Terracon

April 29, 2015

Triad/Fransen, LLC
2801 Alaskan Way, Suite 107
Seattle, Washington 98121

Attn: Mr. Jeff Fransen
P: (425) 344-8833

Re: Geotechnical Report Update Letter
166th Avenue Townhomes
8502 166th Ave NE & 16640 NE 85th St
Redmond, King County, Washington
Terracon Project No. 81155022

Dear Mr. Fransen:

Terracon Consultants, Inc. (Terracon) has completed our geotechnical services for the above referenced project in accordance with our Proposal No. P81150127, dated April 23, 2015. This letter is an addendum to, and should be used in conjunction with both our Geotechnical Report and subsequent Geotechnical Report Addendum that were submitted to Redmond Town Center Condominiums, LLC on January 17, 2007 and September 21, 2007, respectively (Terracon project number 81065239, formerly known as Zipper Zeman Associates). The purpose of this letter is to comment on and update our previous recommendations as they apply to the current development plans for the site. Our understanding of the current project scope is based upon our conversations with Mr. Erich Armbruster of Ashworth Homes, a review of preliminary layout plans (3 sheets, dated March 16, 2015) by Daniel Umbach, Architect, our previous Geotechnical Report and Addendum, and our recent visit to the site on April 28, 2015.

Project Information

Terracon conducted surface and subsurface investigations on the subject site on December 19, 2006 and August 3, 2007 to provide recommendations for the design and construction of a proposed multi-family development at the above listed site. Though development plans had not been completed at the time of our original report, the recommendations provided in the report were based on our understanding that the project construction would likely consist of a four story multi-family structure with 2 levels of below-grade parking. The September 21, 2007 report addendum was completed based on anticipated maximum proposed shoring heights of about 22 feet as shown on the preliminary shoring plans by CG Engineering.

We understand that current plans call for construction of four new townhome structures occupying a majority of the site supported by shallow foundations and consisting of two floors of living space above slab-on-grade garages. Two of the buildings would be comprised of 5 townhomes each and two of the buildings would be comprised of 4 townhomes each. The current planned scope of construction appears to be feasible from a geotechnical perspective based on the project description and site conditions as noted within this letter and our original report.

Surface and Subsurface Conditions

As discussed in our original report, the site is comprised of two parcels. Parcel A is located at the northeast corner of 166th Avenue NE and NE 85th Street and is currently developed with a two-story multi-family residential building (8500 through 8510 166th Ave NE). Parcel B is located immediately to the east of Parcel A and north of NE 85th Street (16640 NE 85th Street).

At the time of our original report, Parcel B was occupied by a single-family residential home and deciduous and conifer trees. During our recent site visit, we observed that the home has since been removed from the site. Based on a review of aerial photographs on the King County iMap interactive mapping tool, it appears that the home was demolished and removed from the site sometime between 2009 and 2012. Parcel B is currently vacant and covered with grass, weeds, and deciduous and conifer trees. Site grades on each parcel appear to roughly match the descriptions provided in our original report, with the exception of a 2 to 6 foot depression in the location of the former residence.

During our previous work on the site, Terracon performed a site reconnaissance and advanced three geotechnical borings (B-1 through B-3) in the northwest, southeast, and northeast corners of the site to aid in classification of subsurface soil conditions. The explorations disclosed between 0 and 4 feet of loose silty, gravelly sand fill soils over dense to very dense, native glacial till soils. In boring B-3, completed for the September 2007 report addendum, hard, gray sandy silt was observed below the glacial till soils from about 28½ feet below the ground surface to the full 51½ feet exploration depth.

Groundwater seepage was observed in borings B-1 and B-2 at depths of 16½ and 6 feet. The deeper groundwater seepage observed in B-1 was interpreted as thin, saturated sand zones within the glacial till. The shallower groundwater observed in B-2 was interpreted as perched groundwater above the very dense glacial till soils. Groundwater was not observed in Boring B-3.

A detailed description of both the surface and subsurface conditions may be found in our original Geotechnical Report and the September 2007 addendum.

Conclusions and Recommendations

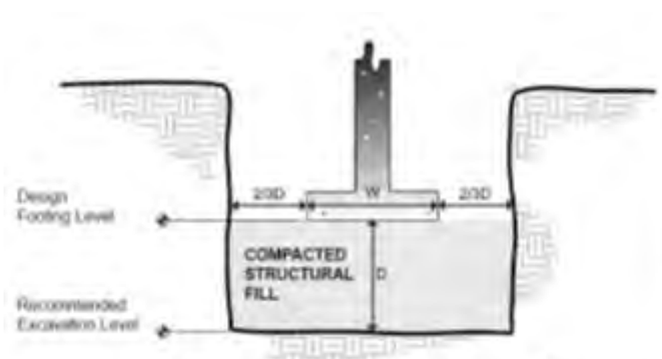
Our previous report provided conclusions and recommendations related to stormwater infiltration feasibility, site preparation, temporary and permanent cut slopes, temporary shoring, structural fill, building foundations, slab-on-grade floors, lateral earth pressures, subgrade foundation walls, and seismic considerations. The September 2007 addendum included recommendations related to design, drainage, and construction monitoring of soldier pile shoring. We understand that below grade construction is not a part of the current development plans and temporary soldier pile shoring will no longer be necessary.

In our opinion, the recommendations provided in our original Geotechnical Report and the September 2007 Geotechnical Report Addendum are still applicable to the current development plans, with minor clarifications and updates. These clarifications are discussed in the following sections.

Building Foundations and Slabs-on-Grade

Our original Geotechnical Report recommended an allowable bearing pressure of 4,000 pounds per square foot (psf) for shallow spread footings bearing on the dense to very dense, native glacial till soils. As discussed in the Surface and Subsurface Conditions section above, existing fill soils to depths of up to about 4 feet were observed in our explorations. Note that locations of deeper existing fill soils may be encountered during construction. We recommend complete removal of existing fill soils below the building footings and floor slabs. Foundations could then bear directly on the exposed native glacial till soils or on compacted structural fill placed atop at the dense to very dense glacial till in accordance with the recommendations provided in our original report. Allowable bearing capacities for conventional spread footings would be 4,000 psf for footings bearing on dense to very dense, native glacial till soils or for footings bearing on no more than 5 feet of compacted structural fill above the native glacial till soils.

If overexcavations are necessary below building footings, the excavations should extend laterally beyond all edges of the footings at least 8 inches per foot of overexcavation depth below footing base elevation. The overexcavation should then be backfilled up to the footing base elevation with structural fill placed in lifts of 8 inches or less in loose thickness and compacted to at least 95 percent of the material's maximum modified Proctor dry density (ASTM D-1557). The overexcavation and backfill procedure is described in the figure below.



Overexcavation / Backfill

NOTE: Excavation shown vertical for convenience; excavations should be sloped as necessary for safety.

Seismic Considerations

Our original report provided a Site Class definition and liquefaction potential evaluation based on the 2003 International Building Code (IBC) and the explorations completed on the site to a maximum depth of about 51½ feet. We understand the current project is being designed based on the 2012 IBC, which indicates that the seismic site classification is based on the average soil and bedrock properties in the top 100 feet. The current scope does not include a 100-foot soil profile determination. Based on the results of our explorations and mapped conditions, however, the 2012 IBC seismic site classification for this site is C. This seismic site class definition considers that soils encountered at depth in our borings continue below the termination depth. Additional exploration to deeper depths would be required to confirm the conditions below the current depth of exploration. Detailed site response spectra based on the 2012 IBC are provided on the attached USGS Design Maps Summary Report.

As noted in our original report, soil liquefaction typically occurs in loose to medium dense, granular soils located below the water table. Our original report concluded that the potential for soil liquefaction at the site is negligible based on the 2003 IBC. Even with the increased seismic ground motion design values between the 2003 and the 2012 IBC, in our opinion the potential for soil liquefaction at the site is still negligible.

It is our opinion that no additional engineering geology investigations or geologic hazard evaluations are necessary relative to seismic hazards for this project.

General Comments

Terracon should be retained to provide observation and testing services during grading, excavation, foundation construction and other earth-related construction phases of the project.

The analysis and recommendations presented in this letter are based upon the data obtained from the borings performed during our previous investigations and from other information discussed in this letter. This letter does not reflect variations that may occur between borings, across the site, or due to the modifying effects of construction or weather. The nature and extent of such variations may not become evident until during or after construction. If variations appear, we should be immediately notified so that further evaluation and supplemental recommendations can be provided.

The scope of services for this project does not include either specifically or by implication any environmental or biological (e.g., mold, fungi, bacteria) assessment of the site or identification or prevention of pollutants, hazardous materials or conditions. If the owner is concerned about the potential for such contamination or pollution, other studies should be undertaken.

This letter has been prepared for the exclusive use of our client for specific application to the project discussed and has been prepared in accordance with generally accepted geotechnical engineering practices. No warranties, either express or implied, are intended or made. Site safety and excavation support are the responsibility of others. In the event that changes in the nature, design, or location of the project as outlined in this letter are planned, the conclusions and recommendations contained in this letter shall not be considered valid unless Terracon reviews the changes and either verifies or modifies the conclusions of this letter in writing.

Conclusion

We appreciate the opportunity to perform these services for you. Please contact us if you have questions regarding this information or if we can provide any additional services.

Sincerely,
Terracon Consultants, Inc.

RYAN MICHAEL SCHEFFLER
STATE OF WASHINGTON
49529
REGISTERED
PROFESSIONAL ENGINEER

Ryan M. Scheffler, PE
Project Engineer

David A. Baska, PhD, PE
Geotechnical Department Manager

Zipper Zeman Associates, Inc.
Geotechnical and Environmental Consulting
A Terracon Company

Job No. 81065239A
September 21, 2007

Redmond Town Center Condominiums, LLC
333 156th St. NE
Arlington, WA 98223

Attention: Mr. Todd Leabman

Subject: Geotechnical Report ***Addendum***
Soldier Pile Temporary Shoring Recommendations
Proposed Multi-Family Development
8502 166th Ave. NE and 16640 NE 85th St.
Redmond, Washington

Dear Mr. Leabman:

Zipper Zeman Associates, Inc. (ZZA) is pleased to present a copy of the above-referenced report. This report presents an Addendum to our original January 17, 2007 geotechnical report completed for the project. The purpose of this Addendum is to provide geotechnical recommendations for soldier pile temporary shoring. Our work was completed in general accordance with the Scope of Services described in our Proposal (P-3923) dated July 19, 2007 and subsequently authorized by Mr. Jesse Molnick on July 23, 2007.

PROJECT DESCRIPTION

Based on our review of preliminary shoring plans provided by the project structural engineer (CG Engineering), we understand that cantilever soldier pile shoring will be utilized on north and east sides of the site to support a cut required for below grade parking. We expect the shoring to have a maximum supported height of about 22 feet. A 1H:1V cut will be made above the shoring to catch existing site grades.

SUBSURFACE CONDITIONS

Subsurface soil conditions at the project site were evaluated by reviewing information from previous borings completed on the site as part of our January 2007 work and completing and additional subsurface exploration near the north east corner of the site. All exploration locations are shown on the attached Figure 1, Site and Exploration Plan. The borings were completed to depths of ranging from approximately 18 to 51.5 feet below the existing ground surface at the exploration locations. The explorations were continuously monitored by a geotechnical engineer from our firm. Soils were visually classified in general accordance with the Unified Soil Classification System. Subsurface exploration procedures and logs for the explorations are enclosed in Appendix A of this report.

