December 9, 2013

D R Horton, Inc.
4380 SW Macadam Avenue, Suite 100
Portland, Oregon 97239

Subject: Geotechnical Investigation
Meinecke Park Subdivision

Attn: Kati Gault

Northwest GEO Consultants is pleased to submit our Geotechnical Investigation for the Meinecke Park Development in Sherwood, Oregon. This report was prepared in accordance with Contract Number 200269 OF 71000/8000 dated October 30, 2013. The report summarizes the work accomplished and provides our recommendations for site development.

PROJECT INFORMATION

We understand that D R Horton will develop the approximate 5.8 acre Meinecke Road site as a residential subdivision. Plans call for site construction of about 18 single family and 15 attached multi-family homes along with a stormwater facility, underground utilities and roadways The site is currently undeveloped. The site relative to surrounding features is shown in Figure 1.

Homes on the project are expected to be up to three stories tall and will be supported on both continuous and isolated spread footings. We expect that sustained dead loads on the foundations will be on the order of 1,500 pounds per linear foot for continuous footings and 1,500 pounds per square foot for spread footings. The preliminary site layout is shown in Figure 2.

SCOPE OF WORK

The purpose of our services was to explore the site and provide recommendations for design and construction. The following describes our specific scope of services:

- Coordinate and manage the field investigation, including utility locates, authorization for site access, access preparation, scheduling of contractors and NGC staff.
- Complete 8 test pits up to 9 feet below the existing ground surface. The test pits were generally located in street or driveway areas.
- Maintain a log of soil, rock, and groundwater conditions encountered during the explorations. We described the soil in general accordance with the Unified Soil classification System (USCS) using ASTM D2488 (Visual-Manual Procedure).
- Obtain grab samples from the sides of the test pits and relatively undisturbed samples using thin wall (Shelby) tubes in general accordance with guidelines presented in ASTM D1587, the Standard for Thin-walled Tube Sampling of Soils. We returned the samples to our laboratory for additional evaluation and testing.
- Determine the moisture content of soil samples and the dry unit weight of samples obtained from the Shelby tubes in general accordance with guidelines provided in ASTM D-2216 and ASTM D1587 respectively.

- Provide a written Geotechnical Investigation Report summarizing our explorations, geotechnical analysis, conclusions, and recommendations. Our report will include figures showing the site location and the location of explorations on the site. Our specific recommendations and opinions will include:
  - A professional opinion stating whether or not each lot meets Form HUD-92541 (4/2001) regarding Foreseeable Hazards and FHA Data Sheet 79G as it relates to Controlled Earthwork requirements.
  - A discussion on the regional geology and the seismic setting of the site that will include the general geologic features of the surface and underlying deposits and tectonic faulting in the area.
  - An evaluation of the seismic hazards that may be present at the site and provide seismic design criteria in accordance with the Oregon Structural Specialty Code.
  - Recommendations for site preparation, grading and drainage, use and reuse of onsite soil and imported material for structural fill, compaction criteria, cut-and-fill slope criteria, and wet-weather earthwork procedures.
  - Recommendations for utility trench excavation, backfill materials, and backfill compaction.
  - Recommendations for design and construction of shallow-spread foundations, including allowable design bearing pressures, minimum footing depth and width, lateral resistance to sliding, and estimates of settlement.
  - Geotechnical engineering recommendations for the design and construction of concrete floor slabs, including an anticipated value for subgrade modulus.
  - A discussion of groundwater conditions on the site and recommendations for subsurface drainage of foundations, floor slabs, and pavement.

SITE CONDITIONS

The project site is an approximate 5.8 acre area located on an upland plain on the southern margin of the Tualatin Valley in the City of Sherwood, Washington County, Oregon. The site relative to surrounding features is shown in Figure 1. The following paragraphs describe the area geology, surface, and subsurface features.

SITE GEOLOGY

The site is located in the Tualatin Basin, a structural basin filled with a thick sequence of sediment (Yeats et al., 1996)[1]. The upper portion of the sediment is the Quaternary age (last 1.6 million years) Willamette Silt, a catastrophic flood deposit associated with repeated glacial outburst flooding of the Willamette Valley, the last of which occurred about 10, 000 years ago. Regionally, these deposits

[1] Yeats, R.S., Graven, E.P., Werner, K.S., Goldfinger, T. and Popowski, 1996, Tectonics of the Willamette Valley, Oregon: in Assessing earthquake hazards and reducing risk in the Pacific Northwest, v. 1, U.S. Geological Survey Professional Paper 1560, p. 183-222, 5 plates, map scale 1:100,000.

consists of horizontally layered silt to coarse sand forming beds up to 3 feet thick[2]. Locally, the flood deposits are mantled by a thin layer of windblown silt (loess) that is difficult to distinguish from the water deposited silt. Based on regional geologic mapping, the thickness of Willamette Silt in the vicinity of the subject site is on the order of 30 to 40 feet.

The Willamette Silt is underlain by an unnamed sequence of continental, fine-grained strata including moderately- to poorly-lithified siltstone, sandstone, mudstone and claystone with wood fragments and minor amounts of volcanic ash and pumice (Yeats et al., 1996; Gannett and Caldwell, 1998). These rocks are tentatively correlated with the Sandy River Mudstone, and the Troutdale and Helvetia Formations.

Underlying the unnamed sedimentary strata is Miocene (about 14.5 to 16.5 million year old) Columbia River Basalt Group (CRBG), a thick sequence of lava flows which forms the crystalline basement of the Tualatin Basin (Yeats et al., 1996). These basalts are a dense, finely crystalline rock that is commonly fractured along blocky and columnar joints. Where highly weathered, the upper surface of the basalt is altered to a distinctive, red-brown, clayey silt known as laterite or residual soil. Structure contour mapping indicates that the top of the CRBG lies about 175 feet below the ground surface in the site area.

SEISMIC SETTING

Seismic Sources

The Tualatin Basin is subject to seismic events stemming from three possible sources: the Cascadia Subduction Zone (CSZ) at the interface between the Juan de Fuca plate and the North American plate; intraslab faults within the Juan de Fuca plate; and crustal faults in the North American plate.

Maximum magnitude for a CSZ event is expected to be in the range of Moment Magnitude (MW) 9.0. Intraslab events have occurred on a frequent basis in the Puget Sound, contributing small magnitude ground motions in Western Oregon.

There are no mapped faults that pass directly through the site. Quaternary faults within 10 miles of the site are the Canby-Molalla Fault about 4.7 miles to the northeast, Beaverton Fault about 7.2 miles to the north, and the Newberg Fault about 6.7 miles to the southwest.

Seismic Design Factors

The contribution of potential earthquake-induced ground motion from all known sources, including the fault described above, are included in the probabilistic ground motion maps developed by the USGS. Seismic site characterization and design recommendations based on USGS mapping and analysis are implemented in the International Building Code/Oregon Structural Specialty Code. Seismic design parameters for the project site are provided in Figure 3.

SURFACE CONDITIONS

The project site is a relatively flat-lying area that lies at an approximate elevation of 200 feet above mean sea level. The property is bordered on the north and west by a 20 to 30 feet deep gully shown

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[2] Gannett and Caldwell, 1998, Geologic framework of the Willamette Lowland aquifer system, Oregon and Washington; U.S. Geological Society Professional Paper 1424-A, 32 pages, 8 plates.

on USGS mapping to be a tributary of Chicken Creek. The property is bordered by residential development on the east and by Meinecke Road on the south.

The majority of ground surface is currently vegetated with low grass with sparse dense brush and trees. Aerial photos available from Google Earth© show the site was previously used in agriculture for production of grass seed or hay.

SUBSURFACE CONDITIONS

We explored subsurface conditions at the site by excavating 8 test pits (TP-1 through TP-9) to depths up to 9 feet below the existing ground surface. The test pits were excavated on October 18, 2013 using a medium size tracked excavator owned and operated by Parker Concrete of Forest Grove, Oregon. Descriptions of the field explorations, exploration logs, and laboratory procedures are included in Attachment A. The approximate locations of the test pits are shown in Figure 2.

Till Zone

We encountered an 18- to 24- inch thick till zone at the ground surface consistent with prior agricultural activity on the site. The till zone consists of soft silt with trace organics. A 4-inch thick heavily rooted zone is present at the ground surface.

