GEOTECHNICAL EVALUATION REPORT

For the

Proposed St. Andrews Condominiums
Lots 9 through 12
Southwest 18th and Southwest Mill Street Terrace,
Portland, Oregon

Prepared for

St. Andrews Development, LLC
3231 Northeast US Grant Place
Portland, Oregon 97212

Prepared by

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PSI Report No. 704-85036-1

April 4, 2008

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Engineering • Consulting • Testing

Charles R. Lane, P.E.
Senior Geotechnical Engineer
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April 4, 2008

Mr. John Reilly
St. Andrews Development
3231 Northeast US Grant Place
Portland, Oregon 97212

Re: Geotechnical Evaluation Report
Proposed St. Andrews Condominiums, Lots 9 through 12
Southwest 18th Avenue and Southwest Mill Street Terrace
Portland, Oregon
PSI Report No. 704-85036-1

Dear Mr. Reilly:

Professional Service Industries, Inc. is pleased to transmit our Geotechnical Engineering Services Report for the referenced project. This report includes our results of field and laboratory testing, as well as recommendations for foundation design and general site development. This report should be used in conjunction with the Site Specific Seismic Evaluation presented in PSI Report #704-85036-2.

We appreciate the opportunity to perform this Geotechnical Study and look forward to continued participation during the design and construction phases of this project. If you have any questions pertaining to this report, or if we may be of further service, please contact the undersigned at (503) 289-1778.

Respectfully submitted,

PROFESSIONAL SERVICE INDUSTRIES, INC.

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Proposed St. Andrews Condominiums, Lots 9 through 12
Southwest 18 Avenue and Southwest Mill Street Terrace
Portland, Oregon

1.0 EXECUTIVE SUMMARY

An exploration and evaluation of the subsurface conditions have been completed for the proposed condominium development located east of the intersection of Southwest Mill Street Terrace and Southwest 18th Avenue in the Goose Hollow area of Portland, Oregon. One soil boring in the only area accessible to our truck mounted drill rig, as well as a number of drive probes, were conducted in the proposed area of development. Groundwater was not encountered during site exploration conducted on March 24, 2008.

Results of this subsurface exploration indicate that shallow foundations bearing on the underlying rock formation can be considered for the proposed building. Details related to site development, foundation, and construction considerations are included in subsequent sections of this report.

Features requiring special consideration at this development site mainly include the demolition of the existing building foundation, proposed rock excavation, and the shallow rock. These features are discussed further in this report.

The owner/designer should not rely solely on this Executive Summary and must read and evaluate the entire contents of this report prior to utilizing our engineering recommendations in preparation of design/construction documents.

2.0 PROJECT INFORMATION

2.1 Project Authorization

Professional Service Industries, Inc. (PSI) has completed a geotechnical evaluation report for the proposed multi-story condominium development to be located on the east side of Southwest 18th Avenue and Southwest Mill Street Terrace in Portland, Oregon. This exploration was accomplished in general accordance with PSI Proposal No. 704-08-P026, dated February 4, 2008. Our services were contracted by Mr. John Reilly of St. Andrews Development, LLC on February 23, 2008.

2.2 Project Description

PSI's understanding of the project is that the proposed construction will consist of a five-story condominium with underground parking. We understand the property was once developed with a single family residence that has since been partially demolished. We assume that the proposed facility will be constructed in accordance with the provisions of the Oregon Structural Specialty Code 2007 Edition (OSSC 2007).

Detailed structural loading information was not provided; however, for the purpose of this report, we have assumed that maximum wall load and column load will be on the order of 10 kips per linear foot and 300 kips, respectively. Also, in our analyses, floor slab loads of less than 150 psf are assumed, and up to 40 feet of cut into rock and less than 2 feet of fill are anticipated for the design grade.

The geotechnical recommendations presented in this report are based on the available project information, building location, and the subsurface materials described in this report. If any of the noted information is incorrect, please inform PSI in writing so that we may amend the recommendations presented in this report if appropriate and if desired by the client. PSI will not be responsible for the implementation of its recommendations when it is not notified of changes in the project.

2.3 Purpose and Scope of Services

The purpose of this study was to explore the subsurface conditions at the site to enable an evaluation of acceptable foundation recommendations for the proposed structure.

Our scope of services included drilling one (1) soil test boring at the site to rock in the northwest corner of the site; as well a number of drive probes, performing select laboratory testing, and preparation of this geotechnical report. This report briefly outlines the testing procedures, presents available project information, describes the site and subsurface

conditions, and provided the recommendations regarding the following:

- Grading procedures for site development.
- Rock Excavation.
- Foundation types, depths, allowable bearing capacities, and an estimate of potential settlement.
- Recommendations for the floor slab support.
- Comments regarding factors that will impact construction and performance of the proposed construction.

The scope of services did not include an environmental assessment for determining the presence or absence of wetlands or hazardous or toxic materials in the soil, bedrock, surface water, groundwater, or air on or below, or around this site. Any statements in this report or on the boring logs regarding odors, colors, and unusual or suspicious items or conditions are strictly for informational purposes.

As directed by the client, PSI did not provide any service to investigate or detect the presence of moisture, mold or other biological contaminates in or around any structure, or any service that was designed or intended to prevent or lower the risk of the occurrence of the amplification of the same. Client acknowledges that mold is ubiquitous to the environment with mold amplification occurring when building materials are impacted by moisture. Client further acknowledges that site conditions are outside of PSI's control, and that mold amplification will likely occur, or continue to occur, in the presence of moisture. As such, PSI cannot and shall not be held responsible for the occurrence or recurrence of mold amplification.

3.0 SITE GEOLOGICAL AND SUBSURFACE CONDITIONS

3.1 Site Location and Description

The proposed multi-story condominium is to be located east of the intersection of Southwest 18th Avenue and Southwest Mill Street Terrace in Portland, Oregon. The vacant property is surrounded by residential development. The approximate location of the site is presented in Figure 1 of this report. The site is steeply sloping and was occupied by a single family residence at one time that has since been partially demolished as the foundation of the residence still exists on the site.

