SFA Design Group, LLC
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September 5, 2013

TerraFirma Foundation Systems
7901 Old Hwy 99 North, Address 2
Roseburg, Oregon 97470

RE: Geotechnical Investigation Report, Proposed Foundation Underpinning
Macomber Residence
44 NW Macleay Blvd
Portland, OR 97210

As requested, SFA Design Group, LLC (SFA) has provided the following report summarizing the findings of: a site visit conducted on June 27, 2013 to observe surface conditions and to evaluate the subsurface soil conditions, and provide pertinent geotechnical/geologic information necessary for the proposed underpinning project.

Scope

The following services were performed for the subject sites:

- Site reconnaissance.
- Sampled and logged (1) hand auger boring to a depth of approximately 7'-0".
- Preparation of this report summarizing current subsurface soil conditions, findings, and recommendations for foundation underpinning.
- Geotechnical review of plans.

Site Description and Project Narrative

The project site is located at 44 NW Macleay Boulevard in Portland, Oregon (See Figure-A). The existing residence is a single family two-story wood frame structure of approximately 2700 with a daylight basement. The house was built in 1927 and lies on a lot of approximately 3500 sqft.

Based upon elevation readings provided by TerraFirma Foundation Systems, SFA understands that the southeastern foundation perimeters have experienced settlements up to 3-3/4". Due to the age of this structure, observed settlements are likely due to a combination of improperly compacted construction fills, poorly designed foundation, and/or poor construction practices. The purpose of the proposed underpinning is to mitigate further settlement of the existing structure. Tie-backs are to be installed at the request of the customer, and are not required to sustain lateral resistance in the North-South direction (Gridline A). Lateral loading of the underpinned portion of the structure in the East-West direction (Gridline 1) shall be resisted by passive pressures acting on concrete backfill encasing the push piers. The subject site has been found to be in an area with slope stability hazards. The helical tie-back has been installed to address this issue. The proposed push pier, tie-back, and concrete backfill system is an acceptable way to address the settlement the existing structure is experiencing.

The existing/proposed foundation plans, retaining walls, slope setbacks, grading plans and/or specifications for this underpinning project have been reviewed and found to be in compliance with the recommendations contained within this report.

Geotechnical Field Investigation

On June 27, 2013 SFA conducted a field exploration at the above referenced property. The exploration involved (1) hand auger boring performed with a 3" diameter auger adjacent to the southern foundation perimeter of the residence. The hand auger borings (HA-1 and HA-2) extended to a depth of approximately 7'-0" below existing grade. Representative soil samples provided by the borings were logged and reviewed. The approximate locations of the boring(s) are indicated on the enclosed Boring Location Map (Appendix-A) and the depths and types of soil samples recovered are indicated on the borehole log (Appendix-B1 and B2). Subsequent to drilling all borings were backfilled with excavated soils.

No groundwater was encountered in the hole.

Geotechnical Findings

Regional Geologic Setting

The subject site is located in Multnomah County, Oregon. According to the Geologic Map of State of Oregon Department of Geology and Mineral Industries, the subject site is underlain by a Winter Water group - one or two flows of entablature-jointed, dark-gray to black, commonly porphyritic basalt. Flows are typically microphyric and phyric to abundantly phyric with uncommon small (<0.3 cm) stellate plagioclase glomerocrysts. Distribution of plagioclase glomerocrysts is uneven. Unit thickness within the area is variable, ranging from 0 to > 40 m-thick with highly permeable catastrophic flood deposits consisting of gravel with silt and coarse sand matrix.

Site Specific Earth Units and Groundwater

Based on our subsurface investigation, the subject site is predominantly underlain by layers of clayey silt (ML) to a depth explored of approximately 7'-0" with rock below. Two hand auger borings (HA-1 and HA-2) were done to ensure the rock layer was at a consistent depth. The soil was most, but groundwater was not encountered in the borings. Seasonal and long-term fluctuations in the groundwater may occur as a result of variations in subsurface conditions, rainfall, run-off conditions, and other factors. Therefore, variations from our observations may occur. Detailed descriptions of the earth units encountered in our borings are presented in the log of the borings in (Appendix B1 and B2).

Stormwater

Site development does not support infiltration of stormwater, however visual inspection of the site has determined that all rain drains appear to be functioning properly and are in good repair. The "Infiltration and Soils" portion of PortlandMaps has determined the potential for onsite infiltration using the Wetted Drainage Class, Hydrologic Soil Group, and Sump Capacity Data by Quarter Section that this area is "Well Drained". There does not appear to be a stormwater distribution concern on site.