Native Soil

Native soil was encountered directly below the till zone in all 8 of the test pits excavated at the site. The native soil is medium stiff to very stiff silt. Laboratory tests indicate an in-situ dry density of 90.5 and 94.5 pcf. Moisture content of the silt varied from 24.1 to 30.8 percent.

Details of the soil layers encountered and the results of laboratory testing are provided in the test pit logs included in Attachment A.

GROUNDWATER CONDITIONS

We explored the site in October during the first third of the wet season. No groundwater was encountered in the explorations to a depth of 9 feet below the ground surface. Shallow perched groundwater should be expected near the ground surface in the winter and spring months.

Regional groundwater studies report a groundwater table depth in the site vicinity of about 20 feet below the ground surface (Gannett and Caldwell, 1998).

RECOMMENDATIONS

Based on the results of our field explorations and our engineering analysis, it is our opinion that the site can be developed as proposed. Expected building loads can be supported on the medium stiff to very stiff undisturbed native silt that underlies the site or on newly placed structural fill supported on undisturbed native silt.

An approximate 18- to 24-inch-thick till zone overlies the entire site. The zone consists of soft to medium stiff silt with a heavily rooted zone extending to approximately 4 inches below the ground surface. We recommend that after stripping the heavily rooted zone, the site surface should be scarified to a depth of 24 inches and compacted as structural fill.

Specific recommendations for project design and construction are provided in the paragraphs that follow.

SITE PREPARATION

The existing heavily rooted zone of grass and organics should be stripped and removed from the site in all proposed building and pavement areas and for a 5-foot margin around such areas. Based on our explorations, the depth of stripping will be about 4 inches although greater stripping depths may be required to remove localized zones of loose or organic soil. The actual stripping depth should be based on field observations at the time of construction. Stripped material should be transported off site for disposal or used in landscaped areas.

An approximate 1-1/2 to 2- foot thick till zone was observed in the explorations. We recommend removing or scarifying the stripped ground surface to the depth of the tilled zone within building and paved fill areas prior to placing structural fill. The scarified soil should be compacted as recommended for structural fill.

The on-site silt can be sensitive to small changes in moisture content and may be difficult to compact adequately during wet weather. Accordingly, scarification and compaction of the subgrade may only be possible during extended dry periods and following moisture conditioning of the soil.

After stripping, scarification and required site cutting have been completed, we recommend proofrolling the subgrade with a fully loaded dump truck or similar size, rubber-tire construction equipment to identify areas of excessive yielding. The proofrolling should be observed by a member of our geotechnical staff, who will evaluate the subgrade. If areas of excessive yielding are identified, the material should be excavated and replaced with compacted materials recommended for structural fill. Areas that appear to be too wet and soft to support proofrolling equipment should be prepared in accordance with the recommendations for wet weather construction presented in the following section of this report.

The test pits excavations were backfilled using relatively minimal compactive effort. Therefore, soft spots can be expected at these locations. We recommend that these relatively uncompacted soils be removed from the test pits located within the proposed building and paved areas to a depth of 3-feet below finished subgrade. The resulting excavation should be brought back to grade with structural fill.

WET WEATHER CONSTRUCTION

Near surface soil on the site can become disturbed during the wet season. Earthwork should be planned and executed to minimize subgrade disturbance.

We recommend that a minimum of 2-inch thickness of granular material be placed at the base of footing excavations in wet weather conditions. The granular material reduces water softening of subgrade soils, reduces subgrade disturbance during placement of forms and reinforcement, and provides a clean environment for reinforcing steel. To be effective, the granular material should be compacted until well keyed using a small vibratory plate compactor.

PERMANENT SLOPES

Permanent cut and fill slopes constructed using on-site soil should not exceed a grade of 2H:1V (Horizontal to Vertical). Slopes that will be maintained by mowing should not be constructed steeper

than 3H:1V. Structures and paved surfaces should be located at least 5 feet from the slope face. The slopes should be planted with appropriate vegetation to provide protection against erosion. Surface water runoff should be collected and directed away from slopes steeper than 3H:1V to prevent water from running down the face of the slope.

UTILITY TRENCH EXCAVATION AND BACKFILL

Trench construction and maintenance of safe working conditions, including temporary excavation stability, is the responsibility of the contractor. All applicable local, state, and federal safety codes should be followed. Temporary excavations should either be shored or sloped in accordance with Safety Standards for Excavation, Oregon Administrative Rules (OARs) 1926.650.

We did not encounter groundwater in our test pit explorations however groundwater could be encountered and dewatering of utility trenches could be required during construction. A sump located within the trench excavation likely will be sufficient to remove the accumulated water, depending on the amount and persistence of water seepage and the length of time the trench is left open. Flow rates for dewatering are likely to vary depending on location, and the season during which the excavation occurs. The dewatering systems should be capable of adapting to variable flows.

If groundwater is present in the base of the excavation, we recommend over excavating the trench by 1-foot and placing trench stabilization material in the base. Trench stabilization material should consist of well-graded crushed rock or crushed gravel with a maximum particle size of 4 inches and with less than 5% fines (percent passing the U.S. Standard No. 200 Sieve). The contractor should be responsible for selecting the excavation and dewatering methods, monitoring the trench excavations for safety, and providing shoring as required to protect personnel and adjacent improvements.

Trench backfill in structural areas should consist of well-graded granular material with a maximum particle size of ¾-inch and less than 8 percent by weight passing the U.S. Standard No. 200 Sieve. The material should be free of roots, organic matter, and other unsuitable materials.

Trench backfill in the bedding zone and pipe zone should be placed and compacted in maximum lifts of 6 inches. Trench backfill above the pipe zone should be placed and compacted with a minimum of two lifts. A minimum cover of 3 feet over the top of the pipe should be placed before compacting with a hydraulic plate compactor (hoe-pack).

Trench backfill should be compacted to at least 90 percent of the maximum dry density at depths greater than 4 feet below finished grade and to 95 percent of the maximum dry density within 4 feet of finished grade. Compaction is based on ASTM D1557, the modified proctor test or as recommended by the pipe manufacturer.

STRUCTURAL FILL

The term “structural fill” refers to any material used for building pads, roadway embankments, detention pond berms, foundations, retaining walls, slab-on-grade floors, sidewalks, pavements, and other similar features. The on-site silt is suitable for use as structural fill provided it can be moisture-conditioned, separated from unsuitable material, and compacted to the specified density. The on-site silt should be placed in lifts with a maximum uncompacted thickness of 8 inches and compacted to not less than 92 percent of the maximum dry density as determined by ASTM D 1557.

We recommend using imported granular material for structural fill if the on-site material cannot be moisture conditioned. Imported granular material for structural fill should be pit-run or quarry-run rock, crushed rock, crushed gravel, or sand. It should be fairly well-graded between coarse and fine material and have less than 5 percent by weight passing the U.S. Standard No. 200 Sieve. The material should be placed in lifts with a maximum uncompacted thickness of 12 inches and compacted to not less than 95 percent of the maximum dry density as determined by ASTM D 1557.

Regardless of material or location, structural fill should be placed over firm, unyielding subgrade prepared in accordance with the "Site Preparation" section of this report. The condition of the subgrade should be verified by a NGC representative before filling or construction begins. Fill soil compaction should be verified by in-place density tests performed during fill placement so that adequacy of soil compaction efforts may be evaluated as earthwork progresses.

SHALLOW FOUNDATIONS

In our opinion, the proposed structures can be supported on continuous or isolated column footings founded on existing compacted structural fill, new structural fill, or on undisturbed native soil.

Continuous wall and spread footings should be proportioned for an allowable bearing pressure of 1,500 pounds per square foot (psf). For this allowable bearing pressure, foundations should be at least 12 inches wide or 12 inches in diameter. The base of the foundations should be at least 12 inches below the lowest adjacent final grade.

The recommended allowable bearing pressure applies to the total of dead plus long-term live loads. The allowable bearing pressure may be increased by a factor of 1/3 for short-term wind or seismic loads.

Differential and total settlement of footings is anticipated to be less than ½-inch and 1-inch under static conditions respectively.

Lateral loads of the proposed buildings founded on undisturbed native soil or on structural fill can be resisted by passive earth pressure on the sides of footings and by friction on the base of the footings but not both. We recommend using the Equivalent Fluid Pressures and Coefficients of Friction provided in Table 1.

