Carlson Geotechnical
A Division of Carlson Testing, Inc.
Phone: (503) 601-8250
Fax: (503) 601-8254

Bend Office (541) 330-9155
Eugene Office (541) 345-0289
Salem Office (503) 589-1252
Tigard Office (503) 684-3460

12-216597-CO 2

Report of Geotechnical Investigation Downstream Warehouse Remodel 735 NW 16th Avenue Portland, Oregon

CGT Project Number G1203772

Prepared for

Mr. Tim Bright
Downstream
1650 NW Naito Parkway Suite 301
Portland, Oregon 97209

November 13, 2012

Carlson Geotechnical
A Division of Carlson Testing, Inc.
Phone: (503) 601-8250
Fax: (503) 601-8254

Bend Office (541) 330-9155
Eugene Office (541) 345-0289
Salem Office (503) 589-1252
Tigard Office (503) 684-3460

November 13, 2012

Mr. Tim Bright
Downstream
1650 NW Naito Parkway Suite 301
Portland, Oregon 97209

Report of
Geotechnical Investigation
Downstream Warehouse Remodel
735 NW 16th Avenue
Portland, Oregon

CGT Project Number G1203772

Dear Mr. Bright:

Carlson Geotechnical (CGT), a division of Carlson Testing, Inc. (CTI), is pleased to submit this report summarizing our geotechnical investigation for the proposed Downstream Warehouse Remodel. The project site is located at 735 NW 16th Avenue in Portland, Oregon. We performed our work in general accordance with CGT Proposal GP5834, dated October 19, 2012. Written authorization was provided on October 23, 2012. We appreciate the opportunity to work with you on this project. Please contact us at 503.601.8250 if you have any questions regarding this report.

Respectfully Submitted,
CARLSON GEOTECHNICAL

PROFESSIONAL ENGINEER
70773PE
OREGON
10, 2007
MICHAEL WILCOX
6/30/2014

Kyle Smetana, E.I.T.
Geotechnical Project Manager
ksmetana@carlsontesting.com

Brad M. Wilcox, P.E.
Senior Geotechnical Engineer
bwilcox@carlsontesting.com

Doc ID: G:\GEOTECH\PROJECTS\2012 Projects\Downstream Warehouse - NW Portland\Report G1203772.doc

TABLE OF CONTENTS

ATTACHMENTS

1.0 INTRODUCTION

Carlson Geotechnical (CGT), a division of Carlson Testing, Inc. (CTI), is pleased to submit this report summarizing our geotechnical investigation for the proposed Downstream Warehouse Remodel project. The existing warehouse is located at 735 NW 16th Avenue in Portland, Oregon, as shown on the attached Site Plan, Figure 1.

1.1 Project Information

CGT developed an understanding of the proposed project based on our correspondence with the project structural engineer, Mr. Randall Toma, P.E., S.E., of ABHT Structural Engineers (ABHT). The project will include remodeling and converting the existing warehouse building to an office building. The existing 1930's era building is one-story in height, approximately 15,000 square feet in plan area, concrete- and wood-framed, and incorporates a slab-on-grade floor. As part of the remodel, plans include installation of new steel braced frames and strengthening existing wall foundations by installing deep foundation elements (e.g. micro-piles and/or helical piles). The project will also include remodeling the storefront entry and construction of a new canopy structure. No detailed structural information has been provided; however, we have assumed that building loads will be typical of these types of structures, with continuous perimeter footing loads of less than 3 kips per lineal foot (klf), interior column loads of less than 50 kips, and uniform floor slab loads of less than 150 pounds per square foot (psf). Geotechnical recommendations for design and construction of the proposed new structural features were requested by ABHT.

1.2 Scope of Work

The purpose of our geotechnical investigation was to explore subsurface conditions at the site in order to provide geotechnical engineering recommendations for the proposed project. Our specific scope of services included the following:

- CGT contacted the Oregon Utilities Notification Center to mark the locations of public utilities at the site within a 15-foot radius of our painted explorations. In addition, CGT subcontracted a private utility locating service to mark the locations of private utilities within a 15-foot radius of the borings.
- Explore subsurface conditions at the site by advancing two, machine-drilled, soil borings, one hand auger boring, and one dynamic cone penetrometer test within the warehouse portion of the existing building. Additional details of the subsurface explorations are presented in Section 3.0 of this report.
- Classify the materials encountered in the borings in general accordance with American Society for Testing and Materials (ASTM) D2488 (Visual-Manual Procedure). Qualified members of CGT's staff observed and maintained a detailed log of each exploration.

- Collect representative, disturbed samples of the soils encountered within the borings in order to perform laboratory testing and to confirm our field classifications. The scope of laboratory testing performed on selected soil samples is described later in this report.
- Provide a site vicinity map and a site plan showing the locations of the borings relative to existing site features.
- Provide logs of the explorations, including groundwater levels observed. Results of laboratory testing are presented on the respective boring logs.
- Provide geotechnical recommendations for site preparation and earthwork, including:
    - Demolition of existing structures.
    - Subgrade preparation.
    - Utility trench excavation and backfill.
    - General grading considerations.
    - Fill type for imported materials.
    - Use of on-site soils as structural fill.
    - Fill compaction criteria.
- Provide geotechnical engineering recommendations for design and construction of new shallow spread foundations, including:
    - An allowable design bearing pressure.
    - Minimum footing and depth requirements.
    - Lateral capacity criteria.
    - An estimate of settlement based on assumed loads.
- Provide geotechnical engineering recommendations for design and construction of micro-piles, including thickness, type (cohesionless or cohesive), and bond strength for each soil layer encountered in the explorations. In addition, guidelines for installation and load testing of micro-piles have been presented.
- Provide geotechnical engineering recommendations for design and construction of helical piles, including thickness, type (cohesionless or cohesive), and strength parameters for each soil layer encountered in the explorations. In addition, guidelines for installation and load testing of helical piles have been presented.
- Provide geotechnical engineering recommendations for design and construction of concrete floor slabs supported on-grade, including:
    - An anticipated value for modulus of subgrade reaction.
    - A capillary break and vapor barrier.
    - An estimate of settlement based on assumed loads.
- Provide recommendations for the Seismic Site Class, mapped spectral response accelerations, site seismic coefficients, and maximum considered earthquake (MCE) response accelerations in accordance with Section 1613.5 of the 2010 Oregon Structural Specialty Code (OSSC).
- Provide a <u>qualitative</u> evaluation of seismic hazards at the site, including liquefaction potential, earthquake-induced settlement and landsliding, and surface rupture due to faulting or lateral spread.
- Provide this written report summarizing our geotechnical investigation and recommendations for the project.

2.0 SITE DESCRIPTION

2.1 Site Geology

The available mapping (Beeson, *et al.*[1]) indicates that the site is underlain by Pleistocene-Age, fine-grained, catastrophic flood deposits (Qff), consisting primarily of silt and sand. The deposits are mapped as being several tens of feet in thickness and are underlain by Columbia River Basalt.

2.2 Site Surface Conditions

The existing building effectively encompassed the entire lot and was bounded by NW Johnson Street to the north, NW 16th Avenue to the east, and established commercial development to the south and west. The building site was generally level and incorporated a concrete slab-on-grade floor. At the time of our field investigation, the building was being used as office, retail, and warehouse space. A layout of the existing building (prepared by others) is shown on the attached Site Plan, Figure 2. The eastern two-thirds (approximate) of the building was occupied by an existing tenant and was not accessible for exploration equipment.

3.0 FIELD EXPLORATION

3.1 Drilled Borings

3.1.1 <u>Overview</u>

CGT observed the advancement of two drilled borings (B-1 and B-2) at the site on October 25, 2012, to depths up to about 40¼ feet bgs. The approximate boring locations are shown on the attached Site Plan, Figure 2. The boring locations shown therein were determined based on measurements from existing site features and should be considered approximate. The borings were conducted using a WS-45, track-mounted, drill rig provided and operated by our subcontractor, Western States Soil Conservation of Hubbard, Oregon. Each boring was advanced using the mud rotary technique to the full depth explored. Upon completion, the borings were backfilled with granular bentonite and surfacing was patched with quickcrete.

3.1.2 <u>Standard Penetration Tests</u>

The drill rig was equipped with a 140-pound, automatic hammer, which was used to conduct Standard Penetration Tests (SPTs). The SPT is performed by driving a 2-inch, outside-diameter, split-spoon sampler into the undisturbed formation located at the bottom of the advanced boring with repeated blows of a 140-pound, automatic hammer falling a vertical distance of 30 inches. The number of blows (N-Value) required to drive the sampler the last 12 inches of an 18-inch sample interval is used to characterize the soil consistency or relative density. SPTs were

---

[1] Beeson, M.H., et al., 1991: Geologic Map of the Portland Quadrangle, Multnomah and Washington Counties, Oregon, and Clark County, Washington. Oregon Department of Geology and Mineral Industries Geological Map Series GMS-75.

generally conducted at a 2½-foot interval to depths of about 10 feet bgs and at 5-foot intervals below that depth to the termination depths of the borings. It should be noted that automatic hammers generally produce lower SPT values than those obtained using a traditional safety hammer (cathead). Studies have generally indicated that penetration resistances may vary by a factor of 0.8 to 1.3 between the two methods[2]. According to the driller, the automatic hammer on the WS-45 drill rig had efficiency (ETR_{hammer}) of 67.3 percent, resulting in an efficiency factor of about 1.12. We have considered these efficiencies in our description of soil relative density and in our evaluation of soil strength and compressibility. The SPT values listed on the attached boring logs are "raw" values and have not been adjusted.

3.2 Hand Explorations

CGT advanced one hand auger boring (HA-1) at the site on October 25, 2012, to a depth of about 3 feet bgs using a manual, 3-inch diameter, hand auger. The approximate location of the boring is shown on the attached Site Plan, Figure 2. The exploration was located in the field using approximate measurements from existing site features shown on the Site Plan. Upon completion, the hand auger boring was loosely backfilled with the excavated materials and the surface was patched with quickcrete.

