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Development Engineering

1ST SUBMIT

EnviroSound Consulting Inc.

Geotechnical & Environmental Consulting

Project Information

Project Name: Proposed Single Family Residence, N. E. Shore Drive
Client: Mr. Jeff Kanyer
Project #: ESC010-G025
Project Location: Indianola, Washington
Date: December 20, 2010

Company Information

3388 Byron Street NW Suite 200
Silverdale, Washington 98383
Phone: 360-698-5950
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December 20, 2010

Mr. Jeff Kanyer
121 Louis Thompson Road NE
Sammamish, Washington 98074

Subject: Geotechnical Engineering Evaluation
Proposed Single-Family Residence
N. E. Shore Drive
Indianola, Washington
Tax Parcel No:4360-001-018-0004

Dear Mr. Kanyer:

EnviroSound Consulting Inc. (ESC) has prepared this letter report to present our findings and limited geotechnical recommendations for the proposed development of a single-family residence (SFR) for the property located off of N. E. Shore Drive (Tax Parcel Identification Number 4360-001-018-0004) in Indianola Washington (Figure 1). Our scope of services included discussion of preliminary project details with yourself, shallow subsurface exploration with three test pit excavations, one hand boring, limited engineering analyses, and preparation of this letter report as well as the review of previous letter reports by Aspect Consulting.

Proposed Construction

Based on our discussions, we understand that the proposed development is to consist of the construction of a single-family residential structure with a partial daylight basement and an attached garage. Cast in place concrete retaining walls are proposed along a buffer interface and select locations on the site. A site plan generated from a drawing by Team-4 Engineering is attached as Figure 2.

Methodology

The subject property soil and slope conditions were visually examined and evaluated by observing soil around the project area. This information provided a basis for a general understanding of the local soil and geologic conditions as discussed below. In addition to the observation of available soil exposures and reconnaissance of the area the following readily available resources were reviewed.

Resources used:

- "Soil Survey of Kitsap County Washington", United States Department of Agriculture, 1977.
- "Slope Stability, Kitsap County, Washington", Jerry Deeter, 1979.
- The Washington State Department of Ecology Coastal Zone Atlas, Kitsap County (Volume 10) dated 1979.

- "Geologic map of surficial deposits in the Seattle 30' by 60' quadrangle, Washington", Yount et al., 1993.
- "Geologic Map of Washington – Northwest Quadrant", Washington State Department of Natural Resources, Division of Geology and Earth Resources, GM – 50, 2002.
- U.S.G.S. 7.5 minute series topographic map "USGS Suquamish Topographic Map".
- Preliminary Drawings by Hecker Architects, dated September 28, 2008.

Site Conditions

The subject property consists of two parcels located in Section 15 Township 26N Range 2E as shown on the Vicinity Map (Figure 1). Access to the subject property is provided by a primitive driveway off of N. E. Shore Drive. At the time of the site visit the site was vacant and undeveloped with construction material stockpiled on the site. The lot was cleared of trees several years ago with the majority of the vegetation composed of secondary growth on the property in the form of blackberry bushes and small trees. The property is bordered by single-family residences to the east and west, N. E. Shore Drive to the north, and Port Madison Bay to the south.

The site has a relatively level plateau area on the northeast portion of the lot with a descending slope toward the west and south. The majority of the slope descends to a swale which is aligned in a general north-south direction. The slope adjacent to the swale ranges from 10 to 20 feet in height at a ratio of 2H:1V (Horizontal : Vertical). The south portion of the slope gradually descends to the shoreline with a height of approximately 26 feet.

There was no visible evidence of significant erosion or slope failure on the subject parcel or adjoining parcels during our site visit.

