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Report of
Geotechnical Investigation
Alta Mira Circle Residence
5424 SW Alta Mira Circle
Portland, Oregon

CGT Project Number G1504162

Prepared for

Arcon Group, Inc.
Attn: Mr. Chris Thelen
PO Box 42292
Portland, Oregon 97242

April 30, 2015

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Dear Mr. Thelen:

Carlson Geotechnical (CGT), a division of Carlson Testing, Inc. (CTI), is pleased to submit this report summarizing our geotechnical investigation for the planned Alta Mira Circle Residence project. The site is located at 5424 SW Alta Mira Circle in Portland, Oregon. We performed our work in general accordance with CGT Proposal GP6643, dated April 10, 2015. Written authorization for our services was provided on April 10, 2015.

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

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Jan. 10, 2007
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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 Alta Mira Circle Residence project. The site is located at 5424 SW Alta Mira Circle in Portland, Oregon, as shown on the attached Site Location, Figure 1.

1.1 Project Description

CGT developed an understanding of the proposed project based on our correspondence and review of the provided architectural plans, prepared by Arcon Group, Inc, and dated January 15, 2015. Based on our review, we understand the project will include the following:

- Construction of a new, two-story, wood-framed, single family residential structure. The structure will be located within the northern portion of the site and include an attached garage. The lower floor of the structure will consist of a slab-on-grade, daylight basement, with concrete retaining walls anticipated to be up to about 9 feet in height. Although no structural information has been provided, we have assumed maximum column, continuous wall, and uniform floor slab loads will be less than 30 kips, 2 kips per lineal foot, and 150 pounds per square foot (psf).
- Construction of a new driveway to provide vehicular access to the residential structure from Alta Mira Circle.
- Installation of appurtenant underground utilities to serve the new residence.
- Permanent grade changes at the site are anticipated to include cuts up to about 10 feet in depth in order to achieve desired basement elevations. No significant fills are proposed at the site.
- We anticipate stormwater collected from new impervious areas of the site will be collected and routed to the nearest storm drain or other suitable discharge point.

1.2 Scope of Work

The purpose of our work was to explore subsurface conditions at the site in order to provide geotechnical engineering recommendations for design and construction of the proposed development. Our scope of work included the following:

- Explore subsurface conditions at the site by observing the excavation of three test pits to depths of about 10 feet below ground surface (bgs).
- Classify the materials encountered in the explorations in general accordance with American Society for Testing and Materials (ASTM) D2488 (Visual-Manual Procedure).
- Collect representative, disturbed samples of the soils encountered within the explorations in order to perform laboratory testing and to confirm our field classifications.
- Provide a site vicinity map and a site plan showing the locations of our test pits relative to existing site features.
- Provide logs of the test pits, including results of in-situ and laboratory testing on collected samples.
- Provide geotechnical recommendations for site preparation and earthwork.
- Provide geotechnical engineering recommendations for design and construction of shallow spread foundations, floor slabs, and retaining walls.

- Provide this written geotechnical report to summarize the results of the geotechnical investigation and recommendations for the project.

2.0 SITE DESCRIPTION

2.1 Site Geology

Available geologic mapping[1] indicates the project area is underlain by Pliocene to Holocene undifferentiated sediments. The sediments are typically fine-grained, massive to finely bedded, and mantle the bedrock in the area. The sediments range in thickness from approximately 15 feet to over 200 feet thick. Surficial sediments in the area of the site typically consist of wind-blown silt (loess) and fine-grained Missoula Flood deposits.

2.2 Site Surface Conditions

The approximate 6,000 square foot lot is bordered by SW Alta Mira Circle to the north, developed residential lots to the east and west, and a paved pathway to the south. At the time of our field investigation, the area of the proposed residential structure was covered with grass and contained trees along the west perimeter of site. The site descended to the south at gradients visually estimated up to about 5H:1V (Horizontal: Vertical). The south portion of the site contained two, relatively short (up to two feet high), segmental rock retaining walls. Site layout, topography, and surface conditions are shown on the attached Site Plan and Site Photographs, Figures 2 and 3, respectively.

3.0 FIELD INVESTIGATION

CGT observed the excavation of three test pits (TP-1 through TP-3) at the site on April 17, 2015. The test pits were excavated to depths of about 10 feet bgs using a Hitachi Zaxis 70, track-mounted excavator with a 12-inch wide toothed bucket provided and operated by our client's representative. The test pits were located in the field using approximate measurements from existing site features shown on Figure 2. Upon completion, the test pits were loosely backfilled with the excavated soils by the client's representative.

Pocket penetrometer readings were taken within the upper 4 feet of the test pits. The pocket penetrometer is a hand-held instrument that provides an approximation of the unconfined compressive strength of cohesive, fine-grained soils. The correlation between pocket penetrometer readings and the consistency of cohesive, fine-grained soils is provided on the attached Figure 4.

A qualified member of CGT's staff logged the soils observed within the test pits in general accordance with the Unified Soil Classification System (USCS) and collected representative samples of the materials encountered. An explanation of the USCS is presented on the attached Soil Classification Criteria and Terminology, Figure 4. Samples collected from the test pits were stored in sealable plastic bags and transported to our laboratory for further examination and testing. Our staff visually examined all samples returned to our laboratory in order to refine the field classifications.

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[1] Beeson, M.H., and others, 1989. Geologic Map of the Lake Oswego Quadrangle, Clackamas, Multnomah, and Washington Counties, Oregon. Oregon Department of Geology and Mineral Industries. GMS-59.

4.0 LABORATORY TESTING

Laboratory testing was performed on samples collected from our test pits in order to refine our initial field classifications and estimate in-situ properties. Laboratory testing included six moisture content determinations (ASTM D2216), one plasticity index test (ASTM D4318), and one percent passing the U.S. Standard No. 200 Sieve test (ASTM D1140). Results of the laboratory tests are shown on the respective test pit logs.