Slope Stability

The structure displays potential sliding along the southeast portion of the residence. The cracks in the existing concrete foundation appear to have originated from translation of the existing structure. There is a significant slope present at the subject property. The slope is inclined in the North-South direction at an angle of approximately 35 degrees. The slope on the property occurs at the southern end of the structure. The subject structure is also located in a landslide area. Map #IMS-33 indicates that there is a historic earth flow on the property which is considered to be shallow and lies primarily on the southeastern portion of the property. Additionally, slope stability maps by the State of Oregon Department of Geology and Mineral Industries indicates that the subject site is located within an area having slope stability concerns (Figure-D and Figure-E). These concerns and hazards are to be addressed with the installation of a helical tie-back in the North-South direction along Gridline A. The helical tie-back is to mitigate further sliding of the structure.

Liquefaction, Differential Seismic Settlement, and Surface Displacement Due to Lateral Spreading

Indications of shallow groundwater were not encountered during the site specific explorations or research, thus the subject site is not located within an area having a potential for earthquake induced liquefaction, differential seismic settlement, or surface displacement due to lateral spreading. (Figure-C)

Geologic Hazards, Surface Displacement Due to Faulting and Regional Seismicity

We consider the most significant geologic hazard to be the potential for moderate to strong seismic shaking that is likely to occur at the subject site. The subject site is located in the highly seismic zones in the Pacific Northwest region within the influence of several faults that are considered to be active or potentially active. An active fault is defined as a "sufficiently active and well defined fault" that has exhibited surface displacement within the Holocene time (about the last 11,000 years). A potentially active fault is defined as a fault with a history of movement within Pleistocene time (between 11,000 and 1.6 million years ago).

These active and potentially active faults are capable of producing potentially damaging seismic shaking at the site. It is anticipated that the subject site will periodically experience ground acceleration as the result of being in a severe risk earthquake zone. Other active faults without surface expression (blind faults) or other potentially active seismic sources that are not currently zoned and may be capable of generating an earthquake are known to be present are shown on (Figure-B). Based on our review of the referenced geologic maps, as well as our field reconnaissance, the subject site is not underlain by known active or potentially active faults (i.e., faults that exhibit evidence of ground displacement in the last 11,000 years and 2,000,000 years, respectively).

Seismic Parameters

When reviewing the 2009 International Building Code and ASCE 7-5 the following seismic data pertain to the subject site.

Latitude (degree)	45.523955
Longitude (degree)	-122.708845
Site Class	D
Site Coefficient, F_a	1.106
Site Coefficient, F_v	1.703
Mapped Spectral Acceleration at 0.2-sec Period, S_S	0.985g
Mapped Spectral Acceleration at 1.0-sec Period, S_1	0.349g
Spectral Acceleration at 0.2-sec Period Adjusted for Site Class, S_{MS}	1.089g
Spectral Acceleration at 1.0-sec Period Adjusted for Site Class, S_{M1}	0.594g
Design Spectral Acceleration at 0.2-sec Period, S_{DS}	0.726g
Design Spectral Acceleration at 1.0-sec Period, S_{D1}	0.396g
Seismic Design Category	D

Seismic Parameters - Table-A

Geotechnical Parameters

Effective Friction Angle	30°
Soil Unit Weight, γ	110 pcf

Geotechnical Parameters - Table-B

The structural consultant should review the above parameters and the 2009 International Building Code to evaluate the seismic design.

Conformance to the criteria presented in the above table for seismic design does not constitute any type of guarantee or assurance that significant structural damage or ground failure will not occur during a large earthquake event. The intent of the code is "life safety" and not to completely prevent damage of the structure, since such design may be economically prohibitive.

General Recommendations

Drainage

All drainage systems disturbed during construction should be repaired prior to final approval. In the absence of adequate drainage SFA recommends that properly functioning positive site drainage should be maintained at all times. This should include the proper routing and discharge of rain drains as well as curtain drains. Drainage should not flow uncontrolled down any descending slope or retaining wall. Water should be directed away from foundations and not allowed to pond and/or seep into the ground. Pad drainage should be directed toward the street/parking or other approved area. Roof gutters and down spouts should be utilized to control roof drainage. Down spouts should outlet a minimum of 5 feet from the proposed structure or into an BES approved drainage point. The effects of improperly or non-existent drainage systems would be likely contributors to typically observed settlement of structures.

Site Specific Recommendations

It is our understanding that the proposed project consists of supporting and lifting the foundation of the existing structure utilizing push piers and laterally stabilizing the structure using helical piers. Lateral loads along Gridline 1 shall be resisted by concrete backfill encasing the push piers acting on passive pressure from the soil. The tie-backs are not required to sustain lateral resistance in the north-south direction and are present to mitigate sliding from the slope on-site. Therefore, based on site exploration and engineering analysis, a push pier system as presented on the foundation plans (SFA Sheet S2.1, 2013) are a suitable method for stabilizing the existing foundation of the building located at the subject site.