Geologic Setting

The subject site lies within the central Puget Lowland. The lowland is part of a regional north-south trending trough that extends from southwestern British Columbia to near Eugene, Oregon. North of Olympia, Washington, this lowland is glacially carved with a depositional and erosional history including at least four separate glacial advance/retreats. The Puget Lowland is bounded on the west by the Olympic Mountains and on the east by the Cascade Range. The lowland is filled with glacial and nonglacial sediments consisting of interbedded gravel, sand, silt, and till

The Washington Division of Geology and Earth Resource (WDGER), Geologic Map of Washington - Northwest Quadrant, dated 2002, and the Geologic Map of Surficial Deposits in the Seattle 30' by 60' quadrangle, Washington Map by Yount et al., dated 1993 both show the site located in an area mapped as Glacial Drift. The drift consists of oxidized sand and gravel with thin discontinuous deposits of interbedded silt and fine grained sand.

A review of "Slope Stability, Kitsap County, Washington", Jerry Deeter, 1979, and the Department of Ecology Shoreline Stability Maps was performed in conjunction with preparing this study. The subject

property and surrounding area has been mapped as Stable slopes. Stable slopes are generally less than 15 percent but can include areas of steeper slopes that are stable due to low groundwater concentration or competent bedrock. It should be noted that the mapping was performed in the 1970's and does not reflect more recent activity.

The United States Department of Agriculture (USDA) Soil Survey of Kitsap County Area, Washington, information indicates the following soil types exist on the project site:

- Ragnar fine sandy loam, 0 to 6 percent slopes.
- Ragnar fine sandy loam, 15 to 30 percent slopes.

The soil survey descriptions of these soil types are summarized in the following table.

USDA Soil Survey Name	**44 – Ragnar fine sandy loam, 0 to 6 percent slopes**	**45 – Ragnar fine sandy loam, 15 to 30 percent slopes**
Typical Profile	*0 to4 inches:* Fine sandy loam *4 to 23 inches:* Fine sandy loam *23 to 60 inches:* Loamy sand, sand	*0 to 4 inches:* Fine sandy laom *4 to 23 inches:* Fine sandy loam *23 to 60 inches:* Loamy sand, sand
Origination	Glacial outwash.	Glacial outwash.
Drainage	Moderately well drained	Moderately well drained
Permeability	Moderately rapid.	Moderately rapid.
Surface Runoff	Slow	Medium.
Erosion Hazard	Moderate.	Severe

Field Exploration and Subsurface Soils

Three exploratory test pits were excavated by a mini trackhoe provided by the client on September 7, 2010. A hand boring was dug on the south portion of the lot on September 8, 2010 to acquire subsurface information in a limited access area. The test pits were excavated on the slope face and on the plateau area (See Figure 2) and logged by a representative of ESC. The test pits and hand boring were placed within the proposed building footprint. Soil samples were obtained from the test pits for additional review and for laboratory testing.

In general, the subsurface soils consisted of loose topsoil and sandy Silt in the upper 2 to 3 feet. There was visible evidence of fill in the upper 2 to 3 feet in the presence of broken bottles and asphalt patch material. There were also thick root mats and remnants of decomposing tree stumps. The silt was underlain by layers of sand with varying amounts of gravels and cobbles. For more complete information, see the attached test pit logs.

Groundwater

Groundwater was not encountered in the test pits or hand boring. The water drainage in the swale appears to be seasonal in nature and does not appear to be of significant concern for the construction of the SFR. Water table elevations can fluctuate with time. Groundwater levels are typically influenced by seasonal precipitation, irrigation, land use, and climatic conditions, as well as other factors.

CONCLUSIONS

Based on the findings of this investigation, it is our opinion that the proposed SFR may be constructed on the site provided the following recommendations are followed.

It is our opinion that the construction of the proposed SFR within the 15-foot building set-back will not adversely impact the stability of the slope. The proposed SFR will have a partial daylight basement which will place foundations in the more dense sands encountered at depth. Portions of the SFR will need to be supported on auger cast pilings or pin piles due to the proximity of the SFR to the edge of the slope face. The southern portion of the proposed building footprint will need to be supported on piles. The pin piles or auger cast footings can be tied to the perimeter foundation through the use of a grade beam.

Foundations

The majority of the proposed construction may be founded on a shallow foundation system bearing in the medium dense silty, sand. Topsoil, organic debris and soft or loose soil should be removed from foundation excavations. Continuous or isolated footings may be designed for a net allowable bearing pressure of 2,000 pounds per square foot (psf). An increase of 1/3 may be applied to the allowable bearing pressure value for short duration loads, such as those associated with wind and seismic conditions.