5.0 SUBSURFACE CONDITIONS

Logs of the test pits are attached as Figures 5 through 7. Surface elevations indicated on the logs were determined based on the topographic contour utility provided by Metro[2]. Elevation contours identified from the referenced utility were compared and appeared consistent with our site observations. Elevations shown on the logs should be considered approximate.

5.1 Soils

Silt topsoil (ML) was encountered within the upper 1 to 1½ feet of all three test pits. This layer was generally stiff to very stiff, brown, and contained organics (roots up to ¾-inch in diameter). Below the silt topsoil, we encountered silty sand to sandy silt (SM/ML) in all three test pits. This soil extended to the full depths explored in the test pits, about 10 feet bgs. This soil was generally brown with occasional orange and gray mottling, moist, and medium dense/stiff to better in terms of relative density/consistency. The sand content was very fine grained and generally decreased with depth. The silt content exhibited low to medium plasticity.

5.2 Groundwater

We did not encounter groundwater within the depths explored at the site on April 17, 2015. To determine approximate regional groundwater levels in the area, we researched well logs available at the Oregon Water Resources Department (OWRD)[3] website for wells located within Township 1 South, Range 1 East, Section 16. Our review indicated that groundwater levels in the area were highly variable and ranged from depths of about 15 feet to in excess of 400 feet bgs in the vicinity of the site. 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 project site. We anticipate that groundwater levels will fluctuate due to seasonal and annual variations in precipitation, changes in site utilization, or other factors. Additionally, the on-site native silty sand to sandy silt (SM/ML) is conducive to formation of perched groundwater.

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2 http://gis.oregonmetro.gov/metromap/
3 Oregon Water Resources Department, 2015. Water well logs obtained from OWRD website http://www.wrd.state.or.us/

6.0 SEISMIC CONSIDERATIONS

6.1 Seismic Design

The 2014 Oregon Residential Specialty Code (2014 ORSC) requires the determination of seismic site class be based on subsurface data in accordance with Chapter 20 of the American Society of Civil Engineers Minimum Design Loads for Buildings and Other Structures (ASCE 7). Based on the results of our subsurface explorations and review of the geologic setting, we have assigned a Site Class D for the subsurface conditions encountered. Seismic ground motion values were determined in accordance with Section R301.2.2 of the 2014 ORSC. The Seismic Design Category was determined from Table R301.2.2.1.1 of the 2014 ORSC. Earthquake ground motion parameters for the site were obtained from the United States Geological Survey (USGS) Seismic Design Values for Buildings - Ground Motion Parameter Calculator[4]. The site Latitude 45.483457° North and Longitude 122.693657° 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 (2014 ORSC)

Parameter		Value
Mapped Acceleration Parameters	Spectral Acceleration, 0.2 second (S_s)	0.995g
	Spectral Acceleration, 1.0 second (S_1)	0.428g
Coefficients (Site Class D)	Site Coefficient, 0.2 sec. (F_A)	1.102
	Site Coefficient, 1.0 sec. (F_V)	1.572
Adjusted MCE Spectral Response Parameters	MCE Spectral Acceleration, 0.2 sec. (S_{MS})	1.097g
	MCE Spectral Acceleration, 1.0 sec. (S_{M1})	0.673g
Design Spectral Response Accelerations	Design Spectral Acceleration, 0.2 seconds (S_{DS})	0.731g
	Design Spectral Acceleration, 1.0 second (S_{D1})	0.449g
Seismic Design Category		D_1

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 approaches zero, and the soil 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.

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

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[4] United States Geological Survey, 2015. Seismic Design Parameters determined using:, "U.S. Seismic Design Maps Web Application - Version 3.1.0," from the USGS website http://earthquake.usgs.gov. 2012 IBC design code reference used as indicated by the 2014 ORSC.

constantly evolving. Current practice[5] 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. 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).

Based the lack of static groundwater, the soils encountered within the depths explored at the site are considered non-liquefiable.

6.2.2 Slope Instability

Review of the Statewide Landslide Information Database for Oregon (SLIDO), available at the DOGAMI website[6], shows no historic or prehistoric landslides mapped at the site. No obvious signs of recent or on-going instability were observed at the site during our investigation, and site topography is generally gently sloping. Recognizing the above, it is our opinion that the risk of seismically-induced slope instability occurring on the site is low.

6.2.3 Surface Rupture

6.2.3.1 Faulting

Although the site is situated in a region of the country with known active faults and historic seismic activity, no known faults exist on or immediately adjacent to the site. The Oatfield fault is located approximately ½-mile to the southwest of the site. The Oatfield fault consists of a 29-kilometer-long steeply dipping reverse fault that forms escarpments in Miocene Columbia River Basalt in the Tualatin Mountains[7]. No fault scarps or displacement of surficial deposits have been described, but exposures within tunnels show offset of Boring Lava, indicating Quaternary activity. The slip rate for the Oatfield fault has been calculated to be about 0.1 mm per year based on the tunnel exposures. Given the very low slip rate and lack of displacement of surficial deposits, this fault is considered to have a very long recurrence interval. Therefore, the risk of surface rupture at the site due to faulting is considered low.

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 lack of liquefiable soils at the site, the risk of surface rupture due to lateral spread is considered negligible.

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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 http://www.oregongeology.org/slido/index.html

7 Personius, S.F., compiler, 2002. Fault number 875, Oatfield fault, in Quaternary fault and fold database of the United States: U.S. Geological Survey website, http://earthquakes.usgs.gov/hazards/qfaults.