The subject building may be underpinned with push piers (steel tubes). The allowable axial capacity of the push piers were evaluated and presented on structural calculations (SFA Structural Calculations, 2013). These capacities include a factor of safety of 2.0. Underpinning as outlined shall halt further vertical settlement of the structure at the underpinned locations if the anchors are installed per the enclosed requirements. Installation of these anchors in no way provides support of the remainder of the structure.

Push Pier Vertical Capacity
Pin piles, consisting of FSI 288 push piers, used to support the vertical loads of the structure through connection to the existing foundation shall be installed to a depth of 10ft and to a minimum resistance of 2100psi imparted by a 3.5 in diameter hydraulic ram. Ultimate capacity of 20.2 kips with a factor of safety of 2.0 results in a design capacity of 10.1 kips. Maximum spacing per structural engineer requirements.

Push Pier Lateral Capacity
Push piers do not have appreciable capacity for lateral loads. Lateral loads are to be resisted by passive pressures acting on concrete backfill encasing the push piers in the East/West direction and a helical tie-back in the North/South direction installed per the requirements presented on the foundation plans (SFA Sheet S2.1, 2013).

Helical Tie-Back Lateral Capacity
A helical tie-back, consisting of a FSI HA150 Helical Anchor, shall be used to resist horizontal loads in the direction of the slope (North-South) at the subject property. The loads will be resisted through connection to the existing foundation and shall be installed to a minimum depth of 15 ft and minimum torque of 3000 ft-lbs. Utilizing an empirical torque conversion factor of $10ft^{-1}$ resulting in an ultimate capacity of 30 kips with a factor of safety of 2.0 yields a design capacity of 15 kips.

Observation/Testing During Construction
Continuous special inspection is required during installation per 2009 IBC Section 1810.4.12. Load testing shall be performed in accordance with ASTM Method D1143 (Quick Method) on 20 percent of push piers but not less than (2) total piers and will be selected by the special inspector. An alignment load (AL) shall be applied to the pile prior to setting the deflection measuring equipment to zero or a reference position. The AL shall be no more than 10% of the DL. Incremental loading shall be in accordance with the following schedule:

Test Loading Schedule	Hold Time	Max Deflection
AL (.10 DL Max)	0 min.	
0.25 DL	Until Stable	
0.50 DL	Until Stable	
0.75 DL	Until Stable	
1.00 DL	Until Stable	
1.25 DL	Until Stable	
1.50 DL	Hold for Creep Test (See Below)	0.04 inches
1.25 DL	Until Stable	
1.00 DL	Until Stable	
0.75 DL	Until Stable	
0.50 DL	Until Stable	
0.25 DL	Until Stable	

Load testing creep acceptance criteria shall be no greater than 0.04 inches within a 10 minute period. If movement is observed greater than 0.04 inches within the 10 minute period the load test shall be held for an additional 50 minutes, the pier is to be deepened and re-tested, or the pier is to be abandoned and replaced with a new pier. If the load test is to be held the pier movements shall be measured at 15, 20, 30, 40, 50, and 60 minutes. The creep versus the logarithm of time shall be plotted. If the creep rate is less than 0.080 inches between 6 and 60 minutes, the load test shall be considered successful.

Limitations and Closure

This report has been prepared for the exclusive use of TerraFirma and their design consultants relative to maintaining the subject site. No portion of this report may be used by other parties or for other purposes. The exploratory work was performed on the southeast corner of the existing building. SFA considered a number of unique, project-specific factors when establishing the scope of services for this report. This report has not been prepared for use by other parties, and may not contain sufficient information for the purposes of other parties.

Our findings were obtained in accordance with generally accepted current professional principles and local practice in geotechnical engineering and reflect our best professional judgment based on experience and gathered data. It is to be understood that geotechnical information is characterized by a degree of uncertainty. Judgments rendered meet current professional standards; no other warranty is issued, either expressed or implied. The findings contained in this report are based upon our evaluation and interpretation of the information obtained from the limited number of test borings and the results of laboratory testing and engineering analysis. As part of the engineering analysis it has been assumed, and is expected, that the geotechnical conditions that exist across the area of study are similar to those encountered in the borings. However, no warranty is expressed or implied as to the conditions at locations or depths other than those explored. If a period of one year elapses since preparation of this report, the geotechnical consultant should verify the current site conditions, and provide any additional recommendations (if necessary) prior to construction.