The concrete retaining wall proposed along the western buffer will need to be founded in medium dense soils with loose topsoil or fill material being removed from excavated areas. The retaining wall footings may be designed for a net allowable bearing pressure of 2,000 psf.

Footing excavations should be observed by a representative of the geotechnical engineer to verify that the foundations will bear on suitable material. If topsoil or loose native soils are observed within foundation excavations, these materials should be removed and/or re-compacted, depending on the depth and nature of the material. The extent of overexcavation and/or re-compaction should be based on the geotechnical engineer's representative's recommendations during construction monitoring.

Footings should have adequate embedment for local frost penetration requirements. In the area of this project, the minimum depths are typically 18 inches for exterior footings and 12 inches for interior footings. We recommend that the footings be a minimum of 12 inches wide, regardless of the design foundation pressures. As a rule of thumb footings should be located a minimum of 5 feet from any adjacent, descending slope. The distance is to be measured horizontally at the base of the footing. In the event a footing needs to be located within 5 feet of a slope face the footing will need to be deepened so that the load bearing zone beneath the footing does not extend onto the slope face.

Total foundation settlements should be less than ¾ of an inch for footings that are constructed as recommended. Differential settlement across a distance of 20 feet should be less than ¾ inch. The majority of the settlement is expected to occur at the time of construction or soon after. Greater than expected post-construction settlement can occur if foundation subgrade soils become saturated.

Lateral footing displacement can be resisted by friction along the base of the foundation and passive pressure acting against the appropriate footing faces. We recommend an allowable friction factor of 0.40 and an allowable passive pressure of 300 psf. These values include a factor of safety of 1.5 for the allowable friction factor and 2.0 for the allowable equivalent fluid passive pressure. Note that the allowable passive pressure assumes that the foundations are located in the dense sand.

Auger Case Piles

Auger cast piles are constructed by placing concrete in a hole formed with a continuous-flight hollow stem auger. When the proper pile length has been determined the auger is extracted and the grout is pumped down the hollow auger stem under a pressure of 150 to 250 pounds per square inch. Steel reinforcement is than placed in the fluid grout column. Pile lengths may range from 15 to 18 feet below the proposed building subgrade. The parameters provided in the following tables may be used to preliminarily determined diameter and depth:

Allowable Pile Capacities (Auger Cast Piles)

14 inch Diameter Pile		
Embedment depth (feet)	Compression (kips)	Uplift (kips)
5	31	13
10	73	77
18 inch Diameter Pile		
Embedment depth (feet)	Compression (kips)	Uplift (kips)
5	61	18
10	102	100
24 inch Diameter Pile		
Embedment depth (feet)	Compression (kips)	Uplift (kips)
5	81.85	27
10	152.7	135

Pile installation should be monitored to verify proper embedment into the presumed bearing layer. Pile lengths can be adjusted in the field if it is determined that the bearing depth varies.

Pin Piles

Pin piles are small diameter steel pipes which are driven by pneumatic hammers into load bearing soils. The pipes can range from 2 to 6 inches in diameter and generally have capacities ranging from 2 to 15 tons. We would project the depth of the pin piles to be a minimum of 20 feet.

Drainage

Runoff from the building and impervious surfaces should be directed into an appropriately designed stormwater disposal system. Design of stormwater disposal systems is primarily the responsibility of a civil engineer. Based on the soils encountered in the test pits and the limited area for infiltration we expect the surface water run-off to be directed toward the shoreline.

A perimeter footing drain should be installed around the SFR at a depth equal to the bottom of the footing. This drain should consist of a minimum 4-inch diameter perforated rigid pipe (with perforations placed down) with a minimum thickness of 6 inches of pea gravel or clean washed rock around the pipe. The backfill soils within 1 foot of the foundation wall should consist of free-draining sand and gravel material. This drainage system should be designed to transport water away from the residence and discharge into an appropriate area.