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. Satisfactory subgrade support for planned shallow foundations, floor slabs, and retaining walls can be achieved on the native, medium dense/stiff to hard, inorganic silty sand to sandy silt (SM/ML), or structural fill that is properly placed and compacted on this material during construction. This soil was encountered at depths of about 1 to 1½ feet bgs within our explorations.

The near-surface native soil is sensitive to small changes in moisture content and can pose challenges for earthwork performed during wet weather. Considerations for wet weather construction are discussed in Section 8.3 of this report.

8.0 RECOMMENDATIONS

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. CGT should be consulted for further recommendations if the design and/or location of the proposed development changes, or variations and/or undesirable geotechnical conditions are encountered during site development.

8.1 Site Preparation

8.1.1 Site Stripping

Existing vegetation and silt topsoil (ML) should be removed from within, and for a 5-foot-margin around, proposed structural fill areas, building pad area, and driveway locations. Based on the results of our field explorations, stripping of topsoil is anticipated to extend to depths of about 1 to 1½ feet bgs. This material may be deeper or shallower at locations away from our explorations. The geotechnical engineer or his representative should provide recommendations for actual stripping depths based on observations during site stripping. Stripped vegetation and topsoil should be transported off-site for disposal, or stockpiled for later use in landscaped areas.

8.1.2 Grubbing

Grubbing of shrubs should include the removal of the root mass and roots greater than 1-inch in diameter. Grubbed materials should be transported off-site for disposal. Where root masses are removed, the resulting excavation should be properly backfilled with imported granular structural fill in conformance with Section 8.4.2 of this report.

8.1.3 Existing Utilities & Below-Grade Structures

All existing utilities at the site should be identified prior to excavation. Abandoned utility lines beneath the new residential structure, driveway, and hardscaping features 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 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. Excavations resulting from demolition of existing structures should be backfilled with structural fill as described in Section 8.4 of this report, as needed to achieve design grades.

8.1.4 Test Pits

The test pits conducted at the site were loosely backfilled during our field investigation. Where test pits are located within finalized structural fill, structure, or driveway areas, the loose backfill materials should be re-excavated. The resulting excavations should be backfilled with structural fill placed and compacted in general accordance with Section 8.4 of this report.

8.1.5 Erosion Control

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

8.2 Temporary Excavations

8.2.1 Overview

Conventional earthmoving equipment in proper working condition should be capable of making necessary excavations for the anticipated cuts at the site as described earlier in this report. 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 <u>not</u> 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 silty sand to sandy silt (SM/ML) encountered at the site. Some instability may occur if groundwater seepage is encountered. If seepage undermines the stability of utility trenches, 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 OSHA Soil Type

For use in the planning and construction of temporary excavations up to 10 feet in depth, an OSHA soil type "C" should be used for the native silty sand to sandy silt (SM/ML).

8.2.4 Excavations near Foundations

Excavations near footings should not extend within a 1H:1V 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

For planning purposes, the wet season should be considered to extend from late September to late June. It is our experience that dry weather working conditions should prevail between early July and the middle of September. Notwithstanding the above, soil conditions should be evaluated in the field by the geotechnical engineer or his representative at the initial stage of site preparation to determine whether the recommendations within this section should be incorporated into construction.

8.3.1 General

The near-surface silty sand to sandy silt (SM/ML) encountered at the site is susceptible to disturbance during wet weather. Trafficability of this soil may be difficult, and significant damage to subgrade soils could occur, if earthwork is undertaken without proper precautions at times when the exposed soils are more than a few percentage points above optimum moisture content. For construction that occurs during wet weather, site preparation activities may need to be accomplished using track-mounted equipment, loading removed material onto trucks supported on granular haul roads, or other methods to limit soil disturbance. The geotechnical representative should evaluate the subgrade during excavation by probing rather than proof rolling. Soils that have been disturbed during site preparation activities, or soft or loose areas identified during probing, should be over-excavated to firm, stable subgrade, and replaced with imported granular structural fill.

8.3.2 Geotextile Separation Fabric

We recommend a geotextile separation fabric be placed to serve as a barrier between the prepared fine-grained subgrade and granular fill/base rock in areas of repeated or heavy construction traffic. The geotextile fabric should be in conformance with Section 02320 of the current Oregon Department of Transportation (ODOT) Standard Specification for Construction.

8.3.3 Granular Working Surfaces (Haul Roads & Staging Areas)

Haul roads subjected to repeated heavy, tire-mounted, construction traffic (e.g. dump trucks, concrete trucks, etc.) will require a minimum of 18 inches of imported granular material. For light staging areas, 12 inches of imported granular material is typically sufficient. Additional granular material and/or geo-grid reinforcement may be recommended based on site conditions and/or loading at the time of construction. The imported granular material should be in conformance with Section 8.4.2 of this report and have less than 5 percent material passing the U.S. Standard No. 200 Sieve. The prepared subgrade should be covered with geotextile fabric prior to placement of the imported granular material. The imported granular material should be placed in a single lift (up to 24 inches deep) and compacted using a smooth-drum, non-vibratory roller until well-keyed.

8.3.4 Footing Subgrade Protection

A minimum of 3 inches of imported granular material is recommended 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, have less than 5 percent material passing the U.S. Standard No. 200 Sieve, and have a maximum particle size limited to 1-inch. The imported granular material should be placed in one lift over the prepared, undisturbed subgrade, and compacted using non-vibratory 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 may take the form of in-place density tests and/or proof roll tests with suitable equipment. Structural fill should be evaluated at intervals not exceeding every 2 vertical feet as the fill is being placed.

8.4.1 On-Site Materials – General Use

Recognizing the limited grading (fill placement) associated with this project, soil moisture sensitivity, and other special considerations, re-use of the on-site soils as structural fill at the site is not recommended. We recommend imported granular material be used for structural fill as described below.