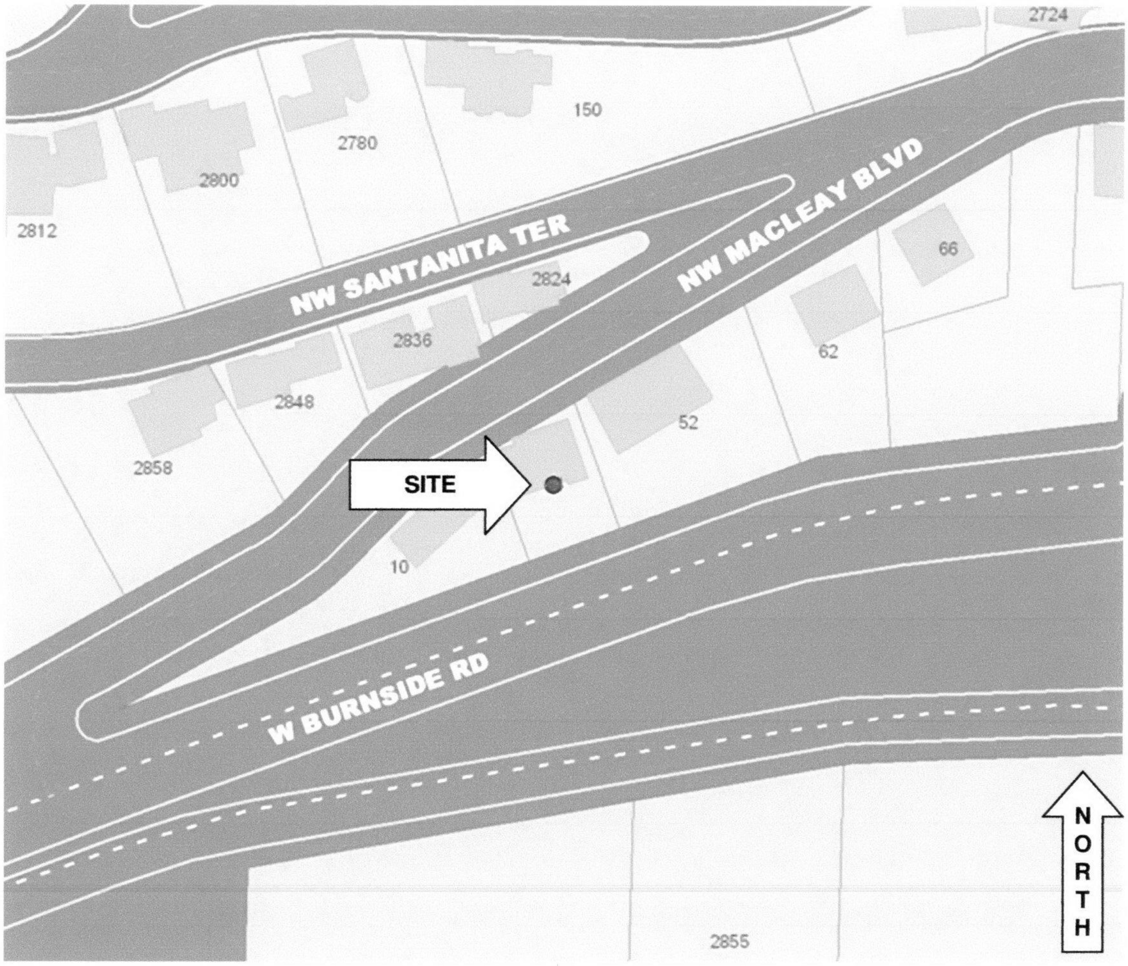
Thank you for the opportunity of providing our services to you on this project.

Sincerely,
SFA Design Group, Inc.