Lateral Earth Pressures

We understand that shallow below grade walls may be included in the proposed construction as well as concrete retaining walls. Lateral pressures will be exerted on below grade (basement) and retaining walls by backfill soils, surcharge loads, and hydrostatic pressures caused by groundwater. Lateral earth pressures on walls depend upon the type of wall, type of backfill material and allowable wall movements. For walls that are restrained at the top, lateral earth pressures should be estimated for an "at rest" condition. For walls that are free to rotate away from the retained soil, lateral earth pressures should be estimated for an "active" earth pressure. For walls that are compressing the retained soil, lateral earth pressures should be estimated for a "passive" earth pressure. Recommended lateral earth pressures coefficients are provided in the following tables along with equivalent fluid pressures. These pressures are calculated assuming a moist unit weight for the backfill soil of 130 pounds per cubic foot (pcf) and an angle of internal friction of 35 degrees, as would be typical for compacted sand backfill soil.

Lateral Earth Pressures, no slope above or below the wall					
"Active" Condition		"At Rest" Condition		"Passive" Condition	
Coefficient (Ka)	Equivalent Fluid Unit Weight (pcf)	Coefficient (Ko)	Equivalent Fluid Unit Weight (pcf)	Coefficient (Kp)	Equivalent Fluid Unit Weight (pcf)
0.27	35	0.43	56	1.85	242

Lateral Earth Pressures, 2H:1V slope above or below the wall					
"Active" Condition		"At Rest" Condition		"Passive" Condition	
Coefficient (Ka)	Equivalent Fluid Unit Weight (pcf)	Coefficient (Ko)	Equivalent Fluid Unit Weight (pcf)	Coefficient (Kp)	Equivalent Fluid Unit Weight (pcf)
0.39	51	0.62	81	0.72	94

The recommended equivalent fluid unit weights do not include hydrostatic pressure due to groundwater accumulated behind walls. The recommended fluid pressures in the first table assume a horizontal ground surface above and below the wall and do not include seismic loading, or any surcharge due to nearby loading from structures, equipment or traffic. The recommended fluid pressures in the second table assume a 2H:1V slope above the wall for the active and at rest cases and a 2H:1V slope below the wall for the passive case. Therefore, for a wall with a 2H:1V slope below it, the active or at-rest pressures from the first table should be used with the passive pressures from the second table. In both tables the passive pressures have been reduced by a factor of 2 to limit wall translation.

The potential seismic force on the wall can be modeled as a uniform pressure (in pounds per square feet-psf) on the back of the wall equal to 7 times the height of the wall (in feet) if there is no slope above the wall. For a wall with a 2H:1V slope above, the potential seismic force should be increased to 14 times the height of the wall. The units for this pressure are pounds per square feet.

Continuous drains with cleanouts should be installed at the base of retaining walls to prevent the buildup of hydrostatic pressure behind the structure. These drains should consist of a minimum 4-inch diameter perforated rigid pipe (with perforations placed down) with a minimum thickness of 6 inches of pea gravel around the pipe. The backfill soils within 1 foot of the walls should consist of free-draining sand and gravel material. This drainage system should be designed to transport water away from the structure and discharge into an appropriate area.

Rockeries

In the event that rockeries are utilized the subject lot as part of the development we offer the following general guidelines. As a general standard, rockeries with a height of less than 6 feet should have a minimum sized 2-man rock at its base with a batter of approximately 6V:1H (vertical to horizontal). Complete details for proper construction are shown in the "Standard Rock Wall Construction Guidelines" by Associated Rockery Contractors. It should be noted that rockeries, however are not engineered structures. The long term performance of rockeries is highly dependent on the construction techniques and the monitoring of the work.

Erosion Control

The soils on the slope may erode in the disturbed state or under conditions of channelized water flow. Therefore, best management practices for erosion control including silt fences, hay bales, etc. should be used to prevent sediment from leaving the site and entering storm water sewer systems or surface waters. Water should not be allowed to flow over the slope in a concentrated manner. Stripping of vegetation on the steep slopes should be limited to the extent possible for the proposed construction. We further recommend that vegetation be replanted on the slopes as soon as practical following completion of grading. We understand that a Habitat Management Plan was generated by Wiltermood & Associates which also provides replanting guidelines.

