PROVIDED.
4. REFER TO SHEET G2.00 FOR SPECIFICATIONS AND DIMENSIONING OF THE ENGINEERED AGGREGATE PIER LAYOUTS. ALL LAYOUTS CENTERED BENEATH FOOTINGS U.N.O.
5. INDICATES DEAD LOAD + LIVE LOAD IN kips (DCI; SEE ALSO FOOTING SCHEDULE).
6. INDICATES DEAD LOAD + LIVE LOAD + SEISMIC IN ksf (PROVIDED BY DCI; APPLIES AT EACH END OF SHEAR WALL FOOTINGS).
7. XXX XXX XXX / XX' 1A — INDICATES ESTIMATED NET UPLIFT IN kips (PROVIDED BY DCI; APPLIES AT EACH END OF SHEAR WALL FOOTINGS).
8. INDICATES PIER LAYOUT NUMBER (SEE SHEET G2.00).
9. INDICATES MINIMUM PIER DRILL DEPTH BELOW BOTTOM-OF-FOOTING ELEVATION.
10. INDICATES TENSION PIER.
11. T (XXX) — INDICATES 30" PIER DRILL DIAMETER FOR FOUNDATION SUPPORT.
12. INDICATES PIER REFERENCE NUMBER.
13. SITE GRADE AT TIME OF GEOTECHNICAL EXPLORATION = ELEV. 30 TO 31.4 (PER GEOTECHNICAL EXPLORATION LOGS).
14. ALL PIER DRILL DEPTHS TO EXTEND TO AT LEAST 8' BELOW BOTTOM-OF-FOOTING. ALL DRILL DEPTHS FOR PIERS DESIGNED FOR TENSION LOADING TO EXTEND TO AT LEAST 10' BELOW BOTTOM-OF-FOOTING.
15. INDICATES APPROXIMATE LOCATION OF GEOTECHNICAL EXPLORATION.
16. B-1 — INDICATES EXPLORATION REFERENCE NUMBER.
17. (6.0 / 24.25') — INDICATES APPROXIMATE ELEVATION / DEPTH BELOW FINISH FLOOR OF BEARING STRATUM INTERFACE (NATIVE SP) AND SHALLOWER COMPRESSION PIER DRILL DEPTH TERMINATION IF ENCOUNTERED.

FOOTING SCHEDULE						
TYPE	LENGTH	WIDTH	THICKNESS	MAX COL. LOAD	PIER LAYOUT	SHAFT LENGTH
F3.11	11'-6"	3'-0"	2'-2"	200K	2-D	8'
F6.0	6'-0"	6'-0"	1'-2"	200K	2-A	8'
F7.5	7'-6"	7'-6"	1'-8"	325K	3-D	8'
F10.0	10'-0"	10'-0"	2'-0"	600K	5-C	12'
PERIM. MATS	-	-	-	200K / 400K	2-D / 3-D	12'
-	-	-	-	-	-	-
-	-	-	-	-	-	-
-	-	-	-	-	-	-
-	-	-	-	-	-	-
-	-	-	-	-	-	-
-	-	-	-	-	-	-
-	-	-	-	-	-	-

0 1" 2"

REGISTERED PROFESSIONAL ENGINEER 85814PE OREGON MARCH 13, 2012 JEREMY JOHN GRAY
EXPIRES: 06/30/2015

GTFC - WEST
www.gtfcwest.com
GEOTECH FOUNDATION COMPANY - WEST -
214 SE WALNUT ST., HILLSBORO, OR 97123
PH: (503) 640-1340 FX: (503) 648-6706

PORTLAND, OREGON

GEORAM® ENGINEERED AGGREGATE PIER PLAN

BLOCK 8L MIXED USE
60 NW DAVIS ST.

DATE:	09/11/2014
REVISED:	
REVISED:	
PERMIT SET:	
DRAWN BY:	JJG/NJD
APPROVED BY:	JPM
SCALE:	1/8" = 1'-0"

G1.00

GEODESIGN INC

Memorandum

To:	Jennifer Jenkins	From:	Tacia Miller, P.E., G.E. and Brett Shipton, P.E., G.E.
Company:	Ankrom Moisan Architects, Inc.	Date:	September 18, 2014
Address:	6720 SW Macadam Avenue, Suite 100 Portland, OR 97219		

cc:	Shirley Chalupa, DCI Engineers (via email only) Jeremy Gray GTFC West (via email only)

GDI Project:	Gerding-184-01
RE:	Block 8L NW 1st Avenue and NW Davis Street; Portland, Oregon Aggregate Pier Uplift Testing

GeoDesign, Inc. has evaluated the uplift capacity of compacted aggregate piers for the proposed Block 8L development. We understand the proposed aggregate piers are 30 inches in diameter and are designed to extend a minimum of 10 feet below the bottom of footing for tensile loads. Each uplift aggregate pier has a design uplift capacity of 40 kips. We understand that the piers are designed with a safety factor of 3.0 against uplift.

The specialty contractor has proposed a cyclic load test with a peak tensile load equal to 200 percent of the design load. A minimum of three cycles have been specified. A description of the test is provided on drawing G2.00, prepared by GeoTech Foundation Company – West. As the geotechnical engineer of record for the project, it is GeoDesign's opinion that a one uplift test, as described above, is sufficient to confirm the uplift capacity for the uplift aggregate piers as shown on drawing G1.00. Our opinion is based on the following:

- Our borings show that soil conditions are uniform across the site.
- The aggregate piers are designed with a safety factor of 3.0 for uplift.

Due to the relatively uniform nature of the site, no specific test location has been selected. The test pier will be representative regardless of the location on the site. Also, the test pile may either be a production pier or an additional sacrificial pier, at the contractor's option.

TCM:BAS:kt

One copy submitted (via email only)

Document ID: Gerding-184-01-091814-geom.docx

REGISTERED PROFESSIONAL ENGINEER
56207
OREGON
July 11, 2000
BRETT A. SHIPTON
EXPIRES: 6/30/16

14-211482 STR 01 CO

GEODESIGN INC

Memorandum

To:	Jennifer Jenkins	From:	Tacia Miller, P.E., G.E. and Brett Shipton, P.E., G.E.
Company:	Ankrom Moisan Architects, Inc.	Date:	September 16, 2014
Address:	6720 SW Macadam Avenue, Suite 100 Portland, OR 97219		

cc:	Shirley Chalupa, DCI Engineers (via email only) Jeremy Gray GTFC West (via email only)

GDI Project:	Gerding-184-01
RE:	Block 8L NW 1st Avenue and NW Davis Street Portland, Oregon Review of Rammed Aggregate Pier Design

GeoDesign, Inc. has reviewed the geotechnical parameters used by GeoTech Foundation Company-West in the design of the rammed aggregate pier foundation system as presented in plans and design calculations dated August 2014. In our opinion, geotechnical parameters used are reasonable and consistent with the findings of our geotechnical engineering study at the locations of our drilled borings. Our exploration program is documented in a report entitled *Report of Geotechnical Engineering Services; Block 8L; NW 1st Avenue and NW Davis Street; Portland, Oregon,* dated July 16, 2014.

TCM:BAS:kt
One copy submitted (via email only)
Document ID: Gerding-184-01-091614-geom.docx

REGISTERED PROFESSIONAL ENGINEER
56207
OREGON
July 11, 2000
BRETT A. SHIPTON
EXPIRES: 6/30/16

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GEOTECH FOUNDATION COMPANY
- WEST -

DESIGN CALCULATIONS

Block 8L Mixed Use
60 NW Davis Street
Portland, Oregon

August, 2014

Included Calculations:

- Soil Model -------------------------------- ☑
- Footing Settlement ----------------- ☑
- Footing Bearing Capacity --------------- ☑
- Pier Compression Capacity ------------ ☑
- Pier Tension Capacity --------------- ☑
- Slab On-Grade Support ---------------- ☐
- Slope Stabilization ---------------- ☐
- Liquefaction ------------------------ ☐