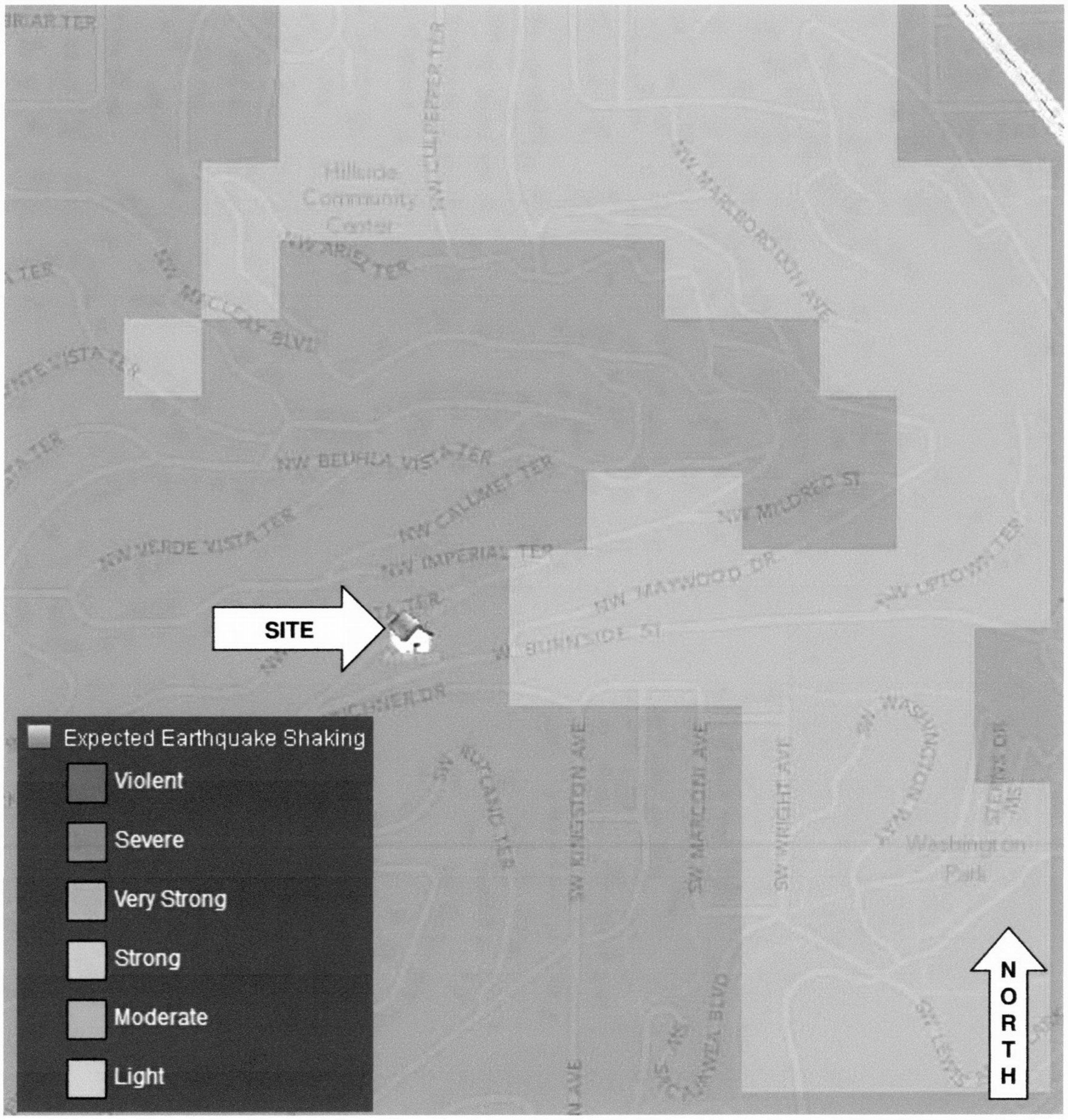
Jeff Fitch, P.E.
Principal

REGISTERED PROFESSIONAL
ENGINEER
59022PE
OREGON
09-09-2003
JEFFREY S. FITCH
EXPIRES: 12-31-13

Attachments:
Figure-A – Site Location Map
Figure-B – Seismic Hazard Zone Map
Figure-C – Liquefaction Hazard Map
Figure-D – Slope Stability Hazard Map
Figure E – Landslide Hazard Map
Appendix-A – Boring Location Map
Appendix-B1 – Log of Boring
Appendix-B2 – Log of Boring
Distribution: (3) to Addressee

