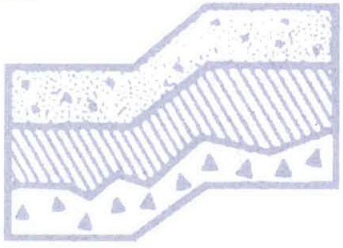
GEOTECHNICAL REPORT

Patton Property
5201-5205 Rainier Avenue South
Seattle, Washington

Project No. T-5982-2



Terra Associates, Inc.

Prepared for:

Mr. Murray Kahn
Seattle, Washington

April 12, 2007

TERRA ASSOCIATES, Inc.

Consultants in Geotechnical Engineering, Geology
and
Environmental Earth Sciences

April 12, 2007
Project No. T-5982-2

Mr. Murray Kahn
2291 NE 60th Street
Seattle, Washington 98115

Subject: Geotechnical Report
Patton Property
5201-5205 Rainier Avenue South
Seattle, Washington

Dear Mr. Kahn:

As requested, we have conducted a geotechnical engineering study for the subject site. The attached report presents our findings and recommendations for the geotechnical aspects of project design and construction.

Soil conditions at the site generally consist of four to eight feet of fill overlying native very soft to very stiff silty clay and clayey silt. In our opinion, the existing uncontrolled fill and very soft silty clay is compressible and will not provide suitable support for spread footing foundations. Given the fill and very soft native soils depth, we recommend supporting the building on piles or on ground conditions improved by the installation of Geopiers.

Detail recommendations regarding foundation support and other geotechnical design considerations are outlined in the attached report.

We trust the information presented is sufficient for your current needs. If you have any questions or require additional information, please call.

Sincerely yours,
TERRA ASSOCIATES, INC.

Thein Aung, M.Sc., P.Eng., F.G.S.
Geotechnical Engineer

4-12-07

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1.0 PROJECT DESCRIPTION

The project will consist of redeveloping the property with a four-story multi-use building. Our review of a site plan provided to us, shows that the proposed building footprint will cover an area of approximately 15,500 square feet. The lower floor elevation for the building will be at Elev. 118 to Elev. 120 feet. This lower-level will be used primarily for parking with some retail and office along the northeast building perimeter. The upper building levels will be constructed for residential use. Grading to establish the desired floor elevation will require excavation of about two to six feet below current grades in the central and southern portions of the property.

Specific building design information was not available at the time of this report. However, based on our experience, we expect the building perimeter wall loads will be approximately 4 to 8 kips per foot, with interior isolated columns carrying loads of 400 to 500 kips. We also expect that lower slab-on-grade floor will carry non-uniform floor loads of between 100 and 200 pounds per square foot (psf).

The recommendations in the following sections of this report are based upon our understanding of the design features. If actual features vary or changes are made, we should review them in order to modify our recommendations, as required. We should review final design drawings and specifications to verify that our recommendations have been properly interpreted and incorporated into project design.

2.0 SCOPE OF WORK

Our work was completed in accordance with our scope of work, as outlined in our proposal, dated January 30, 2007. On March 14, 2007, we drilled 4 test borings at the site to a maximum depth of 56.5 feet below existing surface grades. Using this information, we performed analyses to develop geotechnical recommendations for project design and construction. Specifically, this report addresses the following:

- Soil and groundwater conditions
- Seismic
- Site preparation and grading
- Excavations
- Foundation design criteria
- Floor slabs
- Lateral earth pressures for wall design
- Subsurface drainage
- Utilities

It should be noted that recommendations outlined in this report regarding drainage are associated with soil strength, design earth pressures, erosion, and stability. Design and performance issues with respect to moisture as it relates to the structure environment (i.e., humidity, mildew, mold) is beyond Terra Associates' purview. A building envelope specialist or contractor should be consulted to address these issues, as needed.

3.0 SITE CONDITIONS

3.1 Surface

The site is a triangular shaped parcel located between 39th Avenue South and Rainier Avenue South in Seattle, Washington. The approximate site location is shown on attached Figure 1.

Four small structures associated with an auto repair shop occupy the south and southwestern portion of the property. The remaining area of the site is paved with asphalt concrete pavement. Site grades slope to the north with overall relief on the order of 12 feet.

3.2 Subsurface

The soils observed in the test borings consisted of approximately four to eight feet of fill composed of soft silty clay to clayey silt with occasional organics. Very soft to very stiff clayey silt to silt, was found underlying the surficial fill to the boring termination depths.

The Boring Logs in Appendix A present more detailed descriptions of the subsurface conditions encountered. The approximate boring locations are shown on Figure 2.

3.3 Groundwater

We observed what appears to be perched groundwater at depths of 10 feet and 15 feet below current site grades at Borings B-1 and B-2, respectively. Evidence of groundwater was not observed in Test Borings B-3 and B-4 located in the southern site area.

3.4 Seismic Considerations

Based on soil conditions observed in the test borings and our knowledge of the area geology, per Chapter 16 of the 2003 International Building Code (IBC), site class "D" should be used in structural design. Based on this site class, the following parameters should be used in computing seismic forces in accordance with the 2003 IBC:

Seismic Design Parameters (IBC 2003)

Spectral Response Acceleration (Short Period), Ss	1.530
Spectral Response Acceleration (1 - Second Period), S1	0.525

Seismic Design Parameters (IBC 2003) (continued)

Site Coefficient, Fa	1.000
Site Coefficient, Fv	1.500
Five Percent Damped .2 second period, Sds	1.020
Five Percent Damped 1.0 second period, Sd1	0.525

Soil Liquefaction

Liquefaction is a phenomenon where there is a reduction or complete loss of soil strength due to an increase in pore water pressure induced by vibrations from a seismic event. Liquefaction mainly affects geologically recent deposits of fine-grained sands that are below the groundwater table. Soils of this nature derive their strength from intergranular friction. The generated water pressure or pore pressure essentially separates the soil grains and eliminates this intergranular friction; thus, eliminating the soil's strength. Silts and clays that exhibit undrained cohesive strength typically are not affected by the liquefaction phenomenon.

Given soil conditions observed at the test borings, in our opinion, the risk for liquefaction occurring at this site and its associated affects on structures is negligible.