TABLE 1: LATERAL RESISTANCE FACTORS

SOIL TYPE	EQUIVALENT FLUID PRESSURE (γ_A - PCF)	FRICTION COEFFICIENT (μ_A - %)
ON-SITE SILT/SAND	350	.41
IMPORTED CRUSHED ROCK	820	.61

The tabulated values above are <u>ultimate values</u>. The project structural engineer should apply appropriate factors of safety for static and dynamic conditions. Typical factors of safety values for static conditions are 2 to 3 for equivalent fluid pressure and 1.5 to 2 for friction coefficients. Factors of safety for dynamic conditions are usually 1.1.

In order to develop the tabulated capacities concrete must be poured neatly in excavations, or the adjacent confining structural fill must consist of granular soil compacted to not less than 95% of the dry density as determined by ASTM D 1557. Footing backfill should extend a minimum horizontal distance of two times the footing embedment from base of the footing to bottom of the slab. Adjacent floor slabs, pavements, or the upper 12-inch depth of adjacent, unpaved areas should not be considered when calculating passive resistance.

SLAB-ON GRADE FLOORS

Satisfactory subgrade support for lightly loaded building floor slabs can be obtained on the undisturbed native soil or on engineered structural fill. A 6-inch-thick layer of imported granular material should be placed and compacted over the prepared subgrade to assist as a capillary break. A subgrade modulus of 100 pounds per cubic inch may be used to design the floor slab.

Imported granular material should be crushed rock or crushed gravel and sand that is fairly well-graded between coarse and fine, contain no deleterious materials, have a maximum particle size of 1½ inches, and have less than 5 percent by weight passing the U.S. Standard No. 200 Sieve. The imported granular material should be placed in one lift and compacted until well-keyed, about 85percent of the maximum dry density as determined by ASTM D 1557.

Vapor retarders are often required by flooring manufacturers to protect flooring and flooring adhesives. Many flooring manufacturers will warrant their product only if a vapor retarder is installed according to their recommendations. However, vapor barriers can trap and hold excess moisture when installed in rainy weather. We recommend following ACI 302.1, Chapter 3 with regard to installing a vapor retarder in spaces with floor coverings or coatings.

RETAINING WALLS AND EMBEDDED BUILDING WALLS

The following recommendations assume that the walls consist of conventional, cantilevered retaining walls or embedded building walls, the walls are less than 10 feet in height, the backfill is drained, and the wall backfill consists of free-draining, imported angular crushed quarry rock. Re-evaluation of our recommendations will be required if retaining walls vary from these assumptions.

In general, cantilever retaining walls yield under lateral loads and should be designed with active lateral earth pressures. Restrained walls, such as embedded building walls and vaults should be designed to withstand at-rest lateral earth pressures. We recommend using the lateral earth pressures shown in Table 2. The loads are provided as equivalent fluid density (G). Diagrams showing use of the lateral earth pressures in design calculations are provided in Figure 4.

TABLE 2: EQUIVALENT FLUID DENSITY (G) ACTING ON RETAINING WALLS

WALL TYPE	BACKFILL COMPONENT (PCF)	SURCHARGE COMPONENT (PSF)	SEISMIC COMPONENT (PCF)
YIELDING WALL	20	NA	18
NON-YIELDING WALL	43	NA	12

Retaining wall drains should consist of a perforated drainpipe embedded in a minimum 1-foot-wide zone of drain rock that is wrapped 360 degrees around by a geotextile filter fabric. The fabric should be overlapped a minimum of 6 inches. The drain should outlet an approved outfall.

The drain rock should consist of course sand or gravel containing not more than 3% fines (material by weight passing the U.S. Standard No. 200 Sieve by washed analysis). The geotextile filter should be a non-woven fabric with an AOS between the U.S. Standard No. 70 and No. 100 Sieve size and a water permittivity of greater than 1.5 sec-1.

Backfill for retaining walls should extend a horizontal distance of H, where H is the wall height, and should consist of medium sand, sand and gravel, or well-graded sand or gravel, with not more than 5% fines. Geotextile filter fabric should be placed between the granular materials and the native soil to prevent movement of fines into the clean granular material.

Backfill should be placed and compacted as recommended for structural fill, with the exception of backfill placed immediately adjacent to walls. To reduce pressure on the walls, backfill located within a horizontal distance of 3 feet from the retaining walls should be compacted to approximately 90% of the maximum dry density, as determined by ASTM D 1557. Backfill placed within 3 feet of the wall should be compacted in lifts less than 6 inches thick using hand-operated tamping equipment (such as a jumping jack or vibratory plate compactor).

Static lateral earth pressures acting on a retaining wall should be increased to account for surcharge loadings resulting from any traffic, construction equipment, material stockpiles, or structures located within a horizontal distance equal to the wall height.

BUILDING AND SITE DRAINAGE

We recommend that all roof drains be connected to a tightline leading to storm drain facilities. Pavement surfaces and open space areas should be sloped such that surface water runoff is collected and routed to suitable discharge points. Ground surfaces adjacent to buildings should be sloped to drain away from the buildings.

French drains and or trench should be installed in slopes if groundwater seepage is encountered during construction. The drains should consist of a 4-inch diameter perforated pipe within an envelope of uniformly graded drain rock with a maximum particle size of 3 inches, and less than 2 percent passing the U.S. Standard No. 200 Sieve. The drain rock should extend at least 4 inches on all sides of the pipe.

The gravel envelope should extend upward to the top of the slope and should be wrapped with filter fabric to reduce the migration of fines from the surrounding soil. The geotextile filter should be a non-woven fabric with an AOS between the U.S. Standard No. 70 and No. 100 Sieve size and a water permittivity of greater than 1.5 sec-1. Design details for French /Trench Drains are provided in Figure 5.

ASPHALT PAVEMENT

The pavement subgrade should be prepared in accordance with the previously described recommendations described in the “Site Preparation,” “Wet Weather Construction,” and “Structural Fill” sections of this report.

Our pavement recommendations are based on a subgrade stiffness using a California Bearing Ratio value of 4. We do not have specific information on the frequency and type of vehicles that will use the area; however, we have assumed that post construction traffic conditions will consist of no more than five heavy trucks per day.

Our analysis shows that a pavement section consisting of a minimum of 4 inches of AC pavement underlain by a minimum of 10.0 inches of crushed rock base will be required to support anticipated traffic loads over a design life of 20 years.

These thicknesses are intended to be the minimum acceptable and are based on the assumption that construction will be completed during an extended period of dry weather. Construction of pavement when subgrade soils are wet will require an increased thickness of crushed rock base.

The AC pavement should conform to Section 0074 of the Standard Specification for Highway Construction, Oregon Highway Specifications. We recommend half inch dense graded Hot Mix Asphalt Concrete for Design Level 2 using Performance Grade Asphalt PG-70-22 for the Sherwood area. The aggregate base should conform to Section 02630 of the specifications with the addition that no more than 5 percent of the material by dry weight passes a U.S. Standard No. 200 Sieve.

Aggregate base should be placed in one lift and compacted to not less than 95% of the modified Proctor maximum dry density (ASTM D1557). Aggregate base should be placed in one lift and compacted to not less than 95 percent of the maximum dry density as determined by ASTM D 1557. Aggregate base contaminated with soil during construction should be removed and replaced before paving.

ADDITIONAL SERVICES

Because the future performance and integrity of the structural elements will depend largely on proper site preparation, drainage, fill placement, and construction procedures, monitoring and testing (geotechnical special inspection) by experienced geotechnical personnel should be considered an integral part of the design process. Consequently, we recommend that NGC be retained to provide the following post-investigation services:

- Review construction plans and specifications to verify that our design criteria presented in this report have been properly integrated into the design.
- Attend a pre-construction conference with the design team and contractor to discuss geotechnical related construction issues.
- Observe footing and floor slab subgrade before granular fill material or concrete is placed, in order to verify the soil bearing capacity.
- Prepare a post-construction letter-of-compliance summarizing our field observations, inspections, and test results.

LIMITATIONS

This report was prepared for the exclusive use of D R Horton and members of the design team for this specific project. It should be made available to prospective contractors for information on the

factual data only, and not as a warranty of subsurface conditions such as those interpreted from the explorations and presented in the discussions of the subsurface conditions included in this report.