In conjunction with the hand auger boring, we performed one dynamic cone penetrometer test (WDCP-1) to a depth of about 6½ feet bgs using a Wildcat Dynamic Cone Penetrometer (WDCP). The WDCP test consists of driving 1.1-inch diameter, steel rods with a 1.4-inch diameter, cone tip into the ground using a 35-pound drop hammer with a 15-inch, free-fall height. The number of blows required to drive the steel rods is recorded for each 10 centimeters (3.94 inches) of penetration. The blow count for each interval is then converted to the corresponding Standard Penetration Test (SPT) "N_{60}" values, which are used to estimate the soil relative consistency for cohesive soils, or relative density for non-cohesive soils.

3.3 Material Sampling & Logging

Soil samples were obtained at selected depths during advancement of the drilled borings using the referenced SPT split-spoon sampler. CGT representatives collected the samples and logged the soils in general accordance with the Unified Soil Classification System (USCS). An explanation of the USCS is provided on the attached Soil Classification Criteria and Terminology, Figure 3. All SPT soil samples collected at the site were stored in sealable plastic bags upon completion of our field examination and were transported to our laboratory for further examination and testing. Our geotechnical staff visually examined all samples returned to our laboratory in order to refine the initial field classifications. The logs of the explorations are presented on the attached Boring Logs, Figures 4 through 6. The surface elevations shown on the logs were based on an assumed elevation of 100 feet at the top of slab elevation within the warehouse portion of the building.

---

2 Youd, et al. 2002. Liquefaction Resistance of Soils: Summary Report from the NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils. Journal of Geotechnical and Environmental Engineering.

4.0 LABORATORY TESTING

Laboratory testing was performed on soil samples collected in the field to refine our initial field classifications and determine in-situ parameters. Laboratory testing included ten moisture content determinations (ASTM D2216), four fines content tests to determine the percent passing the U.S. Standard No. 200 Sieve (ASTM C117), and one Atterberg limit (plasticity) test (ASTM D4318). Results of the laboratory tests are shown on the attached Boring Logs, Figures 4 through 6.

5.0 SUBSURFACE CONDITIONS

5.1 Soils

The following subsurface materials were encountered within the borings:

Concrete Slab:	Concrete was encountered at the surface of each exploration and was approximately 5 inches thick.
Elastic Silt (MH):	Elastic silt was encountered beneath the concrete slab. The elastic silt extended to depths of about 8 to 5 feet bgs within B-1 and B-2, respectively, and to the full depth explored within HA-1, about 3 feet bgs. The elastic silt was generally medium stiff to stiff, brown, moist, and exhibited medium plasticity.
Sandy Silt (ML):	Sandy silt was encountered beneath the elastic silt within B-1 and extended to a depth of about 15½ feet bgs. The sandy silt was generally medium stiff to stiff, brown, very moist, and exhibited low plasticity.
Sand (SP) to Silty Sand (SM):	Sand to silty sand was encountered beneath the sandy silt in B-1 and beneath the elastic silt in boring B-2. These sandy soils have been lumped together for the purposes of discussion recognizing their similar index properties. These soils extended to depths of about 29½ feet bgs in those borings. These soils were generally medium dense, brown, moist to very moist, and fine- to coarse-grained.
Sandy Gravel (GP):	Sandy gravel was encountered beneath the sandy soils (SP, SM) described above. The sandy gravel extended to the full depths explored within B-1 and B-2, about 36½ to 40¼ feet bgs. The sandy gravel was generally very dense, wet, gray, sub-round, fine- to coarse-grained, and contained silt.

5.2 Groundwater

Localized zones of wet soils, interpreted as perched groundwater, were encountered at depths of about 7 feet bgs within borings B-1 and B-2. Groundwater was not encountered within the depth explored (3 feet bgs) in hand auger boring HA-1. Based on visual examination of the samples at the time of drilling, we interpret static groundwater to be present at depths of about 29 feet bgs (near the top of the sandy gravel layer described above).

To determine approximate regional groundwater levels in the area, we researched well logs available at the Oregon Water Resources Department (OWRD) website[3] for wells located within Section 33, Township 1 North, Range 1 East. Our review indicated that groundwater levels in the area varied with surface elevations and generally were greater than about 25 feet bgs. It should be noted that groundwater levels vary with local topography. In addition, the groundwater levels reported on the OWRD logs often reflect the purpose of the well, so water well logs may only report deeper, confined groundwater, while geotechnical or environmental borings will often report any groundwater encountered, including shallow, unconfined groundwater. Therefore, the levels reported on the OWRD well logs referenced above are considered generally indicative of local water levels and may not reflect actual groundwater levels at the site. We anticipate that groundwater levels will fluctuate due to seasonal and annual variations in precipitation, changes in site utilization, or other factors. As indicated above, the on-site, elastic silt (MH), sandy silt (ML), and silty sand (SM) soils are conducive to formation of perched groundwater.

6.0 SEISMIC CONSIDERATIONS

6.1 Seismic Design

Based on the results of the explorations and review of geologic mapping, we have assigned the site as Site Class D for the subsurface conditions encountered in accordance with Section 1613.5.2 of the 2010 Oregon Structural Specialty Code (OSSC). Earthquake ground motion parameters for the site were obtained based on the United States Geological Survey (USGS) Seismic Design Values for Buildings - Ground Motion Parameter Calculator[4]. The site Latitude 45.52836° North and Longitude 122.68768° West were input as the site location. The following table shows the recommended seismic design parameters for the site.

Table 1: Seismic Ground Motion Values (Section 1613.5 of 2010 OSSC)

Parameter		Value
Mapped Acceleration Parameters	Spectral Acceleration, 0.2 second (S_s)	0.984g
	Spectral Acceleration, 1.0 second (S_1)	0.346g
Coefficients (Site Class D)	Site Coefficient, 0.2 sec. (F_A)	1.107
	Site Coefficient, 1.0 sec. (F_V)	1.708
Adjusted MCE Spectral Response Parameters	MCE Spectral Acceleration, 0.2 sec. (S_{MS})	1.088g
	MCE Spectral Acceleration, 1.0 sec. (S_{M1})	0.591g
Design Spectral Response Accelerations	Design Spectral Acceleration, 0.2 seconds (S_{DS})	0.726g
	Design Spectral Acceleration, 1.0 second (S_{D1})	0.394g

[3] Oregon Water Resources Department, 2011. Water well logs obtained from OWRD website http://www.wrd.state.or.us/

[4] United States Geological Survey, 2012. Seismic Design Parameters determined using:, "U.S. Seismic Design Maps Web Application - Version 3.0.0," from the USGS website *http://earthquake.usgs.gov*.

6.2 Seismic Hazards

6.2.1 Liquefaction

In general, liquefaction occurs when deposits of loose/soft, saturated, cohesionless soils, generally sands and silts, are subjected to strong earthquake shaking. If these deposits cannot drain quickly enough, pore water pressures can increase, approaching the value of the overburden pressure. The shear strength of a cohesionless soil is directly proportional to the effective stress, which is equal to the difference between the overburden pressure and the pore water pressure. When the pore water pressure increases to the value of the overburden pressure, the shear strength of the soil reduces to zero, and the soil deposit can liquefy. The liquefied soils can undergo rapid consolidation or, if unconfined, can flow as a liquid. Structures supported by the liquefied soils can experience rapid, excessive settlement, shearing, or even catastrophic failure.

The susceptibility of sands, gravels, and sand-gravel mixtures to liquefaction is typically assessed based on penetration resistance, as measured using SPTs, CPTs, or Becker Hammer Penetration tests (BPTs). For fine-grained soils, susceptibility to liquefaction is evaluated based on penetration resistance and plasticity, among other characteristics. Criteria for identifying non-liquefiable, fine-grained soils are constantly evolving. Current practice to identify non-liquefiable, fine-grained soils is based on plasticity characteristics of the soils, as follows: (1) liquid limit greater than 47 percent, (2) plasticity index greater than 20 percent, and (3) moisture content less than 85 percent of the liquid limit[5]. The susceptibility of sands, gravels, and sand-gravel mixtures to liquefaction is typically assessed based on relative density.

Based on its plasticity and lack of fully-saturated conditions, the native elastic silt (MH) is considered non-liquefiable. Based on the lack of fully-saturated conditions, the native sandy soils (ML, SM, SP) are considered non-liquefiable. Based on its very dense relative density, the native sandy gravel (GP) is considered non-liquefiable within the depths explored.

6.2.2 Slope Instability

Given the relatively flat to gently sloped topography on and immediately surrounding the site, the risk of seismically-induced slope instability is very low.

6.2.3 Surface Rupture

6.2.3.1 Faulting

Although the site is situated in a region of the country known for seismic activity, no mapped faults exist on or immediately adjacent to the site. Therefore, the risk of surface rupture at the site due to faulting is considered low.

---

[5] Seed, R.B. et al., 2003. Recent Advances in Soil Liquefaction Engineering: A Unified and Consistent Framework. Earthquake Engineering Research Center Report No. EERC 2003-06.

6.2.3.2 Lateral Spread

Surface rupture due to lateral spread can occur on sites underlain by liquefiable soils that are located on or immediately adjacent to slopes steeper than about 3 degrees (20H:1V), and/or adjacent to a free face, such as a stream bank or the shore of an open body of water. During lateral spread, the materials overlying the liquefied soils are subject to lateral movement downslope or toward the free face. Given the generally flat topography and lack of a free face at or near the site, the risk of surface rupture due to lateral spread is considered very low.

7.0 CONCLUSIONS

Based on the results of our field explorations and analyses, the site may be developed as described in Section 1.1 of this report, provided the recommendations presented in this report are incorporated into the design and development. Geotechnical recommendations for design and construction of new shallow spread foundations are presented in Section 8.5 of this report.

Site subsurface conditions are conducive for installation of the deep foundation elements (micro-piles or helical piles) discussed in Section 1.1, if incorporated into the final design. Geotechnical recommendations for design and construction of deep foundation elements are presented in Sections 8.6 and 8.7 of this report.

8.0 RECOMMENDATIONS

The following paragraphs present geotechnical recommendations for design and construction of the proposed project. The recommendations presented in this report are based on the information provided to us, results of the field investigation, laboratory data, and professional judgment. CGT has observed only a small portion of the pertinent subsurface conditions. The recommendations are based on the assumptions that the subsurface conditions do not deviate appreciably from those found during the field investigation. If the scope of the proposed project changes, or if variations or undesirable geotechnical conditions are encountered during site development, CGT should be consulted for further recommendations.