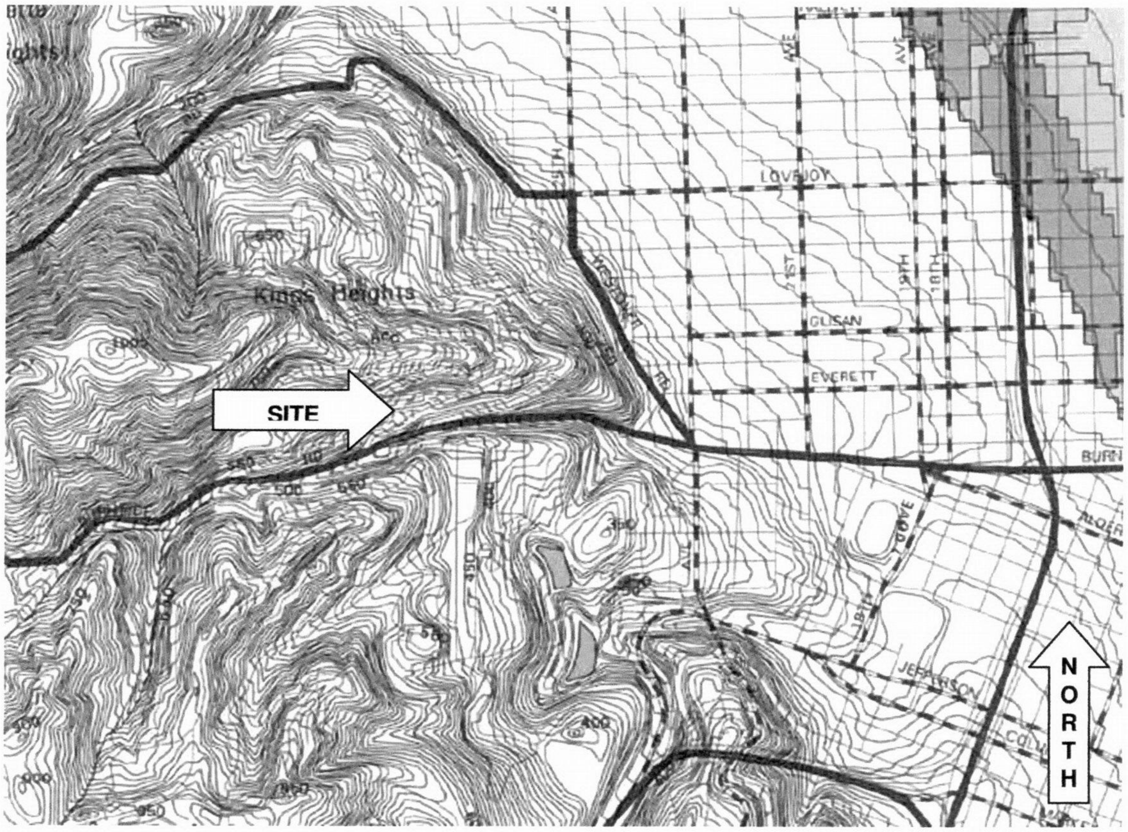


sfa	SITE LOCATION MAP	Project Number: TF13-050
		FIGURE-A



Source: Oregon HazVu: Statewide Geohazards Viewer, http://www.oregongeology.org/sub/hazvu/ (2012)

sfa	**SEISMIC HAZARD ZONE MAP**	Project Number: TF13-050
		FIGURE-B



THICKNESS OF LIQUEFIABLE SEDIMENT:

Greater than 9 m (30 ft)

Between 3 m and 6 m (10 ft and 20 ft)