Soil and groundwater descriptions presented in this report are based on the subsurface conditions encountered at specific exploration locations on the site. Variations in subsurface conditions may exist between the exploration locations and the nature and extent of variations between the explorations may not become evident until construction. If variations then appear, it may be necessary to reevaluate the recommendations of this report. The descriptions of soil conditions provided below are generalized. The exploration logs provided in Appendix A should be referred to for detailed information regarding soil conditions observed at each boring location.

Boring B-1 was completed in the north parking lot area of Parcel A. Subsurface soil conditions observed in boring B-1 generally consisted of 3 inches of asphalt concrete pavement underlain by very dense, moist to wet, silty, gravelly sand. The silty, gravelly sand is interpreted to be glacial till. Boring B-1 was completed at approximately 18 feet below the existing ground surface within the glacial till soil unit.

Boring B-2 was completed at the south end of the driveway for the residence located on Parcel B. Subsurface soil conditions observed in boring B-2 generally consisted of 3 inches of asphalt concrete pavement underlain by loose, wet, silty, gravelly sand (fill) extending to about 4-½ feet below the existing ground surface. The fill was underlain by medium dense to dense, wet, silty, gravelly, sand (weathered glacial till) extending to about 12 feet below the existing ground surface. The weathered glacial till was underlain by very dense, moist, silty, sandy, gravel with thin layers of sand with trace silt (unweathered glacial till). Boring B-2 was completed in the glacial till soil unit at approximately 18 feet below the existing ground surface.

Boring B-3 was completed near the north east corner of the site. Subsurface soil conditions observed in boring B-2 generally consisted of about 4 inches of sod and topsoil underlain by very dense, moist to saturated, gravelly, silty sand (unweathered glacial till) extending to about 28 feet below the existing ground surface. The unweathered glacial till was underlain by hard, moist, sandy silt with a variable gravel content extending to the completion depth of about 51.5 feet below the existing ground surface.

Groundwater Conditions

Groundwater was observed in boring B-1 at about 16.5 feet below the existing ground surface at the time of drilling. Groundwater observed in this boring is interpreted to be thin, saturated sand zones within the glacial till.

Groundwater observed in boring B-2 at about 6 feet below the existing ground surface. Groundwater observed in this boring is interpreted to be perched groundwater. Perched groundwater conditions develop when the downward migration of surface water is impeded by a relatively impermeable soil layer such as the very dense glacial till soils observed in boring B-2. Groundwater was not encountered in boring B-3 at the time of drilling.

The thickness of saturated soil resulting from perched groundwater conditions is typically thin. Perched groundwater is recharged primarily by precipitation. As a result, the saturated zone will tend to be thicker during wet weather.

CONCLUSIONS AND RECOMMENDATIONS

General Summary

Based on the subsurface soil and groundwater conditions observed in our borings, it is our opinion that cantilevered solider pile shoring is a suitable alternative for the project. Specific geotechnical recommendations for shoring are provided in the following sections.

Geotechnical Design Parameters

We recommend temporary shoring for the project be designed using active lateral earth pressures assuming an equivalent fluid pressure of 31 pcf for the proposed shoring configuration. Refer to the attached Figure 2, Soldier Pile Shoring Pressure Diagram for additional recommendations regarding active earth pressures.

Embedment depth of soldier piles below final excavation level must be designed such that the bottoms of the piles are fixed against excessive rotation. Lateral resistance may be computed assuming an equivalent passive fluid pressure of 400 pcf. The upper two feet of soil below the bottom of the excavation should be ignored when calculating passive resistance. The passive pressure may be assumed to act over two pile diameters or the pile to pile spacing, which ever is less.

Due to soil arching effects, timber lagging may be designed for 50 percent of the lateral earth pressure used for shoring design. Prompt and careful installation of lagging will reduce potential loss of ground. The requirements for lagging should be made the responsibility of the shoring subcontractor to prevent soil failure, sloughing and loss of ground and to provide safe working conditions. We recommend any voids between the lagging and soil be backfilled. However, the backfill should not allow potential hydrostatic pressure to build-up behind the wall. Drainage behind the wall must be maintained. A permeable sand slurry may be considered for lagging backfill.

Lateral and Vertical Shoring Deformation

Design of shoring using the active earth pressures provided above assumes that some lateral and vertical deformation of the shoring system can be tolerated. Lateral deformation of the shoring system is expected to be in the range of 0.001 to 0.002 times the height of the supported cut. The lateral deformation will be accompanied by vertical deformation behind the top of the wall on the order of 0.002 times the height of the supported cut. The vertical deformation may extend behind the cut a distance approximately equal to the height of the unsupported cut. If these deformations cannot be tolerated, we should be consulted for additional recommendations. The values provided above for lateral and vertical deformations are "typical" values. The shoring designer should calculate the anticipated deflection using site-specific conditions and make a final determination of site specific deformations.

Shoring Monitoring

As indicated above, a cantilever shoring system designed for active earth pressures will experience some level of lateral and vertical deformations. Even with a well-designed shoring system, the actual deformations may exceed the predicted deformations. For this reason, we recommend a shoring monitoring program be conducted during construction.

The monitoring program should include measurements of the horizontal and vertical movements of the retained improvements and the shoring system itself. At least two reference lines should be established adjacent to the excavation at horizontal distances back from the excavation space of about 1/3H and H, where H is the final excavation height. Monitoring of the shoring system should include measurements of vertical and horizontal movements at the top of each soldier pile. Reference points for horizontal movement should also be selectively placed at various tieback levels as the excavation progresses.

The measuring system used for shoring monitoring should have an accuracy of at least 0.01 foot. All reference points on the retained improvements should be installed and readings taken prior to commencing the excavation. All reference points should be read prior to and during critical stages of construction. The frequency of readings will depend on the results of previous readings and the rate of construction. As a minimum, readings should be taken twice a week throughout construction until the excavation is completed. All readings should be reviewed by Zipper Zeman Associates, Inc.

In addition to the shoring monitoring program, we recommend a pre-construction survey of the adjacent properties to document existing conditions. The survey should include both a visual (photographs and/or videotapes) and written record of the buildings, pavements, utilities, and landscape areas bordering the future excavation. Separate surveys by the owner and contractor are common.

Permanent Drainage Measures

Once the shoring is complete, the permanent basement wall is expected to be cast directly against the temporary shoring wall and lagging. For this reason, it is imperative to include permanent drainage measures into design and construction of the temporary shoring wall. We provide the following recommendations regarding incorporating permanent drainage measures into the temporary wall. Figure 3 provides additional detail regarding connecting the wall drain to a permanent drainage system on the front side of the wall.

1. Any voids created during installation of lagging should be backfilled with a free draining, aggregate containing less than 5% fines.
2. A ¼" gap should be provide between lagging boards to promote drainage through the lagging.
3. A prefabricated drainage board such as Miradrain should be attached to the face of the lagging. The drainage board should extend the full height of the wall and the full width between soldier piles. Between each pile, the drainage board should be connected to a weep hole pipe that extends through the face of the permanent basement wall using a pre-fabricated drain board connector such as DrainGreat. The weep hole pipe should then be tightlined to an interior drainage system place below the basement floor slab.
4. The flow capacity of the prefabricated drainage board should be determined during construction based on groundwater conditions observed in the excavation.

CLOSURE

We have prepared this report for use by Redmond Town Center Condominiums, LLC for this project. The data and report should be provided to prospective contractors for bidding or estimating purposes, but our report, conclusions and interpretations should not be construed as a warranty of the subsurface conditions.

There are possible variations in subsurface conditions across the site and also with time. A contingency for unexpected conditions should be included in the project budget and schedule. Sufficient monitoring, testing and consultation should be provided by our firm during construction to confirm that the conditions encountered are consistent with those indicated by the explorations, to provide recommendations for design changes should the conditions revealed during the work differ from those anticipated, and to evaluate whether or not construction activities comply with the contract plans and specifications.

Within the limitations of scope, schedule and budget, our services have been executed in accordance with generally accepted practices in this area at the time the report was prepared. No warranty or other conditions express or implied should be understood.

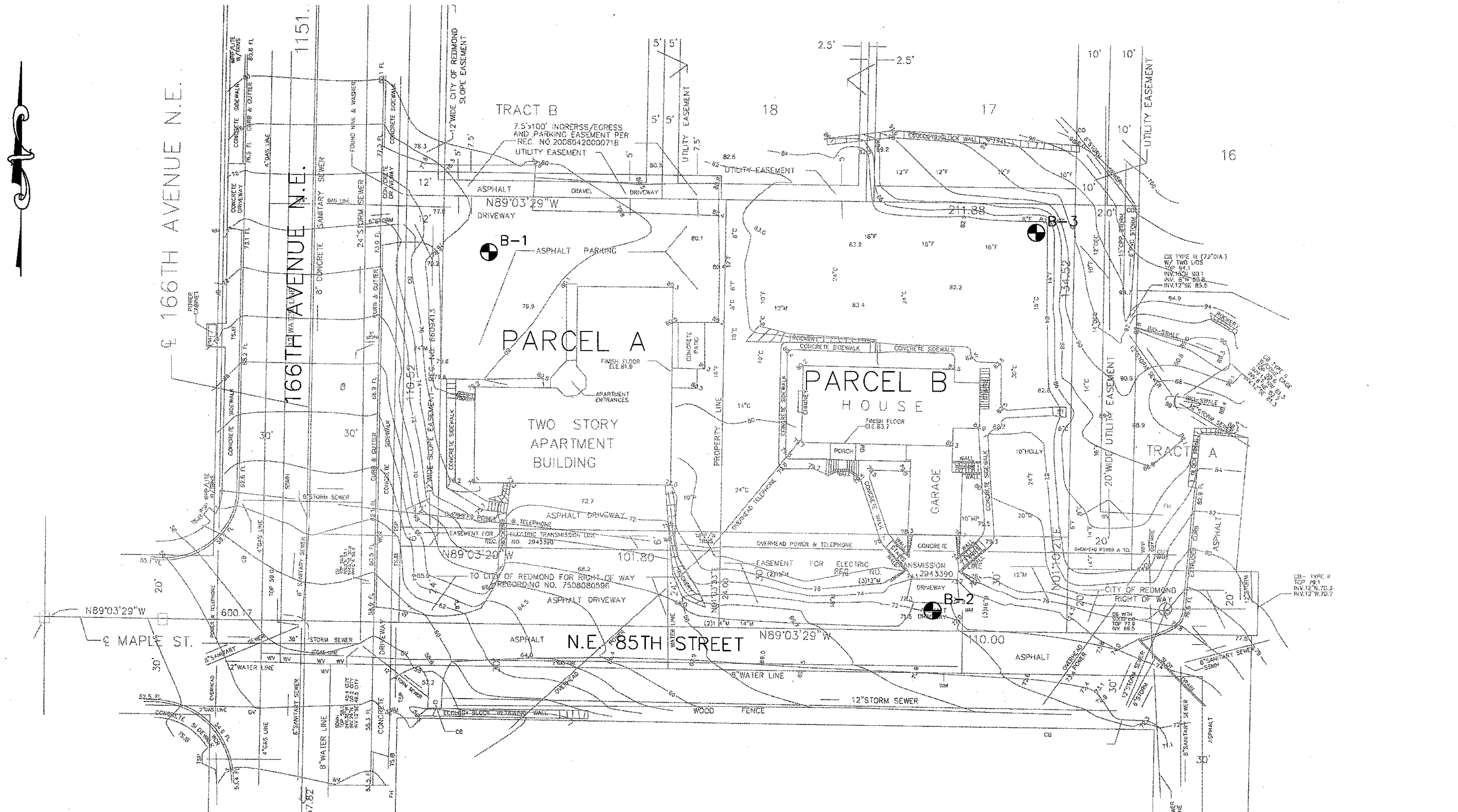
Respectfully submitted,
ZIPPER ZEMAN ASSOCIATES, INC.