3.2 Site Geology and Subsurface Conditions

Located on the flanks of the West Hills (aka Tualatin Mountains, Portland Hills), the site lies on the edge of the Portland Basin, to be discussed in further detail in Section 3.4. Based upon the Geologic Map of the Portland Quadrangle, Multnomah and Washington Counties, Oregon, and Clark County, Washington (DOGAMI, GMS-75, 1991) the property's geologic unit in the north (lower elevation) portion of the site is mapped as Pleistocene aged fine-grained sediments (Qff). This stratum is comprised predominantly of fine grained sediments, ranging in size from coarse sand to silt. Believed to underlie the fine-grained sediments at depths ranging from near surface to 7.5 feet below the existing site grade, based on our subsurface exploration, is the Miocene aged (16.4 million to 15.6 million years ago) Columbia River Basalt Group, Grand Ronde Basalt Winter Water Unit (Tgww) which consists of dark gray to black basalt with flows displaying columnar jointing.

Soil drive probes were conducted throughout the proposed development area. However, in order to confirm depth to rock on the site, one Standard Penetration Test soil boring (B-1) was performed using our CME-75, truck-mounted, drill rig equipped with hollow-stem augers, and an automatic hammer. The soil boring and drive probe locations are shown on the attached Boring and Drive Probe Location Plan, Figure 2. The boring location was marked in the field by PSI. A log of the boring is attached along with a Soil Classification Chart & General Notes.

The boring was advanced utilizing hollow-stem auger methodology. During our drilling processes soil samples were routinely obtained through Standard Penetration Tests using an automatic hammer. The Standard Penetration Test is performed by driving a 2-inch, O.D., split-spoon sampler into the undisturbed formation located at the bottom of the advanced auger with repeated blows of a 140-pound, pin-guided, automatic hammer falling a vertical distance of 30 inches. The number of blow, N-Value, required to drive the sampler one foot, the last 12 inches of an 18-inch sample interval, is used to measure the soil consistency. It should be noted that automatic hammers generally produce lower

standard penetration test values than those obtained using a traditional safety hammer. Studies have generally indicated that penetration resistances may vary by a factor of 1.5 to 2 between the two methods. Drilling and sampling techniques were accomplished in general accordance with ASTM procedures. Soil samples were taken at 2.5-foot intervals for the first 10 feet, and then at 5-foot intervals to the termination depths of the boring. Samples were identified in the field, placed in sealed containers, and transported to the laboratory for further classification and testing.

At the ground surface, a 4 foot thick layer of fill material consisting of gravel with varying amounts of silt was encountered at the borehole location. The soils encountered at the boring location beneath the existing fill may be divided into 2 general strata. These soil strata are discussed separately below.

The upper stratum of fine-grained soil extended from the base of the fill layer to a depth of 7.5 feet at the boring location. The soil is believed to be low plasticity clayey silt (ML) with gravel. Standard penetration N-values within these fine-grained soils generally indicated consistency of soft to medium stiff.

Underlying the soil unit to the terminal depths of the borings, heavily weathered/decomposed basalt rock of the Winter Water unit was encountered. Auger refusal was encountered within this unit at a depth of 8.5 feet below ground surface.

The above subsurface description is of a generalized nature to highlight the major subsurface stratification features and material characteristics. The boring log included in the appendix should be reviewed for specific information at individual boring locations. These records include soil descriptions, stratifications, penetration resistances (safety hammer equivalent), locations of the samples and laboratory test data. The stratifications shown on the boring log represent the conditions only at the actual boring location. Variations may occur and should be expected. The stratifications represent the approximate boundary between subsurface materials and the actual transition may be gradual.

3.3 Groundwater Information

Groundwater was not encountered within the exploration depths of the boring. Variations in groundwater levels should be expected seasonally, annually, and from location to location.

4.0 EVALUATION AND FOUNDATION RECOMMENDATIONS

4.1 Geotechnical Discussion

Based on the results of our field work, the available subsurface data, laboratory test data, and our engineering analysis when considered in conjunction with the structural information, indicates that conventional spread footing foundations in conjunction with tie-back anchors may be used to support the proposed structural loads. However, the primary geotechnical factor influencing the design and construction of the proposed project are possible demolition debris and the necessary slight ground improvement such as overexcavation and replacement of the overexcavated soils with structural fill may be needed in order to achieve an appropriate elevation and bearing capacity. In addition, the use of on grade floor slabs is permissible at this site provided that the site is developed in accordance with the requirements of this report.

Difficult site preparation could be encountered if construction begins during the wet season of the year due to the moisture sensitive, native soil near the site surface. Careful observations should be made to identify any soft, loose, or organic soils. If encountered, the geotechnical engineer of record should be consulted.

4.2 Site Preparation

We recommend that any pavement structure, demolished building material, existing footing foundations, existing fill, and any soft soils in the construction areas should be removed from the site. A representative of the geotechnical engineer should determine the depth of removal to the subgrade and any soft/loose soils at the time of construction. Utilities should be located and rerouted as necessary and any abandoned pipes or utility conduits should be removed to inhibit the potential for subsurface erosion. Utility trench excavations must be backfilled with properly compacted structural fill which is outlined in Section 5.3 of this report.

After stripping and excavating to the proposed subgrade level, the exposed subgrade and/or rock, the bottom of the excavation should be observed to assess that the foundation soils are capable of supporting the design loads and are consistent with the materials discussed in this report.

The upper fine-grained soils encountered at this site are expected to be sensitive to disturbances caused by construction and to changes in moisture content. During wet weather periods, increases in the moisture content of the soils can cause significant reduction in the soil strength and support capabilities. In addition, soils which become wet

may be slow to dry. It will, therefore, be advantageous to perform earthwork and foundation construction activities during dry weather.

It is not uncommon for construction equipment to severely disturb the upper 1 to 2 feet of the subgrade during initial phases of site clearing especially if site preparation work is performed while the soils are wet.

The temporary cut slopes during construction where some sloughing or erosion can be tolerated and life safety is not an issue; such as open excavation slopes for the perimeter of the building, the contractor can cut the slopes up to 1H:1V for slopes where no surcharge is anticipated. In the area where a possible moderate surcharge is anticipated, the slope should not be steeper than 1.5H:1V. The contractor should determine where excavation construction slopes may be required for the areas and have it reviewed by all parties. Permanent earth slopes should be dressed to 2H:1V or flatter and protected from erosion. All vertical rock slopes should be covered by rockfall protection netting (galvanized) with vertical soil slopes restrained by concrete walls.