4.0 DISCUSSION AND RECOMMENDATIONS

4.1 General

Based on our study, in our opinion, there are no geotechnical constraints that would preclude development, as proposed. However, with expected building loads, the upper four- to eight-foot thick layer of uncontrolled compressible clayey silt fill and the underlying soft to very soft clayey silt will consolidate and result in excessive settlement of the building if supported on conventional spread footing foundations. Therefore, we recommend supporting the building on a pile foundation. Alternatively, conventional spread footing foundations could be used with support derived on ground conditions improved with Geopiers.

Considering the relatively light floor loading, proposed design grades, and existing site use, structural support of the lower floor slab should not be required and the floor could be constructed at grade following some improvement in the soil subgrade.

The following sections provide recommendations regarding these issues and other geotechnical design considerations. The recommendations presented should be incorporated into the final design drawings and construction specifications.

4.2 Site Preparation

To prepare the site for new development, demolition activities should include the complete removal of existing building foundations and floor slabs. Buried utilities and other subterranean structures that will be abandoned should be excavated and removed with the grade restored with structural fill.

Following demolition, grading to establish the desired building elevations can be initiated. Soils below the existing pavement surface and buildings are indicated to consist of soft dark brown to black silty clay/clayey silt fill. The soils will be easily disturbed by grading activities and will likely yield and pump significantly under construction traffic loading especially during the normal wet winter season. To prepare a stable subgrade suitable for support of the building floor slab, we recommend excavating and removing the existing fill a minimum depth of 18 inches below the floor slab subgrade elevation and restoring the grade with clean granular structural fill meeting the requirements for wet weather structural fill as described below. Placement of a geotextile reinforcement fabric such as Mirafi 500X or equivalent on the exposed subgrade before filling to prevent fines contamination of the structural fill and to enhance bearing stability is also recommended.

A representative of Terra Associates, Inc. should observe the subgrade preparation to verify stable subgrade suitable for support of the building floor slab is achieved.

In our opinion, on-site existing fill or the underlying native soil will not be suitable for use as structural fill or backfill. We recommend that structural fill imported to the site consist of granular material meeting the following grading requirements:

U.S. Sieve Size	Percent Passing
3 inches	100
No. 4	75 maximum
No. 200	5 maximum*

*Based on the 3/4-inch fraction.

Prior to use, Terra Associates, Inc. should examine and test all materials proposed to be imported to the site for use as structural fill.

Structural fill should be placed in uniform loose layers not exceeding 12 inches and compacted to a minimum of 95 percent of the soil's maximum dry density, as determined by American Society for Testing and Materials (ASTM) Test Designation D-698 (Standard Proctor). The moisture content of the soil at the time of compaction should be within two percent of its optimum, as determined by this ASTM standard. In nonstructural areas or for backfill in utility trenches below a depth of 4 feet, the degree of compaction can be reduced to 90 percent. All structural fill located within City rights-of-way should be placed and compacted in accordance with City of Seattle specifications.

4.3 Excavations

The existing fill and underlying native soils are categorized as Type C soils, in accordance with Occupational Safety and Health Administration (OSHA) regulations. Accordingly, side slopes on temporary excavations greater than 4 feet and less than 20 feet deep should be established at a minimum inclination of 1.5:1 (Horizontal:Vertical).

The above information is provided solely for the benefit of the owner and other design consultants, and should not be construed to imply that Terra Associates, Inc. assumes responsibility for job site safety. Job site safety is the sole responsibility of the project contractor.

Soldier Pile Shoring

With existing grades in the southwestern corner of the site and proposed building elevations, it appears there will be insufficient room to slope excavations, as required without encroaching onto the adjacent south property and western street right-of-way. If excavation easements cannot be obtained temporary shoring would appear necessary. Conventional soldier piles and timber lagging can be used for this purpose. We recommend that the soldier piles have a maximum center-to-center spacing of eight feet. Soil parameters for design of the temporary shoring are presented on Figure 3. To account for arching effects, lateral loads on the lagging can be reduced by 50 percent. Unshored excavation heights should not exceed four feet. No part of the excavation should remain unsupported for over 24 hours.

A monitoring program should be implemented to verify the performance of the shoring system and the effects on adjacent properties. The first step in the monitoring program should consist of documenting the condition of the existing adjacent properties and pavements. This documentation should include a pictorial record.

Shoring system monitoring should be performed twice a week during excavations, then once a week after the excavations are complete. A registered land surveyor should be retained to obtain the baseline data and conduct the readings. The monitoring should include surveying the vertical and horizontal alignment of the top of every other soldier pile. The monitoring data should be reviewed weekly by the shoring designer and geotechnical engineer.

4.4 Foundations

As discussed earlier, the upper existing fills and underlying native silty clay are compressible and in our opinion, will not be suitable for support of conventional spread footing foundations for the expected building loads. Therefore, we recommend supporting the structure on a pile foundation. Considering the area development to avoid potential impacts to adjacent properties and noise disturbance associated with driven piles, we recommend the piles be constructed using augercast techniques.

4.4.1 Augercast Piles

We recommend augercast piles penetrate a minimum depth of five feet into the stiff to very stiff native clayey silt to silt stratum. The piles will derive their capacity through a combination of shaft adhesion/friction and end bearing. Based on test boring data, with the required penetration into the stiffer native soils maximum pile tip elevations equivalent to 40 feet below current site elevations should be planned.

With augercast piles advanced to bearing, as recommended allowable axial and lateral pile capacities for varying pile diameters are as follows:

Maximum Pile Capacities

Allowable Axial Capacity (tons)		Allowable Uplift Capacity (tons)	
Pile Diameter		Pile Diameter	
16 inches	18 inches	16 inches	18 inches
35	40	25	30

Full axial capacity can be used, provided the piles are spaced at a minimum of three pile diameters. Closer spacing in pile groups will require a reduction in the single-pile capacity. This reduction will depend on the number of piles in the pile group and the spacing used. The estimated pile settlement is one-fourth of an inch or less. Elastic shortening of the pile is not included in this value.

Lateral Pile Capacity

Lateral pile capacity analyses were performed for a single-pile foundation using the computer program *GEOPRO 4.0 – Laterally Loaded Pile*. The program uses the soil subgrade modulus to calculate the soil reactions under the lateral loads. Other parameters used in the calculations included pile length, diameter, elastic modulus, and moment of inertia.

The following table provides the lateral pile capacities for different pile head deflections assuming a free head condition:

Lateral Pile Capacities

Pile Head Deflection (inches)	Lateral Pile Load Capacity (kips)	
	Pile Diameter	
	16 inches	18 inches
0.25	9	12
0.5	18	24
0.75	27	36
1.0	36	48

In addition to the lateral pile capacities, lateral resistance will also be provided by earth pressure acting adjacent to the pile caps and grade beams. We recommend calculating the passive resistance using a pressure imposed by an equivalent fluid weight of 300 pcf. This value assumes the pile cap or grade beam will be backfilled with structural fill.