9/21/07

ROBERT A. ROSS
STATE OF WASHINGTON
38384
REGISTERED
PROFESSIONAL ENGINEER
EXPIRES: 09/04/08

Robert A. Ross, P.E.
Senior Project Engineer

Enclosures: Figure 1 – Site and Exploration Plan
Figure 2 – Soldier Pile Shoring Pressure Diagram
Figure 3 – Basement Wall Permanent Drainage
Appendix A –Exploration Procedure and Logs



LEGEND:

B-1 BORING NUMBER AND APPROXIMATE LOCATION

0 30 60
SCALE IN FEET

ZZA
Zipper Zeman Associates, Inc.
Geotechnical and Environmental Consulting
18905 33rd Avenue West, Suite 117
Lynnwood, Washington 98036
Tele: (425) 771-3304 Fax: (425) 771-3549

Project No: 81065239

Drawn by:R. Ross

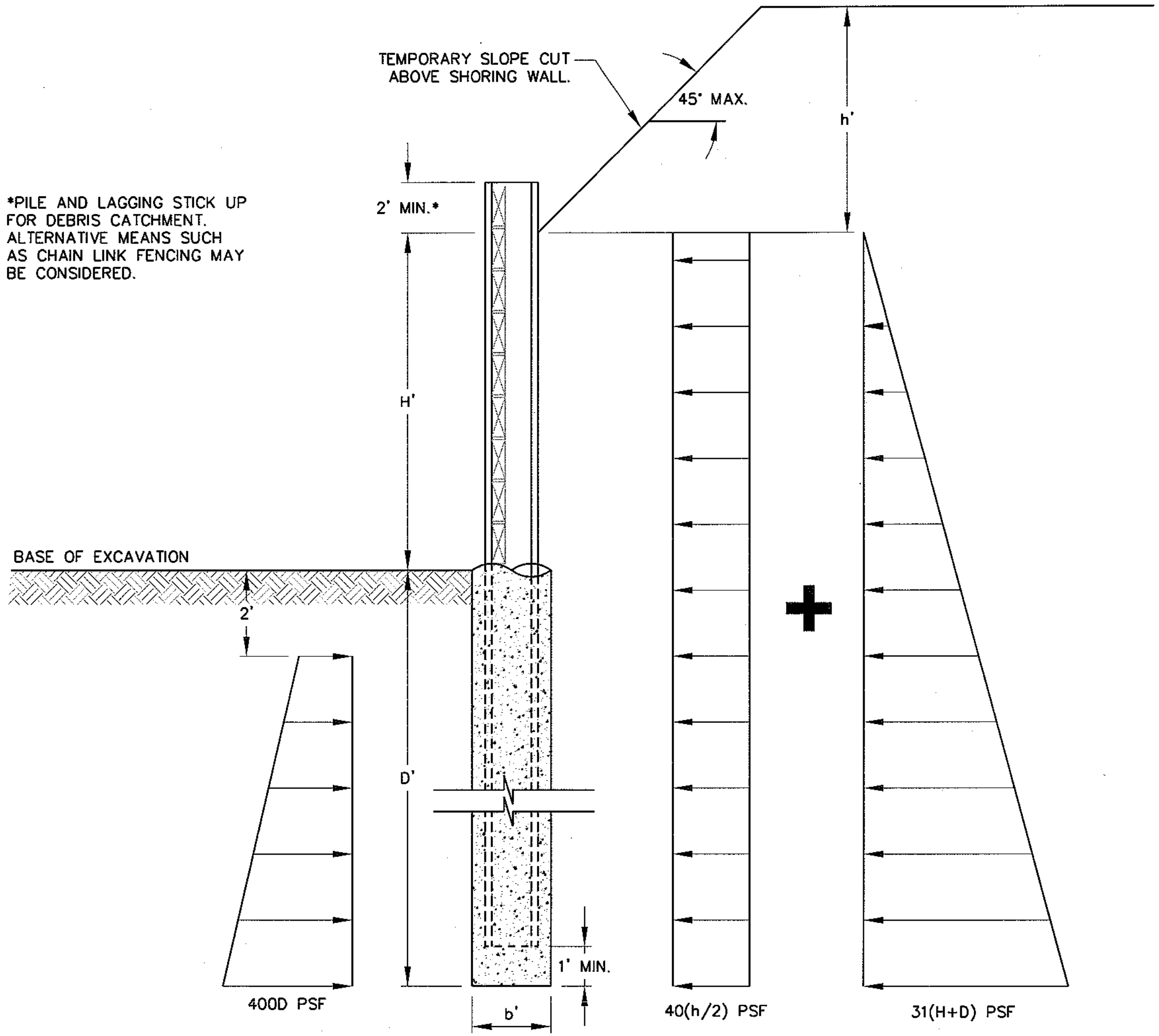
Date: Jan., 2007

Scale: As Noted

Redmond Town Center Condominums
8502 166th Ave. NE and 16933 NE 85th St.
Seattle, Washington

Figure 1: Site and Exploration Plan

Basemap DWG File Provided by others and modified by ZZA



NOTES:

1. ACTIVE PRESSURES MAY BE ASSUMED TO ACT OVER PILE SPACING ABOVE EXCAVATION BASE AND OVER ONE PILE DIAMETER BELOW BASE OF EXCAVATION.
2. PASSIVE PRESSURE MAY BE ASSUMED TO ACT OVER TWICE THE GROUTED SOLDIER PILE DIAMETER OR THE PILE SPACING, WHICH EVER IS SMALLER. PASSIVE PRESSURES INCLUDE A 1.5 FACTOR OF SAFETY.
3. TIMBER LAGGING MAY BE DESIGNED FOR 50% OF THE LATERAL EARTH PRESSURE USED FOR SHORING DESIGN.
4. ALL UNITS ARE IN POUNDS AND FEET.
5. SEE REPORT TEXT FOR ADDITIONAL INFORMATION.

1 / —

CANTILEVER SOLDIER PILE PRESSURE DIAGRAM

NOT TO SCALE

ZZA
Zipper Zeman Associates, Inc.
Geotechnical and Environmental Consulting
18905 33rd Avenue West, Suite 117
Lynnwood, Washington 98036
Tele: (425) 771-3304 Fax: (425) 771-3549

Project No: 81065239A

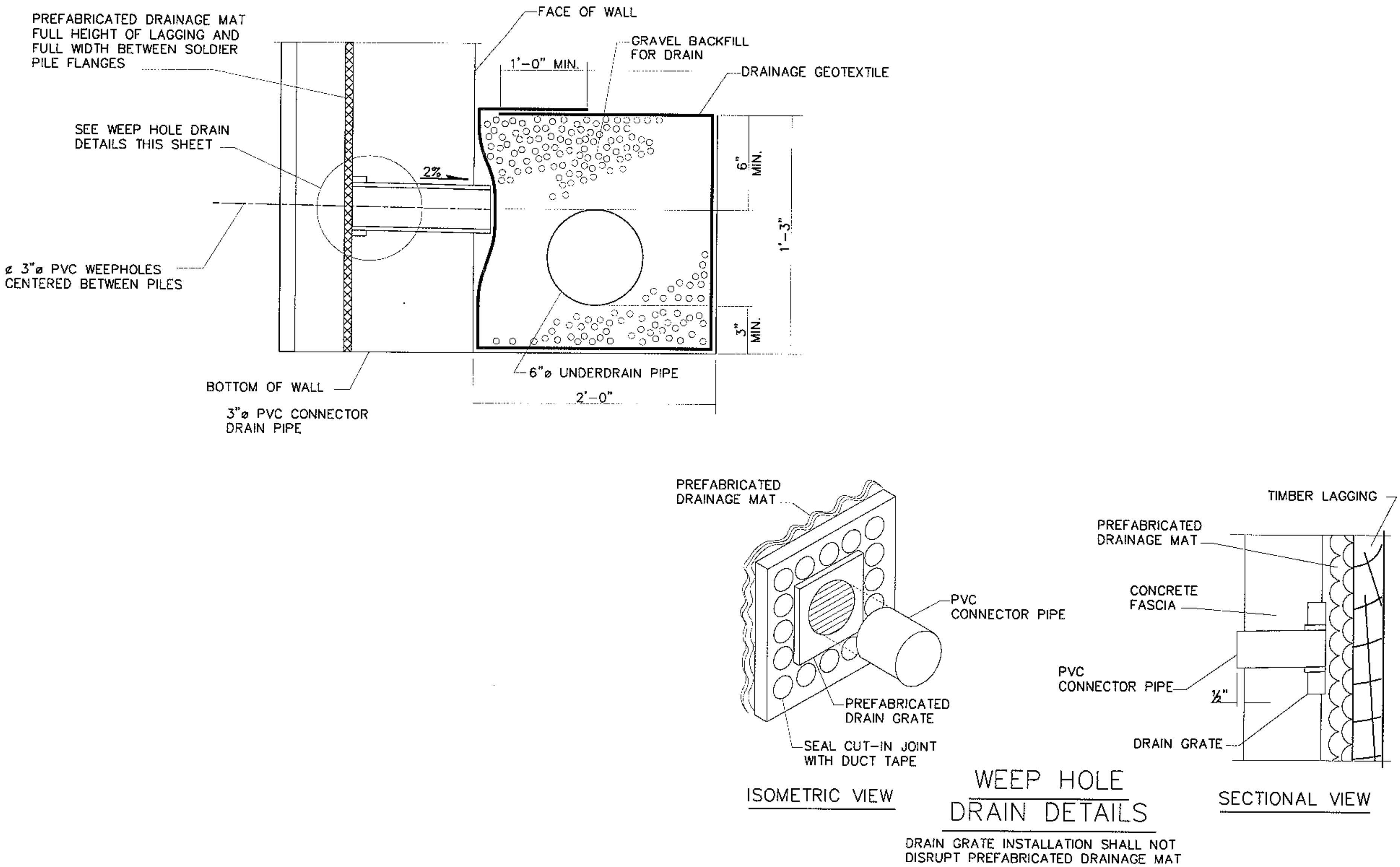
Drawn by: R. Ross

Date: Aug., 2007

Redmond Town Center Condominums
8502 166th Ave. NE and 16933 NE 85th St.
Seattle, Washington

Figure 2: Soldier Pile Shoring Pressure Diagram

ATTACHMENT 17



Zipper Zeman Associates, Inc. Geotechnical and Environmental Consulting 18905 33rd Avenue West, Suite 117 Lynnwood, Washington 98036 Tele: (425) 771-3304 Fax: (425) 771-3549	Project No. 81065239A Date: Sept., 2007 Drawn by: R.A. ROSS	Redmond Town Center Condominiums 8502 166th Ave. NE and 16640 NE 85th St. Redmond, Washington **FIGURE 3: BASEMENT WALL PERMANENT DRAINAGE**

APPENDIX A
FIELD EXPLORATION PROCEDURES AND LOGS
81065239A

Our field exploration for this project included 2 borings completed on December 19, 2006 and 1 boring completed on August 3, 2007. Exploration locations are shown on the Site and Exploration Plan, Figure 1. Exploration locations were approximated in the field using a measuring wheel with reference to existing boundary and topographic survey provided by Redmond Town Center Condominiums, LLC. As such, the exploration locations should be considered accurate to the degree implied by the measurement method. The approximate ground surface elevation at each exploration location was estimated based on the boundary and topographic survey provided by Redmond Town Center Condominiums, LLC. The following sections describe our procedures associated with the exploration. Descriptive logs of the explorations are enclosed in this appendix.