8.1 Site Preparation

8.1.1 Stripping

Concrete should be removed from proposed foundation areas, as required, and within areas where the existing concrete slab will be replaced at the site. Based on the results of our field explorations, the depth of concrete stripping will be about 5 inches. The concrete may be deeper or shallower at locations away from our borings. The geotechnical engineer or his representative should provide recommendations for actual stripping depths based on observations during site stripping. Stripped concrete should be transported off-site for disposal or stockpiled and processed for later use as structural fill as detailed in Section 8.4.1 of this report.

8.1.2 Existing Utilities

All existing utilities at the site should be identified prior to excavation. Abandoned utility lines beneath new foundations and floor slabs should be completely removed or grouted full. Soft, loose, or otherwise unsuitable soils encountered in utility trench excavations should be removed and replaced with structural fill as described in Section 8.4.2 of this report. No below-grade structures were encountered in our explorations. If encountered during site preparation, buried structures (i.e. footings, foundation walls, slabs-on-grade, tanks, etc.) should be completely removed and disposed of off-site. The resulting excavation should be backfilled with imported granular structural fill in conformance with Section 8.4.2 of this report.

8.1.3 Subgrade Preparation – Slab Areas

After site stripping as recommended above, and prior to placement of fill and/or crushed rock base, a geotechnical representative from CGT should probe the exposed subgrade soils in order to identify areas of excessive yielding. If areas of soft soil or excessive yielding are identified, the affected material should be over-excavated to firm, stable subgrade, and replaced with imported granular structural fill in conformance with Section 8.4.2 of this report.

8.1.4 Erosion Control

Erosion and sedimentation control measures should be employed in accordance with applicable County and State regulations regarding erosion control.

8.2 Temporary Excavations

8.2.1 Overview

All excavations should be in accordance with applicable OSHA and state regulations. It is the contractor's responsibility to select the excavation methods, to monitor site excavations for safety, and to provide any shoring required to protect personnel and adjacent improvements. A “competent person”, as defined by OR-OSHA, should be on-site during construction in accordance with regulations presented by OR-OSHA. CGT’s current role on the project does not include review or oversight of excavation safety.

8.2.2 Utility Trenches

Temporary trench cuts should stand near vertical to depths of approximately 4 feet in the native elastic silt (MH) encountered near the site surface. Some instability may occur if groundwater seepage is encountered. If seepage undermines the stability of the trench, or if caving of the sidewalls is observed during excavation, the sidewalls should be flattened or shored. Depending on the time of year trench excavations occur, trench dewatering may be required in order to maintain dry working conditions, particularly if the invert elevations of the proposed utilities are below the groundwater level. Pumping from sumps located within the trench will likely be effective in removing water resulting from seepage. If groundwater is present at the base of utility excavations, we recommend placing trench stabilization material at the base of the excavations. Trench stabilization material should be in conformance with Section 8.4.4 of this report.

8.2.3 <u>OSHA Soil Type</u>

Conventional earthmoving equipment in proper working condition should be capable of making necessary excavations for cuts at the site as described earlier in this report. For use in the planning and construction of temporary excavations up to 5 feet in depth, an OSHA soil type "B" may be used for the native elastic silt (MH).

8.2.4 <u>Excavations Near Foundations</u>

Excavations near footings should <u>not</u> extend within a 1H:1V (horizontal:vertical) plane projected out and down from the outside, bottom edge of the footings. In the event that excavation needs to extend below the referenced plane, temporary shoring of the excavation and/or underpinning of the subject footing may be required. The geotechnical engineer should be consulted to review proposed excavation plans for this design case to provide specific recommendations.

8.3 Wet Weather Considerations

The majority of the proposed project is located within the footprint of the existing building. Accordingly, we do not anticipate that significant portions of the project will be exposed to wet weather conditions. Where new canopy foundations are constructed outside the footprint of the building and exposed to wet weather, we recommend a minimum of 3 inches of imported granular material be placed to protect footing subgrades from foot traffic during inclement weather. The imported granular material should be in conformance with Section 8.4.2 of this report. The maximum particle size should be limited to 1-inch. The imported granular material should be placed in one lift over the prepared, undisturbed subgrade, and compacted using <u>non-vibratory</u> equipment until well keyed.

8.4 Structural Fill

The geotechnical engineer should be provided the opportunity to review all materials considered for use as structural fill (prior to placement). The geotechnical engineer or his representative should be contacted to evaluate compaction of structural fill as the material is being placed. Evaluation of compaction should take the form of in-place density tests conducted at intervals not exceeding every 2 vertical feet as the fill is being placed.

8.4.1 On-Site Soils – General Use

Concrete debris resulting from the demolition of structures can be re-used as structural fill if processed/crushed into material that is fairly well graded between coarse and fine. The processed/crushed concrete and/or asphalt should contain no organic matter, debris, or particles larger than 1½ inches in diameter. The processed/crushed concrete and/or asphalt should be moisture conditioned, placed in lifts with a maximum thickness of about 12 inches, and compacted to not less than 95 percent of the material's maximum dry density as determined in general accordance with ASTM D1557 (Modified Proctor), or approved visual equivalent as determined by the geotechnical engineer.

Recognizing the limited grading planned for this site and their moisture sensitivity, re-use of the near-surface, native soils (MH, ML, SM) as structural fill is not recommended.

8.4.2 Imported Granular Structural Fill – General Use

Imported granular structural fill should consist of angular pit or quarry run rock, crushed rock, or crushed gravel that is fairly well graded between coarse and fine particle sizes. The granular fill should contain no organic matter, debris, or particles larger than 1½ inches, and have less than 5 percent material passing the U.S. Standard No. 200 Sieve. The percentage of fines can be increased to 12 percent of the material passing the U.S. Standard No. 200 Sieve if placed during dry weather, and provided the fill material is moisture-conditioned, as necessary, for proper compaction. Granular fill material should be placed in lifts with a maximum thickness of about 12 inches, and compacted to not less than 95 percent of the material's maximum dry density, as determined in general accordance with ASTM D1557 (Modified Proctor). Proper moisture conditioning and the use of vibratory equipment will facilitate compaction of these materials.

8.4.3 Floor Slab Base Rock

Floor slab base rock should consist of well-graded granular material (crushed rock) containing no organic matter or debris, have a maximum particle size of ¾-inch, and have less than 5 percent material passing the U.S. Standard No. 200 Sieve. Floor slab base rock should be placed in one lift and compacted to not less than 95 percent of the material's maximum dry density as determined in general accordance with ASTM D1557 (Modified Proctor).

8.4.4 Trench Base Stabilization Material

If groundwater is present at the base of utility excavations, trench base stabilization material should be placed. Trench base stabilization material should consist of 1-foot of well-graded granular material with a maximum particle size of 4 inches and less than 5 percent material passing the U.S. Standard No. 4 Sieve. The material should be free of organic matter and other deleterious material, placed in one lift, and compacted until well-keyed.

8.4.5 Trench Backfill Material

Trench backfill for the utility pipe base and pipe zone should consist of granular material as recommended by the utility pipe manufacturer. Trench backfill above the pipe zone should consist of well-graded granular material containing no organic matter or debris, have a maximum particle size of ¾-inch, and have less than 8 percent material passing the U.S. Standard No. 200 Sieve. As a guideline, trench backfill should be placed in maximum 12-inch thick lifts. The earthwork contractor may elect to use alternative lift thicknesses based on their experience with specific equipment and fill material conditions during construction in order to achieve the required compaction. The following table presents recommended relative compaction percentages for utility trench backfill.

Table 2: Utility Trench Backfill Compaction Recommendations

Backfill Zone	Recommended Minimum Relative Compaction	
	Structural Areas[1]	Landscaping Areas
Pipe Base and Within Pipe Zone	90% ASTM D1557 or pipe manufacturer's recommendation	88% ASTM D1557 or pipe manufacturer's recommendation
Above Pipe Zone	92% ASTM D1557	90% ASTM D1557
Within 3 Feet of Design Subgrade	95% ASTM D1557	90% ASTM D1557
[1]Includes proposed floor slabs, hardscaping, etc.		

8.5 Shallow Foundations

8.5.1 Subgrade Preparation

Satisfactory subgrade support for new shallow foundations can be obtained from the native, medium stiff to stiff, elastic silt (MH), the native, medium stiff to stiff, sandy silt (ML), the native, medium dense, silty sand (SM), or new structural fill that is properly placed and compacted on these materials during construction. These materials were first encountered beneath the surface concrete with each of our explorations. The geotechnical engineer or his representative should be contacted to observe subgrade conditions prior to placement of forms, reinforcement steel, or granular backfill (if required). If soft, loose, or otherwise unsuitable soils are encountered, they should be over-excavated as recommended by the geotechnical representative at the time of construction. The resulting over-excavation should be brought back to grade with imported granular structural fill in conformance with Section 8.4.2 of this report. All granular pads for footings should be constructed a minimum of 6 inches wider on each side of the footing for every vertical foot of over-excavation.

8.5.2 Minimum Footing Width & Embedment

Individual spread footings should have a minimum width of 24 inches. Subject to review of the structural engineer, we recommend that continuous wall footings have a minimum width of 18 inches. To help mitigate potential frost action, all perimeter footings should be founded a minimum of 18 inches below the lowest adjacent grade. Interior footings should be founded a minimum of 12 inches below the interior surfacing element (e.g. concrete slab).

8.5.3 Bearing Pressure & Settlement

Footings founded as recommended above should be proportioned for a maximum allowable soil bearing pressure of 2,000 pounds per square foot (psf). This bearing pressure is a net bearing pressure, applies to the total of dead and long-term live loads, and may be increased by one-third when considering seismic or wind loads. For foundations founded as recommended above, total settlement of foundations is anticipated to be less than 1 inch. Differential settlements between adjacent columns and/or bearing walls should not exceed ½-inch.