The recommendations contained in this report are preliminary. They are based on information derived through subsurface sampling. No matter how effective subsurface sampling may be performed, variations between exploration location and the presence of unsuitable materials are possible and cannot be determined until exposed during construction. Accordingly, NGC's recommendations can be finalized only through NGC's observation of the project's earthwork construction. NGC accepts no responsibility or liability for any party's reliance on NGC's preliminary recommendations.

During construction observation NGC will assign persons qualified to recognize unanticipated conditions and observe and report on the contractor's quality of work in order to reduce the risk of problems arising during construction. Note however that construction observation is not insurance, nor does it constitute a warranty or guaranty of any type. NGC's professionals are represented on site solely to observe operations of the contractor identified, to form opinions about the adequacy of those operations, and to report those opinions to our client.

It is our expectation that D R Horton will retain competent contractors who are knowledgeable and experienced in completing the work they are contracted to perform and that contractors will perform irrespective of the presence of our representative or any testing we may conduct. In all cases contractors are assumed to be fully responsible for quality control and quality assurance. Failure by NGC or our client to detect deficiencies in the work or to inform contractor of any deficiencies which may be discovered, shall not relieve any contractor from their responsibility for performance of the end product.

Within the limitations of the scope, schedule and budget, the analyses, conclusions and recommendations presented in this report were prepared in accordance with generally accepted professional geotechnical engineering principles and practice in this area at the time this report was prepared. We make no warranty, either express or implied.

♦ ♦ ♦

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,
Northwest GEO Consultants, LLC

Paul A. Crenna, CEG
Engineering Geologist

Brad L. Hupy, PE, GE
Principal Engineer

Attachments: Figure 1 – Site Location
Figure 2 – Site Layout
Figure 3 – Seismic Site Response
Figure 4 - Retaining Wall Pressures
Figure 5 - French/Trench Drains

A - Field Exploration and Laboratory Testing
Field Exploration Program
Laboratory Testing Program
Key to Boring and Test Pit Logs (4 pages)
Exploration Logs

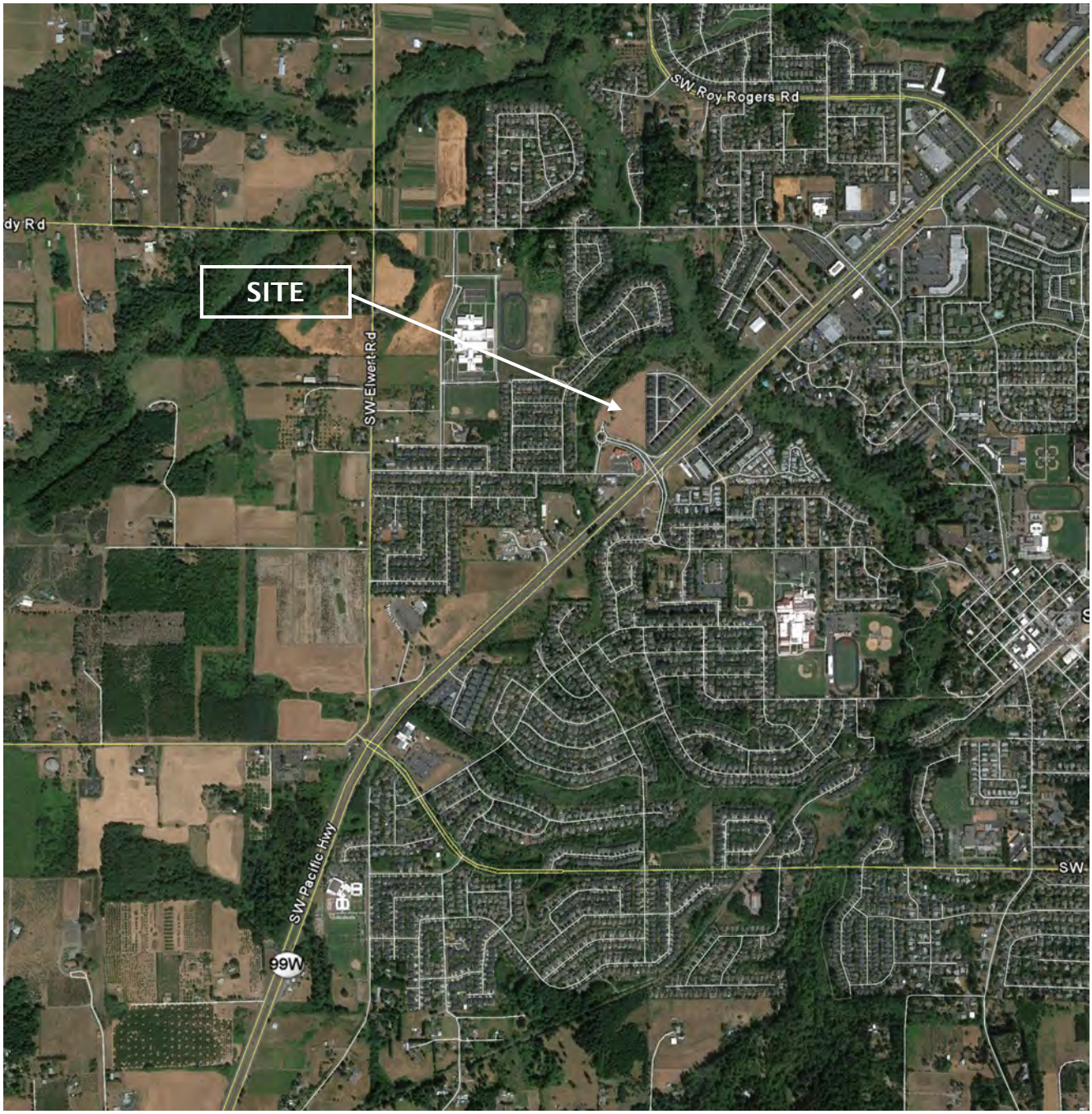


Image August 15, 2012 – Google Earth Pro - Used by permission

45.363N/122.858W - Township 2S, Range 1W, Section 30

N

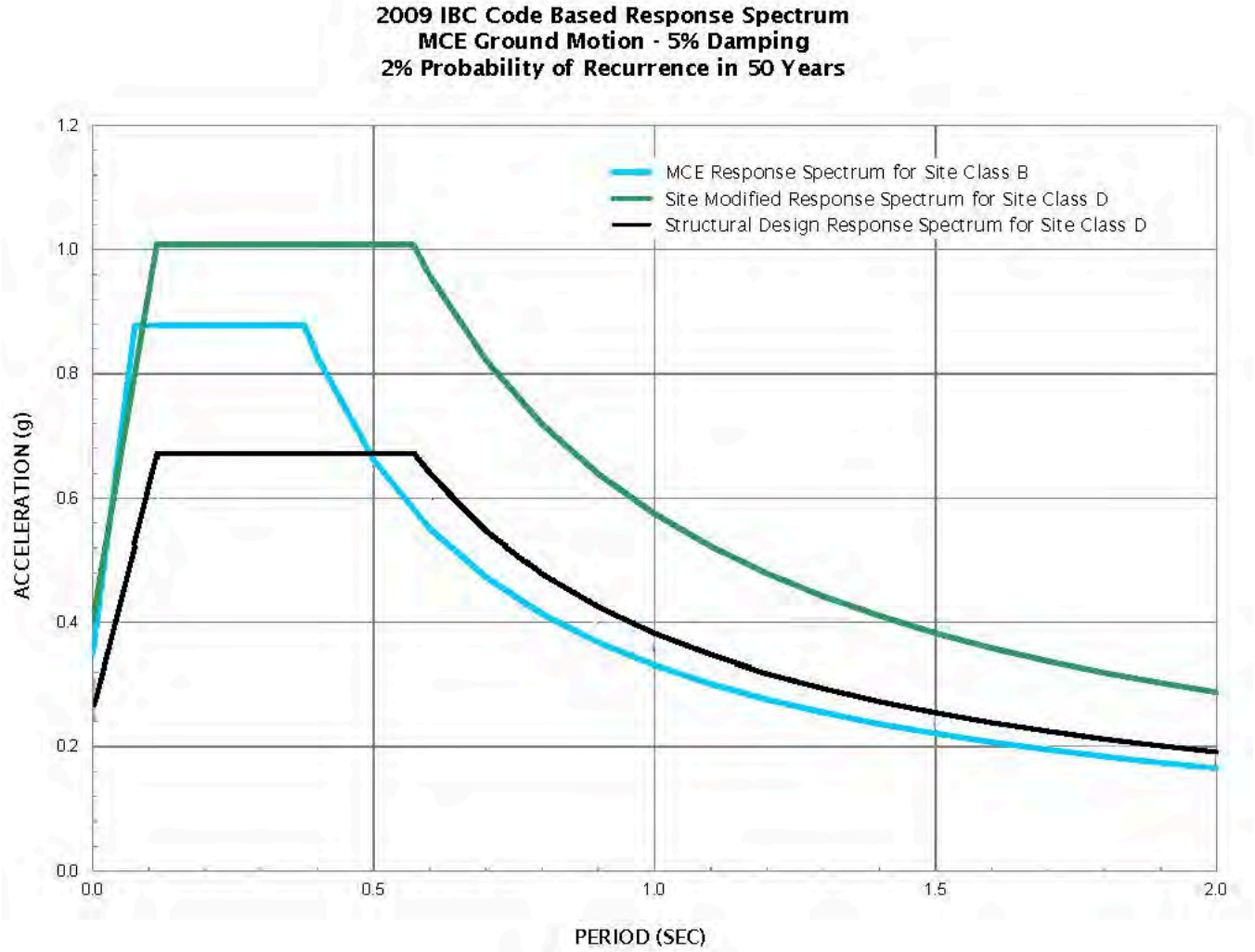
Northwest GEO Consultants, LLC	DEC 2013	**DR HORTON MEINECKE PARK**	
1021 SE 33rd Avenue Portland, OR 97214	Drawn By: TAC	**SITE VICINITY**	**FIGURE 1**

