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GEOTECHNICAL INVESTIGATION REPORT
Landslide Remediation - Adachi Property
1253 Deer Creek Road
Dabob Cove, Washington
Prepared for: Mr. Seiichi Adachi

Project No. 080029-001-02 • October 27, 2008

Aspect Consulting, LLC

Todd S. Parkington, PE
Senior Geotechnical Engineer
tparkington@aspectconsulting.com

John L. Peterson, PE
Senior Associate Geotechnical Engineer
jpeterson@aspectconsulting.com

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1 Introduction

Aspect Consulting, LLC was retained by Mr. Seiichi Adachi to conduct a geotechnical engineering investigation related to a landslide that occurred on 1253 Deer Creek Road located east of Dabob Bay on the Toandos Peninsula in Jefferson County, Washington. The tax parcel identification number for this parcel is 601032020.

1.1 Site Location

The site consists of a single parcel located at the west end of Deer Creek Road. Based on the United States Geologic Survey (USGS), 7.5-minute Quilcene, Washington topographic quadrangle map, the property is located in Jefferson County in Section 3, Township 26 North, Range 1 West, W.M. and at Longitude 122.790 degrees W and Latitude 47.778 degrees N. The location of the site is presented on Figure 1, *Site Location Map*.

1.2 Purpose and Scope

The purpose of this study was to evaluate potential remediation methods for a landslide that occurred on the site the winter of 2007 to 2008. The landslide removed part of the access driveway for the existing house on the site. Remediation of the landslide would be performed to reduce the risk of further damage to the access driveway and possibly to restore the driveway in its original location.

Our study included review of available geologic literature, drilling three borings to evaluate the engineering properties of the site soils, analyzing the stability of the slope using the soil information developed for this report and topography provided by Mr. Adachi, and providing general discussions regarding mitigating the risk of future slope movement and possibly repairing a portion of the site impacted by the landslide. This report presents the results of our investigation. The investigation was conducted in general accordance with our scope of work proposal, dated February 13, 2008, and Contract Change Order 1, dated July 2, 2008.

2 Site Investigation

2.1 Site Description

The site is located on the west side of the Toandos Peninsula overlooking Dabob Bay. The parcel on which the residence is located is just under 7 acres in area. The site slopes down to the west, with the steepest portion of the slope below the house and driveway. The steepest portion of the slope is approximately 100 percent and 80 feet high above the shoreline. Above this, the slope grade gentles to about 40 percent. The slope continues up

to the east beyond the property. The residence sits at the top of the steeper portion of the slope. The access driveway winds across the site from the southeast, curves north of the residence and approaches the residence from the northeast.

The slide occurred to the north of the house at and above the steepest part of the slope. The maximum elevation of the slide is at about elevation 120, which is about 40 feet higher than the top of the steepest portion of the slope. A portion of the upper part of the slide extended into the access driveway. The resulting slide scarp at the head of the slide is approximately 10 feet high and between vertical and 1H:1V (Horizontal:Vertical) in steepness.

The location of our borings and the topography are shown on Figure 2, *Site and Boring Location Plan.*

2.2 Geologic Setting

The Washington Division of Geology and Earth Resources (WDGER), Geologic Map of Washington – Northwest Quadrant, dated 2002, indicates that the site is underlain by Deposits of pre-Fraser Age, Undifferentiated (Qgpc). These deposits are described as clay, silt, sand, gravel, till and peat, generally moderately to deeply weathered and compact.

2.3 Subsurface Exploration

Three borings (B-1 through B-3) were drilled on the site on March 19, 2008 and samples were obtained to aid in the determination of engineering properties of the subsurface materials. The test borings were located on the site as shown on Figure 2. Boring locations were estimated in the field by measuring from existing site features. The borings went to a maximum depth of 41.5 feet below the ground surface using a track-mounted limited access hollow stem auger drill rig. Split spoon (1 3/8-inch inner-diameter) samples were obtained from the borings in general accordance with Standard Penetration Test Procedures (American Society for Testing and Materials [ASTM] D 1586), at 5-foot intervals. The split spoon sampler is driven 18 inches into the soil by a 140 pound hammer falling 30 inches. The number of blows required to drive the sampler are counted for each 6 inches of penetration. The blow count for the first 6 inches of penetration is discarded, and the remaining blow counts are summed to produce the Standard Penetration Resistance (N). Geologic Drill drilled the borings under the direction of Aspect Consulting, LLC.

Logs of the borings indicating the type of soils encountered are presented in Appendix A. The depths indicated on the logs where conditions changed may represent gradational variations between soil types. Changes logged between sample intervals in our borings were interpreted. Soils were classified in general accordance with the ASTM D-2488, "Standard Practice for Description and Identification of Soils (Visual and Manual Procedure)".