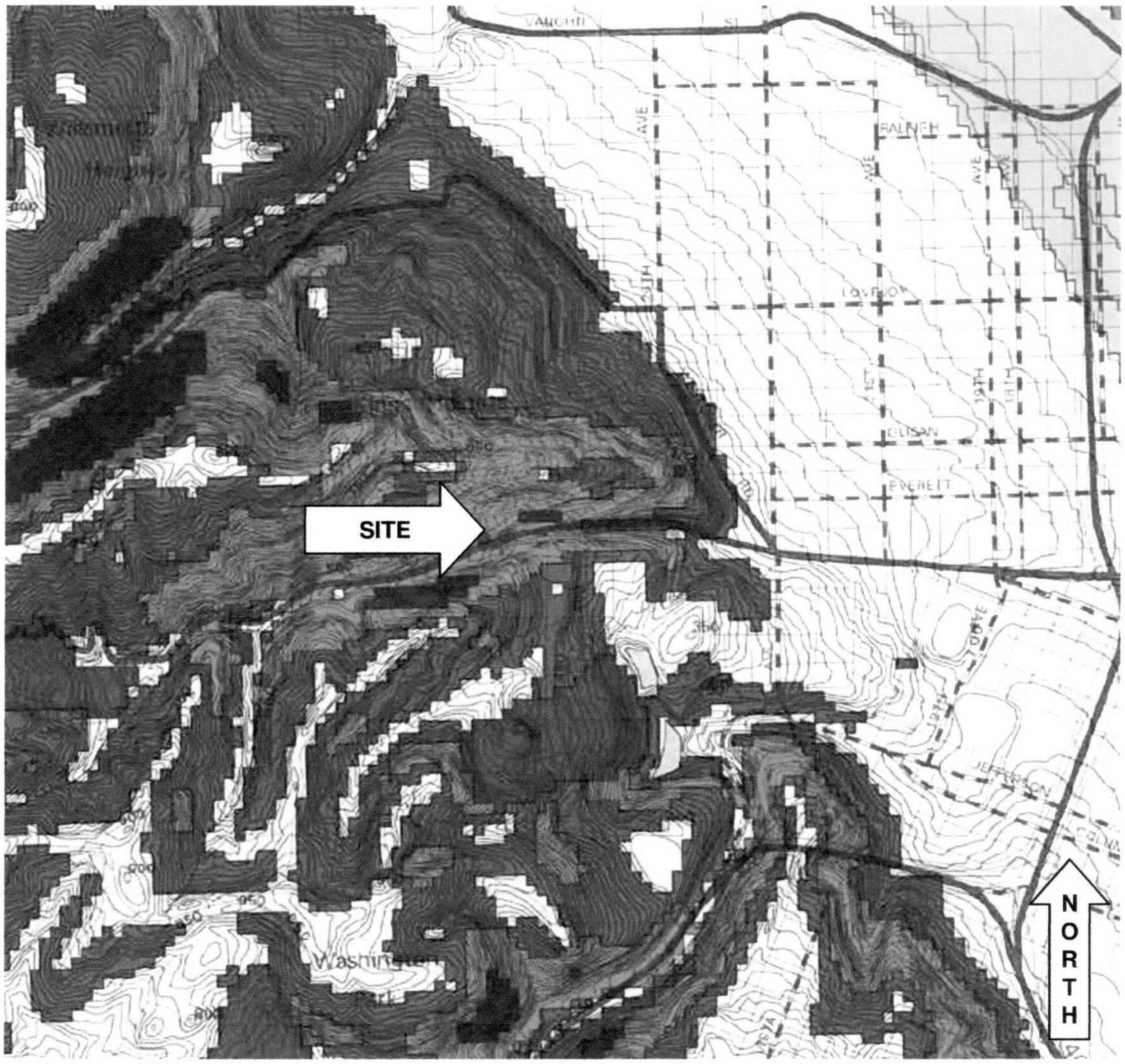
Less than 3 m (10 ft)

Darker color indicates water table less than 4.5 m (15 ft) deep
Lighter color indicates water table between 4.5 m and 6 m (15 ft and 30 ft) deep
Liquefiable sediments seldom occur beneath the water table in areas with a ground water depth greater than 9 m (30 ft)

Indicates areas where ground water could seasonally rise and saturate potentially liquefiable sediment.

Source: Liquefaction Analysis by M. A. Mabey and others, Department of Geology and Mineral Industries (1993)

sfa	LIQUEFACTION HAZARD MAP	Project Number: TF13-050
		FIGURE-C



RELATIVE DYNAMIC SLOPE INSTABILITY

- Ground slope greater than or equal to 15%
- Factor of safety between 2 and 1.25
- Factor of safety less than 1.25
- Existing landslide