TP-1 ■ APPROXIMATE TEST PIT LOCATIONS EXCAVATED ON 10/18/13

BASE DRAWING FROM "BROWNSTONE PROPERTY - OPTION 2" PREPARED BY EMRIO DESIGN

N

Northwest GEO Consultants, LLC	DEC 2013	**DR HORTON MEINECKE PARK**	
1021 SE 33rd AVE Portland, OR 97214	Drawn By: TAC	**SITE LAYOUT & EXPLORATIONS**	**FIGURE 2**

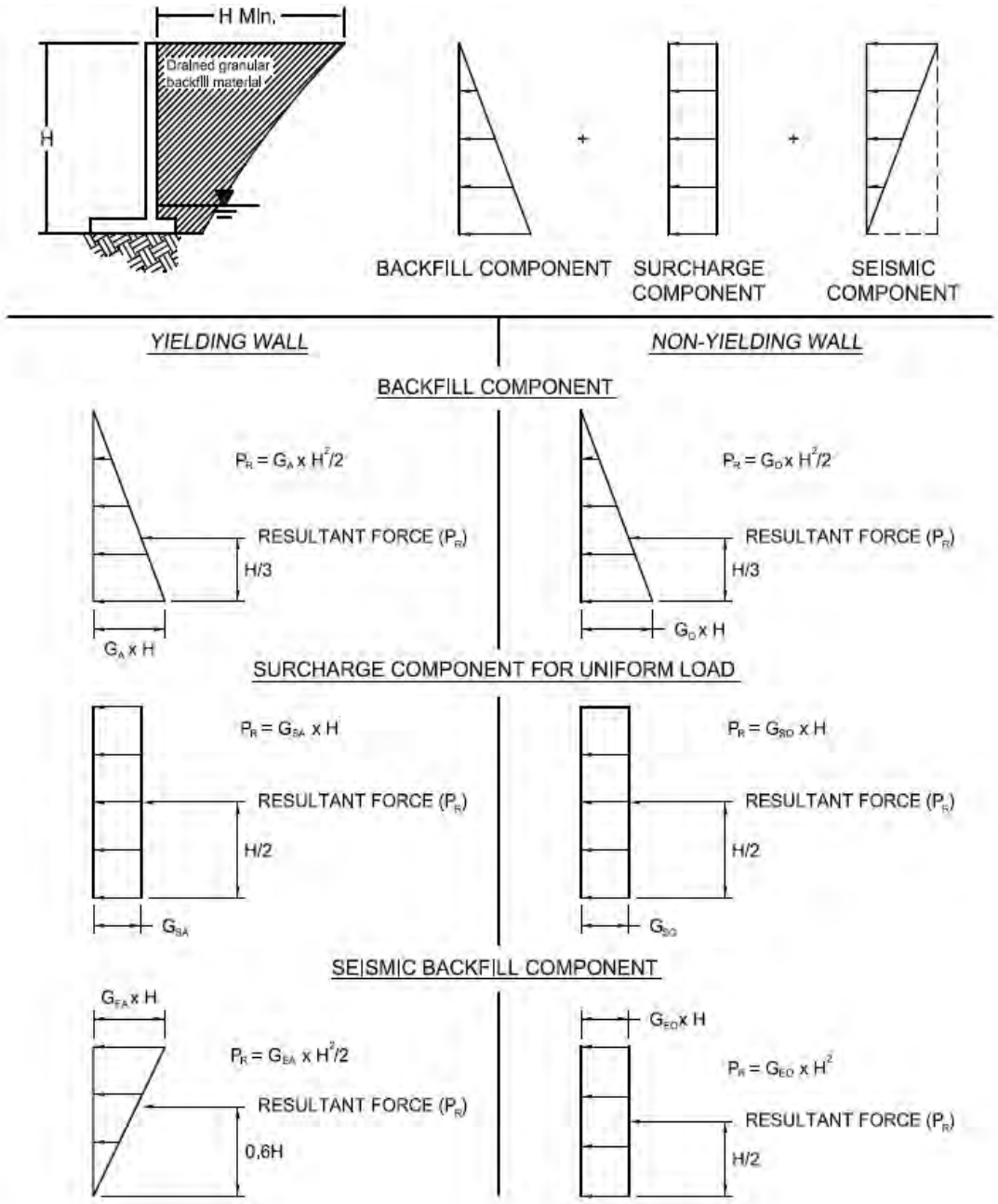


S_s	S1	F_a	F_v	SM_s	SM1	SD_s	SD1
0.878	0.331	1.149	1.738	1.009	0.575	0.673	0.383

Latitude	Longitude	Seismic Site Class	PGA Structural	PGA Geotechnical
45.363 N	122.858 W	D	0.269	0.404

STRUCTURAL PGA FROM NEHRP DESIGN SPECTRUM FOR STRUCTURAL ELEMENTS – GEOTECHNICAL PGA REQUIRES ADDITIONAL MODIFICATION – SEE NCHRP REPORT 611 (2008)

Northwest GEO Consultants, LLC	DEC 2013	DR HORTON MEINECKE PARK	
1021 SE 33rd AVE Portland, OR 97214	Drawn By: BLH	SEISMIC SITE RESPONSE	FIGURE 3