By

GEOTECH FOUNDATION COMPANY – WEST®
214 SE WALNUT STREET
HILLSBORO, OR 97123

REGISTERED PROFESSIONAL ENGINEER
65614PE
OREGON
MARCH 13, 2012
JEREMY JOHN GRAY
EXPIRES: 6/30/2015

GeoRam®

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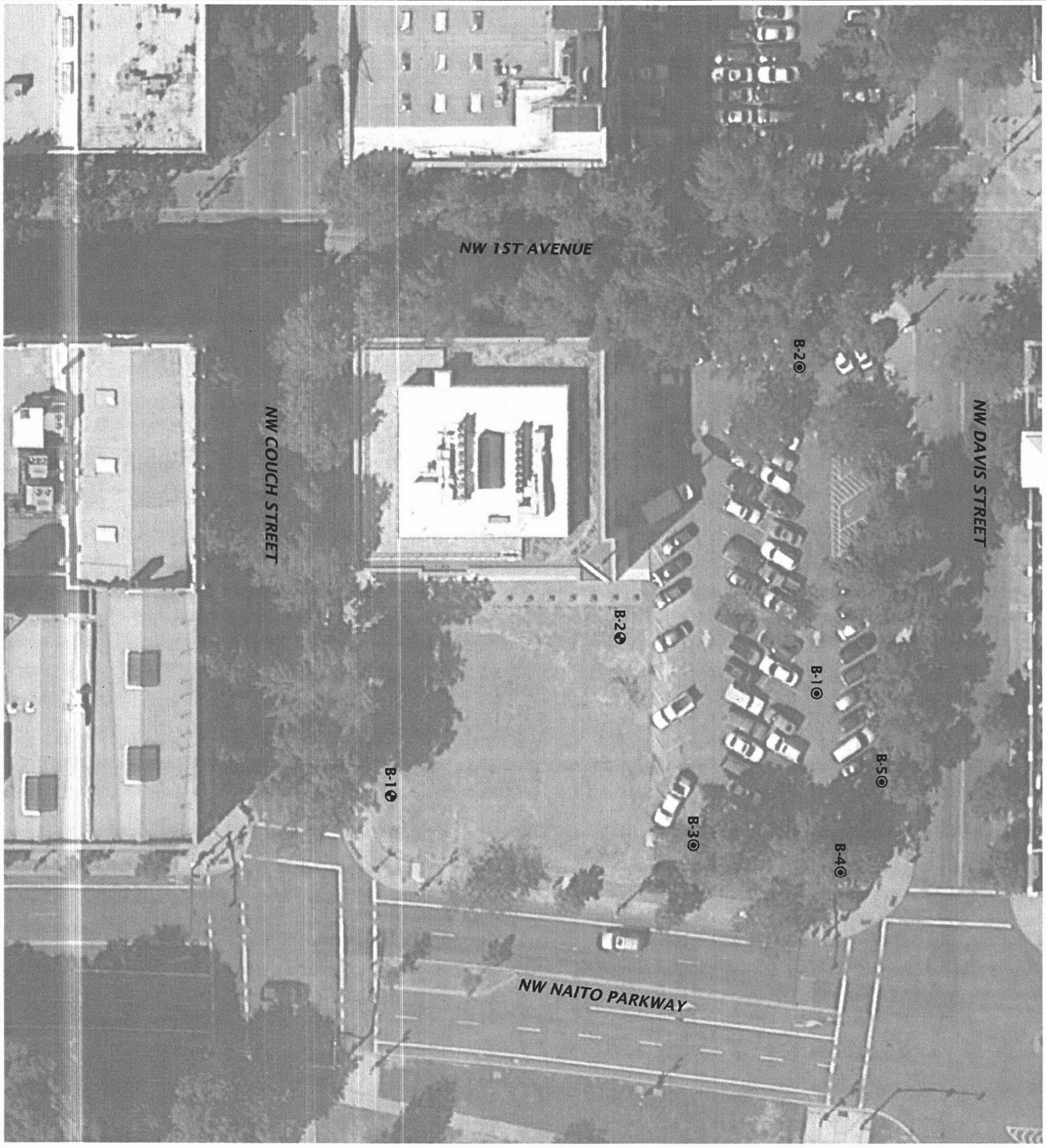
File Name: J:\E-L\Gerding\Gerding-184\Gerding-184-01\Figures\CAD\Gerding-184-01-SP01.dwg | Layout: FIGURE 2



LEGEND:

B-1 ⊙ BORING (SHANNON & WILSON, AUGUST 2003)

B-1 ◐ BORING (GEODESIGN, JUNE 2014)

N

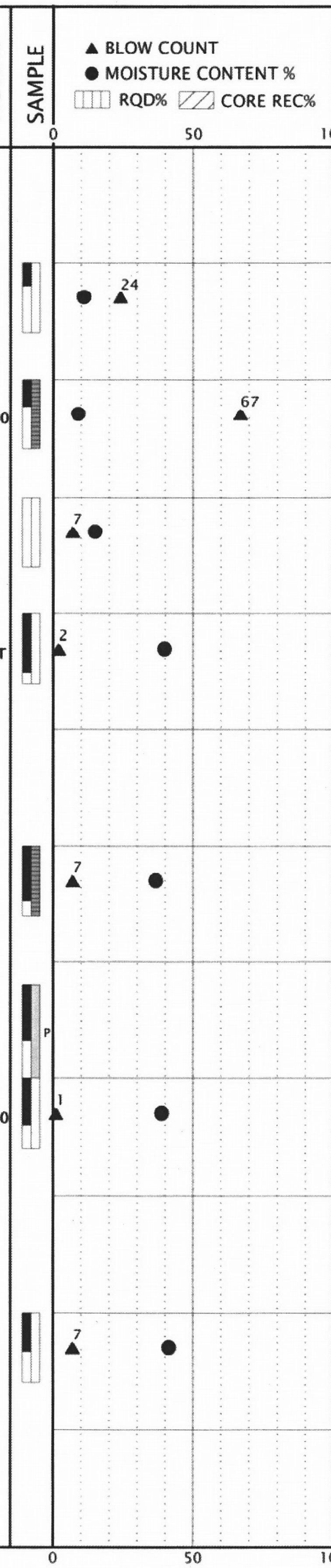
0 40 80

(SCALE IN FEET)

SITE PLAN BASED ON AERIAL PHOTOGRAPH OBTAINED FROM GOOGLE EARTH PRO®, JUNE 17, 2014

GEODESIGN INC 15575 SW Sequoia Parkway - Suite 100 Portland OR 97224 Off 503.968.8787 Fax 503.968.3068	GERDING-184-01	SITE PLAN	
	JULY 2014	BLOCK 8L PORTLAND, OR	FIGURE 2

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	INSTALLATION AND COMMENTS
0.0	Medium dense, gray-brown GRAVEL with sand (GP), trace silt; moist, gravel is subrounded to subangular (0- to 1 1/2-inch-thick root zone) - **FILL.**	30.2		
2.5				
5.0	trace silt at 5.0 feet		P200	Infiltration rate: 60 inches per hour at 5.0 feet. P200 = 5%
7.5	Medium stiff, brown SILT (ML); moist.	22.7 / 7.5		
10.0	soft, light brown, trace sand and organics (rootlets); sand is fine at 10.0 feet		ATT	LL = NP PL = NP
12.5				
15.0	medium stiff, with sand, trace organics (isolated root); moist to wet, root is 1/4-inch diameter at 15.0 feet			Infiltration rate: ~0 inches per hour at 15.0 feet.
17.5				
20.0	soft, gray-brown, sandy; wet at 19.5 feet with gravel; stratified beds of silt at 21.0 feet		P200	P200 = 67%
22.5	Very dense, gray-brown GRAVEL with sand (GP), trace silt; wet.	8.2 / 22.0		
25.0	lens of silty sand from 26.0 to 28.0 feet			
27.5				
30.0				



DRILLED BY: Western States Soil Conservation, Inc. LOGGED BY: JGH COMPLETED: 06/11/14

BORING METHOD: mud rotary and hollow-stem auger (see report text) BORING BIT DIAMETER: 4 7/8-inch/10-inch

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Portland OR 97224
Off 503.968.8787 Fax 503.968.3068

GERDING-184-01	**BORING B-1**	
JULY 2014	BLOCK 8L PORTLAND, OR	**FIGURE A-1**

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	SAMPLE	▲ BLOW COUNT ● MOISTURE CONTENT % ▥ RQD% ▨ CORE REC% (0 – 50 – 100)	INSTALLATION AND COMMENTS
30.0		(continued from previous page)				▲ 70	
32.5							
35.0		dense, with cobbles at 35.0 feet				● ▲ 119	
37.5							
40.0		Very dense, dark gray GRAVEL with sand (GW), trace silt; wet.	-9.8 / 40.0			▲ 30-50/5"	
42.5							
45.0		gravel is subrounded at 45.0 feet				● ▲ 50/3"	
47.5							
50.0		Hard, light brown to gray CLAY (CL), minor gravel and sand; moist.	-20.3 / 50.5			▲ 92	
52.5							
55.0		Very stiff, gray SILT (ML), trace sand and clay; moist, sand is fine.	-24.8 / 55.0			▲ 55	
57.5							
60.0							

DRILLED BY: Western States Soil Conservation, Inc. LOGGED BY: JGH COMPLETED: 06/11/14

BORING METHOD: mud rotary and hollow-stem auger (see report text) BORING BIT DIAMETER: 4 7/8-inch/10-inch

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GERDING-184-01

JULY 2014

BORING B-1
(continued)

BLOCK 8L
PORTLAND, OR

FIGURE A-1

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	▲ BLOW COUNT ● MOISTURE CONTENT % RQD% CORE REC% (0 – 50 – 100)	INSTALLATION AND COMMENTS
60.0		(continued from previous page)				64	
62.5		Very dense, gray-light brown GRAVEL with sand and cobbles (GP), trace silt; moist to wet, tightly packed in light brown-yellow matrix.	32.3 62.5				
65.0						50/3"	
67.5							
70.0						40-50/4"	
72.5							
75.0		Exploration completed at a depth of 75.3 feet.	45.1 75.3			50/4"	
77.5							
80.0							
82.5							
85.0							
87.5							
90.0							

DRILLED BY: Western States Soil Conservation, Inc. LOGGED BY: JGH COMPLETED: 06/11/14

BORING METHOD: mud rotary and hollow-stem auger (see report text) BORING BIT DIAMETER: 4 7/8-inch/10-inch

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GERDING-184-01

JULY 2014

BORING B-1
(continued)

BLOCK 8L
PORTLAND, OR

FIGURE A-1

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	BLOW COUNT / MOISTURE CONTENT %	INSTALLATION AND COMMENTS
0.0	Loose to medium dense, gray-brown SAND with gravel and debris (metal spikes and brick) (SP), trace silt; moist, medium to coarse - **FILL.**	31.4			Flush-mount monument with 1.5 feet of concrete backfill. Top of cut tremie pipe at 0.5 foot. Gravel backfill.
2.5				▲ 10	Bentonite chips.
5.0	Medium dense to dense, gray-brown GRAVEL with sand (GP), trace silt; moist, sand is medium to coarse - **FILL.**	26.4 / 5.0	P200	● ; ▲ 74	Infiltration rate: 6.4 inches per hour at 5.0 feet. P200 = 4%
7.5	dense at 7.5 feet			▲ 40	Cement grout. 1 inch PVC pipe.
9.0	Soft to medium stiff, brown SILT (ML), trace sand; moist.	22.4 / 9.0			
10.0				▲ 4	
12.5			P		
13.5	medium stiff to stiff, light brown, minor sand; moist to wet, sand is fine at 13.5 feet			▲ 8	
15.0	stiff at 15.0 feet		ATT	▲ 17 ; ●	Infiltration rate: ~0 inches per hour at 15.0 feet. LL = 34% PL = 26%
17.5					
20.0	Medium dense, light brown SAND with silt (SP-SM); wet, fine.	11.4 / 20.0		▲ 13	
22.5					
23.0	Very dense, gray-brown GRAVEL with sand (GW), trace silt; wet, stratified beds of brown silty sand.	8.4 / 23.0			
25.0				▲ 63	
27.5	lens of silty sand from 27.5 to 29.0 feet				
30.0					Vibrating wire piezometer #SN140084 set at 29.3 feet.