The excavation contractor for this nearly 40 foot vertical excavation along the east property line should be somewhat involved in the system that is used to make the excavation and construct the permanent wall to retain the east property line excavation. However, we provide a proposed minimum excavation scenario; the area for the excavation is a very steep 1H:1V slope, heavily covered with blackberry vines and brush. It has a 5 to 8 foot strata of silt overlying a weathered to freshly fractured basalt formation. The excavation will be, for practical purposes within a few feet of the property line of the adjacent home built in 1900, which is or within a foot of the property line. We assume, as a worst condition, the foundation of the century old home has been founded in the upper silt formation only a foot below grade.

We recommend the excavation, approximately 3 feet from your property line, be made by constructing a tied-back gunnite foundation wall by using a top down construction procedure. The first excavation would consist of 5 feet in the silt and a row of tie-backs installed on a 5 foot spacing. These tie-backs should be designed for an equivalent fluid pressure of 90 psf. It should be noted the gunnite wall should be placed after the drilled in tie-back anchors and rebar have been installed. The next 5 foot excavation would then be excavated and tie-back anchors installed on 5 foot centers using the 90 pound equivalent fluid pressure design. After the two 5 foot sections are constructed, the excavation should be excavating into the weathered rock formation for the remaining excavation.

The rock formation and its excavated face should be stable and require no additional retention, but should be evaluated by the geotechnical engineer during the excavation. It should be draped with a steel woven (galvanized) wire mesh system, such as Maccaferri

rockfall protection netting, to retain any future, loose rock particles that may occur from freeze/thaw periods; thus directing the small particles to the base of the excavation subsequently allowing future removal and maintenance. The mesh should consist of zinc coated, double twisted steel woven wire mesh manufactured in accordance with ASTM DA975-97. Due to the characteristics of the double twist, the steel wire mesh can withstand the force of falling rocks without unraveling in the event of wire breakage. The nominal mesh opening of Mesh type 8X10, tolerance (distance= D) is 3.25 inches, and Mesh type 6x8, D = 2.5 inches. If for aesthetic reasons or some other reason it may require the rock surface to be covered and the gunnite wall continuing down slope, we would in this case recommend a rock anchor surfacing wall be constructed.

A more detailed design for excavation will be provided in conjunction with the Structural Engineer, Architect, and Contractor prior to construction.

We recommend rock anchors be drilled on a 5 foot center each way in the rock formation using a tri-cone drill with a 4 inch bit. A penetration of only 3 feet into the rock should be sufficient using a No. 6 rebar anchor grouted into the rock and connected to the rebar cage in the wall reinforcing. The design should assume an equivalent fluid pressure of 5 psf for the rock formation design pressure. Vertical drainage mats should be placed between each anchor to prevent the buildup of any water pressure.

Where the excavation will be required to be constructed vertical or near vertical; which is the case along the east property line, a helical anchor tieback could be considered in the first 5 to 10 feet of the excavation in the overlying silt soil unit. The helixes should be installed outside of the theoretical slope failure plane, which should be observed and documented by a representative of the geotechnical engineer.

The helical anchors, reinforcing steel, and shotcrete should be installed the same day the vertical excavation is performed. The exposed vertical excavation should not be allowed to stand overnight.

The face of all excavations should be protected from water seepage. Additionally, the first 3 to 4 feet back (horizontally) from the top of the excavation should be protected from water seepage. Protection may consist of properly secured plastic sheeting. The protection system should be approved and monitored by the geotechnical engineer. No heavy equipment or construction materials should be placed near the top of the vertical excavation.

4.3 Fill Requirements

The excavation of the building site and subgrade basement/garage area should level the

building site excavated into the rock formation. Thus, only a granular subgrade leveling course of 6 to 8 inches will be required.

After the subgrade has been prepared, observed, and approved by the geotechnical engineer, any anticipated fill placement may begin. Fill materials should be free of organic or other deleterious materials have a maximum particle size less than 1 ½ inches, be relatively well graded, and have a liquid limit less than 45 and plasticity index less than 25. Most of the on site soils are not suitable for use as structural fill, however, excavated rock could be considered. The fill materials should be placed after subgrade preparation and observation. The first layer of fill material should be placed in a relatively uniform horizontal lift on the prepared subgrade. Structural fill should be compacted to at least 95 percent of modified Proctor maximum dry density as determined by ASTM Designation D1557.

Fill should be placed in maximum lifts of 8 inches of loose material and should be compacted within the range of 3 percentage points below to 2 percentage points above the optimum moisture content value. If water must be added, it should be uniformly applied and thoroughly mixed. Each lift of compacted engineered fill should be tested by a representative of the geotechnical engineer prior to placement of subsequent lifts. The fill should extend horizontally outward beyond the exterior perimeter of the building and footings a distance equal to the height of the fill. Also, fill should extend horizontally outward from the exterior perimeter of the pavement a distance equal to the height of the fill.

4.4 Foundation Recommendations

Once the site has been properly prepared as discussed above, based on the results of our geotechnical investigation, it is our opinion the planned building can be supported on conventional spread and continuous footing foundations bearing on and penetrated into the dense bedrock. Spread footings for building columns and continuous footings for bearing walls can be designed for allowable soil bearing pressures of 10,000 psf, based on dead load plus design live load. If voids or over excavations are encountered in the footing areas, then these areas can be filled with and leveled with concrete. An allowable bearing capacity of 10,000 psf can be used for this improved subgrade condition.

The allowable bearing pressure includes a safety factor and is intended for dead loads and sustained live loads and can be increased by one-third for the total of all loads, including short-term wind or seismic loads. Minimum dimensions of 36 inches for square footings and 24 inches for continuous footings should be used in the foundation design process.