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 4 inches, and have less than 5 percent material passing the U.S. Standard No. 200 Sieve. For fine-grading purposes, the maximum particle size should be limited to 1½ inches. 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 90 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.

Compaction of granular fill materials with high percentages of particle sizes in excess of 1½ inches should be evaluated by periodic proof-roll observation or continuous observation by the CGT geotechnical representative during fill placement, since it cannot be tested conventionally using a nuclear densometer. Such materials should be "capped" with a minimum of 12 inches of 1½-inch-minus (or finer) granular fill under all structural elements (footings, concrete slabs, etc.).

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 90 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 a minimum 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	88% ASTM D1557 or pipe manufacturer's recommendation	85% ASTM D1557 or pipe manufacturer's recommendation
Above Pipe Zone	90% ASTM D1557	88% ASTM D1557
Within 3 Feet of Design Subgrade	90% ASTM D1557	88% ASTM D1557

[1]Includes proposed residential structure, driveway, structural fill areas, exterior hardscaping, etc.

8.5 Permanent Slopes

Permanent fill and cut slopes constructed at the site should be graded at 2H:1V or flatter. The surface of all slopes should be protected from erosion by seeding, sodding, or other acceptable means. Adjacent on-site and off-site structures should be located at least 5 feet from the top of constructed slopes.

8.6 Shallow Foundations

8.6.1 Subgrade Preparation

Satisfactory subgrade support for shallow foundations associated with the planned residential structure can be obtained from the native, medium dense/stiff to better, silty sand to sandy silt (SM/ML), or on structural fill that is properly placed and compacted on this material during construction. 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. The maximum particle size of over-excavation backfill should be limited to 1½ inches. 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.

8.6.2 Minimum Footing Width & Embedment

Minimum footing widths should be in conformance with the 2014 Oregon Residential Specialty Code (2014 ORSC). As a guideline, we recommend individual spread footings have a minimum width of 24 inches. For one-story and two-story, light-framed structures, we recommend continuous wall footings have a minimum width of 12 inches and 15 inches, respectively. All footings should be founded at least 18 inches below the lowest, permanent adjacent grade.

8.6.3 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. If an increased allowable soil bearing pressure is desired for design, the geotechnical engineer should be consulted.

8.6.4 Horizontal Setback from Descending Slopes

Foundations constructed within or near descending slopes should be setback a minimum of 5 feet from the slope surface. This distance should be measured between the face of the slope and the bottom, outside edge of the respective foundation. Organic topsoil and loose surface soils (if present) should not be included when determining this distance. The geotechnical engineer or his representative should be contacted to observe foundation subgrade conditions and confirm this recommended minimum setback is achieved.

8.6.5 Lateral Capacity

A maximum passive (equivalent fluid) earth pressure of 150 pounds per cubic foot (pcf) is recommended for design of footings confined by native soil described above 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 not 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 on the native soil described above. An ultimate coefficient of friction equal to 0.45 may be used when calculating resistance to sliding for footings founded on a minimum of 6 inches of imported granular structural fill (crushed rock) that is properly placed and compacted during construction.

8.6.6 Subsurface Drainage

Recognizing the fine-grained nature of the native soils, we recommend placing perimeter foundation drains at the exterior, base elevations of continuous wall footings. Foundation drains should consist of a minimum 4-inch-diameter, perforated, HDPE (High Density Polyethylene) drainpipe wrapped with a non-woven geotextile filter fabric. The drains should be backfilled with a minimum of 2 cubic feet of open graded drain rock per lineal foot of pipe. The drain rock should be encased in a geotextile filter fabric in order to provide separation from the surrounding soils. Foundation drains should be positively sloped and should outlet to a suitable discharge point. The geotechnical engineer or his representative should be contacted to observe the drains prior to backfilling. Roof drains should not be tied into foundation drains.

8.7 Floor Slabs

8.7.1 Subgrade Preparation

Satisfactory subgrade support for floor slabs constructed on grade, supporting up to 150 psf area loading, can be obtained from the native, medium dense/stiff to better, silty sand to sandy silt (SM/ML), or on structural fill that is properly placed and compacted on this material during construction. The geotechnical engineer or his representative should observe floor slab subgrade soils to evaluate surface consistencies. 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 as described in Section 8.4.2 of this report.

8.7.2 Crushed Rock Base

Concrete floor slabs should be supported on a minimum 6-inch-thick layer of crushed rock (base rock) in conformance with Section 8.4.3 of this report. For design cases where a vapor barrier or retarder is not placed below the slab, we recommend "choking" the surface of the base rock with fine 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.

8.7.3 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. 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.

8.7.4 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.

8.8 Rigid Retaining Walls

8.8.1 Footings

Retaining wall footings should be designed and constructed in conformance with the recommendations presented in Section 8.6 of this report, as applicable.

8.8.2 Wall Drains

Subject to review of the wall designer, we recommend placing retaining wall drains at the base elevation of the heel of retaining wall footings. Retaining wall drains should consist of a minimum 4-inch-diameter, perforated, HDPE (High Density Polyethylene) drainpipe wrapped with a non-woven geotextile filter fabric. The drains should be backfilled with a minimum of 2 cubic feet of open graded drain rock per lineal foot of pipe. The drain rock should be encased in a geotextile fabric in order to provide separation from the surrounding soils. Retaining wall drains should be positively sloped and should outlet to a suitable discharge point. The geotechnical engineer or his representative should be contacted to observe the drains prior to backfilling. Roof or area drains should not be tied into retaining wall drains.