Legend: ▲ BLOW COUNT ● MOISTURE CONTENT % RQD% CORE REC% (scale 0 – 50 – 100)

DRILLED BY: Western States Soil Conservation, Inc. LOGGED BY: JGH COMPLETED: 06/12/14

BORING METHOD: mud rotary and hollow-stem auger (see report text) BORING BIT DIAMETER: 4 7/8-inch/10-inch

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GERDING-184-01

JULY 2014

BORING B-2

BLOCK 8L
PORTLAND, OR

FIGURE A-2

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	BLOW COUNT	INSTALLATION AND COMMENTS
30.0	(continued from previous page)			63	
35.0	with clay at 35.0 feet			76	
37.5					Bottom of tremie pipe at 37.3 feet.
					Bentonite chips.
40.0	gray at 40.0 feet			98	
45.0	Very dense, gray GRAVEL with silt, sand, and cobbles (GP-GM); wet, sand is fine to coarse.	13.6 / 45.0		26-50/3"	
50.0				70	
52.0	Very stiff, light brown SILT (ML); moist.	20.6 / 52.0			
55.0			ATT	19	LL = 32% PL = 26%
60.0					

▲ BLOW COUNT ● MOISTURE CONTENT % RQD% CORE REC%

0 50 100

DRILLED BY: Western States Soil Conservation, Inc. LOGGED BY: JGH COMPLETED: 06/12/14

BORING METHOD: mud rotary and hollow-stem auger (see report text) BORING BIT DIAMETER: 4 7/8-inch/10-inch

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	SAMPLE	▲ BLOW COUNT ● MOISTURE CONTENT % RQD% CORE REC% (0 – 50 – 100)	INSTALLATION AND COMMENTS
60.0		(continued from previous page)				58	
62.5		Very dense, brown-gray GRAVEL with sand and cobbles (GW), trace silt; moist to wet (Troutdale Formation).	30.6 / 62.0				
65.0						50/3"	
67.5							
70.0						30-120/6"	
72.5							
75.0		Exploration completed at a depth of 75.5 feet.	44.1 / 75.5			50/6"	
77.5							
80.0							
82.5							
85.0							
87.5							
90.0							

0 50 100

DRILLED BY: Western States Soil Conservation, Inc.   LOGGED BY: JGH   COMPLETED: 06/12/14

BORING METHOD: mud rotary and hollow-stem auger (see report text)   BORING BIT DIAMETER: 4 7/8-inch/10-inch

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GERDING-184-01

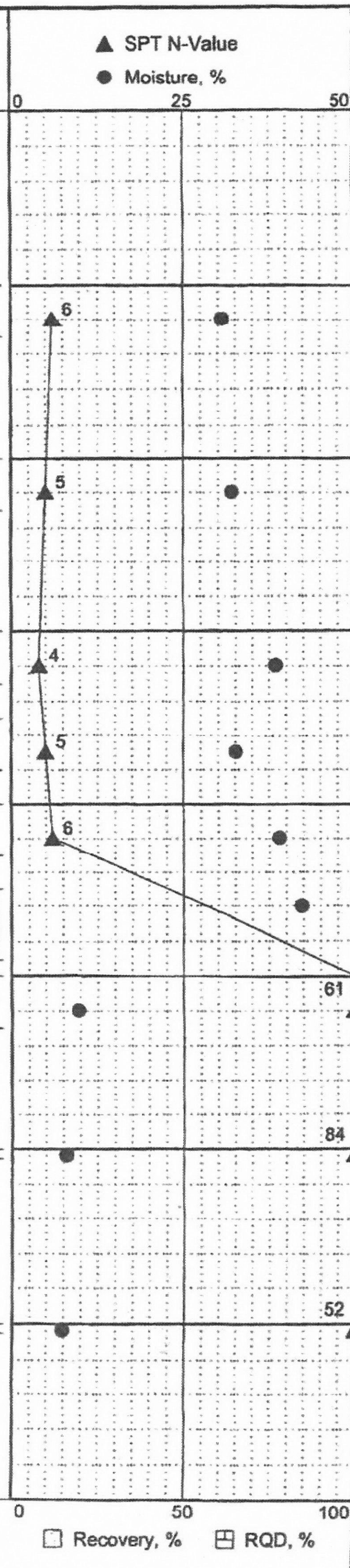
JULY 2014

BORING B-2
(continued)

BLOCK 8L
PORTLAND, OR

FIGURE A-2

Ground Water	Remarks	Elev. Depth Feet 30.0	CLASSIFICATION OF MATERIAL	Log	Depth In Feet	Samples
		29.3 / 0.7	**PAVEMENT**, 4 inches of Asphalt over 4 inches of 3/4-inch minus base rock. (Pavement Section)		0	
			Gravelly Clayey Sandy SILT, medium stiff, gray and brown, moist, low plasticity, sand is fine-grained, gravel is fine-grained and angular. (Fill)		5	S-1
	%-200 = 62	20.0 / 10.0	**Sandy SILT**, slightly clayey, medium stiff, brown, moist, low-plasticity, micaceous, sand is fine-grained. (Recent Alluvium)		10	S-2
	%-200 = 76	17.0 / 13.0	**Clayey SILT**, trace of sand, soft to medium stiff becoming medium stiff, brown with gray and rust mottling, moist to wet, low to medium plasticity, micaceous, sand is fine-grained. (Recent Alluvium)		15	S-3
	%-200 = 88					S-4
▼	7/28/2003 Approximate Water Level %-200 = 45 Dry Density = 77.5 pcf Torvane = 0.1 tsf Pocket Pen. = 0.25 tsf	10.5 / 19.5	**Stratified Clayey SILT and Silty SAND**, 2- to 4-inch lenses, medium stiff, brown, wet, low plasticity clayey silt, no to low plasticity silty sand, micaceous, sand is fine-grained. (Fine-grained Catastrophic Flood Deposits)		20	S-5
		7.5 / 22.5	**Clayey Sandy SILT**, medium stiff, brown, wet, low plasticity, micaceous, sand is fine-grained. (Fine-grained Catastrophic Flood Deposits)			S-6
	%-200 = 78	6.0 / 24.0	Becomes slightly gravelly, fine-grained and rounded. **Silty Sandy GRAVEL**, slightly clayey, dense to very dense, gray, wet, low plasticity matrix, micaceous, fine- to coarse-grained sand and gravel, gravel is subrounded to rounded. (Coarse-grained Catastrophic Flood Deposits)		25	S-7
		0.0 / 30.0	**Sandy GRAVEL**, with scattered to numerous cobbles, very dense, gray, wet, micaceous, fine- to coarse-grained sand and gravel, gravel is subrounded to rounded. (Coarse-grained Catastrophic Flood Deposits)		30	S-8
					35	S-9



LEGEND

- ⌶ = 2.0" O.D. Split Spoon Sample
- ⌶ = 3.0" O.D. Thin-Walled Sample
- * = Sample Not Recovered
- ⌶ = 3.0" O.D. Split Spoon Sample
- ⌶ = Core Rock Sample

Impervious Seal (Bentonite)
Cement Grout
Random Backfill
Granular Backfill
Ground Water Level on Date Shown
Piezometer/Inclinometer Tubing
Perforated Zone

ATTERBERG LIMITS
Liquid Limit
Natural Water Content
Plastic Limit

NOTE:
Lines between soil/rock units are approximate and transition may be gradual.

Portland Fire Station No. 1
Preliminary Sites Assesment, Block 8

LOG OF BORING B-1

page 1 of 2

August 2003 *24-1-03235-001*

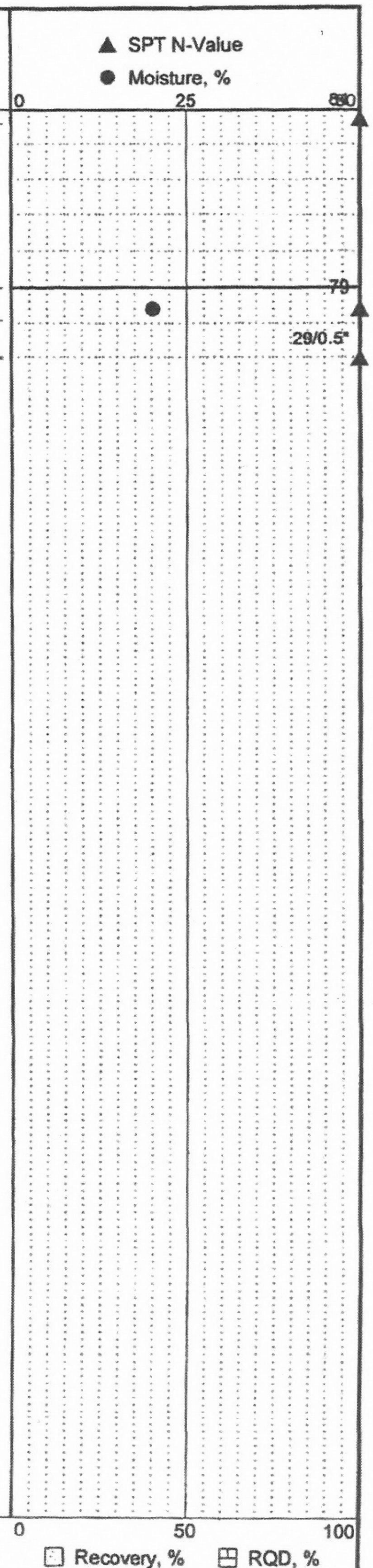
SHANNON & WILSON, INC.
Geotechnical Consultants
Portland, Oregon