Allowable lateral frictional resistance between the base of foundations and the subgrade can be expressed as the applied vertical load multiplied by a coefficient of friction of 0.35. In addition, lateral loads may be resisted by a passive earth pressure based on an equivalent fluid density of 300 pounds per cubic foot (pcf) on footings poured "neat" against in-situ soils or properly backfilled with structural fill. The passive earth pressure recommendation includes a factor of safety of approximately 1.5, which is appropriate due to the amount of movement required to develop full passive resistance.

Exterior footings and foundations in unheated areas should be located at a depth of at least 18 inches below the final exterior grade to provide adequate soil cover for frost protection. If the building is to be constructed during the winter months or if the foundation will likely be subjected to freezing temperatures after foundation construction, then the foundation should be adequately protected from freezing.

Based on the known subsurface conditions and site geology, laboratory testing and past experience, we anticipate that properly designed and constructed foundations supported on the recommended rock formation should experience maximum total and differential settlements between adjacent columns on the order of less than 1/2 inch and 1/4 inches, respectively.

The foundation excavations should be observed by a representative of PSI prior to steel/concrete placement or the structural fill construction to assess that the foundation materials are capable of supporting the design loads and are consistent with the materials discussed in this report. Unsuitable soil zones encountered at the bottom of the foundation excavations should be removed to the level of the native, dense rock or as directed by the geotechnical engineer. Cavities formed as a result of excavation of unsuitable soil zones should be backfilled with lean concrete.

Surface run-off water should be drained away from the excavations and not be allowed to pond. An adequate subsurface drain system should be installed along the parameter footings and behind the foundation walls.

Care should be taken to protect prepared bearing surfaces until footing concrete can be placed. Precautions to achieve this end would consist of either:

- Covering of prepared bearing surfaces with impervious membranes.
- Cessation of work during rainy weather.

4.5 Floor Slab Recommendations

The floor slab can be grade supported on the natural dense rock formation or on properly

compacted structural fill.

Based on the existing soil conditions, the design of slabs-on-grade can be based on a subgrade modulus (k) 150 pci if a minimum 6-inch thick granular mat (crushed rock) is placed below the floor slab as recommended below. These subgrade modulus values represent an anticipated value which would be obtained in a standard in-situ plate test with a 1-foot square plate. Use of these subgrade's moduli for design or other on-grade structural elements should include appropriate modification based on dimensions as necessary.

The floor slab granular mat should consist of well-graded 1½-inch or ¾-inch-minus imported crushed rock aggregates having less than 5 percent material passing the No. 200 sieve, and should be compacted to at least 95 percent of modified Proctor maximum dry density as determined by ASTM Designation D 1557. The crushed rock should provide a capillary break to limit migration of moisture through the slab. If additional protection against moisture vapor is desired, a vapor retarding membrane may also be incorporated into the design. Factors such as cost, special considerations for construction, and the floor coverings suggest that decisions on the use of vapor retarding membranes be made by the architect and owner.

4.6 Excavation Retaining Walls/Foundation Walls

Lateral earth pressures on walls which are not restrained at the top, such as retaining walls, etc., may be calculated on the basis of an equivalent fluid density of 35 pounds per cubic foot (pcf) for level backfill and 60 pcf for sloping backfill less than 2 horizontal (H) to 1 vertical (V) (2H:1V). Basement or other retaining walls that are restrained from yielding at the top may be calculated on the basis of an equivalent fluid density of 55 pcf for level backfill and 90 pcf for sloping backfill less than 2H:1V. The earthquake load can be calculated as 8.5 H^2 (lbs/linear foot), applied at 0.6 H from the base of the retaining wall used in the design.

Lateral loads may be resisted by passive pressures acting against footings and by frictional resistance between foundation elements and supporting soils. An equivalent fluid density of 300 pcf and a friction factor of 0.35 may be used for the design of foundations bearing on and resisted by native soils. The recommended equivalent fluid densities and friction factors included a factor of safety of 1.5, which is appropriate due to the amount of movement required to develop full passive resistance. They are also based on the assumption that retaining walls will be properly drained to prevent hydrostatic buildup behind walls.

All backfill for retaining walls, foundation walls, etc., should be select granular material (sand and/or sandy gravel). We anticipate that on site material will not be suitable for this purpose and that it will be necessary to import material to the project for structural backfill. Native silt soil material can be used for the last 18 to 24 inches of the fill, thus acting as a seal to the granular fill.

An adequate subsurface drain system should be installed behind subsurface walls such as retaining walls, foundation walls, etc. Surface run-off drains and the subsurface drains should be carried to approved discharge areas on the lot.

5.0 CONSTRUCTION CONSIDERATIONS

5.1 Excavation

Excavation and construction operations may expose the on-site soils to inclement weather conditions. The stability of exposed soils may rapidly deteriorate due to a change in moisture content (i.e. wetting or drying) or the action of heavy or repeated construction traffic.

In Federal Register, Volume 54, No. 209 (October 1989), the United States Department of Labor, Occupational Safety and Health Administration (OSHA) amended its "Construction Standards for Excavations, 29 CFR, part 1926, Subpart P". This document and subsequent updates were issued to better insure the safety of workmen entering trenches or excavations. It is mandated by this federal regulation that excavations, whether they be utility trenches, basement excavations or footing excavations, be constructed in accordance with the new OSHA guidelines. It is our understanding that these regulations are being strictly enforced and if they are not closely followed, the owner and the contractor could be liable for substantial penalties. The contractor is solely responsible for designing and constructing stable, temporary excavations and should shore, slope, or bench the sides of the excavations as required to maintain stability of both the excavation sides and bottom. The contractor's "responsible person", as defined in 29 CFR Part 1926, should evaluate the soil exposed in the excavations as part of the contractor's safety procedures. In no case should slope height, slope inclination, or excavation depth, including utility trench excavation depth, exceed those specified in local, state, and federal safety regulations.

We are providing this information solely as a service to our client. PSI does not assume responsibility for construction site safety or the contractor's compliance with local, state, and federal safety or other regulations.

5.2 Construction Dewatering

Due to the absence of groundwater within the upper portion of the soil profile, we anticipate that the need for dewatering during construction is relatively low. However, during wet season and after a prolonged period of precipitation discontinuous zones of perched water may exist within the soil deposits. In this case, the seepage of this perched water condition into the excavations should be low to moderate, and can be controlled by gravity drainage and pumping from strategically placed filtered sumps.