Construction Considerations

Augercast piles are formed by the pressure injection of grout through a hollow-stem auger that is slowly retracted from the ground after advancement to the recommended tip elevation. The grout pressure used will compress the soils within the immediate vicinity of the pile. This will increase, to some extent, the pile diameter and the amount of grout required to construct the pile. For planning purposes, we suggest considering a 50 percent increase in the amount of grout necessary to form the pile.

A higher-than-normal reliance on quality workmanship is required for successful installation of augercast piles. It is extremely important that the grout pressure be consistent and uniform during the installation, and that retraction of the auger occurs at a slow uniform pace beneath a sufficient head of grout in the pile column. The contractor should have adequate means for verifying grout pressure and estimating the volume of grout used in construction of the piles.

Because of the compression effects and the possible influence on adjacent pile construction, the installation sequence should be based on a minimum pile spacing of five pile diameters. Once the grout column has achieved its initial 24-hour set, pile construction between these spacings can be completed.

4.4.2 Geopiers

Considering the soil conditions and the moderate building loads expected, we believe that Geopiers could be an acceptable alternative to augercast pile foundations. Geopiers consist of aggregate columns that are densely compacted in predrilled holes, and are advanced through the unsuitable fill into competent underlying native soil. The procedure improves the foundation subgrade by effectively prestressing the soil both vertically and horizontally. After improvement in the ground conditions, building support can be provided using conventional spread footing foundations. Based on our experience, after installation of the Geopiers, allowable bearing capacity in the range of 4,000 to 5,000 pounds per square foot (psf) is available for dimensioning of conventional spread footings.

The geopier system is patented and exclusive rights for designing and building Geopiers are held by GEOPIER Foundation Company.

4.5 Slab-on-Grade Floors

As discussed earlier, in our opinion, the lower parking and retail floor slab can be constructed at grade following subgrade improvement as recommended in Section 4.2 "Site Preparation". In the northern portion of the site where site grades will be raised slightly, the weight of the added fill will cause some compression of the underlying fill and soft native soil. To mitigate potential impact to the finished floor slab, construction should be delayed to allow this settlement to take place. We estimate the time required for primary compression of the layer due to the fill weight will be about three weeks. Estimated potential floor slab settlement due to the weight of the slab and normal floor loading, following this pre-loading time period, is in the range of one-half to three-fourths inch. If settlement of this magnitude cannot be tolerated, the floor slabs should be structurally supported.

Immediately below the floor slab, we recommend placing a four-inch thick capillary break layer composed of clean, coarse sand or fine gravel that has less than three percent passing the No. 200 sieve. This material will reduce the potential for upward capillary movement of water through the underlying soil and subsequent wetting of the floor slab.

The capillary break layer will not prevent moisture intrusion through the slab caused by water vapor transmission. Where moisture by vapor transmission is undesirable, such as covered floor areas, a common practice is to place a durable plastic membrane on the capillary break layer and then cover the membrane with a layer of clean sand or fine gravel to protect it from damage during construction, and aid in uniform curing of the concrete slab. It should be noted that if the sand or gravel layer overlying the membrane is saturated prior to pouring the slab, it will be ineffective in assisting uniform curing of the slab, and can actually serve as a water supply for moisture seeping through the slab and affecting floor coverings. Therefore, in our opinion, covering the membrane with a layer of sand or gravel should be avoided if floor slab construction occurs during the wet winter months and the layer cannot be effectively drained. We recommend floor designers and contractors refer to the 2003 American Concrete Institute (ACI) Manual of Concrete Practice, Part 2, 302.1R-96, for further information regarding vapor barrier installation below slab-on-grade floors.

4.6 Lower-Level Building Walls

The magnitude of earth pressure development on lower-level building walls will partly depend on the quality of the wall backfill. We recommend placing and compacting wall backfill as structural fill. Below improved areas such as pavements or floor slabs, the backfill should be compacted to a minimum of 95 percent of its maximum dry unit weight, as determined by ASTM Test Designation D-698 (Standard Proctor). In unimproved areas, the relative compaction can be reduced to 90 percent. To guard against hydrostatic pressure development, wall drainage must also be installed. A typical recommended wall drainage detail is shown on Figure 4.

With wall backfill placed and compacted as recommended and drainage properly installed, we recommend designing restrained lower-level building walls for an earth pressure equivalent to a fluid weighing 40 pcf. For evaluation of the wall performance under seismic loading, an additional uniform lateral pressure equivalent to 8H psf, where H is the height of the below-grade portion of the wall, can be used. These values assume a horizontal backfill condition and that no other surcharge loading, such as traffic, sloping embankments, or adjacent buildings, will act on the wall. If these loads are present they must be included in the wall design.

4.7 Drainage

Surface

Surface gradients should be created to direct runoff away from the building and should drain towards suitable discharge facilities. With the exception of paved areas adjacent to the structure, we recommend providing a gradient of at least three percent for a minimum distance of ten feet from the building perimeters. In paved areas adjacent to the building, a minimum gradient of one percent should be provided, unless provisions are included for collection and disposal of surface water adjacent to the structure.

Subsurface

We recommend installing a continuous subsurface drain along the outside lower edge of the perimeter building foundations. The foundation drains and roof downspouts should be tightlined separately to an approved discharge facility. Subsurface drains must be laid with a gradient sufficient to promote positive flow to a controlled point of approved discharge.

4.8 Utilities

Where utility lines are to be excavated and installed in the paved areas, we recommend placing all bedding and backfill in accordance with American Public Works Association (APWA) and all applicable local and state specifications. As a minimum, backfill placement and compaction should be in accordance with the recommendations given in Section 4.2 of this report.