FIG. 3

REV 4

WLG FS-1 BLOCK 8.GPJ 8/29/03 Log: DJH Typ: DJH Rev:

Ground Water	Remarks	Elev. Depth Feet	CLASSIFICATION OF MATERIAL	Log	Depth In Feet	Samples
			Sandy GRAVEL, continued.			*S-10
			Lost circulation at 41.5 feet, approximately 50 gallons of drill mud leaked into formation.			
					45	S-11
		-17.1	Drilling progress became very slow at 46 feet.			S-12
		47.1	Bottom of Boring, Completed 7/28/03			



LEGEND

- = 2.0" O.D. Split Spoon Sample
- = 3.0" O.D. Thin-Walled Sample
- * = Sample Not Recovered
- = 3.0" O.D. Split Spoon Sample
- = Core Rock Sample

- Impervious Seal (Bentonite)
- Cement Grout
- Random Backfill
- Granular Backfill
- Ground Water Level on Date Shown
- Piezometer/Inclinometer Tubing
- Perforated Zone

ATTERBERG LIMITS

- Liquid Limit
- Natural Water Content
- Plastic Limit

NOTE:
Lines between soil/rock units are approximate and transition may be gradual.

Portland Fire Station No. 1
Preliminary Sites Assesment, Block 8

LOG OF BORING B-1

page 2 of 2

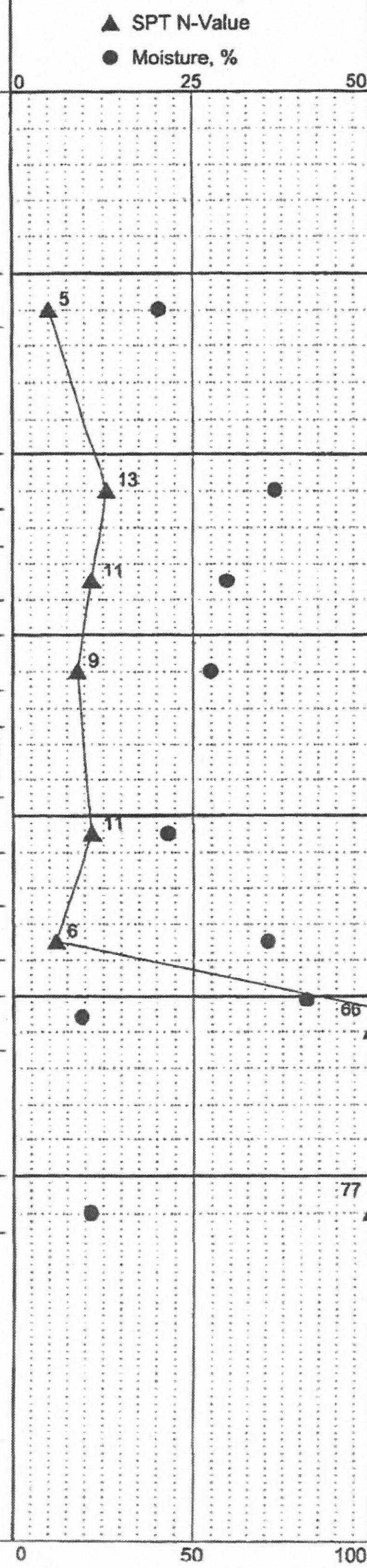
August 2003 *24-1-03235-001*

SHANNON & WILSON, INC.
Geotechnical Consultants
Portland, Oregon

FIG. 3

REV 4

Ground Water	Remarks	Elev. Depth Feet 31.0	CLASSIFICATION OF MATERIAL	Log	Depth in Feet	Samples
			Pavement, 4 inches of Asphalt over 20 inches of 1-inch minus base rock. (Pavement Section)		0	
		29.0 2.0	**Clayey Sandy SILT**, medium stiff to stiff, gray to brown, moist, low plasticity, micaceous; sand is fine grained. (Fill)			
	%-200=64.9				5	S-1
	%-200=64.6				10	S-2
		19.0 12.0	**Sandy SILT** with trace clay, stiff, light brown, moist, low plasticity, micaceous; sand is fine-grained. (Fine-Grained Catastrophic Flood Deposits)			
	%-200=60.3					S-3
	%-200=94.3				15	S-4
		14.0 17.0	**Silty SAND** with trace clay, medium dense grading to loose, light brown, moist grading to wet, non-plastic, micaceous, fine-grained. (Fine-Grained Catastrophic Flood Deposits)			S-5
	%-200=38.3				20	S-6
▼	8/18/2003 %-200=39.3					S-7
		5.7	Clayey silt layer at contact.		25	
	%-200=87.8	25.3	**Sandy GRAVEL** with some cobbles and trace silt, very dense, gray, wet, non-plastic, micaceous; fine- to coarse-grained sand; fine to coarse sub-rounded to rounded gravel. (Coarse-Grained Catastrophic Flood Deposits)			S-8
					30	S-9
		-0.5 31.5	Bottom of Boring, Completed 8/18/03.			



LEGEND

- ⌶ = 2.0" O.D. Split Spoon Sample
- ⌶ = 3.0" O.D. Thin-Walled Sample
- * = Sample Not Recovered
- ⌶ = 3.0" O.D. Split Spoon Sample
- ⌶ = Core Rock Sample

NOTE:
Lines between soil/rock units are approximate and transition may be gradual.

- Impervious Seal (Bentonite)
- Cement Grout
- Random Backfill
- Granular Backfill
- ▽ Ground Water Level on Date Shown
- Piezometer/Inclinometer Tubing
- Perforated Zone

ATTERBERG LIMITS
- Liquid Limit
- Natural Water Content
- Plastic Limit

Portland Fire Station No. 1
Preliminary Sites Assesment, Block 8

LOG OF BORING B-2

page 1 of 1

August 2003 *24-1-03235-001*

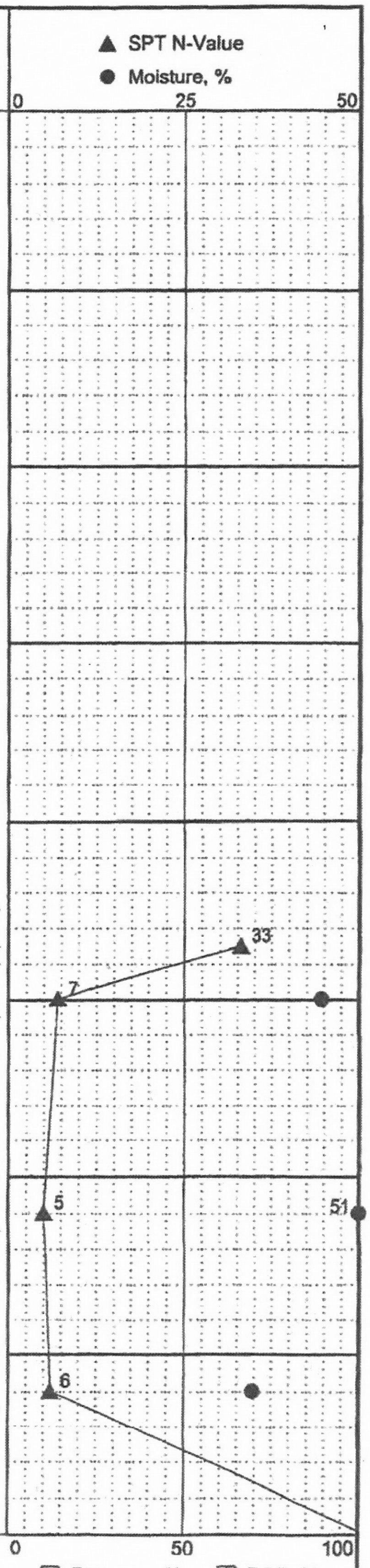
SHANNON & WILSON, INC.
Geotechnical Consultants
Portland, Oregon

FIG. 4

WLG FS-1 BLOCK 8.GPJ 8/29/03 Log: DJH Typ: KEE Rev:

Ground Water	Remarks	Elev. Depth Feet 30.0	CLASSIFICATION OF MATERIAL	Depth In Feet
		28.0 / 2.0	**Pavement**, 4 inches of asphalt over 20 inches of base rock. (Pavement Section)	0
		11.0 / 19.0	**Gravel**, medium dense 0 to 5 feet; loose 5 to 19 ft; hole kept caving, cuttings consited of 1/2 to 1/4 inch subround to sub angular rock. (Sewer Trench Backfill)	5, 10, 15
		2.0 / 28.0	**Silty SAND**, dark brown, wet, non-plastic. (Recent Alluvium) Wood encountered 20 to 25 feet. Poor sample recovery.	20, 25
		-3.0 / 33.0	**Sandy SILT**, medium stiff, dark gray, wet, low to medium plasticity; sand is fine-grained; contains some shredded wood. (Recent Alluvium)	30
		-8.0 / 38.0	**Clayey Sandy SILT**, medium stiff, dark gray, wet, low to medium plasticity; trace wood. (Recent Alluvium)	35
			Sandy GRAVEL with trace cobbles, dense, gray, wet. (Coarse-Grained Catastrophic Flood Deposits)	

Samples: *S-1, S-4, S-3, S-5, S-6



LEGEND

- = 2.0" O.D. Split Spoon Sample
- = 3.0" O.D. Thin-Walled Sample
- * = Sample Not Recovered
- = 3.0" O.D. Split Spoon Sample
- = Core Rock Sample

Impervious Seal (Bentonite)
Cement Grout
Random Backfill
Granular Backfill
Ground Water Level on Date Shown
Piezometer/Inclinometer Tubing
Perforated Zone

ATTERBERG LIMITS
Liquid Limit
Natural Water Content
Plastic Limit

NOTE:
Lines between soil/rock units are approximate and transition may be gradual.