5.3 Drainage Considerations

Water should not be allowed to collect in the foundation excavations or on prepared subgrade for floor slabs and pavement during construction. Positive site drainage should be maintained throughout construction activities. Undercut or excavated areas should be sloped toward one corner to facilitate removal of any collected rainwater, groundwater, or surface runoff.

The site grading plan should be developed to provide rapid drainage of surface water away from the building and pavement areas and to inhibit infiltration of surface water around the perimeter of the building and beneath the floor slabs and pavements. The grades should be sloped away from the building and pavement areas. Careful consideration should be given to the potential impact of landscaped areas and/or sprinkler systems on adjacent foundations, floor slabs, and pavements. The accumulated storm-water can be collected through a system of flow through planters and drained through solid pipes to the drainage points away from the slope or to the bottom of the slope.

An adequate subsurface drain system should be installed behind subsurface walls such as retaining walls or foundation walls. In addition, a parameter footing drainage system should be installed. All gunnite excavation walls should be constructed with a vertical drainage system.

5.4 Construction Monitoring

It is recommended that PSI be retained to examine and identify exposures created during project excavations in order to verify that soil and rock conditions are as anticipated. We further recommend that the structural fills be continuously observed and tested by our representative in order to evaluate the thoroughness and uniformity of their compaction. Samples of fill materials should be submitted to our laboratory for evaluation prior to placement of fills on site.

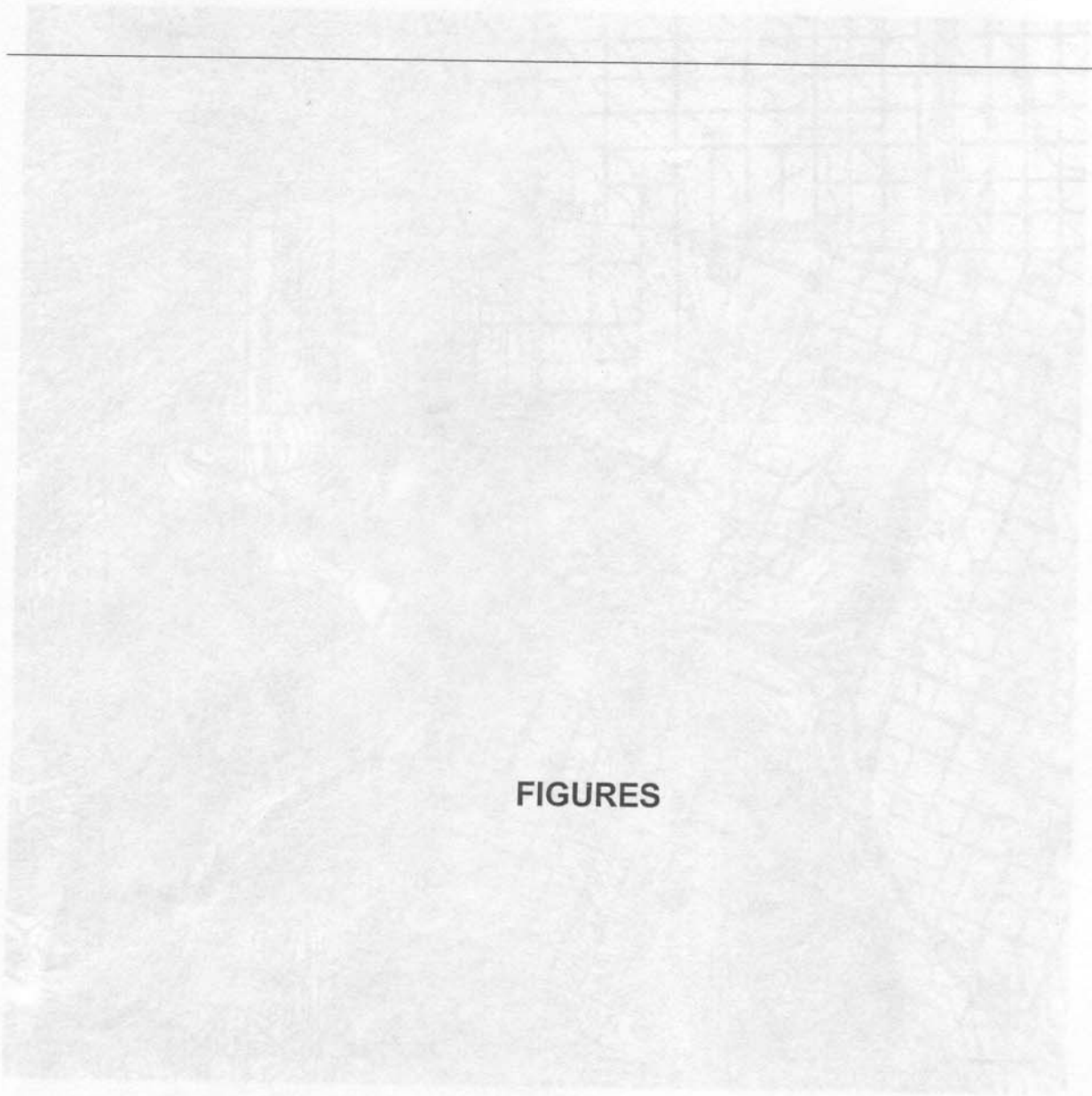
It is also recommended that PSI be retained to provide observation and testing of construction activities involved in the foundation, earthwork, and related activities of this project. PSI cannot accept any responsibility for any conditions which deviate from those described in this report, nor for the performance of the foundation, if not engaged to also provide construction observation and testing for this project.

6.0 REPORT LIMITATIONS

The recommendations submitted in this report are based on the available subsurface information obtained by PSI and design details furnished by the client and his architect/engineer for the proposed project. If there are any revisions to the plans for this project, or if deviations from the subsurface conditions noted in this report are encountered during construction, PSI should be notified immediately to determine if changes in the foundation and/or pavement recommendations are required. If PSI is not retained to review these changes, PSI will not be responsible for the impact of those conditions on the project.