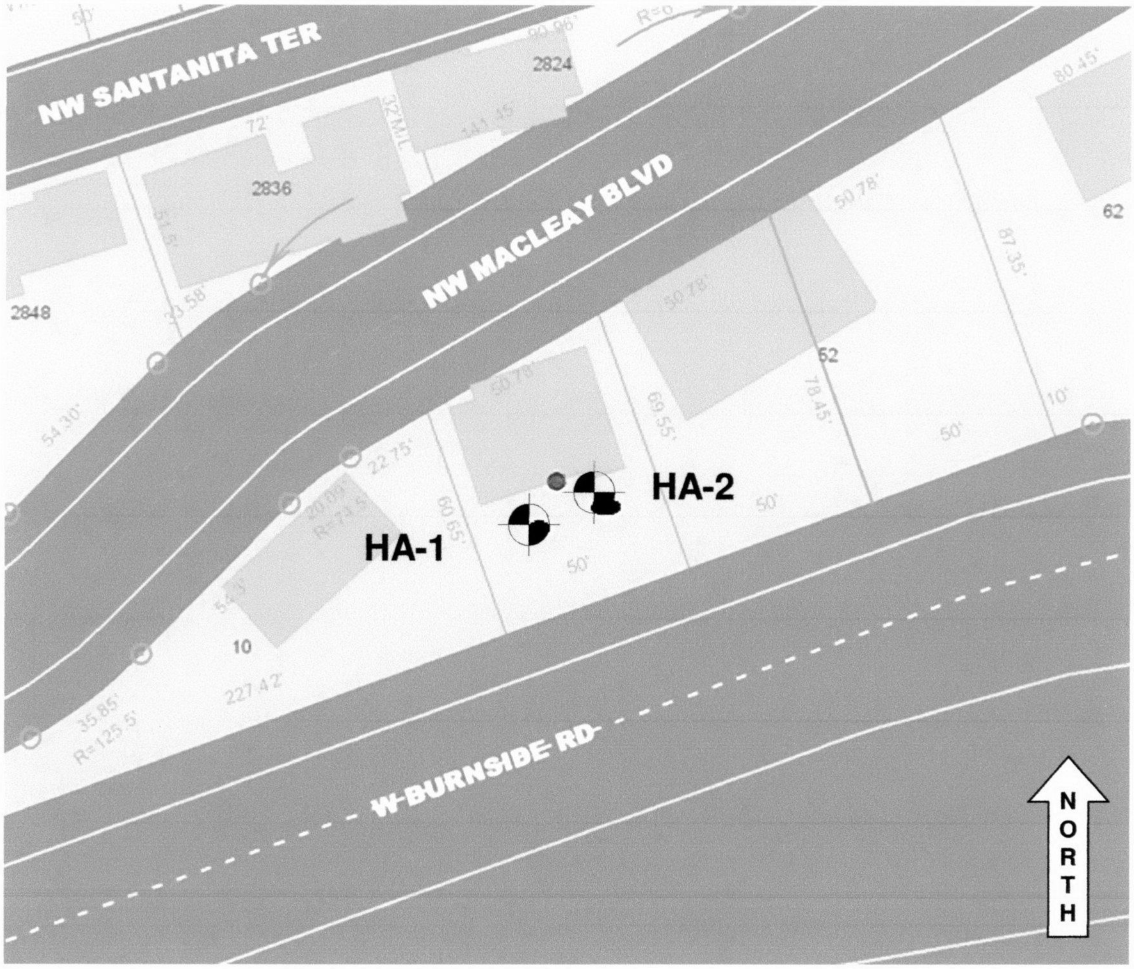
White area implies < .3m liquefaction displacement, factor of safety > 2, and slopes < 15%

Source: Dynamic Slope Stability Analysis by M.A. Mabey and others, Oregon Department of Geology and Mineral Industries (1993)

sfa	SLOPE STABILITY HAZARD MAP	Project Number: TF13-050
		FIGURE-D

Source: Overview of the Landslide Inventory of the Portland Quadrangle, Multnomah and Washington Counties, Oregon, Oregon Department of Geology and Mineral Industries (2010), IMS-33 by William J. Burns and Serin Duplantis

sfa	**LANDSLIDE HAZARD MAP**	Project Number: TF13-050
		FIGURE-E



KEY

HA-1 – Hand Auger Boring

sfa	**BORING LOCATION MAP** **44 NW Macleay Blvd** **Portland, OR 97210**	Project Number: TF13-050 Appendix-A

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BORING LOG

Boring Number:	HA-1	Site Address:	44 NW Macleay Blvd
Boring Date:	June 27th, 2013	Project Name:	Macomber Residence
Boring Method:	3" Diameter - Uncased Temporary Hand Auger	Project Number:	TF13-050
Water Encountered:	No	Weather:	Sunny, +-74 degrees
Total Boring Depth:	7'-0"	Logged By:	Chris Fitch

Depth, ft	Lithology/Remarks	Consistency/ Relative Density	Moisture Condition	Color
0	Clayey Silts (ML)	Soft	Moist - No Visible Water	Light Brown
1	↓	↓	↓	↓
2	↓	↓	↓	↓
3	↓	↓	↓	↓
4	↓	↓	↓	↓
5	↓	↓	↓	↓
6	↓	↓	↓	↓
7	↓	↓	↓	↓
8	ROCK			
9				
10				
11				
12				

Notes: Rock was encountered at a depth of 7' and the boring was unable to go any deeper

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BORING LOG

Boring Number:	HA-2	Site Address:	44 NW Macleay Blvd
Boring Date:	June 27th, 2013	Project Name:	Macomber Residence
Boring Method:	3" Diameter - Uncased Temporary Hand Auger	Project Number:	TF13-050
Water Encountered:	No	Weather:	Sunny, +-74 degrees
Total Boring Depth:	7'-0"	Logged By:	Chris Fitch

Depth, ft	Lithology/Remarks	Consistency/ Relative Density	Moisture Condition	Color
0	Clayey Silts (ML)	Soft	Moist - No Visible Water	Light Brown
1	↓	↓	↓	↓
2	↓	↓	↓	↓
3	↓	↓	↓	↓
4	↓	↓	↓	↓
5	↓	↓	↓	↓
6	↓	↓	↓	↓
7	↓	↓	↓	↓
8	ROCK			
9				
10				
11				
12				

Notes: Rock was encountered at a depth of 7' and the boring was unable to go any deeper