In addition to the borings performed for this investigation, information gathered at the site for construction of the existing house was used to enhance our understanding of the site soils. This information is contained in a report prepared by Shannon & Wilson, Inc.

for Mr. Sei Adachi, titled *Geotechnical Report, Adachi Property on Toandos Peninsula,* dated September, 1998.

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2.3.1 Soil

In general, subsurface conditions on the slope consisted of sand or silty sand overlying silts and clays. The relative density of the sand encountered in the borings drilled within the landslide area was loose. The consistency of the silt and clay was generally hard. The soil descriptions in the Shannon & Wilson report were similar. However, the apparently in-place upper sands described in that report had relative density ranging from loose to medium dense.

2.3.2 Laboratory Testing

To aid in classifying the soils and to estimate general soil characteristics, laboratory tests were performed on selected representative samples. The following tests were performed: grain size distribution (ASTM D422), Atterberg limits (ASTM D4318) and moisture content (ASTM D2216). Analytical Resources, Inc. in Tukwila, Washington was retained to provide geotechnical laboratory analysis. The results of the laboratory testing are presented on the boring logs and in Appendix B.

2.3.3 Groundwater

Groundwater was not noted in the borings prepared for this report. However, there was water flowing out of the slope in the area of the slide and the moisture content laboratory tests indicate that the upper sands have high moisture contents and are likely saturated. The saturation appears to extend into the silts and clays for some distance. The relatively low permeability of the silts and clays would tend to mask the presence of groundwater during drilling. Groundwater was encountered in the borings performed for the referenced Shannon & Wilson report. This report indicated perched groundwater in the sand over the hard clay. Water table elevations fluctuate with time, being dependent on seasonal precipitation, irrigation, land use, and climatic conditions, as well as other factors. Therefore, water level observations at the time of the field investigation may vary from those encountered during the construction phase of the project. The evaluation of such factors is beyond the scope of this report.

2.3.4 Seismic Information

Table 1613.5.2 of the 2006 International Building Code (IBC) is used to classify soils and rock into categories for use in seismic design of structures. One of the criteria by which soil can be classified using the table is the average Standard Penetration Resistance (N-value) within the top 100 feet of soil. Site specific data were not developed to a depth of 100 feet. However, N-values do not typically decrease with depth. Using the final blow count in our borings as representative of the soil beneath the borings, the average blow count for the upper 100 feet of soil ranges from 33 to 50. Therefore, for seismic design of structures, the site should be considered Class D, "stiff soil", as defined in the IBC.

Chapter 16 of the IBC provides equations for converting the mapped earthquake spectral response to the design spectral response. The mapped earthquake spectral response accelerations for short periods (S_s) and for 1-second periods (S_1) were obtained for the

site from the United States Geologic Survey's National Seismic Hazards Mapping Project website using the latitude and longitude given in Section 1.1, above. S_s at the site is 1.21g and S_1 is 0.44g. Site coefficients for this site are $F_a = 1.1$, $F_v = 1.6$. The design earthquake spectral response accelerations adjusted for site class effects are $S_{ds} = 0.89g$, $S_{d1} = 0.47g$. As indicated in Chapter 18 of the IBC, a design peak ground acceleration (PGA) for the site may be estimated from the S_{ds} by dividing S_{ds} by 2.5 which results in 0.36g.

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3 Conclusions and Recommendations

3.1 Slope Stability

Aspect Consulting, LLC performed a slope stability analysis on a cross section drawn through the slope. The analysis was conducted using the commercially available software Slide 4.0 by Rocscience. A groundwater table was assumed to be present in the slope at shallow depth. Soil strength parameters were estimated from the results of the subsurface investigation combined with a back analysis of topography present before the slide and an assumed safety factor of just under 1.0.

The pseudostatic method was used for our slope stability analysis to estimate the safety factor under seismic conditions. The seismic coefficient used in a pseudostatic analysis is typically taken to be about ½ of the PGA that the site is estimated to experience during the design earthquake. For this project, we used a seismic coefficient of 0.18g.

The results of slope stability analyses are expressed as factors of safety against displacement failure. The factor of safety is the ratio of resisting forces to driving forces. A factor of safety of 1.0 is equilibrium; a factor of safety of less than 1.0 indicates failure. Typically, a minimum factor of safety of 1.5 for static conditions and 1.0 for seismic (pseudostatic) conditions is considered adequate. The lower minimum safety factor for seismic conditions is adequate as the probability of occurrence of the seismic conditions analyzed is relatively low.

Based on our observations at the site, the head of the slide or slide scarp does not appear to be very stable. Portions of the scarp are vertical or near vertical with water flowing from the base of the scarp in at least one location. The scarp appears to be 8 to 12 feet high. As the steepness of the scarp appears to be greater than other slopes in the upper sand in this area, we anticipate that over time, if no remediation is performed, the scarp will slough to a shallower angle. The approximate location of the slide scarp on the south (residence) side of the slide is indicated on Figure 2.