The geotechnical engineer warrants that the findings, recommendations, specifications, or professional advice contained herein have been made in accordance with generally accepted professional geotechnical engineering practices in the local area. No other warranties are implied or expressed.

After the plans and specifications are more complete, the geotechnical engineer should be retained and provided the opportunity to review the final design plans and specifications to check that our engineering recommendations have been properly incorporated into the design documents. This report has been prepared for the exclusive use of St. Andrews Development for the specific application to the proposed multi-story condominium unit at the intersection of Southwest 18th Avenue and Southwest Mill Street Terrace in Portland, Oregon



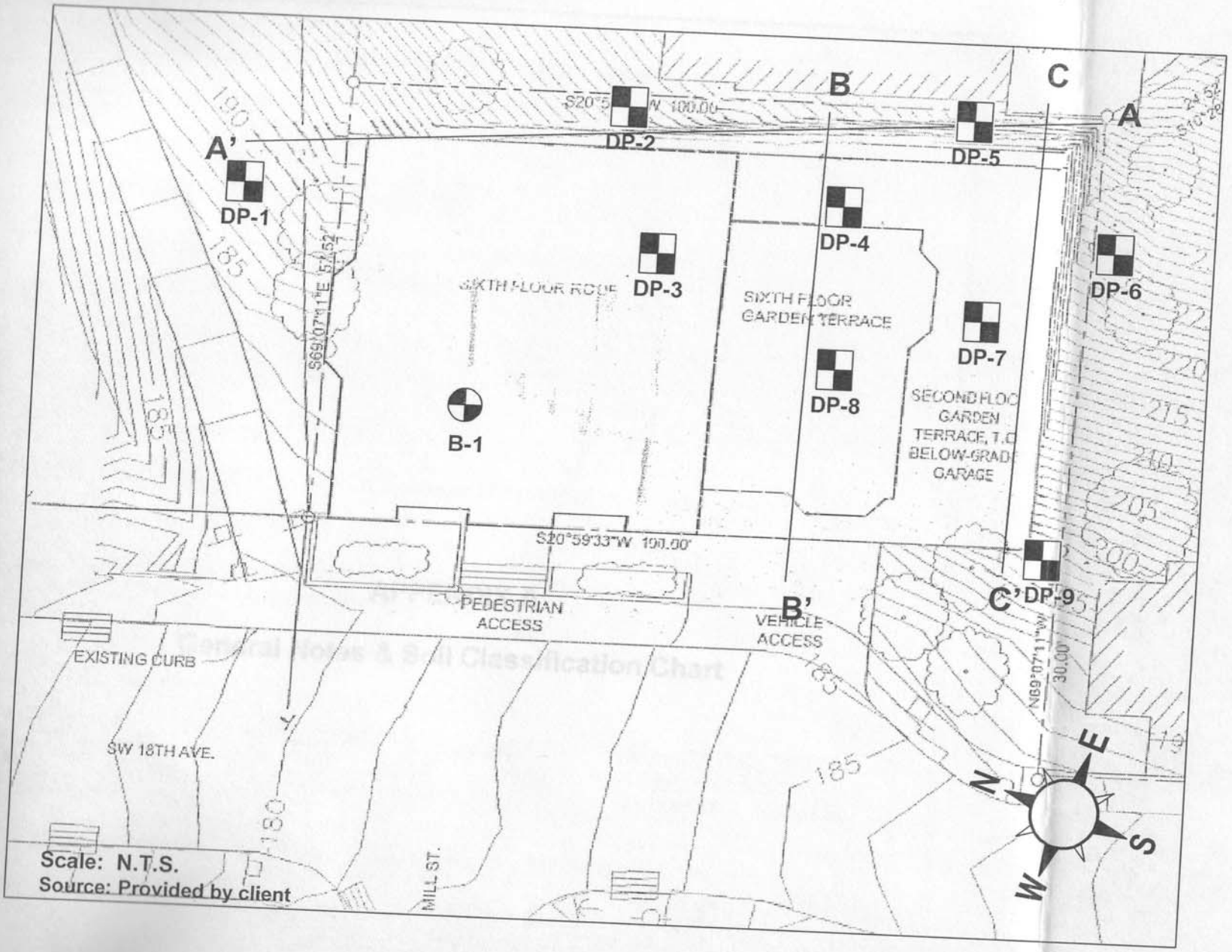
FIGURES

FIGURE 1: SITE LOCATION PLAN

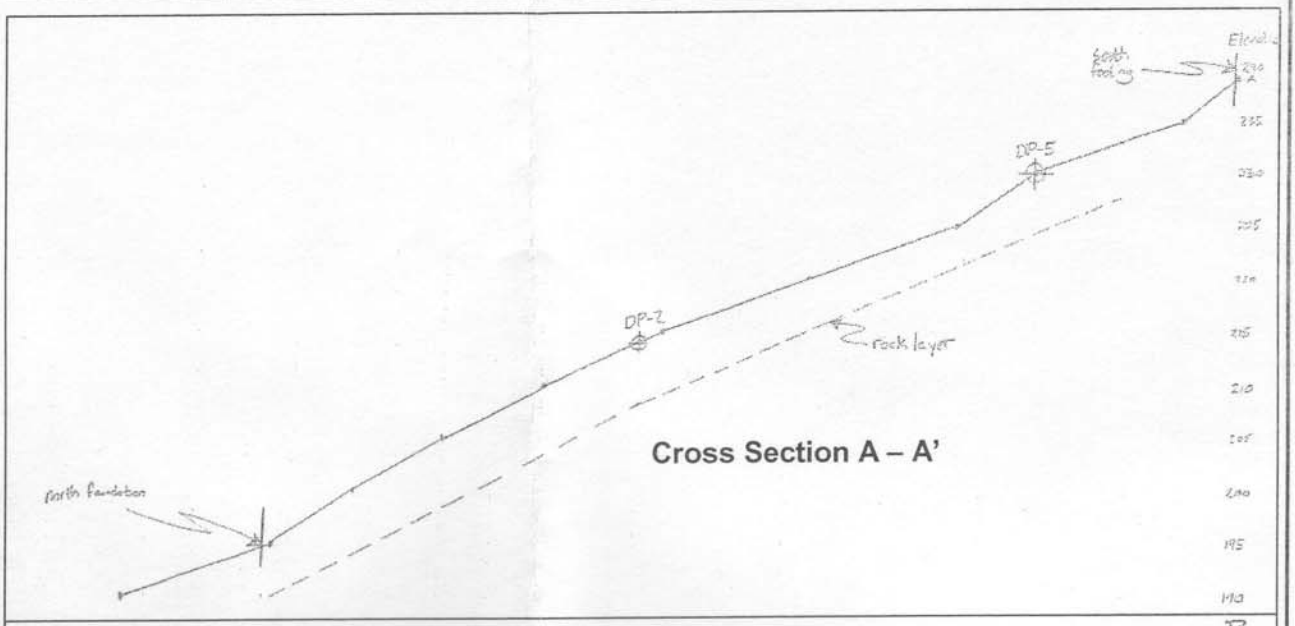
Source: www.TerraServer-USA.com

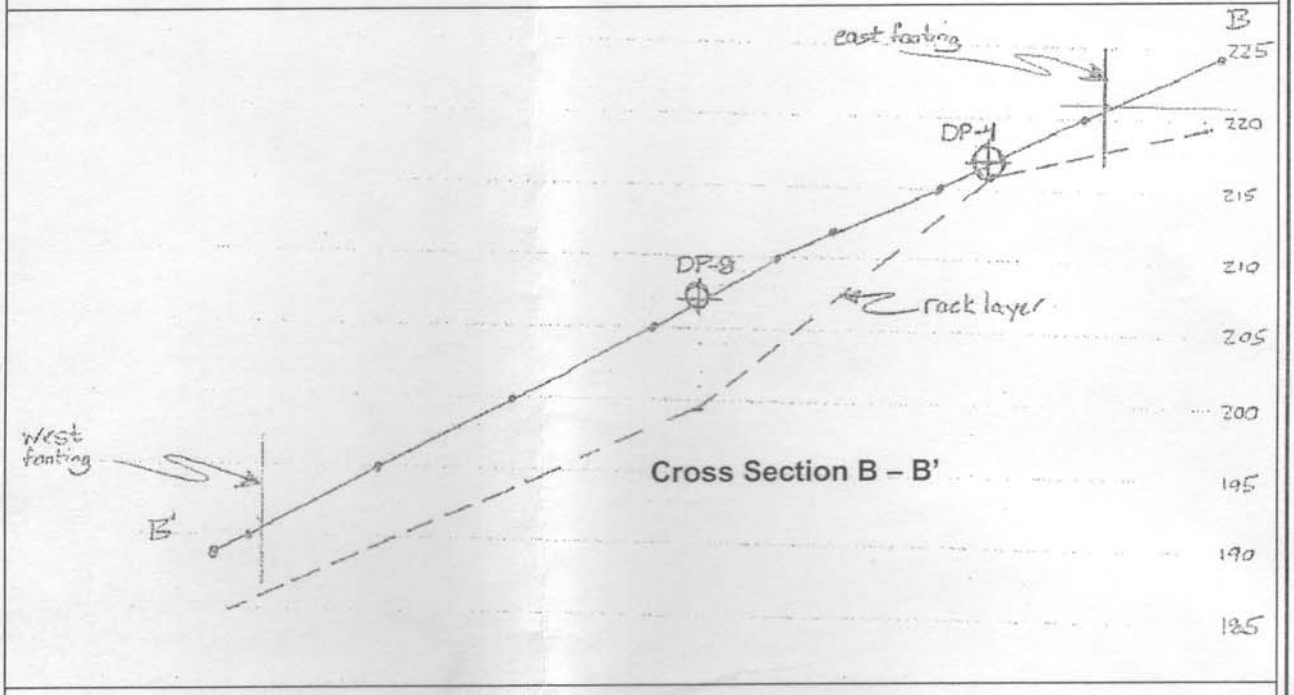
psi Information To Build On Engineering • Consulting • Testing	**Project:** **St. Andrews Condominium Development** **SW18th Avenue and SW Mill Street Terrace** **Portland, Oregon**	**File No.** **704-85045-1** **Date:** **4/3/08**