8.8.3 Wall Backfill

Retaining walls should be backfilled with imported granular structural fill in conformance with Section 8.4.2 and contain less than 5 percent passing the U.S. Standard No. 200 Sieve. The backfill should be compacted to a minimum of 90 percent of the material's maximum dry density as determined in general accordance with ASTM D1557 (Modified Proctor). When placing fill behind walls, care must be taken to minimize undue lateral loads on the walls. Heavy compaction equipment should be kept at least "H" feet from the back of the walls, where "H" is the height of the wall. Light mechanical or hand tamping equipment should be used for compaction of backfill materials within "H" feet of the back of the walls.

8.8.4 Design Parameters & Limitations

For rigid retaining walls founded, backfilled, and drained as recommended above, the following table presents parameters recommended for design.

Table 3: Design Parameters for Rigid Retaining Walls

Retaining Wall Condition	Modeled Backfill Condition	Static Equivalent Fluid Pressure (S_A)	Additional Seismic Equivalent Fluid Pressure (S_{AE})	Surcharge from Uniform Load, q, Acting on Backfill Behind Retaining Wall
Not Restrained from Rotation	Level (i = 0)	30 pcf	10 pcf	0.22*q
Restrained from Rotation	Level (i = 0)	50 pcf	6 pcf	0.38*q

Note 1. Refer to the attached Figure 8 for a graphical representation of static and seismic loading conditions. Seismic component of active thrust acts at 0.6H above the base of the wall.

Note 2. Seismic (dynamic) lateral loads were computed using the Mononobe-Okabe Equation as presented in the 1997 Federal Highway Administration (FHWA) design manual.

The above design recommendations are based on the assumptions that:

(1) the walls consist of concrete cantilevered retaining walls (β = 0 and δ = 24 degrees, see Figure 8).
(2) the walls are 10 feet or less in height.
(3) the backfill is drained and consists of imported granular structural fill (ϕ = 38 degrees).
(4) no line load or point load surcharges are imposed behind the walls.
(5) the grade behind the wall is level, or sloping down and away from the wall, for a distance of 10 feet or more from the wall.
(6) the grade in front of the walls is level or sloping up for a distance of at least 5 feet from the wall.

Re-evaluation of our recommendations will be required if the retaining wall design criteria for the project vary from these assumptions.

8.9 Additional Considerations

8.9.1 Drainage

Subsurface drains should be connected to the nearest storm drain or other suitable discharge point. Paved surfaces and ground near or adjacent to the residential structure should be sloped to drain away from the structure. Surface water from paved surfaces, hardscaped areas, and open spaces should be collected and routed to a suitable discharge point. Surface water should not be directed into foundation drains or onto site slopes.

8.9.2 Expansive Potential

The near surface soil consists of low to medium plasticity silty sand to sandy silt (SM/ML). Based on experience with similar materials, 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 the site.

9.0 RECOMMENDED ADDITIONAL SERVICES

9.1 Design Review

Geotechnical design review is of paramount importance. We recommend the geotechnical design review take place prior to releasing bid packets to contractors.

9.2 Observation of Construction

Satisfactory earthwork, foundation, floor slab, and driveway 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 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 & Grubbing
- Subgrade Preparation for Structural Fills, Shallow Foundations, Floor Slabs & Driveway Areas
- Compaction of Structural Fill and Retaining Wall Backfill
- Compaction of Utility Trench Backfill
- Placement of Foundation Drains, Retaining Wall Drains & Other Drains
- Compaction of Base Rock for Floor Slabs & Driveway Areas

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

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 can 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.

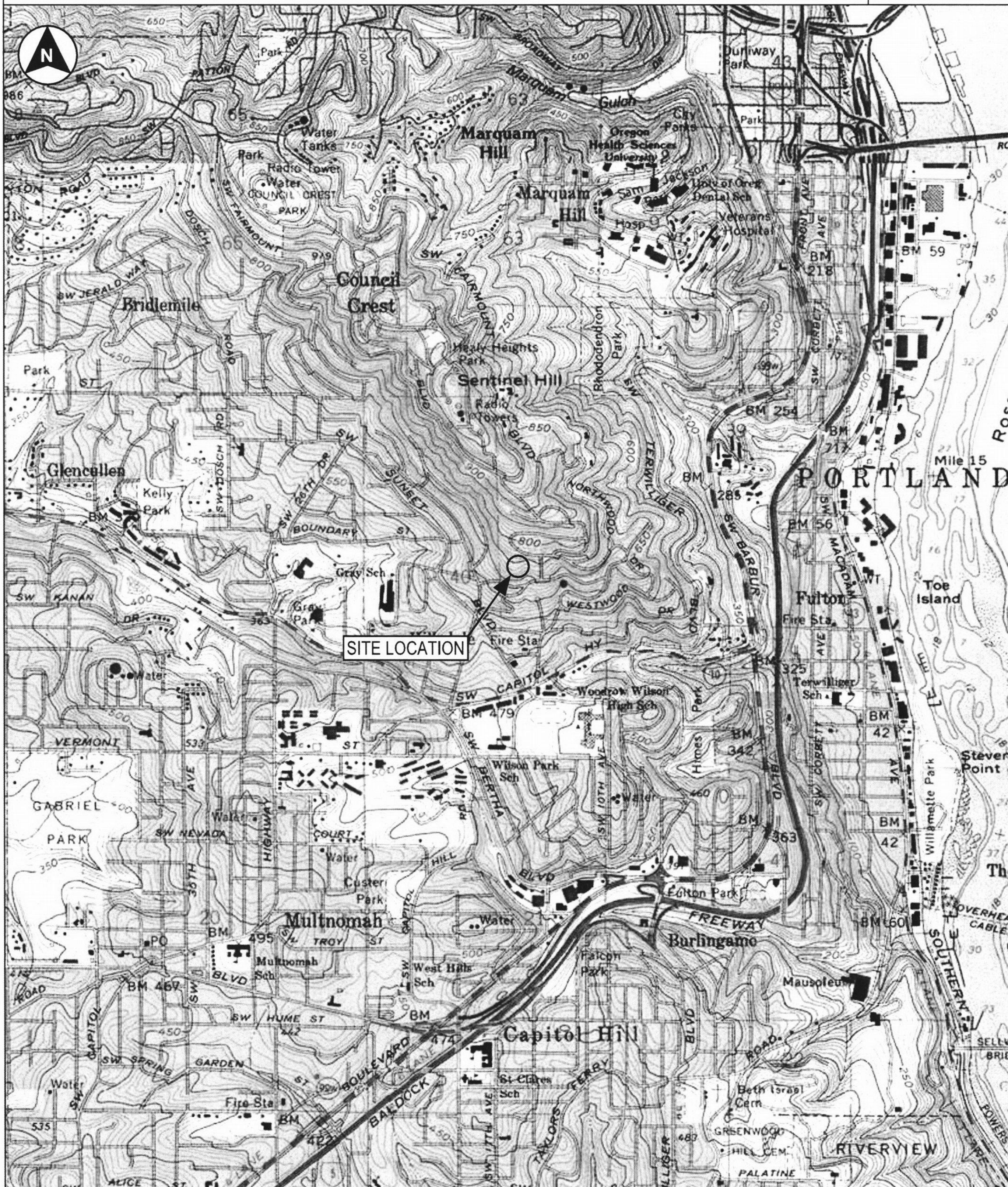
Geotechnical engineering and the geologic sciences are characterized by a degree of uncertainty. Professional judgments presented in this report are based on our understanding of the proposed construction, familiarity with similar projects in the area, and on general experience. Within the limitations 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, expressed or implied, is made. This report is subject to review and should not be relied upon after a period of three years.