Soil Boring Procedures

Our exploratory borings were advanced with a hollow stem auger, using a trailer-mounted portable drill rig operated by an independent drilling firm working under subcontract to our firm. A geotechnical engineer from our firm continuously observed the borings logged the subsurface conditions encountered, and obtained representative soil samples. All samples were stored in moisture-tight containers and transported to our laboratory for further visual classification and testing. After each boring was completed, the borehole was backfilled with soil cuttings, and the surface was patched with bentonite grout.

Throughout the drilling operation, soil samples were obtained at 2.5- to 5-foot depth intervals by means of the Standard Penetration Test (ASTM: D-1586). This testing and sampling procedure consists of driving a standard 2-inch outside diameter steel split spoon sampler 18 inches into the soil with a 140-pound hammer free falling 30 inches. The number of blows required to drive the sampler through each 6-inch interval is recorded, and the total number of blows struck during the final 12 inches is recorded as the Standard Penetration Resistance, or "blow count" (N value). If a total of 50 blows is struck within any 6-inch interval, the driving is stopped and the blow count is recorded as 50 blows for the actual penetration distance. The resulting Standard Penetration Resistance values indicate the relative density of granular soils and the relative consistency of cohesive soils.

The enclosed boring logs describe the vertical sequence of soils and materials encountered in each boring, based primarily upon our field classifications and supported by our subsequent laboratory examination and testing. Where a soil contact was observed to be gradational, our logs indicate the average contact depth. Where a soil type changed between sample intervals, we inferred the contact depth. Our logs also graphically indicate the blow count, sample type, sample number, and approximate depth of each soil sample obtained from the boring, as well as any laboratory tests performed on these soil samples. If any groundwater was encountered in a borehole, the approximate groundwater depth, and date of observation, is depicted on the log. Groundwater depth estimates are typically based on the moisture content of soil samples, the wetted portion of the drilling rods, the water level measured in the borehole after the auger has been extracted, or through the use of an observation well.

The boring logs presented in this appendix are based upon the drilling action, observation of the samples secured, laboratory test results, and field logs. The various types of soils are indicated as well as the depth where the soils or characteristics of the soils changed. It should be noted that these changes may have been gradual, and if the changes occurred between samples intervals, they were inferred.

ATTACHMENT 17

PROJECT: Redmond Condominiums	JOB NO.: 81065239	BORING: B-1	PAGE 1 OF 1
Location: Redmond, WA	Approximate Surface Elevation: 79 feet		

Depth (ft)	Soil Description	Sample Type	Sample Number	Ground Water	Penetration Resistance (▲ Standard, △ Other; Blows per foot 0–50)	N-values	Testing
0	3 inches asphalt over moist, gray/brown, gravelly SAND with some silt						
	Very dense, moist, gray/brown, silty, gravelly SAND (Glacial Till)	2-inch O.D. split spoon	S-1			50/5"	
5	(Blowcount overstated)	2-inch O.D. split spoon	S-2			50/5"	
		2-inch O.D. split spoon	S-3			50/5"	
10	Very dense, moist, gray/brown silty, sandy GRAVEL with thin layers of wet sand with trace silt (Glacial Till)						
		2-inch O.D. split spoon	S-4			50/6"	
15				▼ ATD			
	Becomes wet to saturated.	2-inch O.D. split spoon	S-5			50/6"	
	Boring completed at 18 feet on 12/19/06. Groundwater observed at 16.5 feet at time of drilling.						
20							
25							

Explanation

- 2-inch O.D. split spoon sample
- 3-inch I.D Shelby tube sample
- ⊗ No Recovery
- ▼ ATD Groundwater level at time of drilling or date of measurement

Monitoring Well Key

- Clean Sand
- Bentonite
- Grout/Concrete
- Screened Casing
- Blank Casing

Moisture Content

Plastic Limit — Natural — Liquid Limit

Testing Key

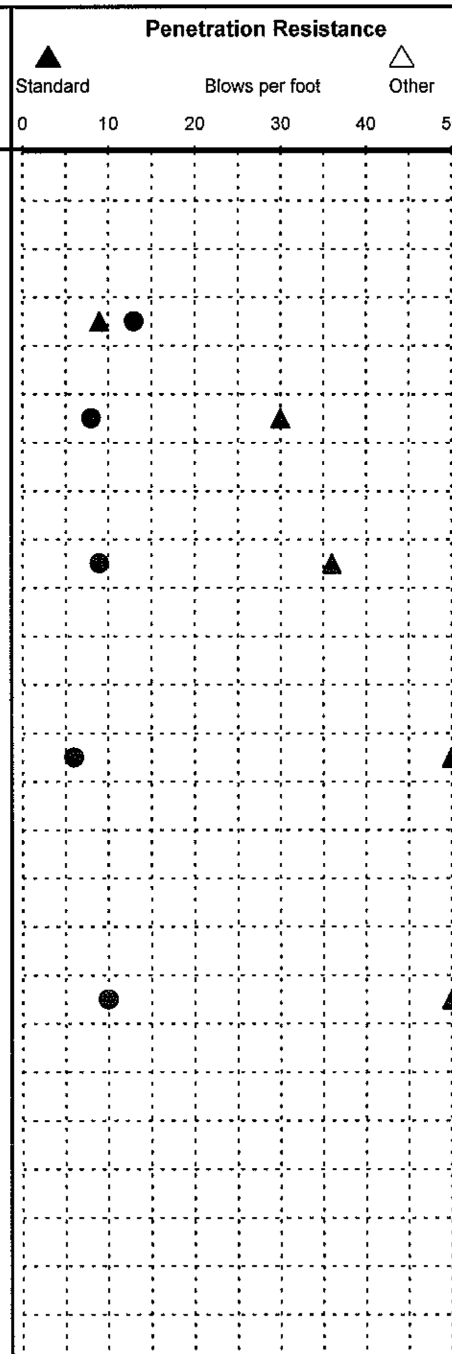
GSA = Grain Size Analysis
200W = 200 Wash Analysis
Att. = Atterberg Limits
Consol. = Consolidation Test

ZZA Zipper Zeman Associates, Inc. Geotechnical and Environmental Consulting	BORING LOG	Figure A-1
	Date Drilled: 12/19/2006	Logged By: RWS

ATTACHMENT 17

PROJECT: Redmond Condominiums	JOB NO.: 81065239	BORING: B-2	PAGE 1 OF 1
Location: Redmond, WA	Approximate Surface Elevation: 72 feet		

Depth (ft)	Soil Description	Sample Type	Sample Number	Ground Water	N-values	Testing
0	3 inches asphalt over loose, wet, black/brown, silty, gravelly SAND (Fill)					
		2-inch O.D. split spoon sample	S-1		9	
5	Medium dense to dense, wet, mottled gray/brown, silty, gravelly SAND (Weathered Till)	2-inch O.D. split spoon sample	S-2	ATD	30	
		2-inch O.D. split spoon sample	S-3		36	
10						
	Very dense, moist, gray/brown silty, sandy GRAVEL with thin layers of wet, sand with trace silt (Glacial Till)	2-inch O.D. split spoon sample	S-4		100/5"	
15						
	Becomes wet to saturated.	2-inch O.D. split spoon sample	S-5		50/5"	
	Boring completed at 18 feet on 12/19/06. Groundwater observed at 6 feet at time of drilling.					
20						
25						



Explanation

- 2-inch O.D. split spoon sample
- 3-inch I.D Shelby tube sample
- No Recovery
- ATD Groundwater level at time of drilling or date of measurement

Monitoring Well Key

- Clean Sand
- Bentonite
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- Screened Casing
- Blank Casing

Moisture Content

Plastic Limit — Natural — Liquid Limit

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GSA = Grain Size Analysis
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Zipper Zeman Associates, Inc. Geotechnical and Environmental Consulting	BORING LOG	A-2
	Date Drilled: 12/19/2006	Logged By: RWS

PROJECT: Redmond Condominiums	JOB NO.: 81065239A	BORING: B-3	PAGE 1 OF 3
Location: Redmond, WA	Approximate Surface Elevation: 84 feet		

Depth (ft)	Soil Description	Sample Type	Sample Number	Ground Water	Penetration Resistance (▲ Standard, △ Other; Blows per foot 0–50)	N-values	Testing
0	4" Sod and Topsoil						
	Very dense, moist, gray-brown, gravelly, silty, fine SAND (glacial till)	I	S-0		▲	80/11"	
5		I	S-1		▲	50/5"	
10		I	S-2		▲	50/5"	
15		I	S-3		▲	50/5"	
	Hard, moist, gray, gravelly SILT with some sand						
20		I	S-4		▲	84	
	Very dense, wet to saturated, gray, gravelly, silty, fine SAND (glacial till)						
25							

Explanation

- 2-inch O.D. split spoon sample
- 3-inch I.D Shelby tube sample
- ⊗ No Recovery
- ▼ ATD Groundwater level at time of drilling or date of measurement

Monitoring Well Key
- Clean Sand
- Bentonite
- Grout/Concrete
- Screened Casing
- Blank Casing

Moisture Content

Plastic Limit — Natural — Liquid Limit

Testing Key
- GSA = Grain Size Analysis
- 200W = 200 Wash Analysis
- Att. = Atterberg Limits
- Consol. = Consolidation Test

ZZA Zipper Zeman Associates, Inc. Geotechnical and Environmental Consulting	BORING LOG	Figure A-3
	Date Drilled: 08/03/2007	Logged By: RAR

ATTACHMENT 17

PROJECT: Redmond Condominimums	**JOB NO.:** 81065239A	**BORING:** B-3	**PAGE 2 OF 3**
Location: Redmond, WA	**Approximate Surface Elevation: 84 feet**		

Depth (ft)	Soil Description	Sample Type	Sample Number	Ground Water	Penetration Resistance (Standard ▲ / Other △, Blows per foot 0–50)	N-values	Testing
25	Very dense, wet to saturated, gray, gravelly, silty, fine SAND (glacial till)	2-inch O.D. split spoon	S-5		▲	50/5"	
	Hard, moist, gray, sandy SILT with trace gravel						
30		2-inch O.D. split spoon	S-6		▲	92	
35		2-inch O.D. split spoon	S-7		▲	82/11"	
40		2-inch O.D. split spoon	S-8		▲	58	
45	Hard, moist, gray, sandy SILT with trace gravel	2-inch O.D. split spoon	S-9		▲	36	
50							