Portland Fire Station No. 1
Preliminary Sites Assesment, Block 8

LOG OF BORING B-3

page 1 of 3

August 2003 *24-1-03235-001*

SHANNON & WILSON, INC.
Geotechnical Consultants
Portland, Oregon

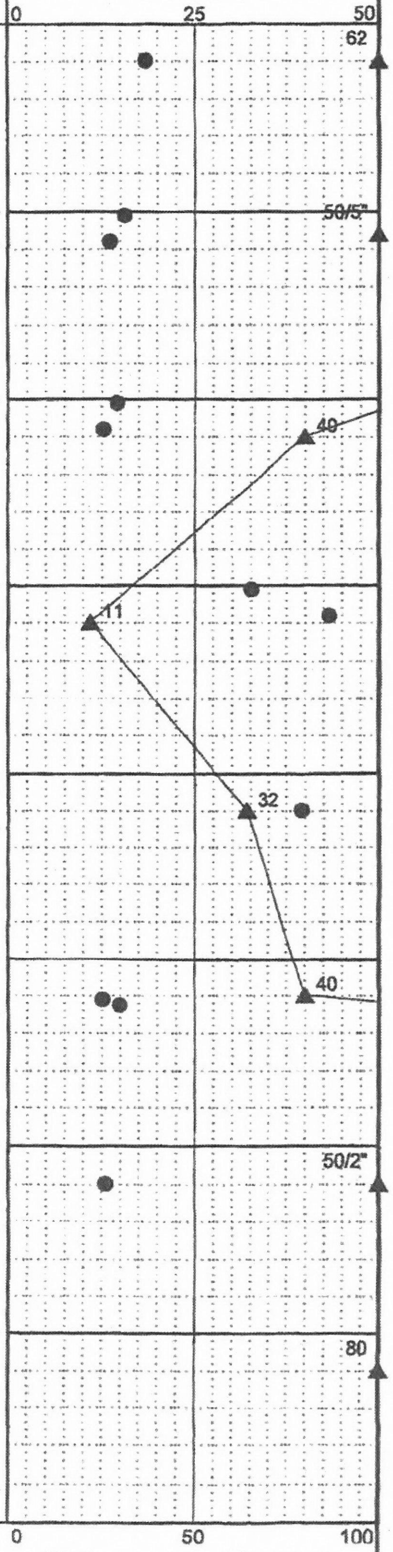
FIG. 5

Log: djh/def/Typ: deh Rev:

WLG FS-1 BLOCK 8.GPJ 8/29/03

Ground Water	Remarks	Elev. Depth Feet	CLASSIFICATION OF MATERIAL	Log	Depth In Feet	Samples
			Sandy GRAVEL, continued.			S-7
		-12.0 42.0	**GRAVEL** with cobbles, very dense, gray, wet, non-plastic, sub angular. (Coarse-Grained Catastrophic Flood Deposits)		45	S-8
					50	S-9
		-23.0 53.0	**Silty SAND**, slightly sandy, medium dense, wet, gray, micaceous. (Fine-Grained Catastrophic Flood Deposits)			
	%-200=28.7 %-200=94.4	-25.1 55.1	Stratified **Clayey SILT**, stiff, gray to brown, moist, medium plasticity. (Older Alluvium)		55	S-10
	%-200=86.8				60	S-11
		-32.5 62.5	Stratified **Sandy GRAVEL to Gravely SAND** with some cobbles, dense to very dense, red-yellow-gray gravel clasts, sand layers are red-brown, moist; sand is medium-grained; gravel is fine and subrounded to rounded. (Coarse-Grained Catastrophic Flood Deposits)		65	S-12
					70	S-13
					75	S-14

▲ SPT N-Value
● Moisture, %



☐ Recovery, % ⊞ RQD, %

LEGEND

- = 2.0" O.D. Split Spoon Sample
- = 3.0" O.D. Thin-Walled Sample
- * = Sample Not Recovered
- = 3.0" O.D. Split Spoon Sample
- = Core Rock Sample

- Impervious Seal (Bentonite)
- Cement Grout
- Random Backfill
- Granular Backfill
- Ground Water Level on Date Shown
- Piezometer/Inclinometer Tubing
- Perforated Zone

ATTERBERG LIMITS
- Liquid Limit
- Natural Water Content
- Plastic Limit

NOTE:
Lines between soil/rock units are approximate and transition may be gradual.

Portland Fire Station No. 1
Preliminary Sites Assesment, Block 8

LOG OF BORING B-3

page 2 of 3

August 2003 *24-1-03235-001*

SHANNON & WILSON, INC.
Geotechnical Consultants
Portland, Oregon

FIG. 5

Log: djh/def/yp: deh Rev:

WLG FS-1 BLOCK 8.GPJ 8/29/03

Ground Water	Remarks	Elev. Depth Feet	CLASSIFICATION OF MATERIAL	Log	Depth In Feet	Samples	▲ SPT N-Value ● Moisture, % (0 – 25 – 50)
			Stratified Sandy GRAVEL to Gravelly SAND continued. Drilling progress becomes very slow at 83.5 feet (Troutdale Formation?)				
		-55.0 85.0	Bottom of Boring, Completed 8/19/03.		85	*S-15	50/1st 3"

0 – 50 – 100
☐ Recovery, % ⊟ RQD, %

LEGEND

- = 2.0" O.D. Split Spoon Sample
- = 3.0" O.D. Thin-Walled Sample
- * = Sample Not Recovered
- = 3.0" O.D. Split Spoon Sample
- = Core Rock Sample

Impervious Seal (Bentonite)
Cement Grout
Random Backfill
Granular Backfill
Ground Water Level on Date Shown
Piezometer/Inclinometer Tubing
Perforated Zone

ATTERBERG LIMITS
Liquid Limit
Natural Water Content
Plastic Limit

NOTE:
Lines between soil/rock units are approximate and transition may be gradual.

Portland Fire Station No. 1
Preliminary Sites Assesment, Block 8

LOG OF BORING B-3
page 3 of 3

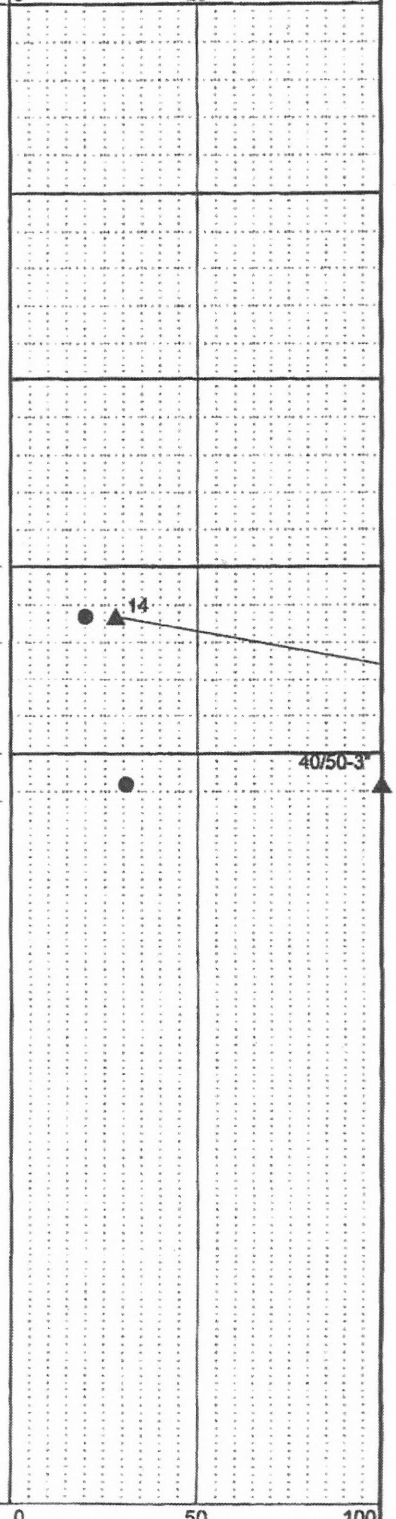
August 2003 *24-1-03235-001*

SHANNON & WILSON, INC.
Geotechnical Consultants
Portland, Oregon

FIG. 5

WLG FS-1 BLOCK 8.GPJ 8/29/03 Log: djh/def/Typ: deh Rev:

Ground Water	Remarks	Elev. Depth Feet 30.0	CLASSIFICATION OF MATERIAL	Log	Depth In Feet	Samples
			Pavement, 4 inches of asphalt over 20 inches of base rock. (Pavement Section)		0	
		28.0 / 2.0	**GRAVEL**, loose, cuttings consisted of 1/2 to 1/4 inch subround to sub angular rock. (Sewer Trench Backfill)			
		25.0 / 5.0	**GRAVEL**, pit run gravel with binder. (Sewer Trench Backfill)		5	
		24.0 / 6.0	**GRAVEL**, loose, cuttings consisted of 1/2 to 1/4 inch subround to sub angular rock. (Sewer Trench Backfill)		10	
		14.0 / 16.0	Stratified**SAND and GRAVEL**, very dense, brown to gray, moist; contains cement dust. (Sewer Trench Backfill)		15	S-1
					20	S-2
		8.5	Hit 48 by 56-inch concrete sewer pipe.			
		21.5	Bottom of Boring, Completed 8/20/03.			