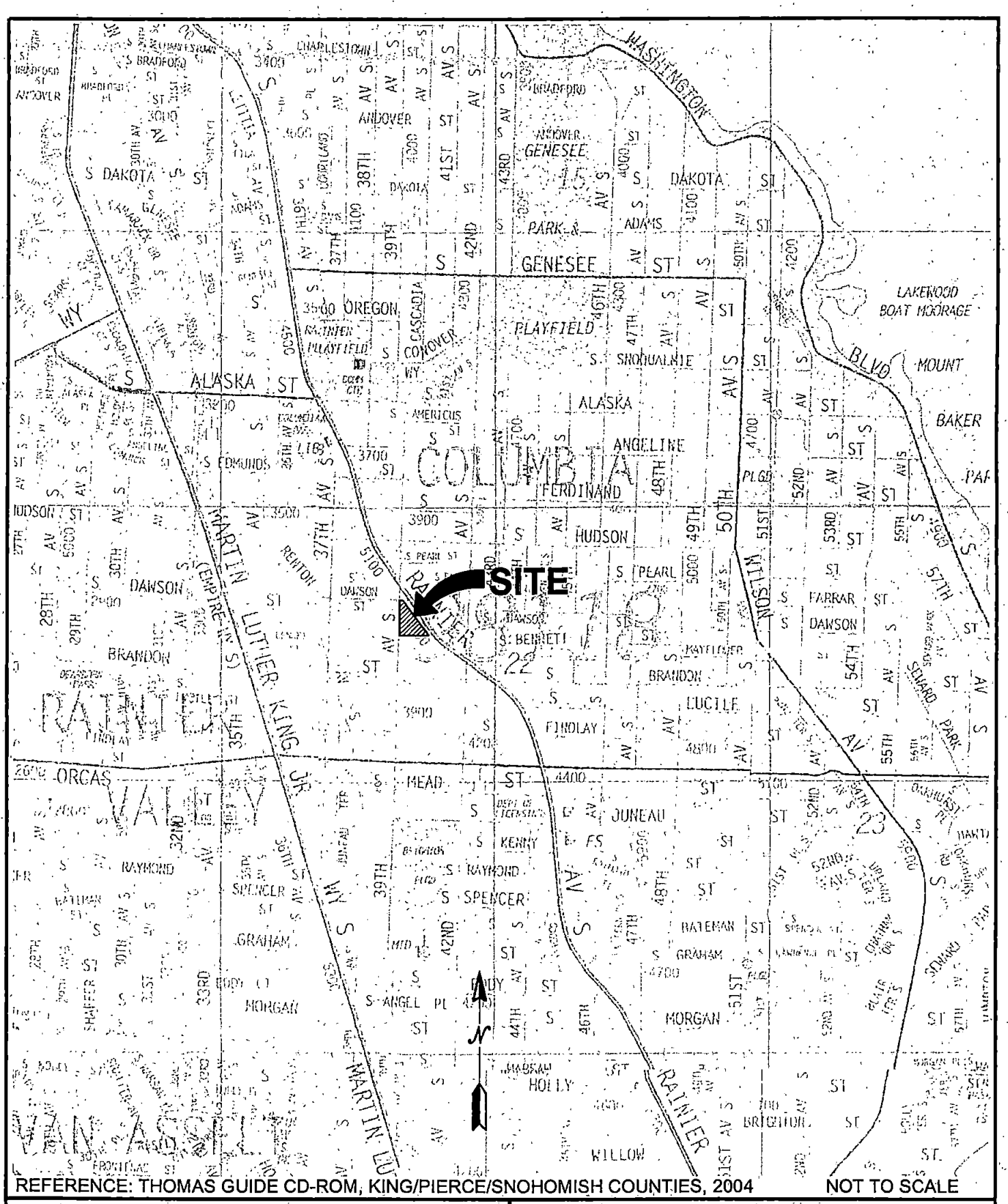
5.0 ADDITIONAL SERVICES

Terra Associates, Inc. should review the final design and specifications in order to verify that earthwork and foundation recommendations have been properly interpreted and incorporated into project design and construction. We should also provide geotechnical services during construction in order to observe compliance with the design concepts, specifications, and recommendations. This will allow for design changes if subsurface conditions differ from those anticipated prior to the start of construction.

6.0 LIMITATIONS

We prepared this report in accordance with generally accepted geotechnical engineering practices. This report is the property of Terra Associates, Inc. and is intended for specific application to the Patton Property project in Seattle, Washington. This report is for the exclusive use of Mr. Murray Kahn and his authorized representatives. No other warranty, expressed or implied, is made.

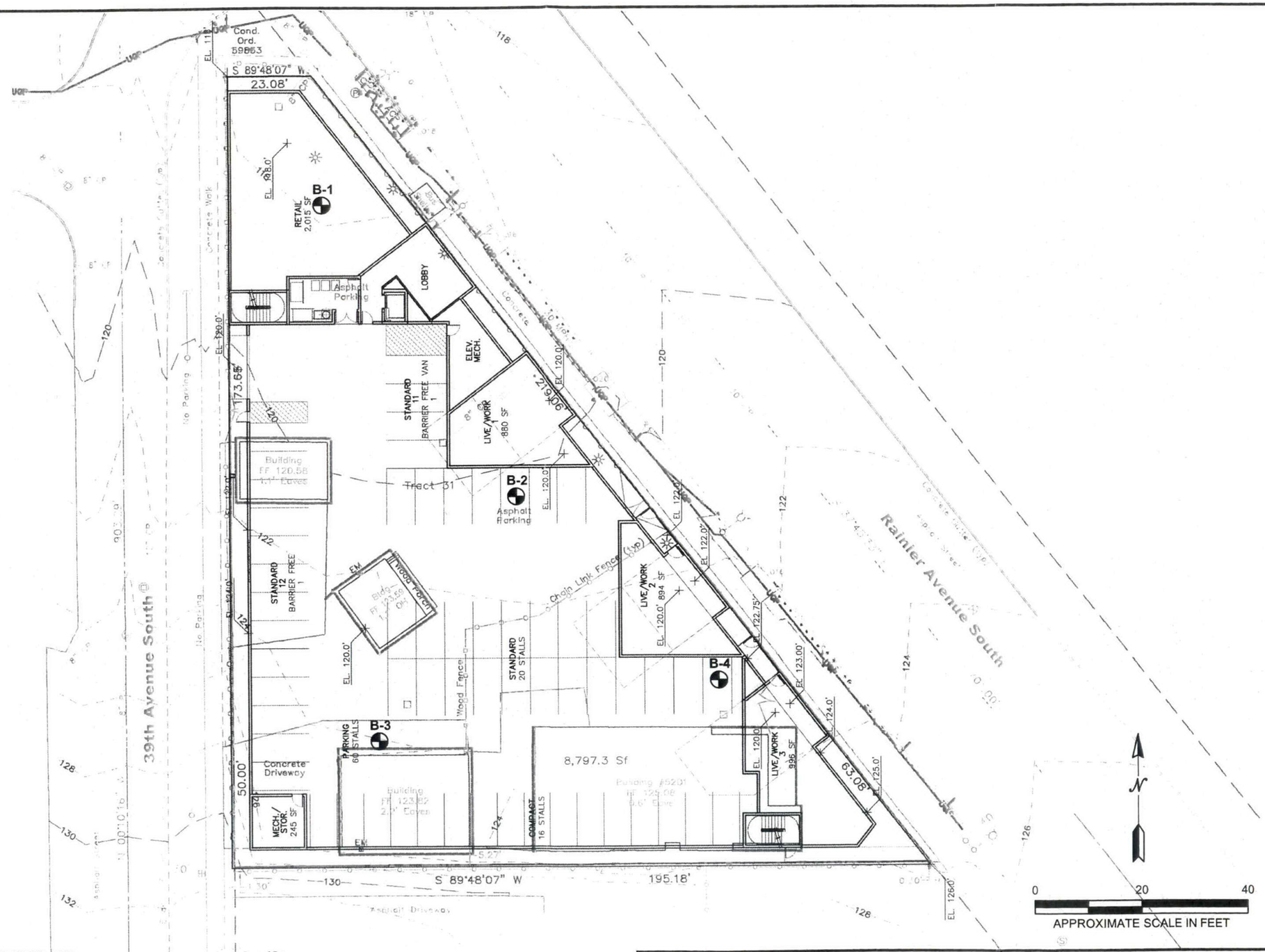
The analyses and recommendations presented in this report are based upon data obtained from the test borings excavated on-site. Variations in soil conditions can occur, the nature and extent of which may not become evident until construction. If variations appear evident, Terra Associates, Inc. should be requested to reevaluate the recommendations in this report prior to proceeding with construction.