8.5.4 Lateral Capacity

A maximum passive (equivalent fluid) earth pressure of 150 pounds per cubic foot (pcf) is recommended for design of footings confined by the native, medium stiff, elastic silt (MH), or imported granular structural fill that is properly placed and compacted during construction. The recommended earth pressure was computed using a factor of safety of 1½, which is appropriate due to the amount of movement required to develop full passive resistance. In order to develop the above capacity, the following should be understood:

1. Concrete must be poured neat in excavations or the foundations must be backfilled with imported granular structural fill,
2. The adjacent grade must be level,
3. The static ground water level must remain below the base of the footings throughout the year.
4. Adjacent floor slabs, pavements, or the upper 12-inch depth of adjacent, unpaved areas should <u>not</u> be considered when calculating passive resistance.

An ultimate coefficient of friction equal to 0.35 may be used when calculating resistance to sliding for footings founded as recommended above. An ultimate coefficient of friction of 0.45 is recommended for footings founded on a minimum of 6 inches of imported granular structural fill that is properly placed and compacted during construction.

8.6 Micro-Piles

8.6.1 Overview

Micro-piles consist of small diameter, high-quality steel, hollow bars typically drilled and grouted into place using a hydraulic rotary (electric or diesel), track- or truck-mounted, drill rig or drill attachment to track-mounted equipment. Compression and/or tension loads applied at the top of the micro-pile are resisted through grout-to-ground bond over a specified length of the micro-pile. Micro-piling is conventionally designed based on skin friction, ignoring tip resistance. For design, the ultimate capacity of the micro-pile is the same in compression and tension (uplift). The means and methods for drilling and grouting of micro-piles are typically subject to the review of specialized contractors and their experience with the specific soil conditions. Accordingly, it is recommended that an experienced, specialty micro-pile contractor be engaged during the planning phase to participate in selecting method(s) of installation, as well as sizing, locations, and other criteria related to micro-piles.

The geotechnical recommendations presented in the following sections are based on design methodology presented in FHWA NHI-05-039[6], and assume that the micro-pile design (including structural capacity) will rest with the structural engineer. If alternative methodology is used, CGT should be consulted.

8.6.2 Geotechnical Bond Capacity

For purposes of design and planning, allowable/design axial loads for micro-piles considering development of grout-to-ground bonding may be based on the following equation[7]:

$$P_{G\text{-allowable}} = [\alpha_{bond} / FS] * [3.14 * D_b * L_b]$$

where:

$P_{G\text{-allowable}}$ = To be determined	(Allowable axial load in tension or compression)
α_{bond} = See table below	(Grout-to-ground bond stress between the subsurface material and the grout.)
FS = 2.0 (minimum)	(factor of safety used in calculating bond length; subject to review of the micro-pile designer)
D_b = To be determined	(diameter of grout column)
L_b = To be determined	(bond length)

[6] Micropile Design and Construction, Publication No. FHWA NHI-05-039, December 2005.
[7] Equation 5-9 of FHWA NHI-05-039.

Table 3: Recommended Soil Descriptions for Micro-Pile Design

Layer	Depth (feet bgs)	Description[1]	Recommended FHWA Micro-Pile Soil Description[2]	Recommended Value for α_{bond}[3]
1	0 to 8	Elastic Silt (MH)	Silt & Clay (some sand), stiff	18 psi
2	8 to 29½	Med. Dense Sandy Soils (ML, SM, SP)	Sand (fine- to coarse grained, loose to medium dense)	18 psi
3	29½ to 40¼	Sandy Gravel (GP)	Gravel (some sand), very dense	35 psi

[1] Refer to attached boring logs. If preliminary design includes extending micro-piles below the depths indicated above, the geotechnical engineer should be consulted.

[2] Per Table 5-3 of FHWA NHI-05-039.

[3] Values presented above for α_{bond} reflect the "middle of range" reported for the soil descriptions listed above and micro-pile "Type B" as shown in Table 5-3 of the referenced FHWA manual. The use of micro-pile "Type B" is presented in conformance with Section 5.9.2 of the FHWA manual and is subject to the micro-pile contractor's preferred grouting method. Accordingly, the values presented for α_{bond} are recommended for preliminary design and planning.

8.6.3 Lateral Capacity

Design procedures for analyzing a single vertical micro-pile for lateral loading and/or overturning moments are presented in Section 5.18 of the referenced FHWA manual. It should be noted that individual vertical micro-piles do not offer much lateral load-carrying capacity. Battered micro-pile groups may be considered to provide additional resistance to lateral loading as detailed in Section 5.19.4 of the referenced FHWA micro-pile manual.

Micro-pile supported (grade beam) foundations can also resist lateral forces by passive earth pressure developing along the side of the foundation. Development of passive resistance assumes that some lateral movement of the foundation is allowed into the surrounding soil, thereby developing a passive soil wedge. The recommendations presented in Section 8.5.4 (passive resistance) are applicable for concrete grade beam footings supported by micro-piles.

8.6.4 Load Testing

As a general guideline, we recommend that proof load test(s) be performed on at least one non-production located pile in order to verify the design allowable axial capacity. The load test program, including actual number of load tests, load increments used during testing, and duration, should be defined by the foundation designer. Guidelines for load testing of micro-piles are provided in Sections 5.9.2 and 7.6 of the referenced FHWA manual. The geotechnical engineer or his representative should witness the installation of test and production piles, and all phases of load testing performed at the site.

8.7 Helical Piles

8.7.1 Overview

Helical piles consist of round or square, steel shafts equipped with welded helical plate(s) and are advanced into the subsurface using a hydraulic torque motor. Helical piles are proprietary foundation systems. The specialty pile contractor, in coordination with the project structural engineer, typically develops design plans for pile supported foundations. Accordingly, it is recommended that experienced, specialty pile contractor(s) be engaged during the planning phase to participate in selecting helix size(s), locations, and other criteria related to helical piles. CGT would be pleased to provide geotechnical recommendations for design, installation, and other criteria related to helical piles, upon request, for an additional fee.

8.7.2 Soil Strength Parameters

CGT has provided recommended values for soil parameters for use in helical pile design in the following table. The parameters provided below were based on the results of the borings, laboratory testing, published correlations with SPT data, and our experience with similar soils.

Table 4: Recommended Soil Strength Parameters for Use in Helical Pile Design

Layer	Depth (feet below ground surface)	Description	Recommended Soil Type	Soil Shear Strength Parameter[1]			
				Φ' (degrees)	c' (psf)	γ' (pcf)	S_u (psf)
1	0 to 8	Elastic Silt (MH)	Cohesive	30	100	110	900
2	8 to 29½	Sandy Soils (ML, SM, SP)	Cohesionless	32	0	115	0
3	29½ to 40	Sandy Gravel (GP)	Cohesionless	38	0	65	0

[1] If additional parameters are required to facilitate design, the geotechnical engineer should be consulted.
[2] For design, groundwater should be modeled at a depth of 29 feet below existing site grades.

Where layers are labeled as cohesive soil, the effective cohesion (c') and friction angle (Φ') are recommended for use in evaluation of helical pile capacity for long-term (fully drained) loading. Similarly, the undrained shear strength, S_u, for cohesive soils is recommended for use in evaluation of helical pile capacity for short-term (undrained) loading (i.e. end of construction). The pile designer is encouraged to analyze both loading conditions when calculating capacities; the analysis resulting in the smaller value of capacity is recommended for use in design.

8.7.3 Installation Depth Consideration

Generally speaking, the helical piles should be installed to a sufficient depth to achieve the required torque specified by the designer. The required installation depth may vary in the field, depending on subsurface conditions at individual helical pile locations. The geotechnical engineer should be contacted to observe helical pile installation.

8.7.4 Lateral Capacity

Helical pile supported (grade beam) foundations can resist lateral forces by passive earth pressure developing along the side of the foundation. Development of passive resistance assumes that some lateral movement of the foundation is allowed into the surrounding soil, thereby developing a passive soil wedge. The recommendations presented in Section 8.5.4 are applicable for concrete grade beam footings supported by helical piles.

8.7.5 Load Testing

As a guideline, we recommend that proof load test(s) be performed on at least one non-production located helical pile in order to verify the design allowable axial capacity. The load test program, including actual number of load tests, load increments used during testing, and duration, should be defined by the foundation designer. The geotechnical engineer or his representative should witness the installation of test and production piles, and all phases of load testing performed at the site. Depending on the decision of the owner and contractor, load testing may be performed on test or production piles. If test pile(s) are used, they should be installed using the same equipment and procedures that will be used to install production piles.

8.8 Floor Slabs

8.8.1 Subgrade Preparation

Subgrade preparation of floor slabs supported on-grade, supporting up to 150 psf area loading, should be in conformance with Section 8.1.3 of this report.

8.8.2 Crushed Rock Base

Concrete floor slabs should be supported on a minimum 6-inch thick layer of crushed rock (base rock). The crushed rock base should conform to the recommendations presented in Section 8.3.3 of this report. The surface of the base rock should be choked with sand just prior to concrete placement. Choking means the voids between the largest aggregate particles are filled with sand, but does not provide a layer of sand above the base rock. Choking the base rock surface reduces the lateral restraint on the bottom of the concrete during curing.

8.8.3 Design Considerations

For floor slabs constructed as recommended, a modulus of subgrade reaction of 100 pounds per cubic inch (pci) is recommended for the design of the floor slab. New floor slabs constructed as recommended will likely settle less than ½-inch. For general floor slab construction, slabs should be jointed around columns and walls to permit slabs and foundations to settle differentially.

8.8.4 Subgrade Moisture Considerations

Liquid moisture and moisture vapor should be expected at the subgrade surface. The recommended crushed rock base is anticipated to provide protection against liquid moisture. Where moisture vapor emission through the slab must be minimized, e.g. impervious floor coverings, storage of moisture sensitive materials directly on the slab surface, etc., a vapor retarding membrane or vapor barrier below the slab should be considered. Factors such as cost, special considerations for construction, floor coverings, and end use suggest that the decision regarding a vapor retarding membrane or vapor barrier be made by the architect and owner.