The Washington State Department of Ecology (DOE) has three publications, which may be helpful in developing long-term slope vegetation maintenance/protection and landscape plans:

- "Slope Stabilization and Erosion Control Using Vegetation: A Manual of Practice for Coastal Property Owners", May 1993, Publication 93-30.
- "Vegetation Management: A Guide for Puget Sound Bluff Property Owners", May 1993, Publication 93-31.
- "Surface Water and Ground Water on Coastal Bluffs: A Guide for Puget Sound Property Owners", June 1995, Publication 95-107.

Although the slope on the property is not a "bluff" the preceding publications have a great deal of general information regarding plants, steep slopes and the use of vegetation to stabilize soil.

Temporary Slopes

It is our opinion that the soils encountered in the test pits (except for the upper 3 feet) are a Type B material as defined by the Washington Industrial Safety and Health Act's (WISHA) regulations on excavation, trenching and shoring. Temporary slopes excavated in Type B material should be inclined no steeper than 1H:1V (horizontal: vertical). A representative of our firm should evaluate temporary slopes to verify that they are appropriate for the soils encountered during construction. In areas where it is not possible to maintain the recommended slopes due to space constraints, temporary shoring may be required. Shoring would need to be properly designed by an engineer.

The Contractor should be familiar with applicable local, state, and federal safety regulations, including the current WISHA regulations on excavation, trenching and shoring. Construction site safety is the sole responsibility of the contractor, who shall also be solely responsible for the means, methods and sequencing of construction operations. ESC is providing this information only as a service to our client. Under no circumstances should the information provided above be interpreted to mean that ESC is assuming responsibility for construction site safety or the contractor's activities; such responsibility is not being implied and should not be inferred.

The soils to be penetrated by any proposed excavations may be variable. ESC's preliminary classifications are based on the materials encountered in the test pits. The Contractor should continually classify the soils that are encountered as excavation work progresses with respect to the WISHA system.

Structural Fill

The majority of the near surface onsite soils will likely be unsuitable for use as structural fill due to the high percentage of organic material such as roots and tree debris. If the earthwork is to take place during the normally wet period of the year, excavations in the sands should not be left open for any significant length of time. Imported structural fill should consist of well-graded gravel and/or sand with a maximum grain size of 3 inches and less than 5 percent fines (material passing the U.S. Standard No. 200 Sieve).

Structural fill should be placed in loose lifts no more than 12 inches thick, moisture conditioned as necessary (moisture content of soil should be within 2 percent of optimum moisture) and the material should be compacted to at least 95 percent of the maximum dry density as determined by ASTM Test Method D-1557. Additional lifts should not be placed if the previous lift did not meet the required dry density or if soil conditions are not stable. Note that, although in place density testing of fill is frequently

used as the primary criterion for acceptance of fill, it should not be the only criterion. If, in the judgment of the geotechnical engineer or his representative, placed fill is not suitable it should be rejected regardless of in place density test results. As an example, fill that is compacted wet of the optimum moisture content may exhibit "pumping" behavior even if in place density test results indicate greater than 95 percent compaction has been achieved. In such a situation, the fill should be removed and replaced with drier material.

Limitations

We have prepared this letter based on standard practices, currently used in this area at the time of preparation. The conclusions are based on the observations made during the site visit. A limited subsurface exploration program was performed. The information presented in this letter was collected and interpreted in a manner consistent with the level of care and skill ordinarily exercised by members of the profession currently practicing under similar conditions. No other warranty, expressed or implied, including (but not limited to) any warranty of merchantability or fitness for a particular use has been made. Should you have any questions or concerns, which have not been addressed, or if we may be of additional assistance, please call our office at (360) 698-5950.

Sincerely,

Shawn E. Williams, LEG.
Senior Engineering Geologist

State of Washington
Licensed Engineering Geologist
Shawn E. Williams
12-21-10

Michael F. Wnek, P.E.
Mike Wnek Engineering.