Terra Associates, Inc.
Consultants in Geotechnical Engineering
Geology and
Environmental Earth Sciences

VICINITY MAP
PATTON PROPERTY
SEATTLE, WASHINGTON

Proj. No.T-5982-2	Date APR 2007	Figure 1



NOTE:

THIS SITE PLAN IS SCHEMATIC. ALL LOCATIONS AND DIMENSIONS ARE APPROXIMATE. IT IS INTENDED FOR REFERENCE ONLY AND SHOULD NOT BE USED FOR DESIGN OR CONSTRUCTION PURPOSES.

REFERENCE:

SITE PLAN PROVIDED BY CLIENT

LEGEND:

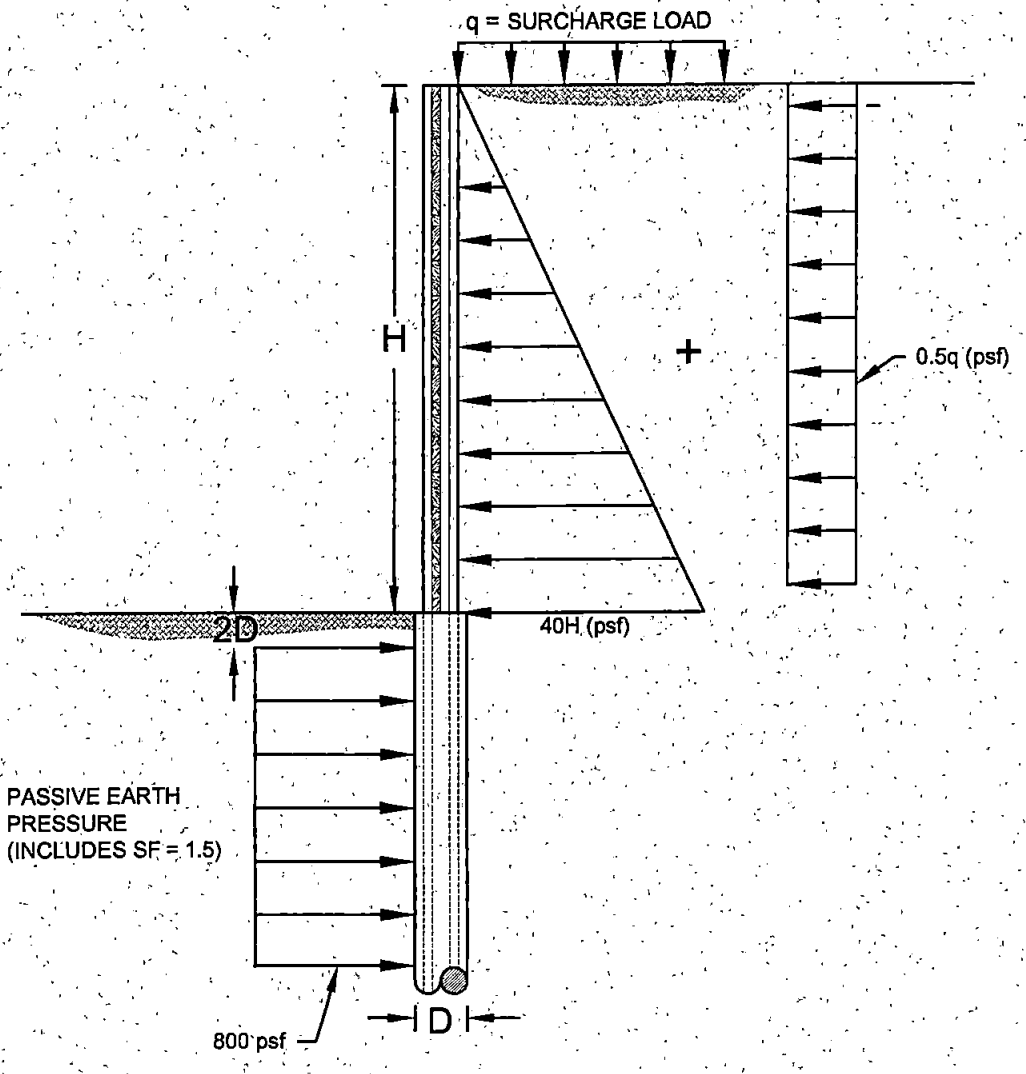
APPROXIMATE BORING LOCATION

Terra Associates, Inc.
Consultants in Geotechnical Engineering Geology and Environmental Earth Sciences