Explanation

- I 2-inch O.D. split spoon sample
- II 3-inch I.D Shelby tube sample
- ⊗ No Recovery
- ▼ ATD Groundwater level at time of drilling or date of measurement

Monitoring Well Key

- Clean Sand
- Bentonite
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- Blank Casing

Moisture Content

Plastic Limit — Natural — Liquid Limit

Testing Key

GSA = Grain Size Analysis
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ZZA Zipper Zeman Associates, Inc. Geotechnical and Environmental Consulting	**BORING LOG**	**Figure A-3**
	Date Drilled: 08/03/2007	**Logged By: RAR**

PROJECT: Redmond Condominiums	JOB NO.: 81065239A	BORING: B-3	PAGE 3 OF 3
Location: Redmond, WA	Approximate Elevation: 84 Feet		

Depth (ft)	Soil Description	Sample Type	Sample Number	Ground Water	Penetration Resistance (▲ Standard, △ Other; Blows per foot 0–50)	N-values	Testing
50	Hard, moist, gray, sandy SILT with trace gravel	I	S-10		▲ (>50)	55	
	Boring B-3 completed at approximatley 51.5 feet on 8/3/07. No groundwater observed at time of drilling.						
55							
60							
65							
70							
75							

Explanation

I	2-inch O.D. split spoon sample
II	3-inch I.D Shelby tube sample
⊗	No Recovery
▼ ATD	Groundwater level at time of drilling or date of measurement

Monitoring Well Key
- Clean Sand
- Bentonite
- Grout/Concrete
- Screened Casing
- Blank Casing

Moisture Content

Plastic Limit — Natural — Liquid Limit

Testing Key
- GSA = Grain Size Analysis
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- Consol. = Consolidation Test

Zipper Zeman Associates, Inc. Geotechnical and Environmental Consulting	BORING LOG	Figure A-3
	Date Drilled: 08/03/2007	Logged By: RAR

ATTACHMENT 17

ZZA Zipper Zeman Associates, Inc.
Geotechnical and Environmental Consulting
A Terracon Company

Job No. 81065239
January 17, 2007

Redmond Town Center Condominiums, LLC
333 156th St. NE
Arlington, WA 98223

Attention: Mr. Todd Leabman

Subject: Geotechnical Report
Proposed Multi-Family Development
8502 166th Ave. NE and 16933 NE 85th St.
Redmond, Washington

Dear Mr. Leabman:

Zipper Zeman Associates, Inc. (ZZA) is pleased to present a copy of the above-referenced report. This report presents results of our geotechnical study of the proposed multi-family residential development on two lots located at 8502 166th Ave. NE and 16933 NE 85th St. Our work was completed in general accordance with the scope of services described in our Proposal (P-3597) dated December 4, 2006 and subsequently authorized Mr. Jesse Molnick on December 6, 2006. The purpose of our services was to complete subsurface explorations as a basis for providing geotechnical recommendations for the project. Our scope of services included completing subsurface explorations, laboratory testing, geotechnical analysis and preparation of this report.

PROJECT DESCRIPTION

Currently, plans have not been developed for the project. However, based on our conversations with you, we understand the project may consist of constructing a four story multi-family residential building with two levels of underground parking. The project will include additional improvements such as underground utilities and stormwater facilities. Detailed plans for the project have not been developed. Once they become available, ZZA should be provided an opportunity to review the plans and revised the recommendations provided in this report if necessary.

SITE CONDITIONS

Surface Conditions

Our observations of surface conditions are based on a site reconnaissance and review of a topographic survey of the properties completed by Harstad Consultants. The project site consists of two parcels totaling approximately 26,000 square feet. Parcel A is located at 8502 166th Avenue NE and Parcel B, which abuts Parcel A to the east, is located at 16933 NE 85th Street Parcel A is currently developed with a two story multi-family residential building and associated parking. Parcel B is currently developed with a single-family residential home. The project site is bordered to the north and east by a residential development currently under construction. The site is bordered to the south by NE 85th Street and to the west by 166th Avenue NE.

A majority of the project site is generally level with ground surface elevations ranging from 80 to 84 feet (NGVD 1929 datum). However, near the west property line, the ground surface slopes downward to 166th Avenue NE. Near the south property line, the ground surface slopes downward to NE 85th Street. There is limited vegetation on Parcel A. However, the existing house on Parcel B is surrounded by deciduous and conifer trees. We did not observe surface water on the project site during our reconnaissance.

Subsurface Conditions

Subsurface soil conditions at the project site were evaluated by completing two geotechnical test borings at the locations shown on Figure 1, Site and Exploration Plan. The borings were completed to depths of approximately 18 feet below the existing ground surface at the exploration locations. The explorations were continuously monitored by a geotechnical engineer from our firm. Soils were visually classified in general accordance with the Unified Soil Classification System. Subsurface exploration procedures and logs for the explorations are enclosed in Appendix A of this report.

Soil and groundwater descriptions presented in this report are based on the subsurface conditions encountered at specific exploration locations on the site. Variations in subsurface conditions may exist between the exploration locations and the nature and extent of variations between the explorations may not become evident until construction. If variations then appear, it may be necessary to reevaluate the recommendations of this report. The descriptions of soil conditions provided below are generalized. The exploration logs provided in Appendix A should be referred to for detailed information regarding soil conditions observed at each boring location.

Boring B-1 was completed in the north parking lot area of Parcel A. Subsurface soil conditions observed in boring B-1 generally consisted of 3 inches of asphalt concrete pavement underlain by very dense, moist to wet, silty, gravelly sand. The silty, gravelly sand is interpreted to be glacial till. Boring B-1 was completed at approximately 18 feet below the existing ground surface within the glacial till soil unit.

Boring B-2 was completed at the south end of the driveway for the residence located on Parcel B. Subsurface soil conditions observed in boring B-2 generally consisted of 3 inches of asphalt concrete pavement underlain by loose, wet, silty, gravelly sand (fill) extending to about 4-½ feet below the existing ground surface. The fill was underlain by medium dense to dense, wet, silty, gravelly, sand (weathered glacial till) extending to about 12 feet below the existing ground surface. The weathered glacial till was underlain by very dense, moist, silty, sandy, gravel with thin layers of sand with trace silt (unweathered glacial till). Boring B-2 was completed in the glacial till soil unit at approximately 18 feet below the existing ground surface.

Groundwater Conditions

Groundwater was observed in boring B-1 at about 16.5 feet below the existing ground surface at the time of drilling. Groundwater observed in this boring is interpreted to be thin, saturated sand zones within the glacial till.

Groundwater observed in boring B-2 at about 6 feet below the existing ground surface. Groundwater observed in this boring is interpreted to be perched groundwater. Perched groundwater conditions develop when the downward migration of surface water is impeded by a relatively impermeable soil layer such as the very dense glacial till soils observed in boring B-2. The thickness of saturated soil resulting from perched groundwater conditions is typically thin. Perched groundwater is recharged primarily by precipitation. As a result, the saturated zone will tend to be thicker during wet weather.

CONCLUSIONS AND RECOMMENDATIONS

General Summary

Based on the subsurface soil and groundwater conditions observed in our borings, it is our opinion that the new building can be adequately supported on conventional shallow foundations that bear on the dense to very dense glacial soils observed in our borings. Specific geotechnical recommendations and discussions are provided in subsequent sections of this report.

Stormwater Infiltration Feasibility Discussions

As part of our scope of services, we evaluated the feasibility of stormwater infiltration for the project. Stormwater infiltration is typically only feasible in soil conditions consisting of sands and gravels with a relatively low silt and clay content (soil finer than the U.S. No. 200 sieve). Soils observed in our explorations consisted of a highly compact mixture of sand, gravel, silt and clay. The soil encountered in our explorations is interpreted to be glacial till, or commonly referred to as hardpan. The infiltration rate of undisturbed glacial till is extremely slow, and for practical purposes, glacial till can be considered impermeable. As a result, it is our opinion that infiltration into the glacial till soils is not feasible.

It should be noted that glacial till soils are typically underlain by a soil unit referred to as advance outwash. The advance outwash typically consists of sand and gravel with a low fines content and is typically well suited for infiltration. However, our borings did not encounter the advance outwash soil unit within the depths explored (approximately 18 feet below the existing ground surface at the exploration locations). One option to further evaluate the feasibility of stormwater infiltration would be to complete an additional boring deeper than the previously completed explorations in an effort to locate the elevation of the advance outwash soil unit. Prior to considering an additional boring, we recommend the project team consult with the City of Redmond regarding stormwater management code requirements and options. For certain types of projects, we understand that storm water requirements can be waived for a fee. Additionally, in some areas of Redmond, stormwater infiltration is not allowed in order to protect a drinking water aquifer located below Redmond.

Seismic Considerations

International Building Code 2003 requires a Site Class definition as well as other properties for development of the building general seismic design response spectrum. Based on soil conditions observed in our borings, we estimate that the average properties of the upper 100 feet of the site profile correspond to Site Class C. Site Class C consists of a "Very dense soil and soft rock" soil profile as defined by the 2003 IBC. A very dense soil and soft rock soil profile is characterized by an average standard penetration resistance (as defined by Section 1615.1.5 of 2003 IBC) greater than 50.

We evaluated the potential for seismic induced soil liquefaction at the site. Liquefaction typically occurs in loose to medium dense, granular soils located below the groundwater table. It is our opinion that the soils observed in our borings are not susceptible to liquefaction.

Site Preparation

Prior to site preparation, temporary erosion and sediment control measures (TESC) should be installed in accordance with the appropriate standards for the project. Once TESC measures have been installed, we expect that site preparation will continue with demolition of existing structures. All elements of existing buildings including concrete foundation elements should be demolished and removed from the site. All existing underground utilities should be properly abandoned by complete removal or capping and filling with cement grout. If existing underground utilities are excavated and removed, all excavations should be backfilled with structural fill as recommended in the **Structural Fill** section of this report.

Once existing structures have been removed, we expect site preparation will continue with clearing and stripping of existing vegetation in the undeveloped areas of the site. All tree stumps and roots larger than ½ inch diameter should be cleared and grubbed from building and pavement areas. All topsoil should be stripped from the site.

Soils observed in our explorations contain a significant fraction of fines (silt and clay sized soil particles). During wet weather, exposed site soils will quickly become unstable and soft. In order to limit subgrade stability problems and grading difficulties, adequate temporary and permanent control of surface water runoff will be required. Excavation, filling, subgrade and grade preparation should be performed in a manner and sequence that will provide drainage at all times and proper control of erosion. Surface water should be pumped or drained to provide a suitable working platform. The site should be graded to prevent water from ponding in construction areas and/or flowing into excavations. Exposed grades should be crowned, sloped, and smooth-drum rolled at the end of each day to facilitate drainage. In order to protect the subgrade, a working surface of quarry spalls may be required. Additionally, temporary cut slopes should be protected from erosion through the use of anchored plastic sheeting.