LEGEND

- = 2.0" O.D. Split Spoon Sample
- = 3.0" O.D. Thin-Walled Sample
- * = Sample Not Recovered
- = 3.0" O.D. Split Spoon Sample
- = Core Rock Sample

Impervious Seal (Bentonite)
Cement Grout
Random Backfill
Granular Backfill
Ground Water Level on Date Shown
Piezometer/Inclinometer Tubing
Perforated Zone

ATTERBERG LIMITS
Liquid Limit
Natural Water Content
Plastic Limit

NOTE:
Lines between soil/rock units are approximate and transition may be gradual.

Portland Fire Station No. 1
Preliminary Sites Assesment, Block 8

LOG OF BORING B-4

page 1 of 1

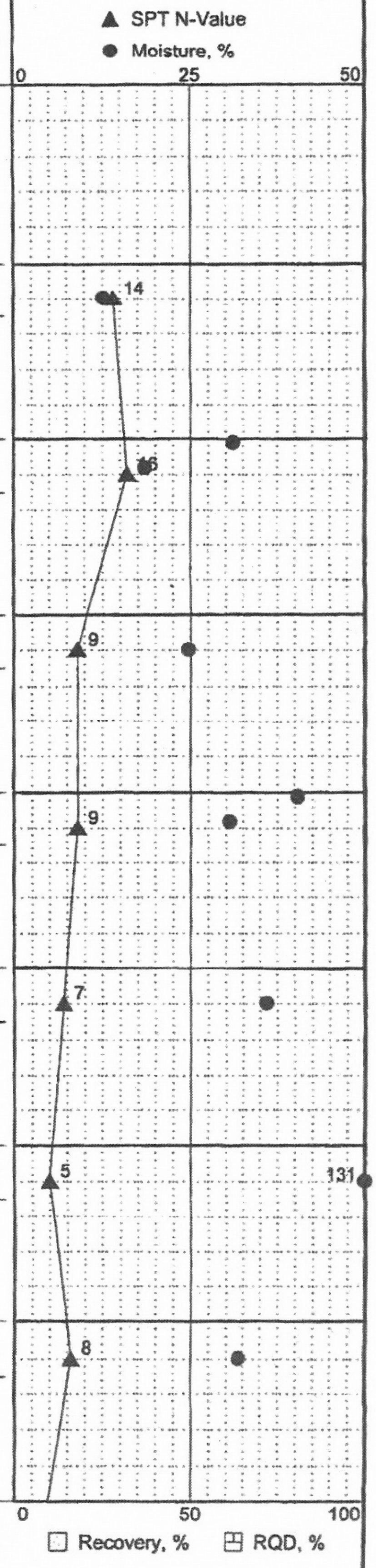
August 2003 *24-1-03235-001*

SHANNON & WILSON, INC.
Geotechnical Consultants
Portland, Oregon

FIG. 6

WLG FS-1 BLOCK 8.GPJ 8/29/03 Log: rbb/def/yp: deh Rev:

Ground Water	Remarks	Elev. Depth Feet 30.0	CLASSIFICATION OF MATERIAL	Log	Depth In Feet	Samples
			Pavement, 4 inches of asphalt over 20 inches of base rock. (Pavement Section)		0	
		28.0 2.0	**Sandy Silty GRAVEL**, medium dense, brown, moist. (Fill)			
					5	S-1
	%-200=83.9 %-200=14.2	19.0 11.0	Stratified **Silty SAND and Sandy Clayey SILT**, loose/soft to medium stiff, gray, moist to wet, non-plastic to medium plasticity; stratified in layers from 3 to 12 inches thick; contains occassional pieces of wood. (Recent Alluvium)		10	S-2
	%-200=22.3				15	S-3
	%-200=42.1 %-200=61.3				20	S-4
	%-200=18.5				25	S-5
	%-200=86.9				30	S-6
	%-200=19.1				35	S-7



LEGEND

- = 2.0" O.D. Split Spoon Sample
- = 3.0" O.D. Thin-Walled Sample
- * = Sample Not Recovered
- = 3.0" O.D. Split Spoon Sample
- = Core Rock Sample

NOTE:
Lines between soil/rock units are approximate and transition may be gradual.

- Impervious Seal (Bentonite)
- Cement Grout
- Random Backfill
- Granular Backfill
- Ground Water Level on Date Shown
- Piezometer/Inclinometer Tubing
- Perforated Zone

ATTERBERG LIMITS
- Liquid Limit
- Natural Water Content
- Plastic Limit

Portland Fire Station No. 1
Preliminary Sites Assesment, Block 8

LOG OF BORING B-5

page 1 of 3

August 2003 — *24-1-03235-001*

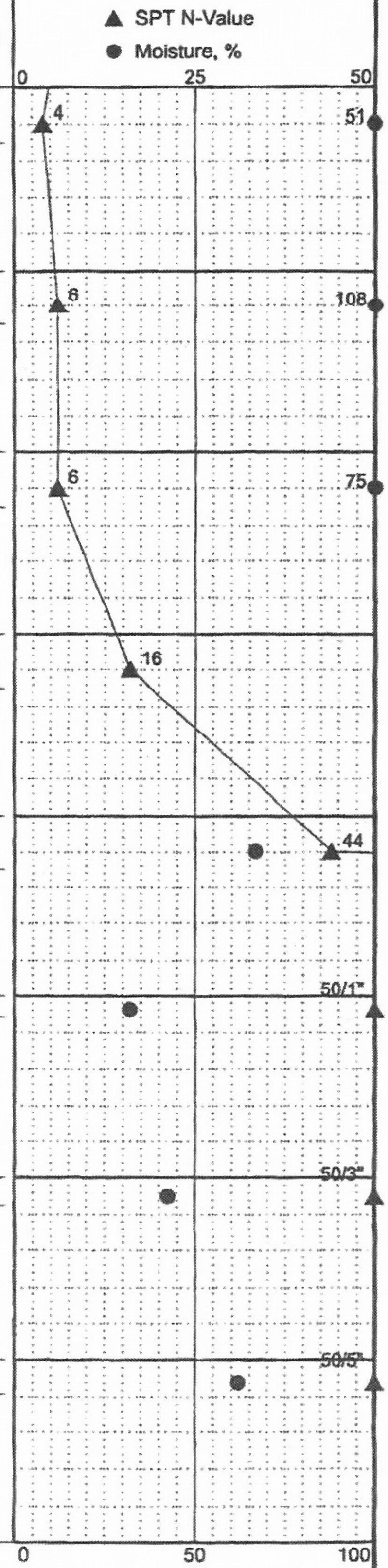
SHANNON & WILSON, INC.
Geotechnical Consultants
Portland, Oregon

FIG. 7

Log: deh Typ: deh Rev:

WLG FS-1 BLOCK 8.GPJ 8/29/03

Ground Water	Remarks	Elev. Depth Feet	CLASSIFICATION OF MATERIAL	Log	Depth In Feet	Samples
	%-200=91.8		Stratified **Silty SAND and Sandy Clayey SILT**, continued.			S-8
	%-200=77.7		Abundant wood fragments at 45 to 50 feet.		45	S-9
	%-200=39.2				50	S-10
		-24.5 54.5	Poor recovery...**Clayey SILT** with lenses of gravel; possibly contains wood. (Recent Alluvium)		55	S-11
		-31.0 61.0	Stratified **Sandy GRAVEL and Gravelly SAND**, very dense, dark gray, wet, non-plastic; sand is medium- to coarse-grained; subrounded to rounded gravel to 2 inches; contains layers of sandy gravel, sand and gravelly sand. (Older Alluvium)		60	S-12
					65	S-13
					70	S-14
					75	S-15



LEGEND

- = 2.0" O.D. Split Spoon Sample
- = 3.0" O.D. Thin-Walled Sample
- * = Sample Not Recovered
- = 3.0" O.D. Split Spoon Sample
- = Core Rock Sample

- Impervious Seal (Bentonite)
- Cement Grout
- Random Backfill
- Granular Backfill
- Ground Water Level on Date Shown
- Piezometer/Inclinometer Tubing
- Perforated Zone

ATTERBERG LIMITS

- Liquid Limit
- Natural Water Content
- Plastic Limit

NOTE:
Lines between soil/rock units are approximate and transition may be gradual.