ALTA MIRA CIRCLE RESIDENCE - PORTLAND, OREGON

CGT Project Number G1504162

FIGURE 1

Site Location



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

Township 1 South. Range 1 East. Section 16 Willamette Meridian

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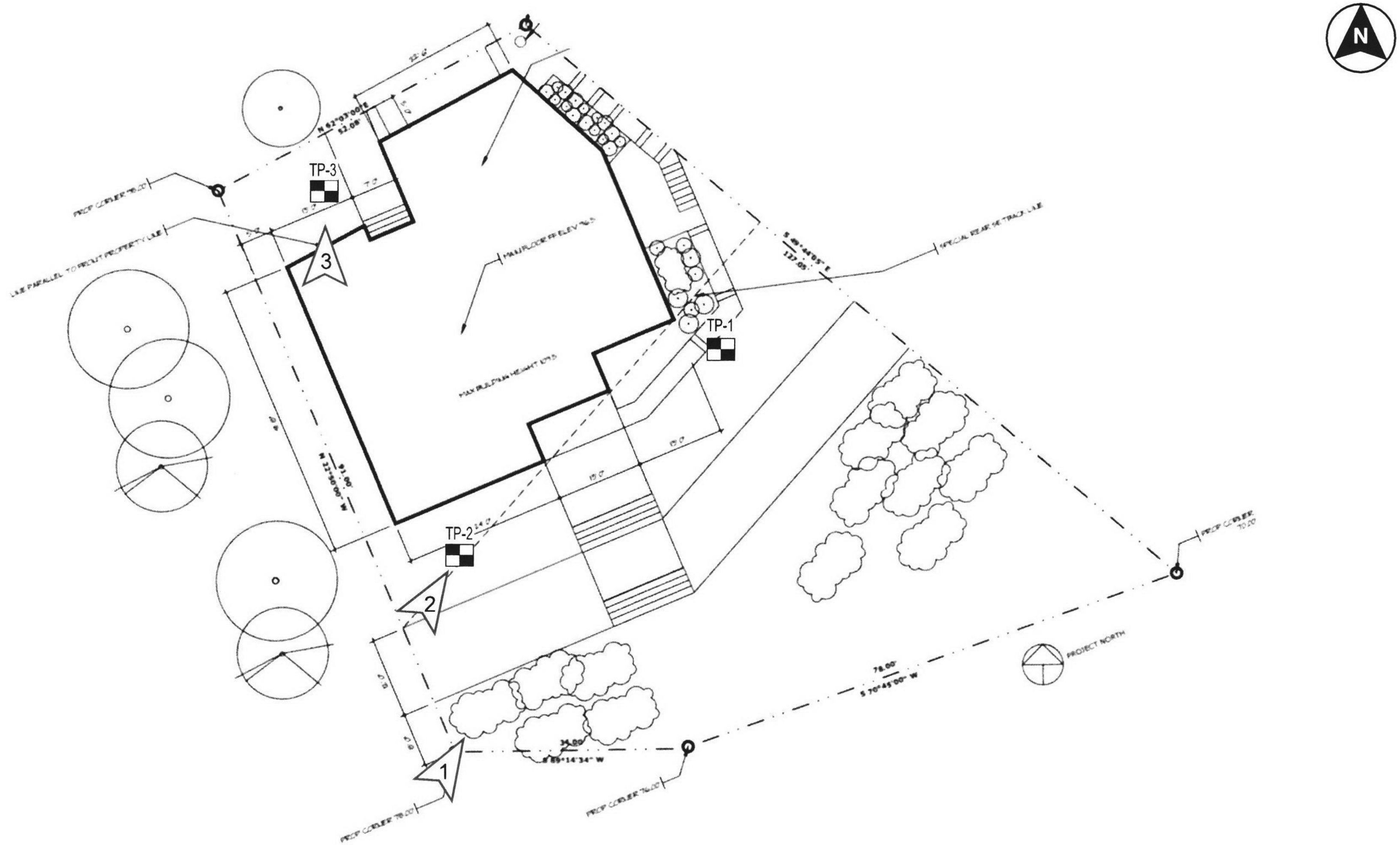
Latitude: 45.483457°
Longitude: -122.693657°

1 Inch = 2,000 feet

0 2000 4000

ALTA MIRA CIRCLE RESIDENCE - PORTLAND, OREGON ***CGT Project Number G1504162***	**FIGURE 2** **Site Plan**



LEGEND

TP-1 Location of test pit excavation

Orientation of site photographs shown on Figure 3

1 Inch = 20 Feet

0 20 40

NOTES: Site plan provided by Chris Thelen of Arcon Group. Inc. The drawing was reproduced and modified by CGT. All exploration locations should be considered approximate.