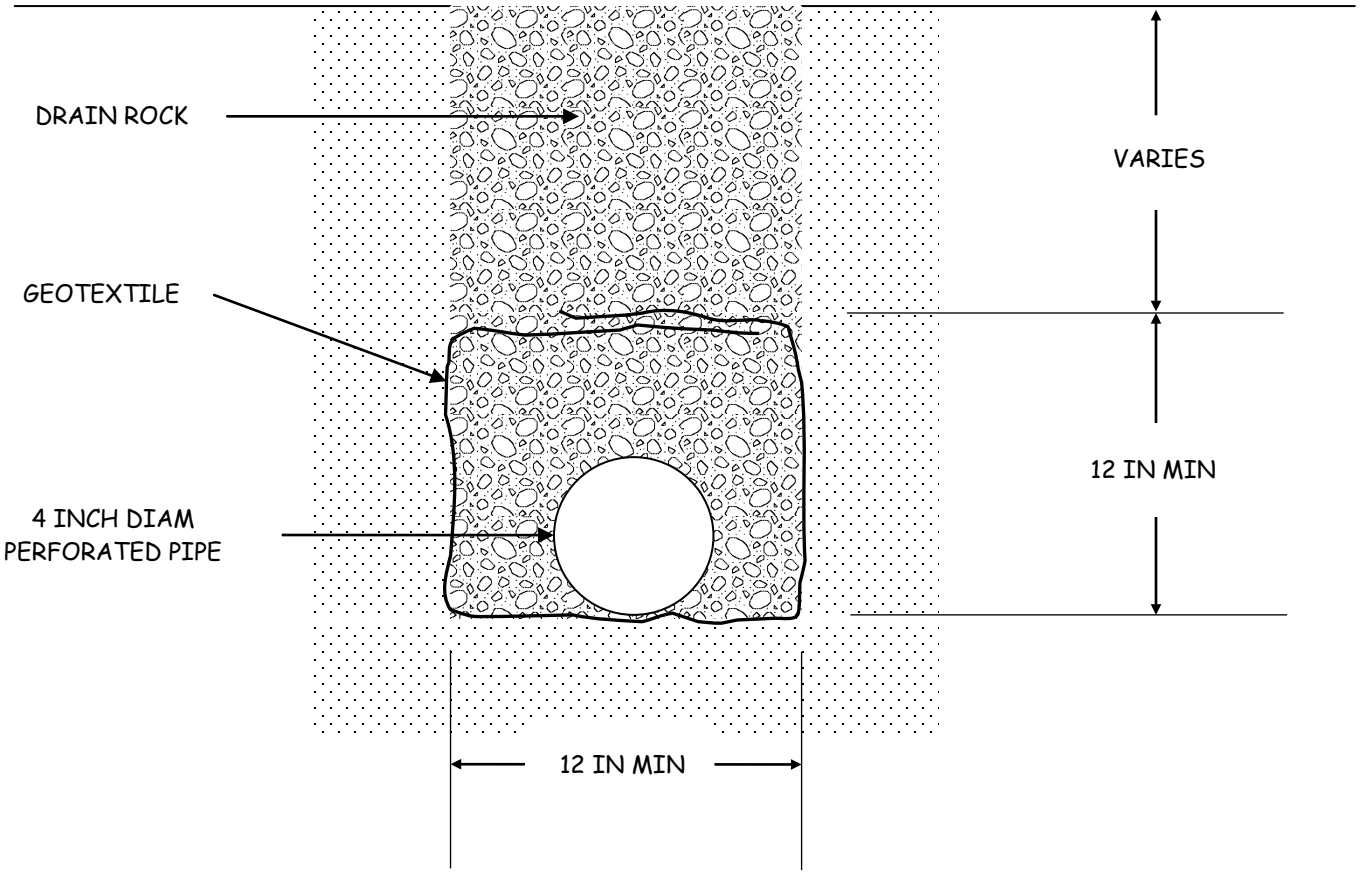


Note:

See report text for values of G.

A: Active Pressure O: At Rest Pressure S: Surcharge Pressure E: Earthquake Pressure

Northwest GEO Consultants, LLC	DEC 2013	DR HORTON MEINECKE PARK	
1021 SE 33rd Avenue Portland, OR 97214	Drawn By: BLH	RETAINING WALL LATERAL EARTH PRESSURES	FIGURE 4



1. Install geotextile filter fabric in trench to wrap drain rock with overlap at the top as shown.
2. Drain rock shall be pea gravel or washed drain rock. Extend to ground surface above geotextile to design subgrade elevation.
3. Install non-woven geotextile filter fabric with an apparent opening size (AOS) equal to the U.S. Standard No. 70 Sieve and a water permittivity of greater than 1.5 sec-1. Fabric meeting ODOT 2008 Standard Specifications - Section 02320 is acceptable.

Northwest GEO Consultants, LLC	DEC 2013	DR HORTON MEINECKE PARK	
1021 SE 33rd Avenue Portland, OR 97214	Drawn By: BLH	FRENCH/TRENCH DRAIN DETAIL	FIGURE 5

ATTACHMENT A

FIELD EXPLORATION PROGRAM

LABORATORY TESTING PROGRAM

KEY TO BORING AND TEST PIT LOGS

TEST PIT LOGS: TP-1 TO TP- 8

FIELD EXPLORATION PROGRAM

General

NGC explored subsurface conditions at the site by observing 8 shallow test pits to depths up to 9 feet below the ground surface (bgs) at the locations shown in Figure 2. The test pits were excavated on October 18, 2013 using a medium size tracked excavator owned and operated by Parker Concrete of Forest Grove, Oregon.

A member of NGC's geotechnical staff was present during the explorations to record soil, rock, and groundwater conditions encountered in our boring and to obtain soil samples for laboratory testing.

Soil Sampling

Representative grab samples of the soil observed in the explorations were obtained from the test pit walls and/or base using the excavator bucket. Relatively undisturbed samples were obtained using a standard 3-inch O.D. Shelby tube in general accordance with ASTM D 1587, the Standard Practice for Thin-walled Tube Sampling of Soils. Samples obtained in the exploration were sealed to retain moisture and returned to our laboratory for additional examination and testing. The test pits were loosely backfilled.

Field Classification

Soil samples were initially classified visually in the field. Consistency, color, relative moisture, degree of plasticity, peculiar odors and other distinguishing characteristics of the soil samples were noted. The terminology used in the soil and rock classifications and other modifiers are defined in the General Notes in this Appendix.

Field Testing

We estimated the undrained shear strength of fine grained soil (silt and clay) using a Pocket Penetrometer (PP) applied to the sidewalls of the test pits. The PP is a hand held device that indicates undrained compressive strength in tons per square foot. The test method is approximate and applicable only to fine grained soil. The results of the tests are presented in the logs included in this appendix.

Exploration Log

Summary exploration t logs follow in this attachment. The left-hand portion of the log provides our interpretation of the soil encountered in the boring, sample depths, and groundwater information. The right-hand, graphic portion of the logs shows the results of laboratory testing.

Soil descriptions and interfaces between soil types shown in the logs are interpretive, and actual changes may be gradual.

LABORATORY TESTING PROGRAM

General

The soil samples obtained during the field explorations were examined in our laboratory. The physical characteristics of the samples were noted and the field classifications were modified where necessary in accordance with terminology presented in the "Key to Boring and Test Pit Logs".

Representative samples were selected during the course of the examination for further testing. The testing program included visual-manual classification, moisture content, and dry unit weight determination. The testing procedures and results of the tests are summarized in the following paragraphs. The phrase "In general accordance with guidelines presented in…" means that certain local and common descriptive practices and methodologies have been followed.

Visual-Manual Classification

The soil samples were classified in general accordance with guidelines presented in ASTM D2488, Standard Practice for Description and Identification of Soils (Visual-Manual Procedure). Certain terminology incorporating current local engineering practice may be used in lieu of ASTM terminology. The term which best described the major portion of the sample was used in determining the soil type. Terminology used is provided in the "Key to Test Pit and Boring Logs" that follows.

Natural Moisture Content

Natural moisture content of selected samples was determined in general accordance with guidelines presented in ASTM D 2216. The natural moisture content is the ratio, expressed as a percentage, of the weight of water in a given amount of soil to the weight of solid particles. The results of these tests are shown on the exploration logs.

In-Situ Dry Density (Dry Unit Weight)

We tested selected soil samples to determine the in-situ dry density (dry unit weight). The tests were performed in general accordance with guidelines presented in ASTM D 2937. The dry density is defined as the ratio of the dry weight of the soil sample to the volume of that sample. The dry density typically is expressed in pounds per cubic foot. The dry densities are presented in the logs included in this appendix.

BORING AND TEST PIT LOGS

DISTINCTION BETWEEN FIELD LOGS AND FINAL LOGS

A field log is prepared for exploration by our field representative. The log contains information concerning soil and groundwater encountered, sampling depths, sampler types used and identification of samples selected for laboratory analysis. The final logs presented in this report represent our interpretation of subsurface conditions based on the contents of the field logs, observations made during explorations, and the results of laboratory testing. Our recommendations are based on the contents of the final logs and the information contained therein and not on the field logs.

SOIL CLASSIFICATION SYSTEM

Soil samples are classified in the field in general accordance with the Unified Soil Classification (USCS) presented in ASTM D 2488 "Standard Practice for Description and Identification of Soils (Visual-Manual Procedure)." Final logs reflect field soil classifications and laboratory testing results. A summary of the USCS is provided on page 3. Classifications and sampling intervals are shown in the logs.