EXPLORATION LOCATION PLAN
PATTON PROPERTY
SEATTLE WASHINGTON

Proj. No.T-5982-2	Date APR 2007	Figure 2

EARTH PRESSURE DIAGRAM
CANTILEVER SOLDIER PILE WALL



NOT TO SCALE

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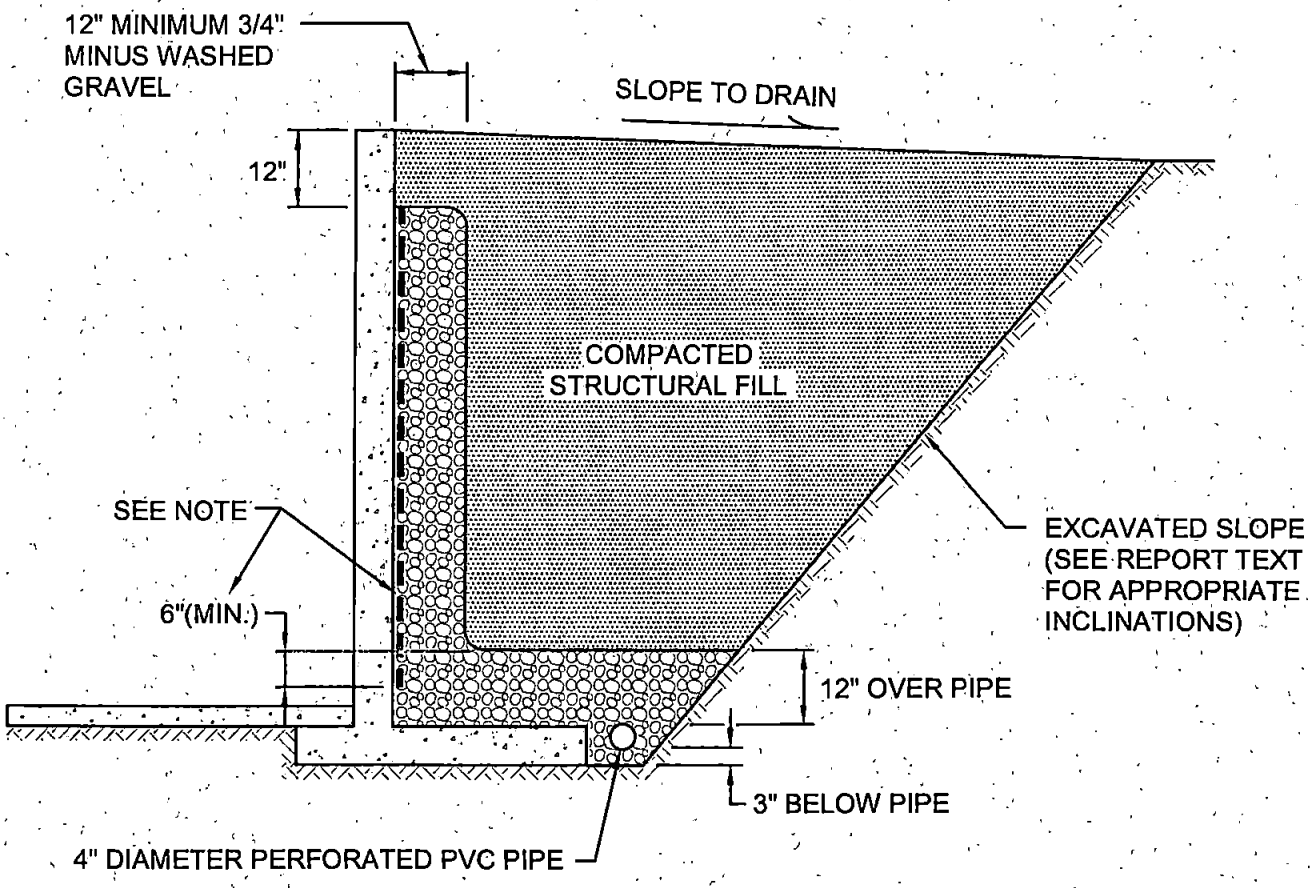
EARTH PRESSURE DIAGRAM
PATTON PROPERTY
SEATTLE, WASHINGTON

Proj. No. T-5982-2	Date APR 2007	Figure 3



NOT TO SCALE

NOTE:

MIRADRAIN G100N PREFABRICATED DRAINAGE PANELS OR SIMILAR PRODUCT CAN BE SUBSTITUTED FOR THE 12-INCH WIDE GRAVEL DRAIN BEHIND WALL. DRAINAGE PANELS SHOULD EXTEND A MINIMUM OF SIX INCHES INTO 12-INCH THICK DRAINAGE GRAVEL LAYER OVER PERFORATED DRAIN PIPE.

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Geology and
Environmental Earth Sciences

TYPICAL WALL DRAINAGE DETAIL
PATTON PROPERTY
SEATTLE, WASHINGTON

Proj. No.T-5982-2	Date SEPT 2007	Figure 4

APPENDIX A
FIELD EXPLORATION AND LABORATORY TESTING

Patton Property
Seattle, Washington

On March 14 and 15, 2007, we observed the drilling of 4 borings to a maximum depth of 56.5 feet below current site grades. Boring locations were determined in the field by measurements from existing site features and buildings. The approximate location of the test borings are shown on the attached Exploration Location Plan, Figure 2. Test Borings Logs are attached as Figures A-2 through A-5.

A geotechnical engineer from our office conducted the field exploration and classified the soil conditions encountered, maintained a log of each test boring, obtained representative soil samples, and observed pertinent site features. During drilling, soil samples were obtained in general accordance with ASTM Test Designation D-1586. Using this procedure, a 2-inch (outside diameter) split barrel sampler is driven into the ground 18 inches using a 140-pound hammer free falling a height of 30 inches. The number of blows required to drive the sampler 12 inches after an initial 6-inch set is referred to as the Standard Penetration Resistance value or N value. This is an index related to the consistency of cohesive soils and relative density of cohesionless materials. N values obtained for each sampling interval are recorded on the Boring Logs, Figures A-2 through A-5. All soil samples were visually classified in accordance with the Unified Soil Classification System (USCS) described on Figure A-1.

Representative soil samples obtained from the test borings were placed in closed containers and taken to our laboratory for further examination and testing. The moisture content of each sample was measured and is reported on the individual Monitoring Well and Boring Logs. Atterberg Limits were determined on seven of the samples. The Atterberg Limit test results are shown on the Boring Logs, opposite the samples on which they were determined.