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ALTA MIRA CIRCLE RESIDENCE - PORTLAND, OREGON CGT Project Number G1504162	FIGURE 3 Site Photographs

Photograph 1: Looking northeast from paved walkway to the south of site.

Photograph 2: Looking northeast at TP-2 excavation.

Photograph 3: Looking north at TP-3 upon completion.

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See Figure 2 for approximate photograph locations and directions. Photographs were taken at the time of our fieldwork.

ALTA MIRA CIRCLE RESIDENCE - PORTLAND, OREGON

FIGURE 4

CGT Project Number G1504162

USCS

Classification of Terms and Content

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. etc.

USCS Grain Size

Fines		<#200 (.075 mm)
Sand	Fine	#200 - #40 (.425 mm)
	Medium	#40 - #10 (2 mm)
	Coarse	#10 - #4 (4.75)
Gravel	Fine	#4 - 0.75 inch
	Coarse	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 ¼ inch
10 - 30	Medium Dense	8 - 15	0.50 - 1.00	1.00 - 2.00	Stiff	Thumb penetrates less than ¼ 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 Medium	Medium to High	None to Slow	Medium
MH	Medium to High	Low to Medium	None to Slow	Low to Medium
CH	Medium to High	High to Very High	None	High

Structure

Stratified: Alternating layers of material or color >6 mm thick

Laminated: Alternating layers < 6 mm thick

Fissured: Breaks along definite 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-2487)

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

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Additional References:
ASTM D2487 Standard Practice for Classification of Soils for Engineering Purposes and
ASTM D2488 Standard Practice for Description and Identification of Soils (Visual-Manual Procedure)

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FIGURE 5

TP-1

CLIENT Chris Thelen
PROJECT NAME Alta Mira Circle Residence
PROJECT NUMBER G1504162
PROJECT LOCATION 5424 SW Alta Mira Circle, Portland, Oregon
DATE STARTED 4/17/15
LOGGED BY HHP
ELEVATION DATUM Metro Map Utility
EXCAVATION CONTRACTOR Client
GROUND ELEVATION 740 ft
EQUIPMENT Hitachi- Zaxis 70
SEEPAGE ---
EXCAVATION METHOD Test Pit
GROUNDWATER AT END ---
NOTES
GROUNDWATER AFTER EXCAVATION ---

ELEVATION (ft)	GRAPHIC LOG	U.S.C.S.	MATERIAL DESCRIPTION	GROUNDWATER	DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N_{SPT} VALUE)	N_{60} VALUE	DRY UNIT WT. (pcf)	▲ SPT N_{SPT} VALUE ▲ / PL MC LL / ☐ FINES CONTENT (%) ☐ (0 20 40 60 80 100)
740		ML	**SILT TOPSOIL:** Stiff, dark brown, with roots up to ¾-inch in diameter, moist, and exhibited low to medium plasticity.		0						
738		SM/ML	**SILTY SAND TO SANDY SILT:** Medium dense/very stiff to hard, brown, with orange and gray mottling, very fine grained sand, moist, and low to medium plasticity fines. Sand content decreases with depth. No mottling below about 2½ feet bgs.		2	GRAB 1					Fines: 27
736					4	GRAB 2					MC: 37
734					6						
732					8						
730					10	GRAB 3					MC: 32
728			• Test pit terminated at a depth of about 10 feet bgs. • No groundwater or caving observed within depth explored. • Test pit loosely backfilled with spoils upon completion.								

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FIGURE 6

TP-2

CLIENT Chris Thelen
PROJECT NAME Alta Mira Circle Residence
PROJECT NUMBER G1504162
PROJECT LOCATION 5424 SW Alta Mira Circle, Portland, Oregon
DATE STARTED 4/17/15 **LOGGED BY** HHP
ELEVATION DATUM Metro Map Utility
EXCAVATION CONTRACTOR Client
GROUND ELEVATION 742 ft
EQUIPMENT Hitachi- Zaxis 70
SEEPAGE ---
EXCAVATION METHOD Test Pit
GROUNDWATER AT END ---
NOTES
GROUNDWATER AFTER EXCAVATION ---

ELEVATION (ft)	GRAPHIC LOG	U.S.C.S.	MATERIAL DESCRIPTION	GROUNDWATER	DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N_{SPT} VALUE)	N_{60} VALUE	DRY UNIT WT. (pcf)	▲ SPT N_{SPT} VALUE ▲ / PL — MC — LL / □ FINES CONTENT (%) □ (0 20 40 60 80 100)
					0						
		ML	**SILT TOPSOIL:** Stiff, brown, with scattered rootlets, moist, and exhibited low plasticity.								
740		SM/ML	**SILTY SAND TO SANDY SILT:** Medium dense/stiff to very stiff, brown, with orange and gray mottling, very fine grained sand, moist, and low to medium plasticity fines. Sand content decreases with depth.		2						
						GRAB 1					
738			No mottling below about 3½ feet bgs.		4	GRAB 2					MC 34
736			Light brown below about 6 feet bgs.		6						
734					8						
						GRAB 3					PL 6, MC 29, LL 32
732					10						
730			• Test pit terminated at a depth of about 10 feet bgs. • No groundwater or caving observed within depth explored. • Test pit loosely backfilled with spoils upon completion.								