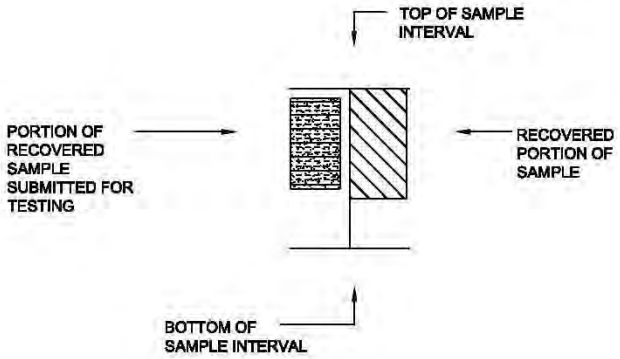
VARIATION OF SOIL BETWEEN EXPLORATIONS

The final logs and related information depict subsurface conditions only at the specific location and on the date(s) indicated. Those using the information contained herein should be aware that soil conditions at other locations or on other dates may differ.

TRANSITION BETWEEN SOIL OR ROCK CLASSIFICATIONS

The lines designating the interface between soil, fill, or rock on the final logs and on the subsurface profiles presented in the report are determined by interpolation and are therefore approximate. The transition between the materials may be abrupt or gradual. Only at specific exploration locations should profiles be considered as reasonably accurate and then only to the degree implied by the notes.

BORING LOG SAMPLES



Northwest GEO Consultants, LLC	2013	KEY TO BORING AND TEST PIT LOGS	
1411 SE 130th Avenue-Suite 6 Portland Oregon 97214	Drawn By: BLH	GENERAL INFORMATION	1/4

EXPLORATION LOG SYMBOLS

- Sample Location with No Sample Recovery
- Sample Location Using Direct Push Sampler (ASTM D 6282)
- Sample Location Using Ring-Lined Barrel Sampler (ASTM D 3550)
- Sample Location Using Split-Barrel Sampler (ASTM D 1586)
- Sample Location Using Thin-Walled Tube Sampler (ASTM D 1587)
- Rock Core Interval
- Grab Sample Location
- Soil Sample Submitted for Chemical Testing
- Soil Sample Submitted for Physical Property Testing
- Water Sample Screened Interval
- Water Sample Submitted for Chemical Testing
- Water Sample Tested in the Field
- Groundwater Level Encountered While Drilling
- Static Groundwater Level
- Perched Groundwater
- Groundwater Level at Time of Sampling

SOIL CHARACTER

Granular Soil		Cohesive Soil		
Density	Standard Penetration Test *	Consistency	Standard Penetration Test*	Unconfined Compressive Strength (tsf)
Very Loose	0 - 4	Very Soft	Less Than 2	Less Than 0.25
Loose	4 - 10	Soft	2 - 4	0.25 - 0.5
Medium Dense	10 - 30	Medium Stiff	4 - 8	0.50 - 1.0
Dense	30 - 50	Stiff	8 - 16	1.0 - 2.0
Very Dense	Greater Than 50	Very Stiff	16 - 32	2.0 - 4.0
Blows Required to Drive a Split-Barrel Sampler 12 inches		Hard	Greater Than 32	Greater Than 4.0

DEFINITINIONS AND ABBREVIATIONS

AT	ATTERBERG LIMITS TEST	ND	NON DETECT	PPB	PARTS PER BILLION
BGS	BELOW GROUND SURFACE	NEG	NEGATIVE RESULT	PPM	PARTS PER MILLION
CO	CONSOLIDATION TEST	NS	NO VISIBLE SHEEN	PSF	POUNDS PER SQUARE FOOT
DS	DIRECT SHEAR TEST	OC	ORGANIC CONTENT	RS	SOIL RESISTIVITY TEST
DW	DRY UNIT WEIGHT	P	PUSHED SAMPLE	S4	SUDAN IV SOIL TEST
GS	MECHANICAL GRAIN SIZE TEST	P200	P200 FINES CONTENT TEST	SG	SPECIFIC GRAVITY TEST
HS	HEAVY SHEEN	PCF	POUNDS PER CUBIC FOOT	SPT	STD. PENETRATION TEST
HYD	HYDROMETER TEST	PH	SOIL pH	SS	SLIGHT SHEEN
MC	MOISTURE CONTENT	PID	PHOTOIONIZATION DETECTOR	TO	TOREVANE
MG/KG	MILLIGRAMS PER KILOGRAM	POS	POSITIVE RESULT	TSF	TONS PER SQUARE FOOT
MS	MODERATE SHEEN	PP	POCKET PENETROMETER	UV	ULTRAVIOLET LIGHT TEST

GRAIN SIZE DEFINITIONS			MINOR FRACTIONS IN FINE GRAINED SOIL		GROUNDWATER SEEPAGE	
SAND	FINE	No. 200 to No. 40	No Mention (CLAY, SILT)	< 15 percent	Slow	< 1 gpm
	MEDIUM	No. 40 to No. 10	With Sand, With Gravel	15 to 30 percent	Moderate	1-3 gpm
	COARSE	No. 10 to No. 4	Sandy, Gravelly	30 to 49 percent	Rapid	> 3 gpm
GRAVEL	FINE	No. 4 to 3/4-inch	FIELD MOISTURE OBSERVATION		CAVING	
	COARSE	3/4- to 3-inch	Dry	Absence of moisture, dusty, dry to touch	Minor	
COBBLE		3-inches to 12-inches	Moist	Damp but no visible water.	Moderate	
BOULDER		> 12-inches	Wet	Saturated, below groundwater	Severe	

Northwest GEO Consultants,	2013	KEY TO BORING AND TEST PIT LOGS	
1411 SE 130th Avenue-Suite 6 Portland Oregon 97214	Drawn By: BLH	SYMBOLS	2/4

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

MAJOR DIVISIONS			SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	CLEAN GRAVELS		GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
		(LITTLE OR NO FINES)		GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
	MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	GRAVELS WITH FINES		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
		(APPRECIABLE AMOUNT OF FINES)		GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	SAND AND SANDY SOILS	CLEAN SANDS		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
		(LITTLE OR NO FINES)		SP	POORLY-GRADED SANDS, GRAVELLY SAND, LITTLE OR NO FINES
	MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	SANDS WITH FINES		SM	SILTY SANDS, SAND - SILT MIXTURES
		(APPRECIABLE AMOUNT OF FINES)		SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY
				OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

Northwest GEO Consultants, LLC	2013	KEY TO BORING AND TEST PIT LOGS	
1411 SE 130th Avenue-Suite 6 Portland Oregon 97214	Drawn By: BLH	SOIL CLASSIFICATION CHART	3/4

ROCK CLASSIFICATION GUIDELINES

HARDNESS		DESCRIPTION
Very soft	(RH-0)	For plastic material only
Soft	(RH-1)	Carved or gouged with a knife
Moderate	(RH-2)	Scratched with a knife
Hard	(RH-3)	Difficult to scratch with a knife
Very hard	(RH-4)	Rock scratches metal; rock cannot be scratched with a knife

STRENGTH	DESCRIPTION
Plastic	Easily deformable with finger pressure
Friable	Crumbles by rubbing with fingers
Weak	Crumbles only under light hammer blows
Moderately Strong	Few heavy hammer blows before breaking
Strong	Withstands few heavy hammer blows and yields large fragments
Very Strong	Withstands many heavy hammer blows, yields dust and small fragments

WEATHERING	DESCRIPTION
Severe	Rock decomposed; thorough discoloration; all fractures extensively coated with clay, oxides, or carbonates.
Moderate	Intense localized discoloration of rock; fracture surfaces coated with weathering minerals.
Little	Slight and intermittent discoloration of rock; few stains on fracture surfaces.
Fresh	Rock unaffected by weathering

FRACTURING	FRACTURE SPACING
Crushed	Less than 5/8 inch to contains clay
Highly Fractured	5/8 inch to 2 inches
Closely Fractured	2 inches to 6 inches
Moderately fractured	6 inches to 1 foot
Little Fractured	1 foot to 4 feet
Massive	Greater than 4 feet

JOINT SPACING	DESCRIPTION
Papery	Less than 1/8 inch
Shaley or Platey	1/8 inch to 5/8 inch
Very Close	5/8 inch to 3 inches
Close	3 inches to 2 feet
Blocky	2 to 4 feet
Massive	Greater than 4 feet

DEPTH (feet bgs)	GRAPHIC LOG	USCS SYMBOL	SOIL DESCRIPTION	SAMPLE	GROUND WATER	WATER CONTENT (percent)	FIELD TESTING	TESTING AND LABORATORY DATA
		ML	Soft, brown **SILT** with trace organics (roots to 1/8 inch diameter, 10%, carbonized wood fragments), abundant air voids, (3- to 4-inch thick heavily rooted zone at the ground surface); moist.	1		26	PP=0.5	
			TILL ZONE				PP=2.0	
		ML	Stiff to very stiff, brown to light brown **SILT** with trace fine sand; moist.	2		29	PP=2.5	
							PP=3.5	
5			**WILLAMETTE SILT**	3		30		
				4		30		
			End at 9 feet in stiff native silt.					
10			No caving and no groundwater observed to the depth explored.					