MICHAEL F. WNEK
STATE OF WASHINGTON
026660
12-20-10
REGISTERED
PROFESSIONAL ENGINEER
EXPIRES 07-13-11

Attachments:
Figure 1 – Vicinity Map
Figure 2 – Site Plan

Appendix A – Test Pit Logs

EnviroSound Consulting

FIGURE 1 Site Map
Project Name: North Shore Lot
Location: Indianola, Washington
Project: ESC10-G025
Client: Mr. Jeff Kanyer
Date: September 2010

N

(Printed Scale 1" = 30')

GRAPHIC SCALE

20 0 10 20

(IN FEET)
1 inch = 20 feet

The elevation datum is
Mean Lower Low Water
Based on an uncorrected observation of the local tide

Kingston Street

Shore Drive

Bench Mark
Set railroad spike 1.0' up North face of utility pole
Elevation = 56.2'

FUTURE ACCESS FOR LOT 17?

I.E. = 48.2'

I.E. = 45.4'

EX WATER METER

Concrete parking

Water valve

Utility pole

Wood wall

Timber wall

Rock wall

6' High board fence

S 01°58'41" W

208.45'

208.98'

CONCRETE RETAINING WALL ALONG BUFFER DESIGNED BY OTHERS

CEDAR PLANK FENCE TO BE INSTALLED ALONG CREEK BUFFER ON TOP OF CONCRETE WALL

CB#1 TYPE 2 W/FLOWGARD INSERT IE=52.0 IE=40.0

11LF 12"ø, N-12, SL=0.0% IE=40.0 W/ROCK PAD

EX CULVERT TO BE REMOVED

FF GARAGE =53.5

8' DRIVE

DRAIN FIELD

House

6" ADS FOOTING DRAIN & 6" PVC ROOF DRAIN TO CB#1.

6" ADS FOOTING DRAIN & 6" PVC ROOF DRAIN TO DISCHARGE TO ONSITE DRAINAGE PATH VIA ENERGY DISSIPATING ROCK PAD.

TP-1

TP-2

TP-3

HB-1

(50% BUFFER)

Foundation Top of wall Elev. = 43.5'

Rock wall

Grave path

CONCRETE RETAINING WALL

CONCRETE RETAINING WALL ALONG BUFFER

15' BUILDING SETBACK

50' SHORELINE BUFFER

Landscape block retaining wall

Concrete slab

Shed

Concrete wall

Concrete steps

Rock bulkhead (Toe el.=12'±)

VIEW LINE

Wood stairs

LOCUS TREES TO BE REMOVED TYPICAL OF 3

MHHW LINE AT ELEVATION 11.40' BASED ON GPS SURVEY SHOTS OF BEACH THAT ARE RELATED TO TIDAL BENCHMARK 7 1973 VM#9072

Concrete landing and stairs

NOTE:
SHORELINE (AS SHOWN) WAS ESTABLISHED BY TEAM 4 ENGINEERING, THIS WAS UNDERTAKEN SEPARATE FROM OTHER TOPOGRAPHIC MAPPING AND WAS DONE UTILIZING A SEPARATE DATUM, AS NOTED.

Legend:
TP = Test Pit
HB = Hand Bore

Note: Figure modified from Team 4 drawing 09/21/10.

Figure 2. SITE MAP
Kanyer Lot
Indianola, Washington
Project: ESC10-G025
Client: Mr. Jeff Kanyer
Date: September 2010

TEST PIT HB- 1

Project Name: Kayner Lot
Client: Jeff Kayner
Project Number: ESC10-G025

Test Pit Elevation:
Test Pit Location: South portion of parcel.
Depth to Groundwater: None encountered.