If a vapor retarder or vapor barrier is placed below the slab, its location should be based on current American Concrete Institute (ACI) guidelines, ACI 302 Guide for Concrete Floor and Slab Construction. In some cases, this indicates placement of concrete directly on the vapor retarder or barrier. Please note that the placement of concrete directly on impervious membranes increases the risk of plastic shrinkage cracking and slab curling in the concrete. Construction practices to reduce or eliminate such risk, as described in ACI 302, should be employed during concrete placement.

8.9 Additional Considerations

8.9.1 Drainage

Subsurface drains, if any are incorporated in the project, should be connected to the nearest storm drain or other suitable discharge point. Paved surfaces and ground near or adjacent to the building should be sloped to drain away from the building. Surface water from paved surfaces and open spaces should be collected and routed to a suitable discharge point. Surface water should not be directed into foundation drains, if incorporated or existing.

8.9.2 Expansive Potential

The near surface native soil consists of medium plastic elastic silt (MH). Based on experience with similar soils in the area of the site, this soil is not considered to be susceptible to appreciable movements from changes in moisture content. Accordingly, no special considerations are required to mitigate expansive potential of the near surface soils at this site.

9.0 RECOMMENDED ADDITIONAL SERVICES

9.1 Design Review

Geotechnical design review is of paramount importance. CGT recommends that the geotechnical design review take place prior to releasing bid packets to contractors.

9.2 Observation of Construction

Satisfactory earthwork and foundation performance depends to a large degree on the quality of construction. Sufficient observation of the contractor's activities is a key part of determining that the work is completed in accordance with the construction drawings and specifications. Subsurface conditions observed during construction should be compared with those encountered during subsurface explorations, and recognition of changed conditions often requires experience. We recommend that qualified personnel visit the site with sufficient frequency to detect whether subsurface conditions change significantly from those observed to date and anticipated in this report. We recommend that the geotechnical engineer or their representative attend a pre-construction meeting coordinated by the contractor and/or developer. The project geotechnical engineer or their representative should provide observations and/or testing of at least the following earthwork elements during construction:

- Site Stripping and Demolition.
- Subgrade Preparation for Structural Fills, Shallow Foundations, and Floor Slabs.
- Installation and Load Testing of Deep Foundation Elements.
- Compaction of Structural Fill and Floor Slab Base Rock.

It is imperative that the owner and/or contractor request earthwork observations and testing at a frequency sufficient to allow the geotechnical engineer to provide a final letter of compliance for the earthwork activities.

10.0 LIMITATIONS & CLOSURE

We have prepared this report for use by the owner/developer and other members of the design and construction team for the proposed development. The opinions and recommendations contained within this report are not intended to be, nor should they be construed as, a warranty of subsurface conditions, but are forwarded to assist in the planning and design process.

We have made observations based on our explorations that indicate the soil conditions at only those specific locations and only to the depths penetrated. These observations do not necessarily reflect soil types, strata thickness, or water level variations that may exist between or away from our explorations. If subsurface conditions vary from those encountered in our site explorations, CGT should be alerted to the change in conditions so that we may provide additional geotechnical recommendations, if necessary. Observation by experienced geotechnical personnel should be considered an integral part of the construction process.

The owner/developer is responsible for ensuring that the project designers and contractors implement our recommendations. When the design has been finalized, prior to releasing bid packets to contractors, we recommend that the design drawings and specifications be reviewed by our firm to see that our recommendations have been interpreted and implemented as intended. If design changes are made, we request that we be retained to review our conclusions and recommendations and to provide a written modification or verification. Design review and construction phase testing and observation services are beyond the scope of our current assignment, but will be provided for an additional fee.

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.

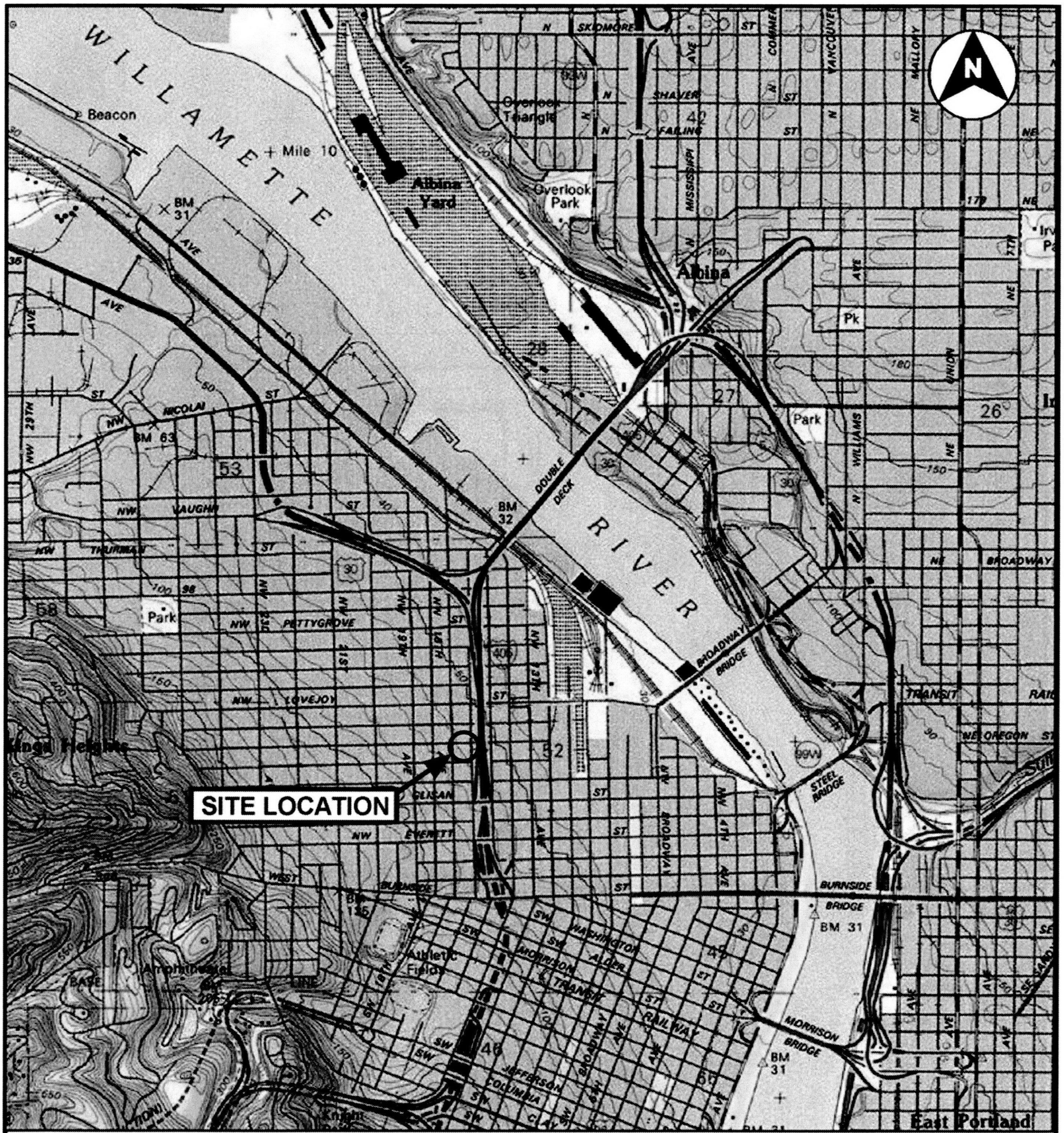
Geotechnical engineering and the geologic sciences are characterized by a degree of uncertainty. Professional judgments presented in this report are based on our understanding of the proposed construction, familiarity with similar projects in the area, and on general experience.

Within the limitations of scope, schedule, and budget, our services have been executed in accordance with the generally accepted practices in this area at the time this report was prepared. No warranty or other conditions, expressed or implied, should be understood. Information contained herein is not to be reproduced, except in full, without prior authorization from this office.

Should you have any questions regarding the recommendations or opinions presented in this report, please contact us at (503) 601-8250.

DOWNSTREAM WAREHOUSE REMODEL - PORTLAND, OREGON

SITE LOCATION



Map created with TOPO!™, © 2006 National Geographic Holdings
USGS 7.5 Minute Topographic Map Series, Portland, OR Quadrangle.