All areas to receive new structural fill should be evaluate by Zipper Zeman Associates to asses the suitability of subgrade conditions. Any loose or otherwise unsuitable soils should be removed and replaced with structural fill as recommended in the **Structural Fill** section of this report. Additionally, sloping ground surfaces should be terraced prior to placing structural fill. Each terrace should penetrate the slope at least 5 feet and not be more than 5 feet high. The horizontal face of each terrace should slope outward at approximately 0.05 foot per foot.

Temporary and Permanent Cut and Fill Slopes

Temporary cut slopes may be required for various aspects of the project. Temporary slope stability is a function of many factors, including the following:

- The presence and abundance of groundwater;
- The type and density of the various soil strata;
- The depth of cut;
- Surcharge loadings adjacent to the excavation;
- The length of time the excavation remains open.

It is exceedingly difficult under the variable circumstances to pre-establish a safe and "maintenance-free" temporary cut slope angle. Therefore, it should be the responsibility of the contractor to maintain safe temporary slope configurations since the contractor is continuously at the job site, able to observe the nature and condition of the cut slopes, and able to monitor the subsurface materials and groundwater conditions encountered. It may be necessary to drape temporary slopes throughout the site with plastic sheeting or other means to protect the slopes from the elements and minimize sloughing and erosion. Unsupported vertical slopes or cuts deeper than 4 feet are not recommended if worker access is necessary. The cuts should be adequately sloped, shored, or supported to prevent injury to personnel from local sloughing and spalling. The excavation should conform to applicable federal, state, and local regulations.

For planning purposes, we recommend temporary cut slopes completed in the dense to very dense glacial soils be planned no steeper than 0.75H:1V (horizontal to vertical). Flatter temporary slopes may be required if groundwater seepage is encountered, or for temporary cuts made in fill or the weathered glacial till soils observed in our borings. All temporary cut slopes should be constructed in general accordance with the Washington Administrative Code, Section 296-115; Part N, Excavation and Shoring.

We recommend that all new unsupported permanent cut or fill slopes be designed at a 2H:1V (Horizontal:Vertical) inclination or flatter. Cut or fill slopes in areas that will experience periodic wetting such as stormwater ponds or bio-swales should be designed at 3H:1V. All permanent slopes must be adequately protected from erosion.

Temporary Shoring

Temporary shoring may be required for various aspects of the project including underground utilities and possibly the building excavation depending on the desired levels of below grade parking. For underground utility excavations, we expect the use of trench boxes will be appropriate. However, it should be noted that the purpose of a trench box is to provide safety for workers inside the excavation, and not for excavation support. The side walls of temporary excavations must become stable prior to installation of a trench box. The upper fill and weathered glacial till soils may tend to cave prior to installation of trench boxes. Such caving may tend to destabilize adjacent existing facilities. In such situations, other methods of temporary shoring may be required.

Depending on the desired levels of below grade parking, temporary structural shoring may be required for the project. Several methods of shoring could be considered. However, based on soil and groundwater conditions observed in our borings, it is our opinion that soldier pile and lagging or soil nailing would best suite the project. Soil nailing is typically more economical compared to soldier pile and lagging.

Soldier pile shoring is constructed from the top down. Construction begins by installing vertical members consisting of steel I-beams in pre-augured holes typically spaced at about 5 to 10 feet on center. The holes are then backfilled with structural concrete extending up to the bottom of the proposed cut elevation at the face of the wall. The remaining depths of the holes are filled with lean mix concrete. Once the concrete has set, the excavation begins from the top down in lifts. As each lift is completed, timber lagging, typically consisting of 3" by 6" or 4" by 6" treated timbers, are installed between the flanges of the I-beams. Depending on the shored height of the wall, horizontal members, or tie-backs may be required. Typically, soldier pile shoring can be constructed to maximum shored heights of about 15 feet without tie-backs. However, if settlement sensitive facilities are located close to the shoring, tie-backs may be required.

Soil nail shoring is also constructed from the top down. Soil nail shoring consists of excavating soils in vertical lifts and installation of nearly horizontal elements called soil nails back through the soil cut face typically at a horizontal spacing of 5 to 6 feet. Once a lift is complete and the nails installed, reinforcing steel is placed on the cut face and a fascia consisting of pneumatically placed concrete, or shotcrete is placed on the cut face. The procedure is repeated until the bottom of the excavation is reached. Soil nailing is most applicable in soils that exhibit a significant standup time when cut vertically. However, some methods can be used to stabilize soils with marginal standup times. Soil nailing is almost always more economical as compared to solider pile shoring.

As indicated above, the project is currently at a conceptual level. Once the project moves further into design, we can provide additional consultation, design parameters, and design services for temporary shoring upon request.

Structural Fill

All structural fill should be placed in accordance with the recommendations presented herein. Structural fill includes any fill material placed under footings, below pavement subgrades, and utility trench backfill. Prior to the placement of structural fill, all surfaces to receive fill should be prepared as previously recommended in the **Site Preparation** section of this report.

We expect structural fill will be required to backfill foundation excavations and for utility trench backfill. The suitability of soil for use as structural fill will depend on the time of year of construction, the moisture content of the soil, and the fines content (that portion passing the U.S. No. 200 sieve) of the soil. As the amount of fines increases, the soil becomes increasingly sensitive to small changes in moisture content. Soils containing more than about 5% fines cannot be consistently compacted to the appropriate levels when the moisture content is more than approximately 2% above or below the optimum moisture content (per ASTM D-1557). Optimum moisture content is that moisture which results in the greatest compacted dry density with a specified compactive effort.

The soils encountered in our explorations are estimated to contain between 20 to 30 percent fines. During wet weather, site soils may not be suitable for reuses as structural fill. During extended periods of dry, warm weather, we expect site soils to be suitable for reuses as structural fill. Site soils will not be suitable for backfill directly against subgrade foundation walls because the soil is not free draining. Additional recommendations for backfilling subgrade walls are presented in the **Subgrade Walls** section of this report.

If required, imported structural fill should consist of material meeting the requirements of WSDOT 9-03.04.14(1) Gravel Borrow during wet weather. During dry weather, a lesser quality material meeting the requirements of 9-03.14(3) Common Borrow could be used. However, the use of lesser quality imported fill should be based on site specific conditions during construction and recommendations provided by ZZA. Requirements regarding the recommended fill types can be found in the 2006 Washington State Department of Transportation Standard Specifications for Road, Bridge, and Municipal Construction.

Structural fill should be placed in lifts not exceeding 12 inches in loose thickness. Individual lifts should be compacted such that a minimum density of at least 95 percent of the modified Proctor (ASTM D1557) is achieved and the fill is firm and unyielding. We recommend that a representative from ZZA be present during the placement of structural fill to observe the work and perform a representative number of in-place density tests. In this way, the adequacy of earthwork may be evaluated as grading progresses.

It should be noted that the effort required for successful placement of structural fill is weather-dependent and delays due to inclement weather are common even when using Gravel Borrow. Excess soils may require stockpiling for extended periods of time before they can be used. We recommend that all stockpiled soils intended for reuse as structural fill be protected with anchored polyethylene sheet plastic strong enough to withstand local wind conditions.

Building Foundations

Based on soil conditions observed in our explorations, it is our opinion that the proposed building could be supported on conventional spread footings founded on the dense to very dense glacial till soils observed in our explorations.

Spread footings founded on the dense to very dense glacial till soils observe in our explorations may be designed for an allowable bearing pressure of 4,000 pounds per square foot (psf). The allowable bearing values provided above apply to the sum of all dead and long-term live loads, excusive of the weight of the footing and any backfill above the footing. For total loads including wind or seismic, a one-third increase on the above-recommended allowable bearing value may be used. All footings should be embedded at least 1.5 feet below finished exterior grades for frost protection. We recommend that continuous and isolated pad footings have minimum widths of 18 and 24 inches, respectively. Provided that spread footings are founded as recommended in this report, we estimate that total foundation settlement will be on the order of 1 inch and differential settlements on the order of ½ inch in 50 feet.

The allowable bearing values and predicted settlements discussed above are based on an undisturbed subgrade. As discussed above, the native soils will easily become disturbed during wet weather, and also may become disturbed due to construction traffic. Any disturbance to footing subgrades should be repaired prior to placement of reinforcing steel by overexcavating the disturbed areas and replacing with crushed rock meeting the requirements of WSDOT 9-03.9(3) Crushed Surfacing Base Course. We recommend that footing subgrades be evaluated by ZZA prior to the placement of reinforcing steel.

Slab On Grade Floors

It is our opinion that slab-on-grade floors can be supported on site soils prepared in accordance with the **Subgrade Preparation** section of this report, or on structural fill placed in accordance with the recommendations of this report. We recommend that floor slabs be underlain by a minimum 6-inch thickness of ¾-inch washed crushed rock to serve as a working surface and a capillary break. This capillary break layer should be compacted to a firm and unyielding condition, and achieve a uniform compaction level of at least 95 percent of the maximum dry density per the ASTM D-1557 test method. Placement of the capillary break material should be sequenced such that the potential for the material to become saturated by rainfall or other water sources can be limited.

Where transmission of water vapor through slabs is undesirable, we recommend a vapor retarder be installed. The vapor retarder should consist of polyethylene sheet plastic that is at least 10-mil thick or a suitable proprietary product approved by the owner. The slab designer and slab contractor should refer to ACI 302 for procedures and cautions regarding the use and placement of a vapor retarder. In addition, the moisture protection details should be reviewed by the architect and owner and additional, more stringent moisture protection details should be specified if required for protection of floor finishes. If the roof membrane will be in place prior to pouring the slab, the vapor barrier should be placed below the capillary break. If the roof membrane will not be in place, the vapor barrier should be placed directly below the slab. The following additional recommendations are provided for vapor barrier installation:

- All joints should be lapped and sealed.
- All penetrations through the vapor barrier should be sealed.
- The vapor barrier should be lapped over footings, sealed to foundation walls or both.
- Any damage to the vapor barrier should be repaired prior to pouring the slab

Lateral Resistance

Lateral loads can be resisted by a combination of passive pressures acting on the face of buried foundation elements and base friction on the bottom of foundation elements. The allowable passive pressures on the face of foundation elements at least 1.5 feet below finished grade and cast neat against site soils may be computed using and equivalent fluid density of 400 pounds per cubic foot (pcf) (triangular distribution) for a level ground surface. The above passive pressure value includes a factor of safety of 1.5. We recommend using an ultimate base friction coefficient of 0.40 for concrete in contact with the soils observed in the explorations or structural fill placed in accordance with this report.

Subgrade Foundation Walls

The lateral soil pressures acting on subgrade walls will depend on the nature and density of the soil behind the wall, and the amount of lateral wall movement that can occur as backfill is placed. For walls that are free to yield at the top at least one-thousandth of the height of the wall, soil pressures will be less than if movement is limited by such factors as wall stiffness or bracing. Assuming that walls are backfilled and drained as described in the following paragraphs, we recommend that yielding walls supporting horizontal backfill be designed using an equivalent fluid density of 35 pcf. Non-yielding walls should be designed using an equivalent fluid density of 50 pcf. Passive soil resistance and base friction values for design of subgrade foundation walls are provide above in the **Lateral Resistance** section of this report.