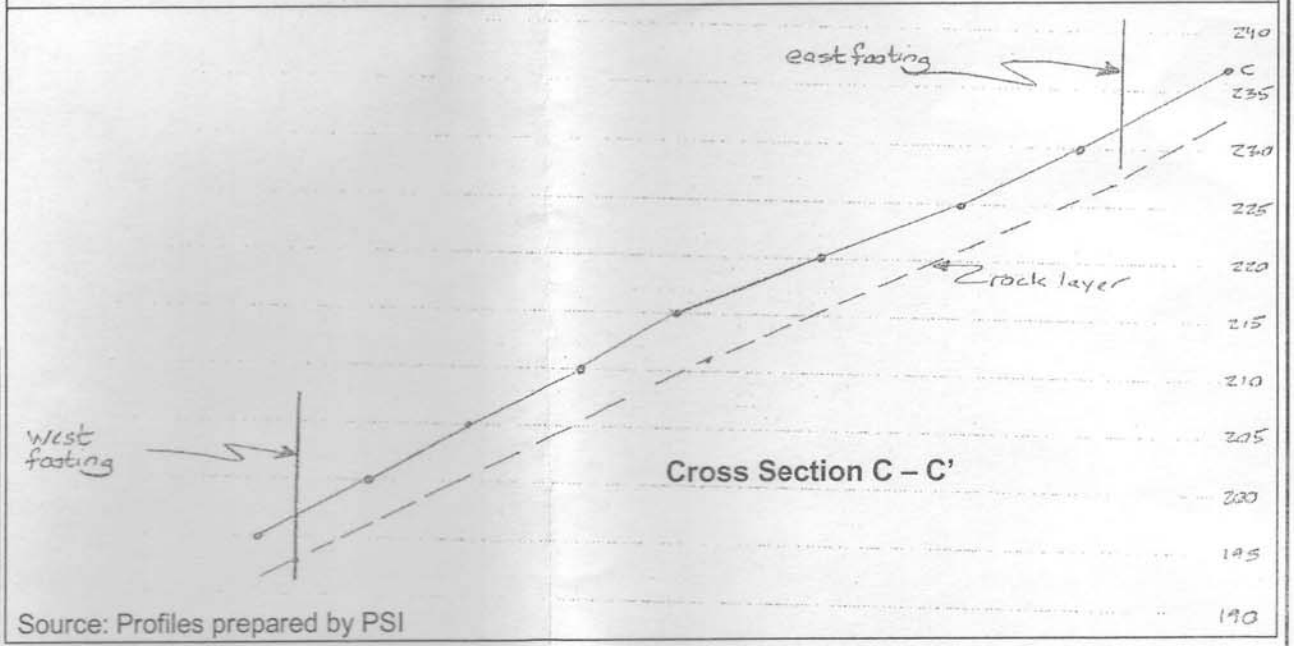
FIGURE 2
BORING AND DRIVE PROBE LOCATION PLAN



Scale: N.T.S.
Source: Provided by client







Source: Profiles prepared by PSI

No:	Date:
6	4/3/08

APPENDIX A

General Notes & Soil Classification Chart

GENERAL NOTES

SAMPLE IDENTIFICATION

The Unified Soil Classification System is used to identify the soil unless otherwise noted.

SOIL PROPERTY SYMBOLS

N: Standard "N" penetration: Blows per foot of a 140 pound hammer falling 30 inches on a 2-inch O.D. split-spoon.

Qu: Unconfined Compressive Strength, TSF.

Qp: Penetrometer value, unconfined compressive strength, TSF.

Mc: Water Content, %.

LL: Liquid Limit, %.

PI: Plasticity Index, %.

δd: Natural Dry Density, PCF.

▼ Apparent Groundwater Level at time noted after completion of boring.

DRILLING AND SAMPLING SYMBOLS

SS: Split-Spoon – 1 3/8" I.D., 2" O.D., except where noted.
ST: Shelby Tube – 3" O.D., except where noted.
AU: Auger Sample.
DB: Diamond Bit.
CB: Carbide Bit.
WS: Washed Sample.

TERM (NON-COHESIVE SOILS)	STANDARD PENETRATION RESISTANCE (SAFETY HAMMER)	STANDARD PENETRATION RESISTANCE (AUTOMATIC HAMMER)
Very Loose	0-4	0-3
Loose	4-10	3-7
Medium	10-30	7-20
Dense	30-50	20-33
Very Dense	Over 50	Over 33

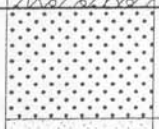
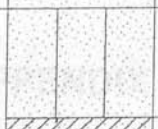
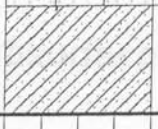
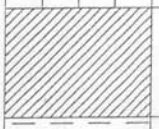
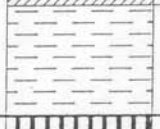
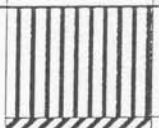
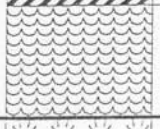
TERM (COHESIVE SOILS)	Qu – (TSF)
Very Soft	0-0.25
Soft	0.25-0.50
Firm (Medium)	0.50-1.00
Stiff	1.00-2.00
Very Stiff	2.00-4.00
Hard	4.00+

PARTICLE SIZE

Boulders	8 in.+	Coarse Sand	5mm-0.6mm	Silt	0.074mm-0.005mm
Cobbles	8 in.-3 in.	Medium Sand	0.6mm-0.2mm	Clay	-0.005mm
Gravel	3 in.-5mm	Fine Sand	0.2mm-0.074mm		

SOIL CLASSIFICATION CHART

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

MAJOR DIVISIONS			SYMBOLS		TYPICAL DESCRIPTIONS
			GRAPH	LETTER	
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	CLEAN GRAVELS		GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
	MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	(LITTLE OR NO FINES)		GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
		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
	MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	(LITTLE OR NO FINES)		SP	POORLY-GRADED SANDS, GRAVELLY SAND, LITTLE OR NO FINES
		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

psi

APPENDIX B

Record of Subsurface Exploration

LOG OF TEST BORING NO. B-1

CLIENT: **St. Andrews Development**

PROJECT: **St. Andrews Condominiums**

LOCATION: SW 18th and SW Mill St. Terrace, Portland, OR

PSI PROJECT NUMBER: 704-85036

DATE OF EXPLORATION: 3/24/2008

EQUIPMENT: CME-75 Hollow Stem Auger w/Auto SPT Hammer

LOGGED BY: T. Carlson

BORING LOCATION: See Boring/Drive Probe Location Plan

SURF. ELEV.:' GROUNDWATER: ' TERMINATION DEPTH: 9'

The soil boring was backfilled with auger cuttings and granular bentonite at the end of exploration

DEPTH (FT)	SAMPLES	RECOVERY	SYMBOL	U.S.C.S. CLASS	SOIL DESCRIPTION	MOISTURE CONTENT(%)	% PASSING #200 SIEVE	DRY UNIT WEIGHT (PCF)	BLOWS/6"	POCKET PEN (TSF)	TORVANE SHEAR (TSF)	LIQUID LIMIT	PLASTIC LIMIT
0	SPT 1				**FILL-** silty gravel/gravelly silt, brown, moist, wood layer at 4 feet, very dense at surface, becomes soft at 2.5 feet bgs				18-35/4"				
	SPT 2								5-2-2				
5	SPT 3				**CLAYEY SILT W/GRAVEL-**red-brown, moist to wet, soft to medium stiff				3-1-2				
	SPT 4				**DECOMPOSED ROCK-**heavily weathered, orange-brown, some original fabric-strong rock strength, some vesicles, very dense				13-35/4"				
	SPT 5								17-50/3"				
10					Auger refusal at 9 feet below ground surface. Groundwater not encountered during site explorations.								
15													

PENETRATION RESISTANCE
(blows/foot)
140 pound hammer/30 inch drop

5 10 20 30 40 50 60

BL_PDX_DCP 704-85036.GPJ CURRENT PORTLAND GEOTECH TEMPLATE.GDT 4/4/08

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