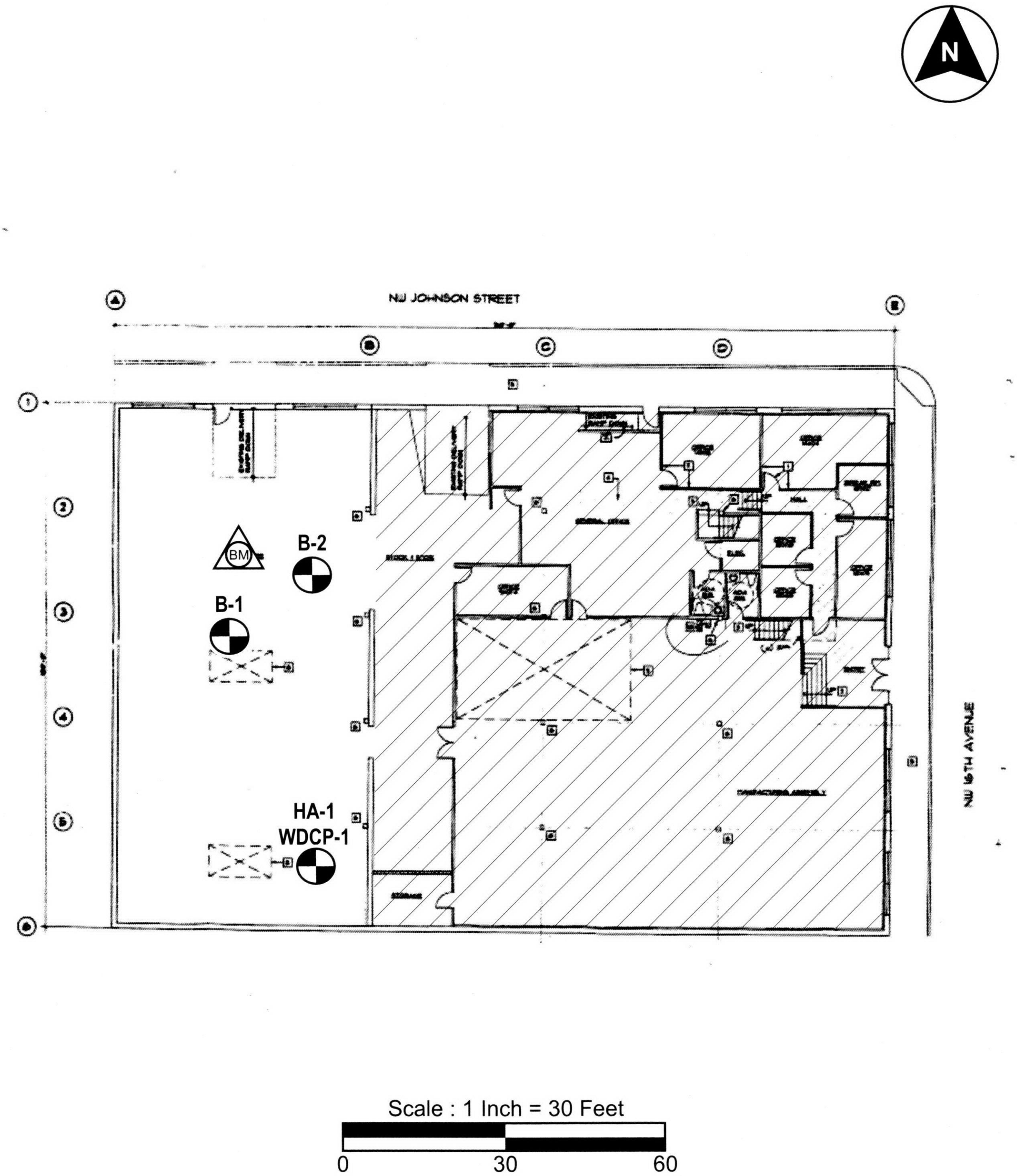
Scale 1 Inch = 2,000 feet

0 2000 4000

Township 1 North, Range 1 East, Section 33 Willamette Meridian

Carlson Geotechnical ***P.O. Box 23814*** ***Tigard, Oregon 97281***	**CGT Job No. G1203772**	**FIGURE 1**

DOWNSTREAM WAREHOUSE REMODEL - PORTLAND, OREGON
SITE PLAN



NOTES: Drawing based on site plan prepared by David Bissett & Associates. Boring locations and site details based on field observations.

LEGEND

Symbol	Description
B-1	Approximate location of mud rotary boring.
HA-1 WDCP-1	Approximate location of hand auger boring and cone penetrometer test.
(hatched)	Approximate boundary of area inaccessible to track mounted drill rig.
BM	Temporary elevation benchmark: Assigned 100-foot elevation at the top of the existing slab

CGT Job No. G1203772

CARLSON CGT GEOTECHNICAL 503-601-8250

Carlson Geotechnical
P.O. Box 23814
Tigard, Oregon 97281

FIGURE 2

DOWNSTREAM WAREHOUSE REMODEL - PORTLAND, OREGON
SOIL CLASSIFICATION CRITERIA AND TERMINOLOGY

Classification of Terms and Content	USCS Grain Size		
NAME : MINOR Constituents (12-50%); MAJOR Constituents (>50%); Slightly (5-12%) Relative Density or Consistency Color Moisture Content Plasticity Trace Constituents (0-5%) Other: Grain Shape, Approximate gradation, Organics, Cement, Structure, Odor... Geologic Name or Formation: Fill, Willamette Silt, Till, Alluvium...	Fines		<#200 (.075 mm)
	Sand	Fine Medium Coarse	#200 - #40 (.425 mm) #40 - #10 (2 mm) #10 - #4 (4.75)
	Gravel	Fine Coarse	#4 - 0.75 inch 0.75 inch - 3 inches
	Cobbles		3 to 12 inches; scattered <15% est., numerous >15% est.
	Boulders		> 12 inches

Relative Density or Consistency

Granular Material		Fine-Grained (cohesive) Materials				
SPT N-Value	**Density**	**SPT N-Value**	**Torvane tsf Shear Strength**	**Pocket Pen tsf Unconfined**	**Consistency**	**Manual Penetration Test**
		<2	<0.13	>0.25	Very Soft	Thumb penetrates more than 1 inch
0 - 4	Very Loose	2 - 4	0.13 - 0.25	0.25 - 0.50	Soft	Thumb penetrates about 1 inch
4 - 10	Loose	4 - 8	0.25 - 0.50	0.50 - 1.00	Medium Stiff	Thumb penetrates about 1/4 inch
10 - 30	Medium Dense	8 - 15	0.50 - 1.00	1.00 - 2.00	Stiff	Thumb penetrates less than 1/4 inch
30 - 50	Dense	15 - 30	1.00 - 2.00	2.00 - 4.00	Very Stiff	Readily indented by thumbnail
>50	Very Dense	>30	>2.00	>4.00	Hard	Difficult to indent by thumbnail

Moisture Content

Dry: Absence of moisture, dusty, dry to the touch
Damp: Some moisture but leaves no moisture on hand
Moist: Leaves moisture on hand
Wet: Visible free water, likely from below water table

	Plasticity	Dry Strength	Dilatancy	Toughness
ML	Non to Low	Non to Low	Slow to Rapid	Low, can't roll
CL	Low to Med.	Medium to High	None to Slow	Medium
MH	Med to High	Low to Medium	None to Slow	Low to Medium
CH	Med to High	High to V. High	None	High

Structure

Stratified: Alternating layers of material or color >6 mm thick
Laminated: Alternating layers < 6 mm thick
Fissured: Breaks along definate fracture planes
Slickensided: Striated, polished, or glossy fracture planes
Blocky: Cohesive soil that can be broken down into small angular lumps which resist further breakdown
Lenses: Has small pockets of different soils, note thickness
Homogeneous: Same color and appearance throughout

Unified Soil Classification Chart (Visual-Manual Procedure) (Similar to ASTM Designation D-2488)

Major Divisions			Group Symbols	Typical Names
Coarse Grained Soils: More than 50% retained on No. 200 sieve	Gravels: 50% or more *retained* on the No. 4 sieve	Clean Gravels	GW	Well graded gravels and gravel-sand mixtures, little or no fines
			GP	Poorly-graded gravels and gravel-sand mixtures, little or no fines
		Gravels with Fines	GM	Silty gravels, gravel-sand-silt mixtures
			GC	Clayey gravels, gravel-sand-clay mixtures
	Sands: more than 50% *passing* the No. 4 Sieve	Clean Sands	SW	Well-graded sands and gravelly sands, little or no fines
			SP	Poorly-graded sands and gravelly sands, little or no fines
		Sands with Fines	SM	Silty sands, sand-silt mixtures
			SC	Clayey sands, sand-clay mixtures
Fine-Grained Soils: 50% or more Passes No. 200 Sieve	Silt and Clays Low Plasticity Fines		ML	Inorganic silts, rock flour, clayey silts
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, lean clays
			OL	Organic silt and organic silty clays of low plasticity
	Silt and Clays High Plasticity Fines		MH	Inorganic silts, clayey silts
			CH	Inorganic clays of high plasticity, fat clays
			OH	Organic clays of medium to high plasticity
Highly Organic Soils			PT	Peat, muck, and other highly organic soils

CARLSON CGT GEOTECHNICAL 503-601-8250
Carlson Geotechnical
P.O. Box 23814
Tigard, Oregon 97281

CGT Job No. G1203772

FIGURE 3

Carlson Geotechnical
PO Box 23814
Tigard, OR 97281
503-601-8250

FIGURE 4

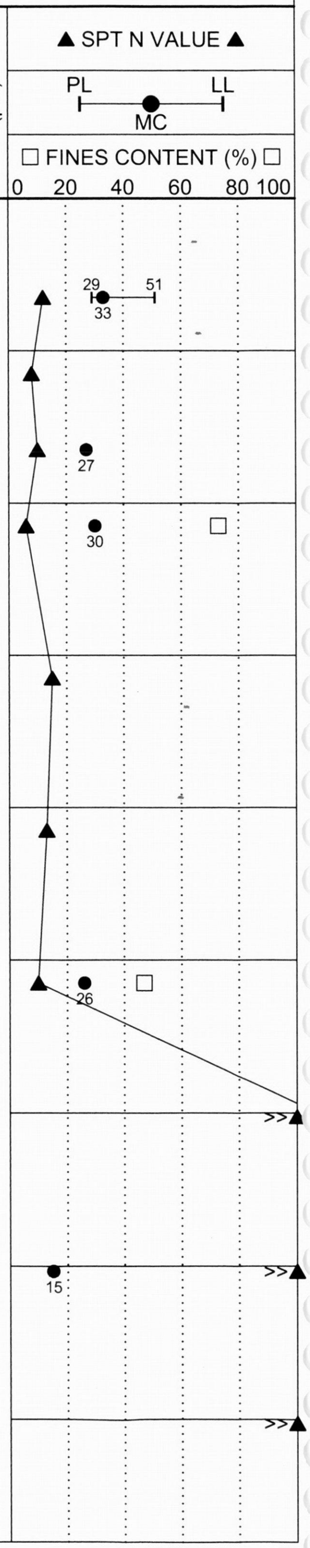
Boring B-1

CLIENT Downstream
PROJECT NAME Downstream Warehouse Remodel
PROJECT NUMBER G1203772
PROJECT LOCATION 735 NW 16th Avenue - Portland, Oregon
DATE STARTED 10/25/12
ELEVATION DATUM Top of existing slab - See Figure 2
DRILLING CONTRACTOR Western States Soil Conservation
GROUND ELEVATION 100 ft
DRILLING METHOD Mud Rotary
GROUND WATER LEVELS:
LOGGED BY M. David Irish **CHECKED BY** Kyle Smetana
▽ **AT TIME OF DRILLING** 7.0 ft / Elev 93.0 ft
NOTES WS-45 track-mounted, drill rig
▼ **AFTER DRILLING** 29.5 ft / Elev 70.5 ft

ELEVATION (ft)	U.S.C.S.	MATERIAL DESCRIPTION
100		**CONCRETE:** Approximately 5 inches thick.
	MH	**ELASTIC SILT:** Stiff, brown, moist and exhibited medium plasticity. Perched groundwater encoutered at about 7 feet bgs.
	ML	**SANDY SILT:** Medium stiff to stiff, brown, very moist, and exhibited low plasticity.
	SP	**SAND:** Medium dense, brown, very moist, poorly graded, and fine- to coarse-grained.
	SM	**SILTY SAND:** Medium dense, brown, very moist, and fine- to coarse-grained. Groundwater level interpretted at a depth of about 29½ feet bgs.
	GP	**SANDY GRAVEL:** Very dense, wet, gray, sub-rounded, fine to coarse (up to 1-inch diameter), and with silt. Sand was fine- to coarse-grained.
		-Boring terminated at about 40¼ feet bgs -No caving observed. -Boring backfilled with granular bentonite and capped with quickcrete patch.

SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)
SS 1	100	4-5-7 (12)		
SS 2	100	3-3-5 (8)		
SS 3	89	3-4-6 (10)		
SS 4	72	4-3-3 (6)		
SS 5	67	4-6-9 (15)		
SS 6	67	6-5-8 (13)		
SS 7	67	6-5-5 (10)		
SS 8	0	50/3"		
SS 9	50	50/4"		
SS 10	67	50/3"		

Elevation scale: 95, 90, 85, 80, 75, 70, 65, 60. Depth (ft) scale: 0, 5, 10, 15, 20, 25, 30, 35, 40.

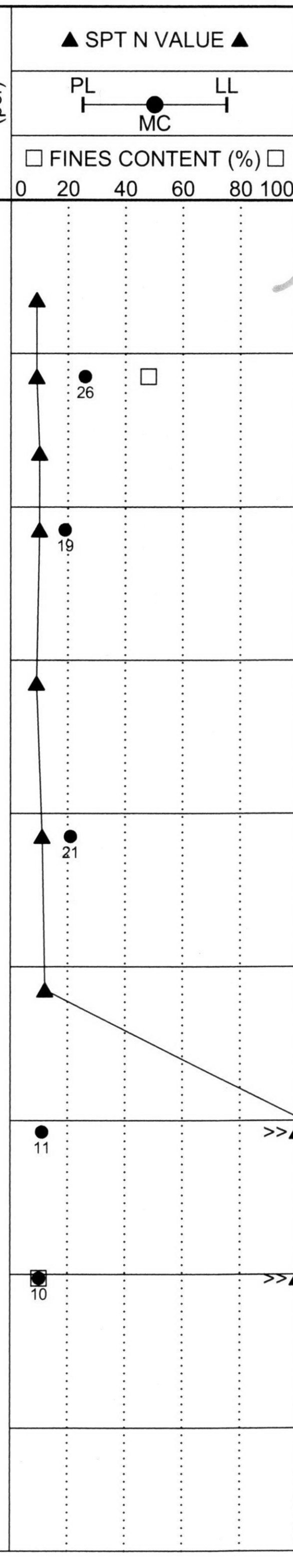


Carlson Geotechnical
PO Box 23814
Tigard, OR 97281

FIGURE 5
Boring B-2

CLIENT Downstream
PROJECT NAME Downstream Warehouse Remodel
PROJECT NUMBER G1203772
PROJECT LOCATION 735 NW 16th Avenue - Portland, Oregon
DATE STARTED 10/25/12
ELEVATION DATUM Top of existing slab - See Figure 2
DRILLING CONTRACTOR Western States Soil Conservation
GROUND ELEVATION 100 ft
DRILLING METHOD Mud Rotary
GROUND WATER LEVELS:
LOGGED BY M. David Irish **CHECKED BY** Kyle Smetana
AT TIME OF DRILLING 7.0 ft / Elev 93.0 ft
NOTES WS-45 track-mounted, drill rig
AFTER DRILLING 29.5 ft / Elev 70.5 ft

ELEVATION (ft)	U.S.C.S.	MATERIAL DESCRIPTION	DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MC
100		**CONCRETE:** Approximately 5 inches thick.	0						
	MH	**ELASTIC SILT:** Stiff, brown, moist and exhibited medium plasticity.		SS 1	100	3-4-5 (9)			
95	SM	**SILTY SAND:** Loose to medium dense, brown, moist, and fine- to medium-grained. Perched groundwater encoutered at about 7 feet bgs.	5	SS 2	44	3-4-5 (9)			26
				SS 3	44	4-4-6 (10)			
90	SP	**SAND:** Loose to medium dense, brown, very moist, poorly graded, and fine- to coarse-grained.	10	SS 4	44	5-5-5 (10)			19
85	SM	**SILTY SAND:** Medium dense, brown, very moist to wet, and fine- to coarse-grained.	15	SS 5	56	6-4-5 (9)			
80	SP	**SAND:** Medium dense, brown, very moist, poorly graded, and fine- to coarse-grained.	20	SS 6	56	8-5-6 (11)			21
75			25	SS 7	78	6-5-7 (12)			
		Groundwater level interpretted at a depth of about 29½ feet bgs.							
70	GP	**SANDY GRAVEL:** Very dense, wet, gray, sub-rounded, fine to coarse (up to 1-inch diameter), and with silt. Sand was fine- to coarse-grained.	30	SS 8	67	25-50/3"			11
65			35	SS 9	133	50/3"			10
60		-Boring terminated at about 36½ feet bgs -No caving observed. -Boring backfilled with granular bentonite and capped with quickcrete patch.	40						



Carlson Geotechnical
PO Box 23814
Tigard, OR 97281

FIGURE 6

Boring HA-1

CLIENT Downstream
PROJECT NAME Downstream Warehouse Remodel
PROJECT NUMBER G1203772
PROJECT LOCATION 735 NW 16th Avenue - Portland, Oregon
DATE STARTED 10/25/12
ELEVATION DATUM Top of existing slab - See Figure 2
DRILLING CONTRACTOR CGT
GROUND ELEVATION 100 ft
DRILLING METHOD Hand Auger & WDCP
GROUND WATER LEVELS:
LOGGED BY Kyle Smetana **CHECKED BY** Brad Wilcox
AT TIME OF DRILLING ---
NOTES 3-inch diameter, manual hand auger
AFTER DRILLING ---

ELEVATION (ft)	GRAPHIC LOG	U.S.C.S.	MATERIAL DESCRIPTION	GROUNDWATER	DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	▲ SPT N VALUE ▲ / PL MC LL / ☐ FINES CONTENT (%) ☐ 0 20 40 60 80 100
			CONCRETE: Approximately 5 inches thick.		0.0			7			
		MH	**ELASTIC SILT:** Medium stiff, brown, moist and exhibited medium plasticity.					7 7 7 8			
			Stiff to very stiff below about 2 feet bgs.		2.5			12 11 12 12			
95			-Boring advanced to confirm presence of native soils below the floor slab and to compare to the near surface profiles in borings B-1 and B-2. -Boring terminated at about 3 feet bgs -WDCP-1 advanced to a depth of about 7 feet bgs. -No caving or groundwater observed. -Boring backfilled with cuttings and capped with quickcrete patch.		5.0			12 9 12 14 16 14 11 12 13 14 15			
					7.5						
90					10.0						

SCAN

ABHT
STRUCTURAL ENGINEERS
1640 NW Johnson Street
Portland, OR 97209
Tel 503.243.6682
Fax 503.243.6622
www.abht-structural.com

ASCE 31 Seismic Evaluation Report

for

735 NW 16th Ave, Portland, OR

ABHT Structural Engineers
Project No. 16812

December 17, 2012

Prepared for:

Downstream
1650 NW Naito Parkway, Suite 301
Portland, Oregon 97209

1.0 INTRODUCTION

ABHT Structural Engineers was retained by Downstream to conduct an ASCE 31 seismic evaluation of the existing one-story building located at 735 NW 16th Ave in Portland, Oregon. The seismic evaluation was triggered by the proposed change of occupancy from its current B/S occupancy to a B occupancy for the majority of the building. Based on the requirements of Chapter 24.85 of the City of Portland Title 24 City Code and Charter, this change in occupancy requires that a seismic evaluation as well as a seismic upgrade be performed in accordance with the ASCE 31 standard (including both Tier 1 and 2 Phases). This report is limited to the ASCE 31 Evaluation along with a list of deficient items and requirements that will need to be addressed during the design and construction phase of the project.

The evaluation is based on the American Society of Civil Engineers (ASCE) Standard 31, *Seismic Evaluation of Existing Buildings*, commonly referred to as ASCE 31. The purpose of ASCE 31 is to provide guidance in the review of an existing building's response to a predetermined level of earthquake based on a selected performance level. This evaluation is based on a Life Safety performance level defined by ASCE 31 as "building performance that includes damage to both structural and nonstructural components during a design earthquake, such that: (a) partial or total structural collapse does not occur, and (b) damage to nonstructural components is non-life threatening." A Life Safety performance level is commonly used for buildings of this construction type and occupancy.

The ASCE 31 evaluation consists of three phases, or tiers. Tier 1 is the Screening Phase which consists of completing a series of checklists and in some cases performing Quick Check structural calculations to quickly identify potential deficiencies. The Tier 2 Phase involves further evaluation of deficiencies identified in the Tier 1 Phase evaluation where required by ASCE 31. The Tier 3 Phase consists of evaluations which are very detailed and involved and only required for certain types of structures. This evaluation does not require a Tier 3 evaluation and therefore is limited to the Tier 1 Screening Phase and Tier 2 Evaluation Phase only.

Our evaluation includes a limited walkthrough of the existing building to observe the general physical status of the structure, an evaluation of the building's lateral force resisting system, identification of potential deficiencies based on our physical observations readily available to view, calculations, and provides the deficiencies and recommendations for seismic upgrades. The evaluation was based on information gathered from limited as-built drawings obtained from the City of Portland Archives, information gathered from our site observation, and site investigations and testing performed to determine the extent of reinforcing and compressive strength of existing concrete walls (performed by Carlson Testing on October, 25, 2012, See Appendix C).

Observations, analysis, conclusions and recommendations contained in this report reflect our best engineering judgment. Concealed problems with the construction of the building may exist that cannot be revealed through our review. ABHT Structural Engineers in no way guarantees the condition of the existing construction of the building and the building site.