DEPTH (FT.)	USGS Classification	VISUAL PHYSICAL DESCRIPTION	SAMPLE NO.	SAMPLE TYPE	SAMPLE DEPTH (FT.)	LABORATORY TESTING RESULTS FOR SAMPLE
0		0-0.5' Loose dark brown sandy (fine) SILT, dry, loose, (Fill) Roots and scattered gravels.	S-1	Grab	1.0'	
		0.5'-3.5' Loose reddish tan silty fine to medium SAND, (Fill), scattered gravels and roots, trace cobbles.				
		Pieces of asphalt at approximately 3.0 feet.	S-2	Grab	3.5'	
		3.5'-4.5' Loose gray slightly sandy SILT (Original Topsoil), dry with roots.				
5		4.5'-5.5' Loose reddish tan silty SAND, moist, scattered gravels.				
		5.5'—8.0 Loose gray silty fine to medium SAND, moist.				
10		Total .Depth. 8.0 feet.				
		No groundwater seepage.				
		Sloughing in upper 2.0 feet of hole.				
15						

Excavation Contractor: EnviroSound
Equipment: Hand Auger
Operator: SEW

Excavation Date: 09/8/10
ESC Representative: SEW
Page 1 of 1

TEST PIT TP- 1

Project Name: Kanyer Lot
Client: Mr. Jeff Kanyer
Project Number: ESC10-G025

Test Pit Elevation: @
Test Pit Location: Slope face
Depth to Groundwater: none encountered

DEPTH (FT.)	USGS Classification	VISUAL PHYSICAL DESCRIPTION	SAMPLE NO.	SAMPLE TYPE	SAMPLE DEPTH (FT.)	LABORATORY TESTING RESULTS FOR SAMPLE
0		0-2' Loose, brown, sandy SILT, dry with scattered gravels and cobbles, some roots.	S-1	Grab	1.0	
		TD 2.0' Sloughing No seepage				
5						
10						
15						

Excavation Contractor: RCNW Excavation Equipment: Mini-Track hoe Operator: Jeff Stern	Excavation Date: 09/07/10 ESC Representative: SEW Page 1 of 1

TEST PIT TP- 2

Project Name: Kanyer Lot
Client: Mr. Jeff Kanyer
Project Number: ESC10-G025

Test Pit Elevation: @
Test Pit Location: Western portion of upper plateau.
Depth to Groundwater: none encountered

DEPTH (FT.)	USGS Classification	VISUAL PHYSICAL DESCRIPTION	SAMPLE NO.	SAMPLE TYPE	SAMPLE DEPTH (FT.)	LABORATORY TESTING RESULTS FOR SAMPLE
0		0-1' Loose, dark gray, sandy SILT (Fill), dry with scattered gravels and cobbles, with roots.				
		1'-3.5' Loose reddish tan silty fine SAND, moist, large roots and root mats.	S-1	Grab	2.0	
		Large chunk of tree trunk at @ 2.5 feet-east portion of test pit.				
		3.5'-8.0' Loose to medium dense gray slightly silty fine to medium SAND, moist, scattered gravels.	S-2	Grab	3.5	
5						
			S-3	Grab	7.5	
10		TD 8.0' Some sloughing upper 3.0 feet. No seepage. Decomposed tree stump in the western sidewall of test pit.				
15						

Excavation Contractor: RCNW
Excavation Equipment: Mini-Track hoe
Operator: Jeff Stern

Excavation Date: 09/07/10
ESC Representative: SEW
Page 1 of 1

TEST PIT TP- 3

Project Name: Kanyer Lot
Client: Mr. Jeff Kanyer
Project Number: ESC10-G025

Test Pit Elevation: @
Test Pit Location: Eastern portion of upper plateau.
Depth to Groundwater: none encountered

DEPTH (FT.)	USGS Classification	VISUAL PHYSICAL DESCRIPTION	SAMPLE NO.	SAMPLE TYPE	SAMPLE DEPTH (FT.)	LABORATORY TESTING RESULTS FOR SAMPLE
0		0-1.0' Loose, dark brown, sandy SILT (Fill), dry with scattered gravels and cobbles, some roots. 1.0'-2.5' Loose reddish tan sandy SILT, dry, some roots. 2.5'-4.0' Loose medium dense tan gray silty SAND, dry to moist, scattered gravels.				
5		TD 4.0' Some sloughing upper 2.0 feet. No seepage T-probed to a depth of 5.5 feet, became medium dense.				
10						
15						

Excavation Contractor: RCNW
Excavation Equipment: Mini-Track hoe
Operator: Jeff Stern

Excavation Date: 09/07/10
ESC Representative: SEW
Page 1 of 1