Portland Fire Station No. 1
Preliminary Sites Assesment, Block 8

LOG OF BORING B-5

page 2 of 3

August 2003 — *24-1-03235-001*

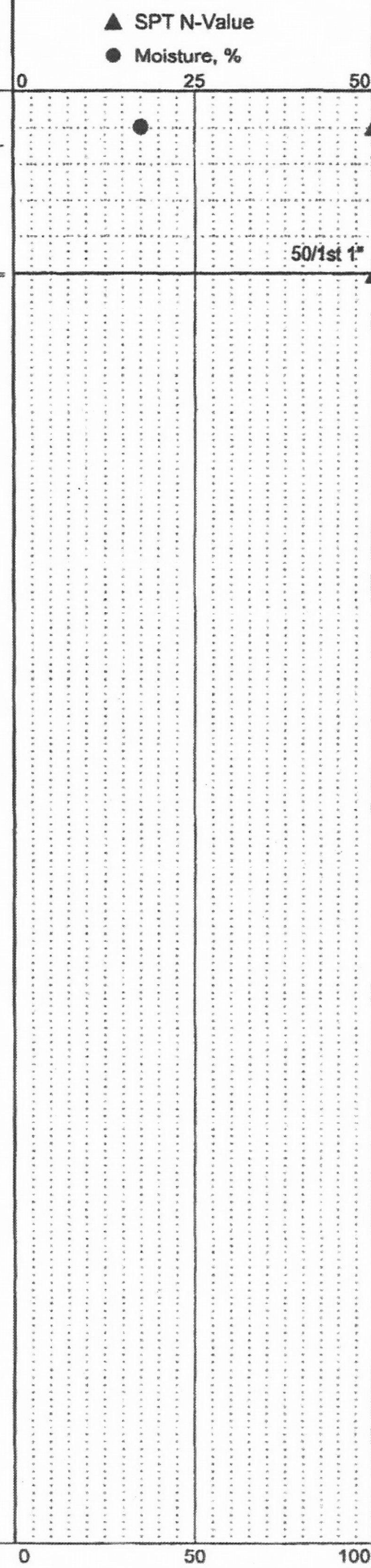
SHANNON & WILSON, INC.
Geotechnical Consultants
Portland, Oregon

FIG. 7

WLG FS-1 BLOCK 8.GPJ 8/29/03

Log: deh Typ: deh Rev:

Ground Water	Remarks	Elev. Depth Feet	CLASSIFICATION OF MATERIAL	Log	Depth In Feet	Samples
	%-200=3.4		**Stratified Sandy GRAVEL and Gravelly SAND**, continued.			S-16
		-52.0 82.0	**Sandy GRAVEL and COBBLES**, very dense; poor recovery. (Troutdale Formation?) Drilling progress became very slow at 46 feet.		85	*S-17
		-55.1 85.1	Bottom of Boring, Completed, 8/25/03.			



LEGEND

- = 2.0" O.D. Split Spoon Sample
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- * = Sample Not Recovered
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Impervious Seal (Bentonite)
Cement Grout
Random Backfill
Granular Backfill
Ground Water Level on Date Shown
Piezometer/Inclinometer Tubing
Perforated Zone

ATTERBERG LIMITS
Liquid Limit
Natural Water Content
Plastic Limit

NOTE:
Lines between soil/rock units are approximate and transition may be gradual.

Portland Fire Station No. 1
Preliminary Sites Assesment, Block 8

LOG OF BORING B-5

August 2003 *24-1-03235-001*

SHANNON & WILSON, INC.
Geotechnical Consultants
Portland, Oregon

FIG. 7

Log: deh Typ: deh Rev:

WLG FS-1 BLOCK 8.GPJ 8/29/03

GEOTECH FOUNDATION COMPANY - WEST ®

ENGINEERED AGGREGATE PIER SYSTEM

Sheet No. 1 of 1

Job No.

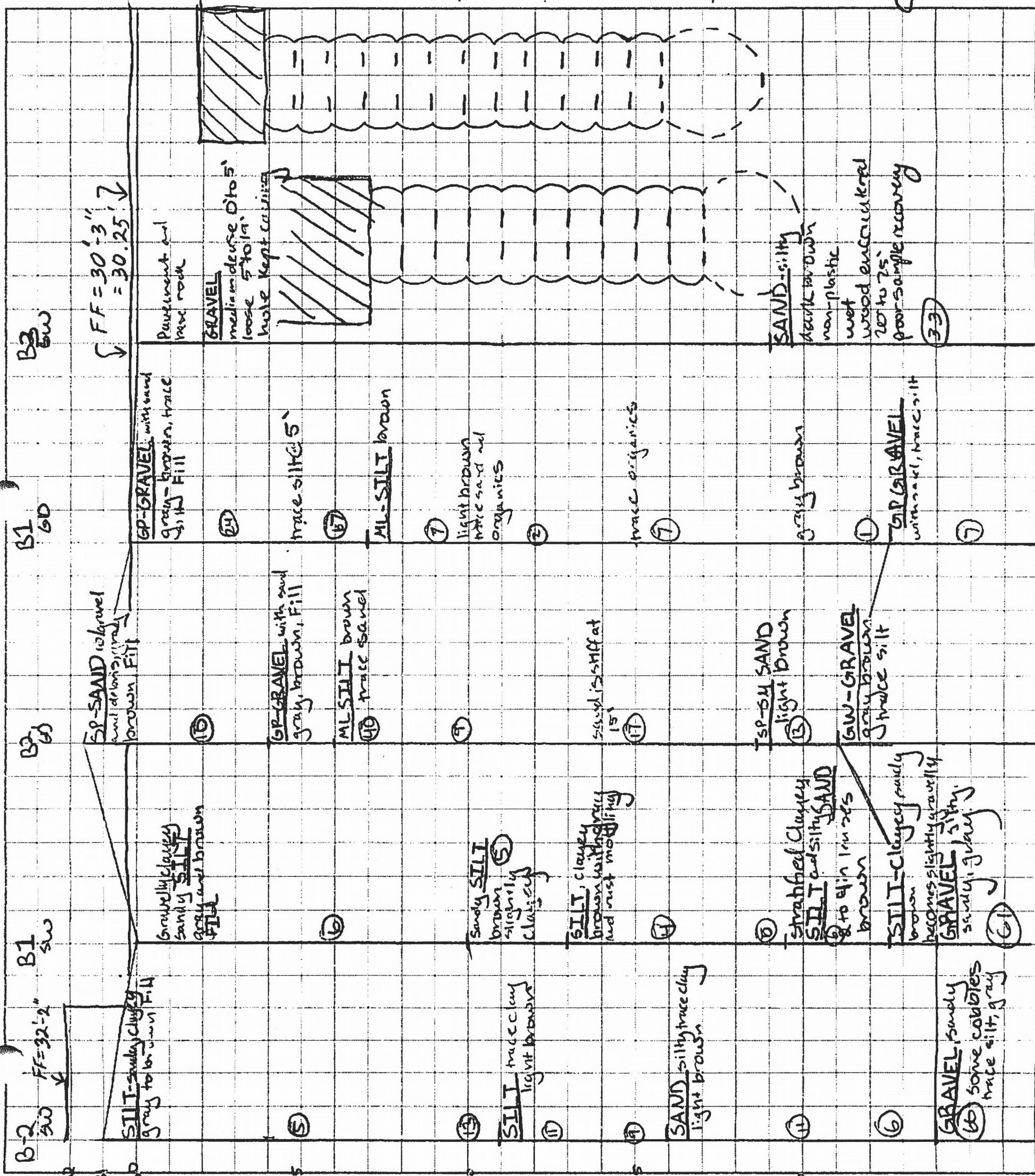
Job Block 8L

By ELG

Date 8/12/14

Client

Subject Soil profile for B2, B1, B2, and Boring 1



SOIL MODEL

Project: Block 8L Mixed Use Portland, Oregon

GeoRam

Reference Borings:

existing grade = 30.25
finish floor elev = 30.25

Depth to GWL = 10 feet
Layer Thickness* = 1 feet

*Recommend Layer Thickness 1' or 2' for accuracy

Depth to Bearing Layer = 30 feet

elev @ top of layer	Soil Type	Stratum Number	Depth Below FF to Bottom of Stratum	Average Depth	Over Consolidation Ratio	Compression Index: Recompression (strain based)	Compression Index: Virgin (strain based)	Soil Behavior (Clay/Sand) C or S	Unsubmerged Unit Weight -pcf- γ	Soils With Internal Frictional Component, Predominantly SILTY and SANDY soils (S): Friction Angle, Drained Φ_d	Friction Angle, Undrained Φ_u	Cohesion -psf- c	CLAY (C) Undrained Strength -psf- S_u	E_s (psi)	E_s (ksf)	SPT N
30.25	ML	1	1.00	0.50				s	110	27				0		
29.25	ML	2	2.00	1.50				s	110	27				0		
28.25	ML	3	3.00	2.50				s	110	27				0		
27.25	ML	4	4.00	3.50				s	110	27				0		
26.25	ML	5	5.00	4.50				s	110	27				0		
25.25	ML	6	6.00	5.50				s	110	27				0		
24.25	ML	7	7.00	6.50				s	110	27				0		
23.25	ML	8	8.00	7.50				s	110	27				0		
22.25	ML	9	9.00	8.50				s	110	27				0		
21.25	ML	10	10.00	9.50				s	110	27				0		
20.25	ML	11	11.00	10.50				s	110	27				348	50	2
19.25	ML	12	12.00	11.50				s	110	27				348	50	2
18.25	ML	13	13.00	12.50				s	110	27				348	50	2
17.25	ML	14	14.00	13.50				s	110	27				565	81	7
16.25	ML	15	15.00	14.50				s	110	27				565	81	7
15.25	ML	16	16.00	15.50				s	110	27				565	81	7
14.25	ML	17	17.00	16.50				s	110	27				565	81	7
13.25	ML	18	18.00	17.50				s	110	27				565	81	7
12.25	ML	19	19.00	18.50				s	110	27				304	44	1
11.25	ML	20	20.00	19.50				s	110	27				304	44	1
10.25	ML	21	21.00	20.50				s	115	27				304	44	1
9.25	ML	22	22.00	21.50				s	115	27				304	44	1
8.25	GP	23	23.00	22.50				s	130	40				9743	1403	50
7.25	GP	24	24.00	23.50				s	130	40				9743	1403	50
6.25	GP	25	25.00	24.50				s	130	40				9743	1403	50
5.25	GP	26	26.00	25.50				s	130	40				9743	1403	50
4.25	ML	27	27.00	26.50				s	115	30				565	81	7
3.25	ML	28	28.00	27.50				s	115	30				565	81	7
2.25	GP	29	29.00	28.50				s	130	40				9743	1403	50
1.25	GP	30	30.00	29.50				s	130	40				9743	1403	50