In addition to active or at-rest earth pressures as recommended above, permanent subgrade foundation walls should be designed to resist seismic lateral earth pressures. Figure 2 provides our recommendations for analysis of subgrade foundation walls subject to seismic earth pressures for both yielding and non-yielding walls.

The above-recommended lateral earth pressures do not include the effects of sloping backfill surfaces, surcharges such as traffic loads, other surface loading, or hydrostatic pressures. If such conditions exist, we should be consulted to provide revised earth pressure recommendations.

Adequate drainage measures must be installed to collect and direct subsurface water away from subgrade walls. The appropriate drainage system for subgrade foundation walls will depend on whether or not temporary shoring is required for the project. We can provide final recommendations regarding subgrade foundation wall drainage once the project moves further into design.

Wall backfill should be compacted to between 90 and 92 percent of the maximum dry density as determined by the ASTM D 1557 test method. Measures should be taken to prevent the buildup of excess lateral pressures due to overcompaction of the backfill behind the wall. This can be accomplished by placing the backfill within 24 inches of the wall in lifts not exceeding 6 inches in loose depth and compacting with hand-operated or self-propelled, light compaction equipment.

Use of the recommended reduced compaction levels for wall backfill may result in some backfill settlement with time. If sidewalks, planters, or other features are constructed above the backfill and cannot tolerate differential settlement in the range of 1 to 2 inches, higher compaction levels should be specified for the backfill below these features. Care should be taken where utilities penetrate through basement walls. Minor settlement of the backfill can put significant soil loading on utilities, and some form of flexible connection may be appropriate at backfilled wall penetrations.

CLOSURE

We have prepared this report for use by Redmond Town Center Condominiums, LLC for this project. The data and report should be provided to prospective contractors for bidding or estimating purposes, but our report, conclusions and interpretations should not be construed as a warranty of the subsurface conditions.

There are possible variations in subsurface conditions across the site and also with time. A contingency for unexpected conditions should be included in the project budget and schedule. Sufficient monitoring, testing and consultation should be provided by our firm during construction to confirm that the conditions encountered are consistent with those indicated by the explorations, to provide recommendations for design changes should the conditions revealed during the work differ from those anticipated, and to evaluate whether or not construction activities comply with the contract plans and specifications.

Within the limitations of scope, schedule and budget, our services have been executed in accordance with generally accepted practices in this area at the time the report was prepared. No warranty or other conditions express or implied should be understood.

Respectfully submitted,
ZIPPER ZEMAN ASSOCIATES, INC.

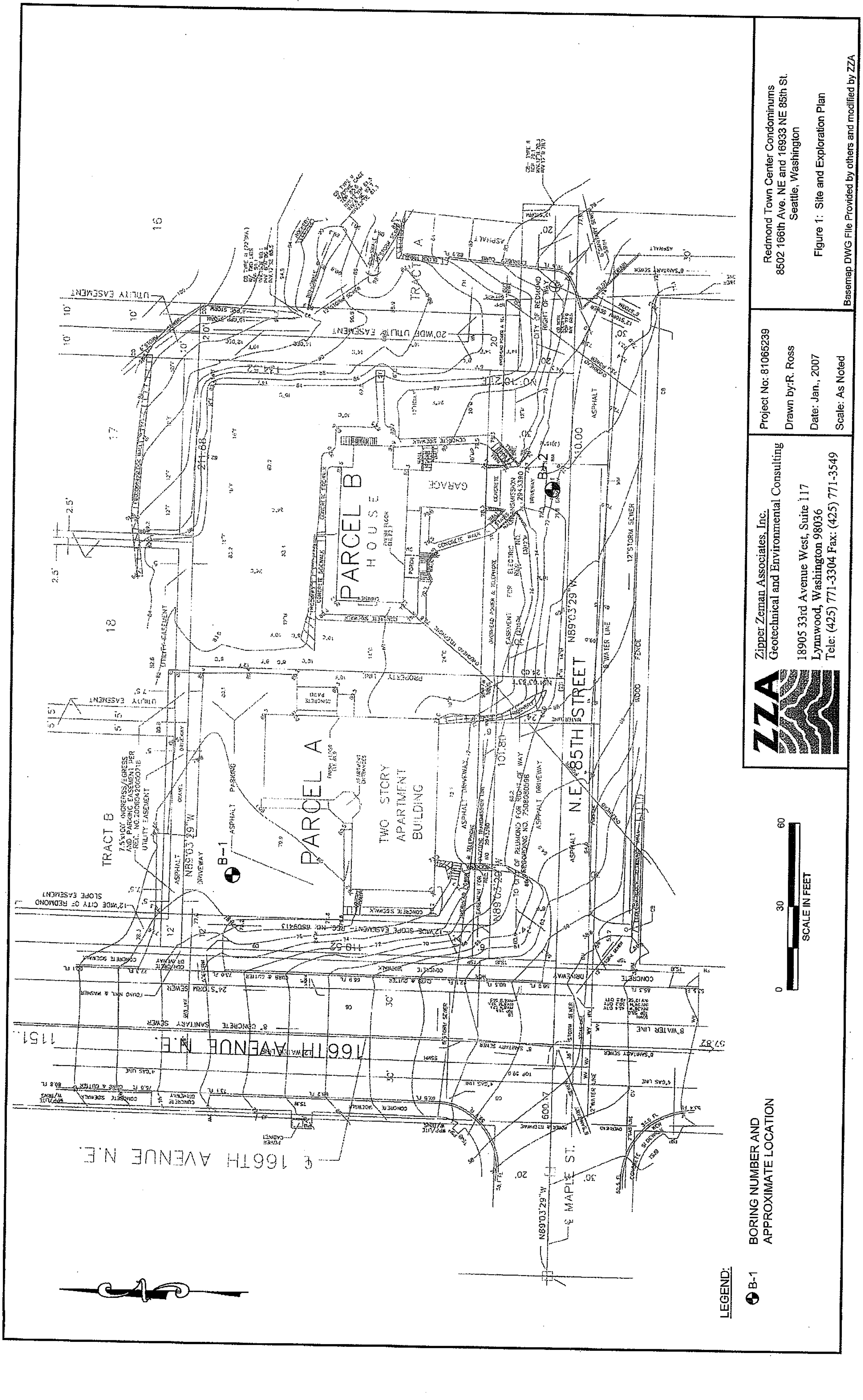
1/17/07

ROBERT A. ROSS
STATE OF WASHINGTON
38384
REGISTERED
PROFESSIONAL ENGINEER
EXPIRES: 09/04/08

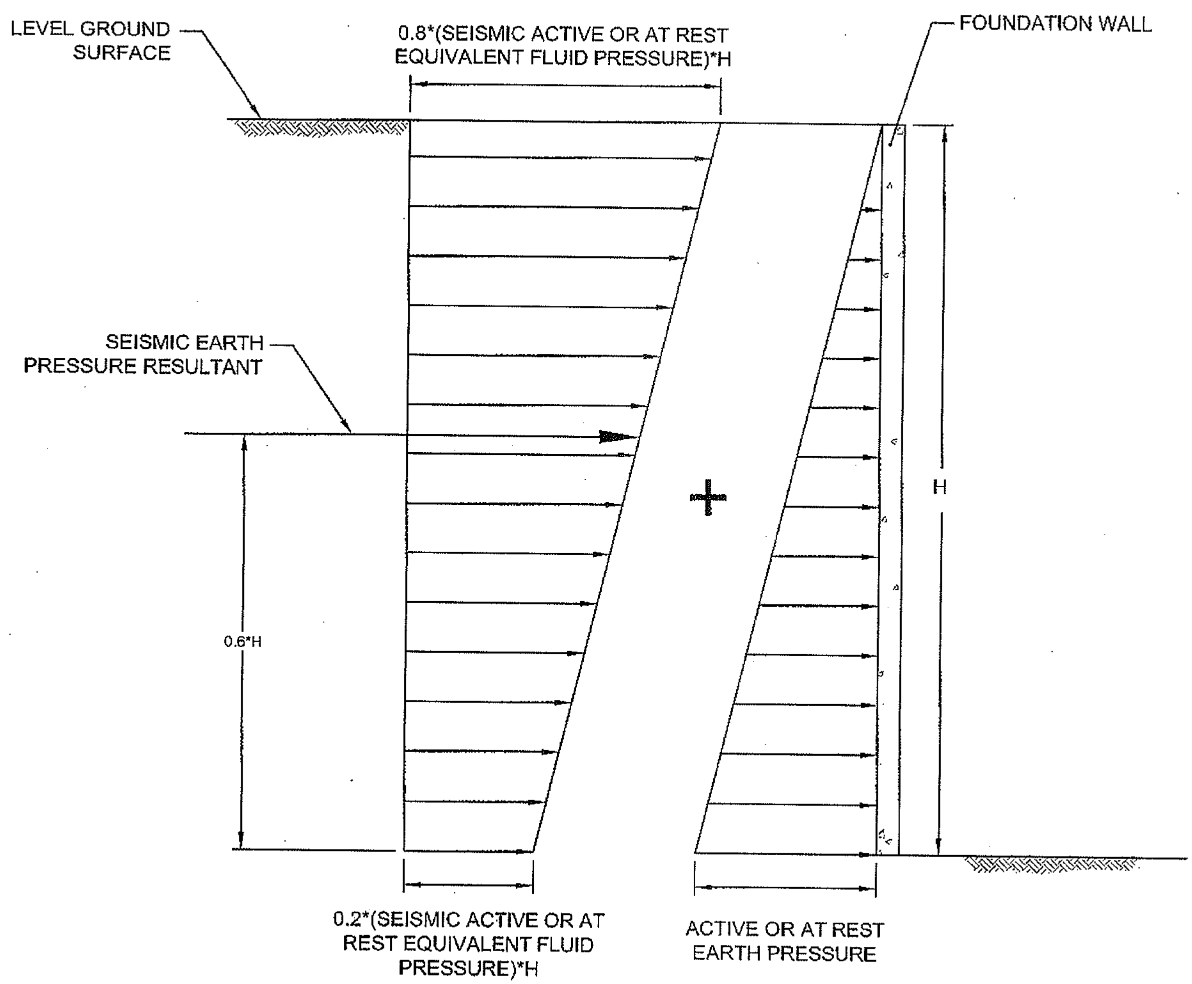
Robert A. Ross, P.E.
Senior Project Engineer

Enclosures: Figure 1 – Site and Exploration Plan
Figure 2 – Application of Seismic Earth Pressures
Appendix A – RZA Exploration Procedure and Logs
Appendix B – Laboratory Testing Procedures and Results

ATTACHMENT 17



ATTACHMENT 17



NOTES:

1. ALL SURCHARGES SHOULD BE CONSIDERED AS APPROPRIATE. THIS FIGURE DOES NOT INCLUDE SURCHARGE AFFECTS.

2. RECOMMENDED EQUIVALENT FLUID EARTH PRESSURES ARE AS FOLLOWS:

ACTIVE: 35 PCF
AT REST: 50 PCF
SEISMIC ACTIVE: 13.5 PCF
SEISMIC AT REST: 39 PCF

Zipper Zeman Associates, Inc. Geotechnical and Environmental Consulting 18905 33rd Avenue West, Suite 117 Lynnwood, Washington 98036 Tele: (425) 771-3304 Fax: (425) 771-3549	Project No. 81065239 Date: Jan. 2007 Drawn by: R.A.R.	Redmond Town Center Condominums 8502 166th Ave. NE and 16933 NE 85th St. Seattle, Washington **Figure 2: Seismic Earth Pressures**

APPENDIX A
FIELD EXPLORATION PROCEDURES AND LOGS
81065239

Our field exploration for this project included 2 borings completed on December 19, 2006. Exploration locations are shown on the Site and Exploration Plan, Figure 1. Exploration locations were approximated in the field using a measuring wheel with reference to existing boundary and topographic survey provided by Redmond Town Center Condominiums, LLC. As such, the exploration locations should be considered accurate to the degree implied by the measurement method. The approximate ground surface elevation at each exploration location was estimated based on the boundary and topographic survey provided by Redmond Town Center Condominiums, LLC. The following sections describe our procedures associated with the exploration. Descriptive logs of the explorations are enclosed in this appendix.