Station: See Figure 2 **Logged By: Paul Crenna, CEG**

Approximate Elevation:

Excavation Started: 10/18/13 **Excavation Completed: 10/18/13**

LOG OF TEST PIT TP-1

DEPTH (feet bgs)	GRAPHIC LOG	USCS SYMBOL	SOIL DESCRIPTION	SAMPLE	GROUND WATER	WATER CONTENT (percent)	FIELD TESTING	TESTING AND LABORATORY DATA
		ML	Soft, brown **SILT** with trace fine organics, abundant air voids, (3- to 4-inch thick heavily rooted zone at the ground surface); moist.	1		26	PP=0.25	
			TILL ZONE				PP=2.5	
		ML	Stiff to very stiff, brown to light brown **SILT** with trace fine sand; moist.	2		27	PP=2.0	
							PP=3.5	
5			**WILLAMETTE SILT**					
			End at 8 feet in stiff native silt.	3		28		
10			No caving and no groundwater observed to the depth explored					

Station: See Figure 2 **Logged By: Paul Crenna, CEG**

Approximate Elevation:

Excavation Started: 10/18/13 **Excavation Completed: 10/18/13**

LOG OF TEST PIT TP-2

D R Horton

Meinecke Park

Northwest GEO Consultants
1021 SE 33rd Avenue
Portland, Oregon 97214
Tel 503-702 8437
Fax

NGC, LLC

LOGS OF TEST PITS

DEPTH (feet bgs)	GRAPHIC LOG	USCS SYMBOL	SOIL DESCRIPTION	SAMPLE	GROUND WATER	WATER CONTENT (percent)	FIELD TESTING	TESTING AND LABORATORY DATA
		ML	Soft, brown **SILT** with trace fine organics, abundant air voids, (3- to 4-inch thick heavily rooted zone at the ground surface); moist. **TILL ZONE**	1		29	PP=0.25	
		ML	Stiff to very stiff, brown to light brown **SILT** with trace fine sand; moist. **WILLAMETTE SILT**	2		24	PP=2.0 PP=2.5 PP=3.0	
5			End at 5 feet in very stiff silt.					
10			No caving and no groundwater observed to the depth explored					

Station: See Figure 2 **Logged By: Paul Crenna, CEG**

Approximate Elevation:

Excavation Started: 10/18/13 **Excavation Completed: 10/18/13**

LOG OF TEST PIT TP-3

DEPTH (feet bgs)	GRAPHIC LOG	USCS SYMBOL	SOIL DESCRIPTION	SAMPLE	GROUND WATER	WATER CONTENT (percent)	FIELD TESTING	TESTING AND LABORATORY DATA
		ML	Soft, brown **SILT** with trace fine organics, abundant air voids, (3- to 4-inch thick heavily rooted zone at the ground surface); moist. **TILL ZONE**				PP=0.5	
		ML	Stiff to very stiff, brown to light brown **SILT** with trace fine sand; moist. **WILLAMETTE SILT**	1 2		29	PP=2.0 PP=2.5 PP=3.5	DD = 94.5 pcf
5			End at 6 feet in very stiff silt.					
10			No caving and no groundwater observed to the depth explored					

Station: See Figure 2 **Logged By: Paul Crenna, CEG**

Approximate Elevation:

Excavation Started: 10/18/13 **Excavation Completed: 10/18/13**

LOG OF TEST PIT TP-4

DEPTH (feet bgs)	GRAPHIC LOG	USCS SYMBOL	SOIL DESCRIPTION	SAMPLE	GROUND WATER	WATER CONTENT (percent)	FIELD TESTING	TESTING AND LABORATORY DATA
		ML	Soft, brown **SILT** with trace fine organics, abundant air voids, (3- to 4-inch thick heavily rooted zone at the ground surface); moist.	1		25	PP=0.25	
			TILL ZONE				PP=2.0	
		ML	Stiff to very stiff, brown to light brown **SILT** with trace fine sand; moist.	2		25	PP=2.5	
							PP=3.0	
5			**WILLAMETTE SILT**	3		30		
			End at 7 feet in stiff native silt.					
10			No caving and no groundwater observed to the depth explored					

Station: See Figure 2 **Logged By: Paul Crenna, CEG**

Approximate Elevation:

Excavation Started: 10/18/13 **Excavation Completed: 10/18/13**

LOG OF TEST PIT TP-5

DEPTH (feet bgs)	GRAPHIC LOG	USCS SYMBOL	SOIL DESCRIPTION	SAMPLE	GROUND WATER	WATER CONTENT (percent)	FIELD TESTING	TESTING AND LABORATORY DATA
		ML	Soft, brown **SILT** with trace fine organics, abundant air voids, (3- to 4-inch thick heavily rooted zone at the ground surface); moist.	1		27	PP=0.5	
			TILL ZONE				PP=2.0	
		ML	Stiff to very stiff, brown to light brown **SILT** with trace fine sand; moist.	2		29	PP=2.5	
							PP=3.0	
5			**WILLAMETTE SILT**	3		31		DD = 90.5 pcf
			End at 8 feet in very stiff silt.					
10			No caving and no groundwater observed to the depth explored					

Station: See Figure 2 **Logged By: Paul Crenna, CEG**

Approximate Elevation:

Excavation Started: 10/18/13 **Excavation Completed: 10/18/13**

LOG OF TEST PIT TP-6

D R Horton

Meinecke Park

Northwest GEO Consultants
1021 SE 33rd Avenue
Portland, Oregon 97214
Tel 503-702 8437
Fax

NGC, LLC

LOGS OF TEST PITS

DEPTH (feet bgs)	GRAPHIC LOG	USCS SYMBOL	SOIL DESCRIPTION	SAMPLE	GROUND WATER	WATER CONTENT (percent)	FIELD TESTING	TESTING AND LABORATORY DATA
		ML	Soft, brown **SILT** with trace fine organics, abundant air voids, (3- to 4-inch thick heavily rooted zone at the ground surface); moist. **TILL ZONE**	1		25	PP=0.5	
		ML	Stiff to very stiff, brown to light brown **SILT** with trace fine sand; moist. **WILLAMETTE SILT**	2		29	PP=1.5 PP=2.5 PP=3.5	
5				3		27		
			End at 6 feet in stiff native silt.					
10			No caving and no groundwater observed to the depth explored					

Station: See Figure 2 **Logged By: Paul Crenna, CEG**

Approximate Elevation:

Excavation Started: 10/18/13 **Excavation Completed: 10/18/13**

LOG OF TEST PIT TP-7

DEPTH (feet bgs)	GRAPHIC LOG	USCS SYMBOL	SOIL DESCRIPTION	SAMPLE	GROUND WATER	WATER CONTENT (percent)	FIELD TESTING	TESTING AND LABORATORY DATA
		ML	Soft, brown **SILT** with trace fine organics, abundant air voids, (3- to 4-inch thick heavily rooted zone at the ground surface); moist. **TILL ZONE**	1		26	PP=0.5	
		ML	Stiff to very stiff, brown to light brown **SILT** with trace fine sand; moist. **WILLAMETTE SILT**	2		27	PP=1.5 PP=2.5 PP=3.5	
5			End at 5 feet in very stiff native silt.					
10			No caving and no groundwater observed to the depth explored					

Station: See Figure 2 **Logged By: Paul Crenna, CEG**

Approximate Elevation:

Excavation Started: 10/18/13 **Excavation Completed: 10/18/13**

LOG OF TEST PIT TP-8