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FIGURE 7

TP-3

CLIENT Chris Thelen	**PROJECT NAME** Alta Mira Circle Residence
PROJECT NUMBER G1504162	**PROJECT LOCATION** 5424 SW Alta Mira Circle, Portland, Oregon
DATE STARTED 4/17/15 **LOGGED BY** HHP	**ELEVATION DATUM** Metro Map Utility
EXCAVATION CONTRACTOR Client	**GROUND ELEVATION** 748 ft
EQUIPMENT Hitachi- Zaxis 70	**SEEPAGE** ---
EXCAVATION METHOD Test Pit	**GROUNDWATER AT END** ---
NOTES	**GROUNDWATER AFTER EXCAVATION** ---

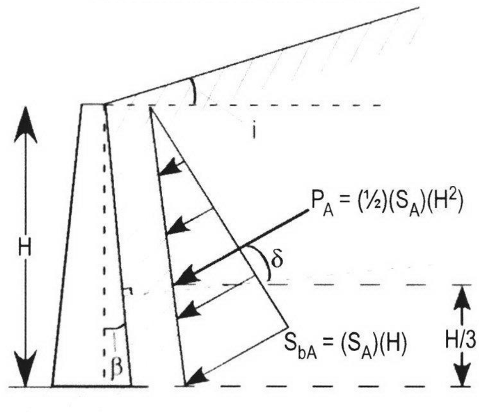
ELEVATION (ft)	GRAPHIC LOG	U.S.C.S.	MATERIAL DESCRIPTION	GROUNDWATER	DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N_{SPT} VALUE)	N_{60} VALUE	DRY UNIT WT. (pcf)	▲ SPT N_{SPT} VALUE ▲ / PL — MC — LL / □ FINES CONTENT (%) □ (0 20 40 60 80 100)
		ML	**SILT TOPSOIL:** Stiff to very stiff, brown, with rootlets, moist, and exhibited low plasticity.		0						
746		SM/ML	**SILTY SAND TO SANDY SILT:** Medium dense/stiff to very stiff, brown, with orange and gray mottling, very fine grained sand, moist, and low plasticity fines. Sand content decreases with depth.		2	GRAB 1					MC 23
744			No mottling below about 4½ feet bgs.		4						
742					6						
740					8	GRAB 2					MC 34
738					10						
736											

• Test pit terminated at a depth of about 10 feet bgs.
• No groundwater or caving observed within depth explored.
• Test pit loosely backfilled with spoils upon completion.

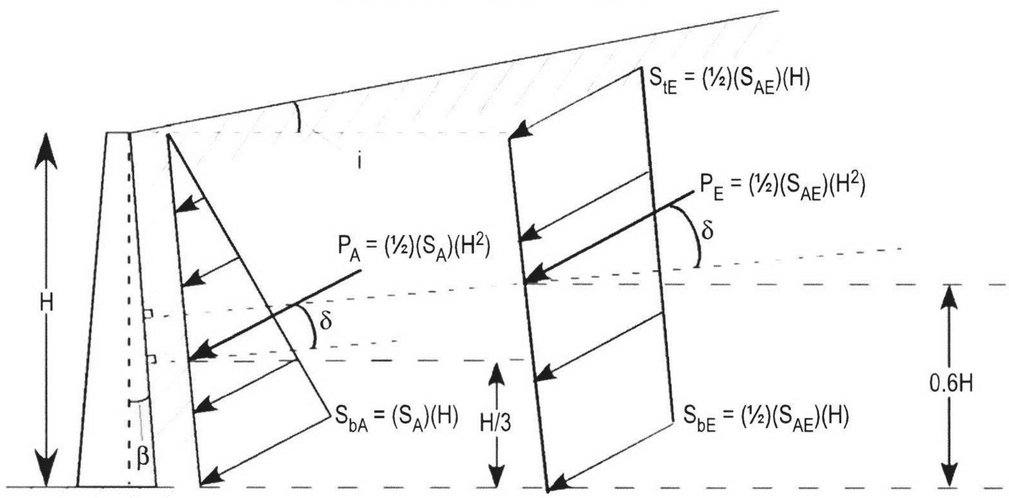
ALTA MIRA CIRCLE RESIDENCE - PORTLAND, OREGON Project Number G1504162	FIGURE 8 RETAINING WALLS

ACTIVE LATERAL PRESSURE DISTRIBUTION

STATIC LOADING CONDITIONS



SEISMIC LOADING CONDITIONS



LEGEND

P_A = Static active thrust force acting at a triangular distribution on wall (lb/ft^3)

P_E = Dynamic component of active thrust force acting at a uniform distribution on wall (lb/ft)

i = Slope of backfill (degrees)**

S_A = Active (static) component of equivalent fluid pressure (lb/ft^3)*

S_{tE} = Active earth pressure (dynamic) at the top of the wall (lb/ft^3)

S_{bA} = Active earth pressure (static) at the bottom of the wall (lb/ft^3)

ϕ = Internal angle of friction for backfill (degrees)**

δ = Angle from normal of back of wall (degrees). Based on friction developing between wall and backfill**

β = Slope of back of wall (degrees)**

S_{AE} = Dynamic component of equivalent fluid pressure (lb/ft^3)*

S_{bE} = Active earth pressure (dynamic) at bottom of the wall (lb/ft^3)*

*Refer to report text for calculated values **Refer to report text for modeled/assumed values

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Notes

1. Uniform pressure distribution of seismic loading is based on empirical evaluations [Sherif et al. 1982 and Whitman. 1990].
2. Placement of seismic resultant force at 0.6H is based on wall behavior and model test results [Whitman. 1990].