2.0 BUILDING DESCRIPTION

The building is a one-story, approximate 15,000-sf, concrete and wood-framed structure situated on a gently sloping site. Constructed circa 1930, the building is rectangular in shape with approximate plan dimensions of 150' (in the East-West direction) and 100' (in the North-South direction). The building appears to consist of two separate structures constructed at separate times. The original structure appears to be a 100' x 100' square shape on the East side of the property constructed circa 1929. A 50' x 100' structure was added later to the West, utilizing similar framing. The added structure appears to utilize the West concrete wall of the original structure for lateral resistance of forces in the North-South direction. The structural framing system consists of the following:

<u>Roof – 100' x 100' East Structure</u>

- The roof is framed with 1x straight wood decking supported by 2x12 joists @ 20" on center spanning in the North-South direction. The joists are supported at the East and West ends by exterior cast-in-place concrete walls and at the interior by 10x timber beams spanning East-West spaced at approximately 20'-0" on center. The 10x beams are supported at the exterior concrete walls on integral pilasters and at the interior by timber columns.

<u>Roof – 50' x 100' West Structure</u>

- The barrel shaped roof is framed with 1x straight wood decking supported by 2x12 joists @ 16" on center spanning in the North-South direction. The joists are supported at the East and West ends by exterior cast-in-place concrete walls and at the interior by timber bow-string trusses spanning East-West spaced at approximately 20'-0" on center. The trusses are supported at the West end by integral concrete pilasters and at the East end by timber column placed adjacent to the original west wall of the original 100'x100' structure.

<u>Walls</u>

- The South and West exterior walls consist of 9" thick cast-in-place concrete walls with no openings. The North and East exterior walls consist of 8" to 9" cast-in-place concrete walls with large openings for windows and doors. An internal 9" thick cast-in-place concrete wall exists in the North-South direction where the two structures join and contains a few large openings for traffic pass-through. On site ground-penetrating radar (GPR) testing concluded that the existing walls have no steel reinforcing within.

<u>Floor</u>

- The first floor appears to consist of a 5" thick concrete slab-on-grade. We were not able to confirm the slab reinforcing (if reinforcing exists) due to limited information on the as-built drawings.

<u>Foundations</u>

- Based on the original as-built drawings for the original 100'x100' building, the wall foundation appears to be a "property line type" continuous 12" wide footing poured as an integral thicker section of wall with localized larger separate concrete spread footings at the integral pilasters. The interior timber columns are founded on 2'-6" x 2'-6" x 1'-0" thick concrete spread footings with reinforcing (if present) unable to be determined.

<u>Lateral Force Resisting System</u>

- Lateral loads are resisted by the concrete shear walls noted above. A straight-sheathed wood roof diaphragm transmits the lateral loads to the shear walls.

3.0 VISUAL OBSERVATION

On September 19, 2012, a representative of ABHT (Justin Lyons) walked through the building and reviewed the general condition of the structure and its contents. The primary objectives of the site observation were to evaluate the structure readily exposed to view, to look for signs of distress, settlement, or deterioration, and to become familiar with the building and its construction. Additionally, an attempt was made to verify that the construction of the building structure was consistent with the design represented on the original drawings. No material testing or exploratory demolition occurred during this observation visit to evaluate the existing construction. Our observation revealed the following:

- Adjacent structures abutted the building under evaluation along portions of the South and West exterior concrete walls. The roof level of the building to the West was at the approximate same elevation, while the building to the South was about 7'-6" lower than the building's roof that is being evaluated (see Appendix B, Figure 5).

- The roof of the original 100' x 100' structure is slightly higher than the roof of the added 50' x 100' building where they meet at the interior concrete wall.

- Roughly 30' of the North parapet of the added 50' x 100' structure, the parapet height to thickness ratio may exceed a 2.5 height-to-thickness ratio. Subsequent GPR testing concluded that the concrete walls and parapets are unreinforced (see Appendix B, Figure 6).

- A wood framed mezzanine was added to the original structure at the NE corner of the building circa 2000, however, it is our understanding that the mezzanine will be demolished as part of the planned renovation.

- A vertical crack was noted in the center of the concrete header above the large opening in the interior concrete wall. A steel channel header exists on each side at the bottom of the header and appears to act as the tension reinforcement in the header (see Appendix B, Figure 1). This concrete header also does not support any significant building load other than its own self weight.

- No significant spalling of the concrete was observed.

- No significant cracks were observed to be propagating from the corners of openings in the concrete walls.

- No significant settlement of foundations was observed, however, a few vertical cracks were noted in walls that appeared to be due to settlement from additional loads put on the soil from adjacent structures built subsequently. No horizontal cracks (indicative of cracking due to tension) were observed in walls (see Appendix B, Figure 2). We would recommend that all cracks be filled with epoxy.

- The 10x timber beams did not appear to be positively anchored to the exterior concrete walls or pilasters (see Appendix B, Figure 2).

- No strapping between the 10x timber beam ends was observed. This indicates that there is a lack of continuous cross-ties between the roof diaphragm chords.

- The 10x beams did not appear to be positively anchored for uplift to their supporting columns or pilasters.

- The web members of the bowstring trusses had signs of checking, however this is not a significant issue for members in compression (see Appendix B, Figure 3).

- We did not observe any positive out-of-plane or in-plane wall anchorage of the concrete walls to the roof diaphragm.

- The equipment hanging from the roof above the main entrance did not appear to be adequately braced (see Appendix B, Figure 4).

4.0 EVALUATION SUMMARY AND LIST OF DEFICIENCIES

We have performed an ASCE 31 Tier 1 Screening Phase and Tier 2 Evaluation Phase of the existing structure. The required checklists per Table 3-2 of the ASCE 31 were completed for a building type C2A; *Concrete Shear Wall Building with Flexible Wood Diaphragm*, using a High Level of Seismicity and a Life Safety Level of Performance. Please reference Appendix A for the completed ASCE 31 required checklists and the supporting structural calculations provided with this report. Please reference the attached Table A: ASCE 31 Evaluation Report Deficiencies for all deficiencies based on our ASCE 31 Evaluation.

5.0 UPGRADE RECOMMENDATIONS/REQUIREMENTS

Reference structural drawings S001 through S701, dated December, 14, 2012 for structural upgrade requirements based on deficiencies described in Table A.

Attached Appendices are as follows: Appendix A: ASCE 31 Tier 1 Evaluation Checklists
Appendix B: Site Photographs
Appendix C: Carlson Testing Report

TABLE A: ASCE 31 Evaluation Report Deficiencies

Deficiency Item # and Deficiency Label	Checklist	Checklist Subsection	Description	Added Notes
1. Adjacent Buildings	Basic Structural Checklist	Building System	The existing building to the west has a very minimal separation and the buildings to the southwest have no separation and an approximate 2-1/2". All of these separations are too small and will require the upgrade to account for potential "pounding" from adjacent buildings.	
2. Mezzanines	Basic Structural Checklist	Building System	Based on the existing drawings that were available, along with the observations readily available to view during our walkthrough, it cannot be determined if the mezzanine was adequately attached to the existing building concrete walls.	We understand that the existing mezzanine will be removed during any building renovation. Additionally, the mezzanine was built well after the original construction (circa 2000), and therefore is not dependent on by the structure for bracing.
3. Wall Stress/Reinforcing Steel	Basic Structural Checklist	Lateral Force Resisting System	Carlson Testing was hired and performed GPR testing, which concluded that no reinforcing steel existed in the concrete walls. While the Tier 1 quick shear stress would prove adequate, the Tier 2 analysis was not adequate for the North and East walls due to the number of openings within these walls.	The North and East walls will be strengthened with new reinforced concrete shear walls.
4. Wall Anchorage	Basic Structural Checklist	Connections	It does not appear, based on observations readily available to view and our experience for buildings of this vintage, that any out-of-plane anchorage from the roof diaphragm was present.	
5. Transfer to Shear Walls	Basic Structural Checklist	Connections	It does not appear, based on observations readily available to view and our experience for buildings of this vintage, that adequate in-plane anchorage from the roof diaphragm was present.	
6. Foundation Dowels	Basic Structural Checklist	Connections	Wall foundations appeared to be poured integral as a thickend element of the wall above, however no foundation dowels were evident on the existing drawings or during site tests.	
7. Coupling Beams	Supplemental Structural Checklist	Lateral Force Resisting System	The coupling beams within the existing concrete walls on the north and east side of the building are not adequate.	This is not a concern for the upgrade since these walls will need to be strengthened with a new concrete shearwall.
8. Diaphragm Continuity	Supplemental Structural Checklist	Diaphragms	The west 50-ft portion roof diaphragm elevation of existing building intersects the adjacent 100-ft building roof at the interior north-south concrete wall approximately 2-ft below the 100-ft building roof diaphragm elevation.	
9. Cross Ties	Supplemental Structural Checklist	Diaphragms	The existing roof diaphragm does not have the adequate roof diaphragm ties and strapping.	
10. Spans	Supplemental Structural Checklist	Diaphragms	The existing roof diaphragm span requires that wood structural panel sheathing be provided.	
11. Overturning	Geologic Site Hazards and Foundations Checklist	Capacity of Foundations	While the Tier 1 aspect ratio is adequate, the Tier 2 analyais for the existing foundations for the existing concrete walls on the interior, north, and east sides of the building are not adequate and will need to be strengthened. The interior wall had a bit larger bearing than 2x the allowable bearing and therefore the footing at the interior wall will be increased in width to reduce the overall bearing.	
12. Tall Narrow Contents	Basic Nonstructural Component Checklist	Building Contents and Furnishing	Some existing tall storage racks did not appear to have positive base anchorage or anchorage to concrete walls.	We understand these storage racks will most likely be removed in the event of a renovation so this should not be a problem.
13. Attached Equipment/Flexible Couplings	Basic Nonstructural Component Checklist	Mechanical and Electrical Equipment	Based on our readily available to view site observations, it appears that equipment hanging directly above the entry did not have adequate bracing.	
14. Concrete Parapets	Intermediate Nonstructural Component Checklist	Parapets, Cornices, Ornamentation, and Appendages	Approximate 30-ft length of the north parapet will require bracing or adhesive reinforcement drilled from the top of the concrete parapet.	