Soil Boring Procedures

Our exploratory borings were advanced with a hollow stem auger, using a trailer-mounted portable drill rig operated by an independent drilling firm working under subcontract to our firm. A geotechnical engineer from our firm continuously observed the borings logged the subsurface conditions encountered, and obtained representative soil samples. All samples were stored in moisture-tight containers and transported to our laboratory for further visual classification and testing. After each boring was completed, the borehole was backfilled with soil cuttings, and the surface was patched with bentonite grout.

Throughout the drilling operation, soil samples were obtained at 2.5- to 5-foot depth intervals by means of the Standard Penetration Test (ASTM: D-1586). This testing and sampling procedure consists of driving a standard 2-inch outside diameter steel split spoon sampler 18 inches into the soil with a 140-pound hammer free falling 30 inches. The number of blows required to drive the sampler through each 6-inch interval is recorded, and the total number of blows struck during the final 12 inches is recorded as the Standard Penetration Resistance, or "blow count" (N value). If a total of 50 blows is struck within any 6-inch interval, the driving is stopped and the blow count is recorded as 50 blows for the actual penetration distance. The resulting Standard Penetration Resistance values indicate the relative density of granular soils and the relative consistency of cohesive soils.

The enclosed boring logs describe the vertical sequence of soils and materials encountered in each boring, based primarily upon our field classifications and supported by our subsequent laboratory examination and testing. Where a soil contact was observed to be gradational, our logs indicate the average contact depth. Where a soil type changed between sample intervals, we inferred the contact depth. Our logs also graphically indicate the blow count, sample type, sample number, and approximate depth of each soil sample obtained from the boring, as well as any laboratory tests performed on these soil samples. If any groundwater was encountered in a borehole, the approximate groundwater depth, and date of observation, is depicted on the log. Groundwater depth estimates are typically based on the moisture content of soil samples, the wetted portion of the drilling rods, the water level measured in the borehole after the auger has been extracted, or through the use of an observation well.

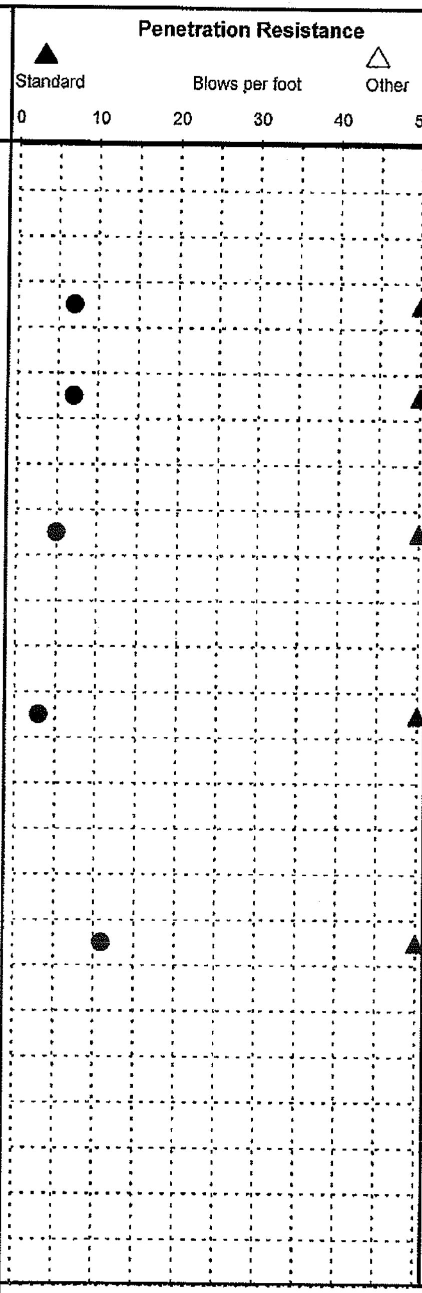
The boring logs presented in this appendix are based upon the drilling action, observation of the samples secured, laboratory test results, and field logs. The various types of soils are indicated as well as the depth where the soils or characteristics of the soils changed. It should be noted that these changes may have been gradual, and if the changes occurred between samples intervals, they were inferred.

ATTACHMENT 17

PROJECT: Redmond Condominiums | **JOB NO.:** 81065239 | **BORING:** B-1 | **PAGE 1 OF 1**

Location: Redmond, WA | **Approximate Surface Elevation: 79 feet**

Depth (ft)	Soil Description	Sample Type	Sample Number	Ground Water	Penetration Resistance (▲ Standard, △ Other; Blows per foot 0–50)	N-values	Testing
0	3 inches asphalt over moist, gray/brown, gravelly SAND with some silt						
	Very dense, moist, gray/brown, silty, gravelly SAND (Glacial Till)	I	S-1		▲	50/5"	
5	(Blowcount overstated)	I	S-2		▲	50/5"	
		I	S-3		▲	50/5"	
10	Very dense, moist, gray/brown silty, sandy GRAVEL with thin layers of wet sand with trace silt (Glacial Till)	I	S-4		▲	50/6"	
15	Becomes wet to saturated.			▼ ATD			
		I	S-5		▲	50/6"	
	Boring completed at 18 feet on 12/19/06. Groundwater observed at 16.5 feet at time of drilling.						
20							
25							



Explanation

- I — 2-inch O.D. split spoon sample
- II — 3-inch I.D Shelby tube sample
- ⊗ — No Recovery
- ▼ ATD — Groundwater level at time of drilling or date of measurement

Monitoring Well Key

- Clean Sand
- Bentonite
- Grout/Concrete
- Screened Casing
- Blank Casing

Moisture Content

Plastic Limit — Natural — Liquid Limit

Testing Key

GSA = Grain Size Analysis
200W = 200 Wash Analysis
Att. = Atterberg Limits
Consol. = Consolidation Test

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Geotechnical and Environmental Consulting

BORING LOG	Figure A-1
Date Drilled: 12/19/2006	Logged By: RWS

ATTACHMENT 17

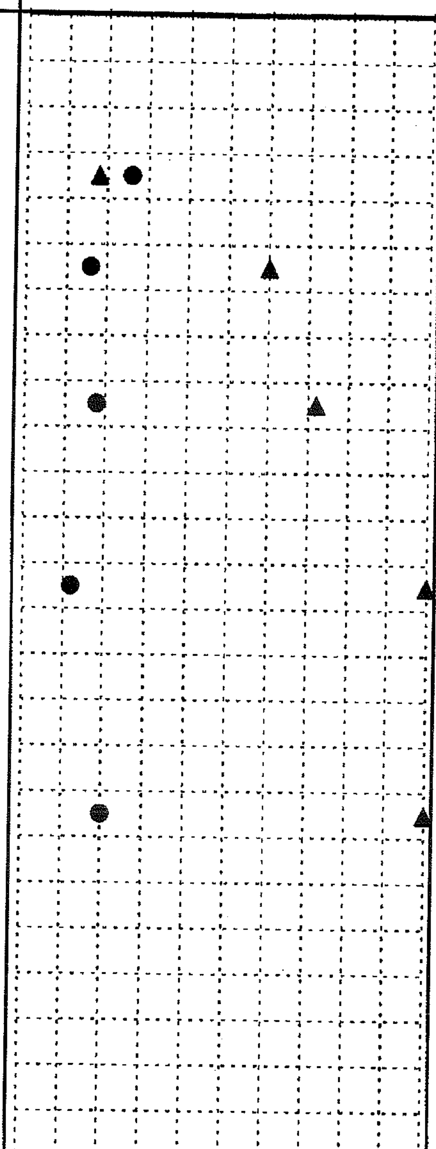
PROJECT: Redmond Condominiums **JOB NO.:** 81065239 **BORING:** B-2 **PAGE 1 OF 1**

Location: Redmond, WA **Approximate Surface Elevation: 72 feet**

Depth (ft)	Soil Description	Sample Type	Sample Number	Ground Water	N-values	Testing
0	3 inches asphalt over loose, wet, black/brown, silty, gravelly SAND (Fill)					
~4		2-inch O.D. split spoon	S-1		9	
5	Medium dense to dense, wet, mottled gray/brown, silty, gravelly SAND (Weathered Till)	2-inch O.D. split spoon	S-2	▼ ATD	30	
~9		2-inch O.D. split spoon	S-3		36	
~11	Very dense, moist, gray/brown silty, sandy GRAVEL with thin layers of wet, sand with trace silt (Glacial Till)					
~13		2-inch O.D. split spoon	S-4		100/5"	
~17	Becomes wet to saturated.	2-inch O.D. split spoon	S-5		50/5"	
18	Boring completed at 18 feet on 12/19/06. Groundwater observed at 6 feet at time of drilling.					

Penetration Resistance

▲ Standard Blows per foot △ Other



Explanation

- 2-inch O.D. split spoon sample
- 3-inch I.D Shelby tube sample
- No Recovery
- ▼ ATD Groundwater level at time of drilling or date of measurement

Monitoring Well Key

- Clean Sand
- Bentonite
- Grout/Concrete
- Screened Casing
- Blank Casing

Moisture Content

Plastic Limit — Natural — Liquid Limit

Testing Key

GSA = Grain Size Analysis
200W = 200 Wash Analysis
Att. = Atterberg Limits
Consol. = Consolidation Test

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BORING LOG	**A-2**	
Date Drilled: 12/19/2006	**Logged By: RWS**	

APPENDIX B
LABORATORY TESTING PROCEDURES AND RESULTS
81065239

A series of laboratory tests were performed during the course of this study to evaluate the index and geotechnical engineering properties of the subsurface soils. Descriptions of the types of tests performed are given below.

Visual Classification
Samples recovered from the exploration locations were visually classified in the field during the exploration program. Representative portions of the samples were carefully packaged in moisture tight containers and transported to our laboratory where the field classifications were verified or modified as required. Visual classification was generally done in accordance with the Unified Soil Classification system. Visual soil classification includes evaluation of color, relative moisture content, soil type based upon grain size, and accessory soil types included in the sample.

Moisture Content Determinations
Moisture content determinations were performed on representative samples obtained from the exploration in order to aid in identification and correlation of soil types. The determinations were made in general accordance with the test procedures described in ASTM: D-2216. Results of the moisture content determinations are shown on the boring logs provided in Appendix A.