NOTE: if **S** : only Φ_d and C are required
if **C** : Φ_u and C, or S_u are required
(if a value is input for S_u, it supercedes both Φ_u and C)

Project: Block 8L Mixed Use
Location: Portland, Oregon
Date: 9/3/2014 9:21

GeoRam®

REINFORCED ZONE

Pier drill diameter	(d) =	30	in
Effective diameter	(d_{eff}) =	36	in
Pier	K_{pier} =	200	pci
Matrix Soil	K_{soil} =	15	pci
Pier Unit Weight	$\gamma pier$ =	135	lbs/ft^3
Pier Friction Angle	$\phi pier$ =	49	
Stiffness Ratio	=	13.3	

LOWER ZONE

Effective Overburden removal =	0	psf
Bottom Bulb =	1.25	Pier Diameters
Height of Bottom Bulb =	3.13	ft
Max Allowable Settlement =	1.00	in
Allowable Settlement Variance =	0.044 in	
Minimum Bearing Capacity FS =	1.50	

REFERENCE BORINGS: 0.000

Minimum FS = 1.5

3/11 - 3/12/2014

Ftg Grid Coordinates or Ftg Mark	Column/Wall Load (DL+LL) -kips-	Footing Dimensions W -ft-	Footing Dimensions L -ft-	Bearing Pressure $q_{footing}$ -psf-	Depth to BOF D_f -ft-	Pier Shaft Length H_s -ft-	Reinforced Depth Hs+1.25d -ft-	RZ/W (Hs+1.25d)/W	Number of piers per Ftg.	Individual Pier-Cell Load Q_{pier} -kips-	Area Ratio R_a	Bearing Stress Top of pier q_{pier} -psf-	Bearing Stress Matrix soil q_{soil} -psf-	Load Top of pier Q_{pier} -kips-	Compression Capacity (Check Shaft Length by Comparing Q_{pier} vs Pier Shaft Friction + End Bearing) Shaft Fric. -kips-	End Bearing -kips-	TOTAL -kips-	CHECK -kips-	Bearing Capacity Shearing Below Tip Indiv. Piers -FS-	W/in Reinf. Matrix -FS- (Minimum FS = 1.5)	Below Reinf. Matrix -FS- (Minimum FS = 1.5)	Settlement Reinforced Zone Settlement S_{RZ} -in-	Lower Zone Settlement C_c[1] S_{LZ} -in-	Lower Zone Settlement E_s[2] S_{LZ} -in-	Max. Allowable = 1 in, Total RZ + LZ Settlement RZ+C_c S_{TOT} -in-	RZ+E_s S_{TOT} -in-
F2.12	200	2.67	12.50	5993	4.5	11.0	14.1	5.3	2	100.0	0.42	12,837	963	90.7	135.3	31.5	166.8	166.8	2.2	4.0	32.0	0.45	n/a	0.00	n/a	0.45
F6.0	200	6.00	6.00	5556	3.0	9.0	12.1	2.0	2	100.0	0.39	12,677	951	89.6	92.8	28.2	121.0	116.0	1.8	3.7	20.9	0.44	n/a	0.00	n/a	0.44
F7.5	325	7.50	7.50	5778	4.0	9.0	12.1	1.6	3	108.3	0.38	13,636	1,023	96.4	102.3	29.1	131.4	127.8	1.7	4.6	17.2	0.47	n/a	0.42	n/a	0.90
F10.0	600	10.00	10.00	6000	4.0	12.0	15.1	1.5	5	120.0	0.35	14,928	1,120	105.5	145.5	31.9	177.4	177.4	2.0	3.6	16.1	0.52	n/a	0.44	n/a	0.96
				0				0.0		0.0	0.00	0	0	0.0	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
	2,709	27.00	40.00	2508	5.0	13.0	16.1	0.6	34	79.7	0.22	8,932	670	63.1	168.0	34.0	202.0	202.0	3.7	15.0	17.1	0.31	n/a	0.50	n/a	0.81
	741	14.00	34.00	1557	5.0	10.0	13.1	0.9	16	46.3	0.24	5,281	396	37.3	124.7	31.0	155.7	155.7	5.0	17.9	27.1	0.18	n/a	0.63	n/a	0.82
	2,329	32.00	46.00	1582	5.0	11.0	14.1	0.4	47	49.6	0.23	5,576	418	39.4	139.2	31.9	171.1	171.1	5.1	26.0	23.2	0.19	n/a	0.79	n/a	0.98
	683	24.00	36.00	791	5.0	10.0	13.1	0.5	31	22.0	0.25	2,553	192	18.0	124.7	31.0	155.7	155.7	10.2	44.7	47.4	0.09	n/a	0.38	n/a	0.47
	717	27.00	42.00	632	6.0	10.0	13.1	0.5	34	21.1	0.21	2,333	175	16.5	131.4	31.9	163.3	163.3	11.4	64.9	59.6	0.08	n/a	0.29	n/a	0.37

** CHECK reduces TOTAL to allow only 20% contribition from End Bearing. Applicable only for "floating" piers where drilling does not terminate in Stiff/Dense bearing stratum.

** Shearing Below the Tips of Individual Piers applies only to single-pier footings. Shearing Within and Below the Reinforced Matrix controls for multiple-pier footings.

1. Based upon Consolidation Theory
2. Based upon Elastic Theory

All piers to achieve Design Shaft Length noted above unless bearing stratum is encountered at shallower depth.

Engineered Aggregate Pier Tension Capacity Calculations

Individual Piers

03-Sep-14

Project: Block 8L Mixed Use — Portland, Oregon

Note: All depths are below finish floor grade

Factor of Safety to apply to the pull-out resistance of the supporting soil = 3.0

Factor of Safety to apply to soil weight within the pull-out block = 1.2

Pier Data:

Depth to footing bottom (D_f)	=	5	feet
Shaft length (L)	=	10	feet
Shaft nominal diameter (D)	=	30	inches
Shaft effective diameter (D_{eff})	=	33	inches
Unsubmerged unit wt.	=	135	lbs/ft^3

shaft length above gwl = 5 *feet*
shaft length below gwl = 5 *feet*
pier crossection area = 4.909 *sq. ft.*

Soil Data:

Design depth to groundwater = 10 feet

divide pier shaft length into 8 layers of equal thickness

	Depth Below Ground Surface (or finish floor) (ft)	Soil Type (see NOTE) S or C	Unsubmerged Unit Weight -pcf- γ	Soils With Internal Frictional Component — Predominantly SILTY and SANDY soils (S): Friction Angle, Drained Φ_d	Friction Angle, Undrained Φ_u	Cohesion -psf- c	CLAY (C) Undrained Str. -psf- S_u	Theoretical Angle of Pier-Group Failure Plane (β) -degrees- 45-Φ/2	Equivalent Undrained Str. of "s" layers based on σ_h (Tan Φ)
	0.00								
		s	110	27					373
top of pier	5.00								
		s	110	27				31.5	840
	6.25								
		s	110	27				31.5	1026
	7.50								
		s	110	27				31.5	1213
	8.75								
		s	110	27				31.5	1399
	10.00								
		s	110	27				31.5	1529
	11.25								
		s	110	27				31.5	1529
	12.50								
		s	110	27				31.5	1529
	13.75								
		s	110	27				31.5	1529
	15.00	*Average =*	*110.0*				*Average =*	*31.5*	

NOTE: if **S** : typically SP, SM, SC, ML — Φ_d and C are required
if **C** : typically CL, CH, MH, OH — Φ_u and C, or S_u are required
(if a value is input for S_u, it supercedes both Φ and C)

Layer thickness (8 layers) l_t = *1.25* feet
Passive pressure coefficient (K_p) = $Tan^2(45+\Phi/2)$
At-Rest pressure coeficient (K_o) = 1-Sin Φ

	Depth Below Ground Surface (or finish floor) (ft)	Average Depth (ft)	Vertical Effective Stress (σ_v) (psf)	Soil Type behavior within the layer	Passive Pressure Coefficient K_p	At-Rest Pressure Coefficient K_o	Horizontal Stress (σ_h) (see note) (psf)	Side Frictional Resistance $r = \Pi D_{eff} (c+\sigma_h T$ (lbs/ft)	Side Frictional Resistance $R = r\, l_t$ (lbs)	Pier Weight (lbs)	Weight of replaced soil (lbs)
	0.00										
		2.50	275.0	granular	2.663	0.546	732.3	-	-	-	
top of pier	5.00										
		5.63	618.8	granular	2.663	0.546	1647.7	7,253	9,066	501	*408*
	6.25										
		6.88	756.3	granular	2.663	0.546	2013.8	8,865	11,081	1,002	*817*
	7.50										
		8.13	893.8	granular	2.663	0.546	2380.0	10,477	13,096	1,002	*817*
	8.75										
		9.38	1031.3	granular	2.663	0.546	2746.2	12,089	15,111	1,002	*817*
	10.00										
		10.63	1129.8	granular	2.663	0.546	3000.0	13,206	16,507	771	*353*
	11.25										
		11.88	1189.3	granular	2.663	0.546	3000.0	13,206	16,507	539	*353*
	12.50										
		13.13	1248.8	granular	2.663	0.546	3000.0	13,206	16,507	539	*353*
	13.75										
		14.38	1308.3	granular	2.663	0.546	3000.0	13,206	16,507	539	*353*
	15.00										
									114,384	5,896	*4,272*

Note:
The results of Ko Stepped Blade tests suggest that the amount of lateral stress buildup as a result of rammed pier installation is limited to about 2500-3000 psf.

design side friction (FS=3) = 38 kips
pier dead weight (FS=1.2) = 5 kips
Design Pull-Out Capacity = 44 kips

Average side friction (FS=3) = *441 psf*
Total resistance from layers having a designated Undrained Str. (FS=3) = - *kips*