MAJOR DIVISIONS			LETTER SYMBOL	TYPICAL DESCRIPTION
COARSE GRAINED SOILS More than 50% material larger than No. 200 sieve size	GRAVELS More than 50% of coarse fraction is larger than No. 4 sieve	Clean Gravels (less than 5% fines)	GW	Well-graded gravels, gravel-sand mixtures, 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, non-plastic fines.
			GC	Clayey gravels, gravel-sand-clay mixtures, plastic fines.
	SANDS More than 50% of coarse fraction is smaller than No. 4 sieve	Clean Sands (less than 5% fines)	SW	Well-graded sands, gravelly sands, little or no fines.
			SP	Poorly-graded sands or gravelly sands, little or no fines.
		Sands with fines	SM	Silty sands, sand-silt mixtures, non-plastic fines.
			SC	Clayey sands, sand-clay mixtures, plastic fines.
FINE GRAINED SOILS More than 50% material smaller than No. 200 sieve size	SILTS AND CLAYS Liquid limit is less than 50%		ML	Inorganic silts, rock flour, clayey silts with slight plasticity.
			CL	Inorganic clays of low to medium plasticity, (lean clay).
			OL	Organic silts and organic clays of low plasticity.
	SILTS AND CLAYS Liquid limit is greater than 50%		MH	Inorganic silts, elastic.
			CH	Inorganic clays of high plasticity, fat clays.
			OH	Organic clays of high plasticity.
HIGHLY ORGANIC SOILS			PT	Peat.

DEFINITION OF TERMS AND SYMBOLS

COHESIONLESS

Density	Standard Penetration Resistance in Blows/Foot
Very loose	0-4
Loose	4-10
Medium dense	10-30
Dense	30-50
Very dense	>50

COHESIVE

Consistency	Standard Penetration Resistance in Blows/Foot
Very soft	0-2
Soft	2-4
Medium stiff	4-8
Stiff	8-16
Very stiff	16-32
Hard	>32

- 2" OUTSIDE DIAMETER SPLIT SPOON SAMPLER
- 2.4" INSIDE DIAMETER RING SAMPLER OR SHELBY TUBE SAMPLER
- ▼ WATER LEVEL (DATE)
- Tr TORVANE READINGS, tsf
- Pp PENETROMETER READING, tsf
- DD DRY DENSITY, pounds per cubic foot
- LL LIQUID LIMIT, percent
- PI PLASTIC INDEX
- N STANDARD PENETRATION, blows per foot

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UNIFIED SOIL CLASSIFICATION SYSTEM
PATTON PROPERTY
SEATTLE, WASHINGTON

LOG OF BORING NO. B-1

Figure No. A-2

Project: Patton Property **Project No:** T-5982-2 **Date Drilled:** March 14, 2007

Client: _____ **Driller:** Holocene **Logged By:** TA

Location: Seattle, Washington **Approx. Elev:** 118

Depth (ft)	Sample Interval	Soil Description	Consistency/ Relative Density	Moisture Content % Wp |-----x-----| Wl 10 30 50 70 90	Pocket Penetrometer TSF 1 2 3 4 / SPT (N) Blows/ft 10 20 30 40	Monitor Well
1–6		2 inches ASPHALT CONCRETE over FILL: dark brown to black clayey silt to silty clay, moist. (ML-CL)	Soft to Medium Stiff	40.1 at 3 ft; 37.8 at 6 ft	2 at 3 ft; 5 at 6 ft	
7–16		Gray clayey SILT, moist to wet. (ML) Oxidation staining below 10 feet. One-half inch sand seam at 12 feet.	Medium Stiff Very Soft	34.1 at 8 ft; 41.5 at 11 ft; 40.2 at 14 ft; 52.7 at 15 ft	5 at 8 ft; 3 at 11 ft; 1 at 14 ft; 0 at 16 ft	
17–26		Gray clayey SILT, moist to wet. (ML) Wet sand layer at 26 feet.	Very Soft Stiff	36.2 at 21 ft; 30.5 at 26 ft	1 at 21 ft; 9 at 26 ft	
27–30		Brown elastic SILT, moist. (MH) *Continued on Next Page.	Stiff			

Note: This borehole log has been prepared for geotechnical purposes. This information pertains only to this boring location and should not be interpeted as being indicative of other areas of the site.

Terra Associates, Inc.
Consultants in Geotechnical Engineering, Geology and Environmental Earth Sciences

LOG OF BORING NO. B-1

Figure No. A-2

Project: Patton Property **Project No:** T-5982-2 **Date Drilled:** March 14, 2007

Client: _____ **Driller:** Holocene **Logged By:** TA

Location: Seattle, Washington **Approx. Elev:** 118

Depth (ft)	Sample Interval	Soil Description	Consistency/ Relative Density	Moisture Content % Wp |-----x-----| Wl 10 30 50 70 90	Pocket Penetrometer △ TSF △ 1 2 3 4 ● SPT (N) Blows/ft ● 10 20 30 40	Monitor Well
31				31.2	12	
32						
33		Brown elastic SILT. (MH)				
34						
35				30.4	14	
36						
37						
38						
39			Stiff to			
40				24.2	27	
41		Sand pockets and sand seams below 40 feet.	Very Stiff			
42						
43						
44				20.8	42	
45						
46						
47						
48						
49						
50				28.1	29	
51						
52		Boring terminated at 51.5 feet. Groundwater encountered at 10 feet on 3-19-07. 2-inch diameter well installed to 25 feet.				
53						
54						
55						
56						
57						
58						
59						
60						

Note: This borehole log has been prepared for geotechnical purposes. This information pertains only to this boring location and should not be interpeted as being indicative of other areas of the site.

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LOG OF BORING NO. B-2

Figure No. A-3

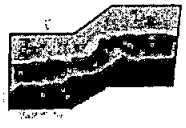
Project: Patton Property **Project No:** T-5982-2 **Date Drilled:** March 14, 2007

Client: _____ **Driller:** Holocene **Logged By:** TA

Location: Seattle, Washington **Approx. Elev:** 120

Depth (ft)	Sample Interval	Soil Description	Consistency/ Relative Density	Moisture Content % Wp |-----x-----| Wl 10 30 50 70 90	Pocket Penetrometer △ TSF △ 1 2 3 4 / SPT (N) ● Blows/ft ● 10 20 30 40	Monitor Well
1		2 inches ASPHALT CONCRETE over FILL: dark brown silty clay, moist. (CL)	Soft			
5				48.4	4	
6		6 inches black organic soils over brown mottled gray clayey SILT with sand seams and oxidation staining, moist. (ML/CL)	Very Soft			
10				40.0	2	
15				41.0	1	
16		Gray between 15 and 20 feet.				
17		1-inch sand seam at 16 feet.				
20		Brown below 20 feet.		44.9	2	
25		Gray wet sand seam at 26 feet.		39.5	4	
27		Brown below 30 feet.	Medium Stiff			
28		*Continued on Next Page.				
30						

Note: This borehole log has been prepared for geotechnical purposes. This information pertains only to this boring location and should not be interpeted as being indicative of other areas of the site.



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Consultants in Geotechnical Engineering, Geology and Environmental Earth Sciences

LOG OF BORING NO. B-2

Figure No. A-3

Project: Patton Property **Project No:** T-5982-2 **Date Drilled:** March 14, 2007

Client: _____ **Driller:** Holocene **Logged By:** TA

Location: Seattle, Washington **Approx. Elev:** 120

Depth (ft)	Sample Interval	Soil Description	Consistency/ Relative Density	Moisture Content % Wp |-----x-----| Wl 10 30 50 70 90	Pocket Penetrometer △ TSF △ 1 2 3 4 ● SPT (N) Blows/ft ● 10 20 30 40	Monitor Well
31				34.9	7	
32			Medium Stiff			
33						
34						
35		Gray clayey SILT. (ML/CL)		29.7	16	
36			Stiff			
37						
38						
39						
40			Very Stiff	27.1	23	
41						
42						
43						
44				32.0	12	
45						
46			Stiff			
47						
48						
49						
50		Vertical sand seams at 50 feet.		30.1	24	
51			Very Stiff			
52						
53						
54						
55				25.7	39	
56						
57		Boring terminated at 56.5 feet. Perched groundwater encountered at 15 feet.				
58						
59						
60						

Note: This borehole log has been prepared for geotechnical purposes. This information pertains only to this boring location and should not be interpeted as being indicative of other areas of the site.

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LOG OF BORING NO. B-3

Figure No. A-4

Project: Patton Property **Project No:** T-5982-2 **Date Drilled:** March 14, 2007

Client: _____ **Driller:** Holocene **Logged By:** TA

Location: Seattle, Washington **Approx. Elev:** 124

Depth (ft)	Sample Interval	Soil Description	Consistency/ Relative Density	Moisture Content % Wp |-----x-----| Wl 10 30 50 70 90	Pocket Penetrometer △ TSF △ 1 2 3 4 ● SPT (N) Blows/ft ● 10 20 30 40	Monitor Well
1–4		2 inches ASPHALT CONCRETE over FILL: dark gray silty clay, moist. (CL)	Soft			
5–10		Brown gray clayey SILT with oxidation staining and some seams, moist. (ML)	Stiff	32.0	9	
10–15				26.9	9	
15–22			Very Stiff	30.8	16	
20–22				31.9	20	
22–25		Gray massive elastic SILT, moist. (MH) *Continued on Next Page.	Very Stiff			

Note: This borehole log has been prepared for geotechnical purposes. This information pertains only to this boring location and should not be interpeted as being indicative of other areas of the site.

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Consultants in Geotechnical Engineering, Geology and Environmental Earth Sciences

LOG OF BORING NO. B-3

Figure No. A-4

Project: Patton Property **Project No:** T-5982-2 **Date Drilled:** March 14, 2007

Client: _____ **Driller:** Holocene **Logged By:** TA

Location: Seattle, Washington **Approx. Elev:** 124

Depth (ft)	Sample Interval	Soil Description	Consistency/ Relative Density	Moisture Content % Wp |-----x-----| Wl 10 30 50 70 90	Pocket Penetrometer △ TSF △ 1 2 3 4 ● SPT (N) Blows/ft ● 10 20 30 40	Monitor Well
26		Gray elastic SILT. (MH)	Very Stiff to Hard	31.4	21	
30				31.5	20	
35				29.6	21	
40				28.9	34	
44		2-inch sandy GRAVEL layer at 46 feet.		29.5	41	
46		Boring terminated at 46.5 feet. No groundwater was encountered.				
47						
48						
49						
50						

Note: This borehole log has been prepared for geotechnical purposes. This information pertains only to this boring location and should not be interpeted as being indicative of other areas of the site.

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LOG OF BORING NO. B-4

Figure No. A-5

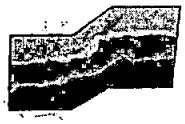
Project: Patton Property **Project No:** T-5982-2 **Date Drilled:** March 15, 2007

Client: _____ **Driller:** Holocene **Logged By:** TA

Location: Seattle, Washington **Approx. Elev:** 123

Depth (ft)	Sample Interval	Soil Description	Consistency/ Relative Density	Moisture Content % Wp |-----x-----| Wl 10 30 50 70 90	Pocket Penetrometer △ TSF △ 1 2 3 4 / SPT (N) • Blows/ft • 10 20 30 40	Monitor Well
1–8		3 inches ASPHALT CONCRETE over FILL: brown sand with silt to silty clay, moist. (SM/CL)	Loose to Medium Stiff	33.6	6	
8–18		Brown mottled gray clayey SILT with sand seams and oxidation staining, moist. (ML)	Soft	35.9 49.0	4 1	
18–30		Dark brown massive elastic SILT, moist. (MH) *Continued on Next Page.	Very Stiff to Stiff	21.6 34.6	23 13	

Note: This borehole log has been prepared for geotechnical purposes. This information pertains only to this boring location and should not be interpeted as being indicative of other areas of the site.



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LOG OF BORING NO. B-4

Figure No. A-5

Project: Patton Property **Project No:** T-5982-2 **Date Drilled:** March 15, 2007

Client: _____ **Driller:** Holocene **Logged By:** TA

Location: Seattle, Washington **Approx. Elev:** 123

Depth (ft)	Sample Interval	Soil Description	Consistency/ Relative Density	Moisture Content % Wp |-----x-----| Wl 10 30 50 70 90	Pocket Penetrometer △ TSF △ 1 2 3 4 / ● Blows/ft ● SPT (N) 10 20 30 40	Monitor Well
31–60		Brown elastic SILT. (MH)	Stiff to Very Stiff	31.0; 32.0; 32.7; 28.7; 29.3	16; 9; 19; 21; 19	
52		Boring terminated at 51.5 feet. No groundwater was encountered.				

Note: This borehole log has been prepared for geotechnical purposes. This information pertains only to this boring location and should not be interpeted as being indicative of other areas of the site.

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