3.1.1 Slope Stability Back Analysis

Our first slope stability analysis for the project was performed using topography available from a survey done for the site prior to construction of the existing house. This topography was developed by Clark Land Office in 1998. Using this topography, we varied the soil parameters of the loose sand layer until a safety factor of about 1 was obtained for a slip surface similar in location and shape to the slide was achieved.

We also used topography developed by Main-Line Surveying earlier this year to analyze the slope in its current state. We again varied the soil parameters of the loose sand layer until a safety factor slightly greater than 1 was obtained. This would represent the minimum possible for the slope to be holding in its current configuration.

Based on these analyses, we used an angle of internal friction of 30 degrees and a cohesion of 100 pounds per square foot (psf) for the loose sand layer. Graphical representations of our analysis are presented in Appendix C.

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3.1.2 Slope Mitigation

Slope mitigation refers to any method used to increase the stability of a slope or any method used to reduce the risk of damage to people or property from slope movement. Methods used to increase the stability of the slope could include regrading the slope to a shallower angle, placement of a soil buttress at the toe of the slope, removal of soil from the top of the slope, or construction of shoring walls on the slope. Methods to reduce the risk of damage to people or property from slope movement include locating structures away from unstable slopes or constructing barriers to limit soil movements toward structures. For purposes of this study we have focused on methods to increase the stability of the slope, as the existing house is already present and there is little available space to move the access driveway.

The limitations of space on the site make regrading the slope or removing soil from the top of the slope infeasible. As the potentially unstable portion of the slope is in the loose sand, with a steeper hard clay slope below it, a toe buttress would likely not be workable. Therefore, construction of a shoring wall of some type appears to be the most feasible option for increasing the stability of the slope.

We considered the merits of several types of walls as a remediation for the landslide including mechanically reinforced earth (MRE) walls, gravity walls (concrete or concrete block), soldier pile walls and soil nail walls. MRE walls consist of compacted soil reinforced with plastic geogrid. Gravity walls are reinforced concrete, or concrete, masonry, or gabion blocks. Soldier pile walls consist of steel beams placed vertically in drilled holes backfilled with concrete. Wood or concrete lagging is placed in the space between adjacent soldier piles. Soil nails are steel pins placed in holes drilled at an angle into the slope with the holes backfilled with cement grout.

Any wall can be subject to a failure that results from movement of the soil beneath the wall. Therefore, for remediation of this slide, any wall must extend through the loose sand to or into the hard clay soil beneath. From the borings drilled for this report and for the Shannon & Wilson report, we anticipate that up to 30 feet of loose sand may be present. A 30-foot deep excavation for construction of either a gravity wall or an MRE wall would likely be prohibitively expensive.

Soil nails work by increasing the total strength of the potentially unstable soil. We anticipate that several rows of nails would need to be installed to sufficiently increase the soil strength. This would require equipment moving on the steep face of the slope where the landslide occurred. We anticipate that the installation requirements for this site and the number of soil nails needed would make this option more expensive than a soldier pile wall.

Installation of a soldier pile wall requires relatively large drill equipment to excavate the holes and install the piles. However, the incremental cost of increasing the lengths of the piles to reach a bearing layer such as the hard clay is generally not prohibitive. Further, the piles can be installed in such a way as to allow backfill soil to be placed to re-

establish the access drive in its pre-slide condition. Soldier pile walls that have an exposed height of greater than 10 to 12 feet are typically reinforced by adding a row or rows of tiebacks. A tieback is a hole drilled from the face of the slope at a near horizontal angle. A steel cable is placed in the hole and the hole is filled with cement grout. The cable is then attached to the soldier pile to increase its load capacity for retained soil.

One further possible option for the project is to not attempt any remediation of the landslide. The access driveway has been moved to the south away from the landslide at this time, and no other structures were directly affected by the slide. The primary disadvantage to this option is that the post landslide topography does not appear to be very stable as discussed in the slope stability section above. We understand that some of the utilities for the residence are within a few feet of the top of the slide scarp. The access driveway has been moved essentially as far to the south as is reasonably possible for the site. For these reasons, any further movement of the landslide towards the structures is likely to damage utilities and possibly cut vehicle access to the residence.

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3.2 Soldier Pile Design

Using the results of our slope stability back analysis described above, we analyzed the slope with soldier piles in place. Based on this analysis, each soldier pile must extend through the loose sand into the hard clay and be capable of withstanding a shear force of 33.3 kips per linear foot of spacing between center of piles (200 kips for a 6-foot spacing). The pile must be capable of withstanding this force at any location within the loose sand layer. In addition, the wall must be designed to withstand the lateral earth forces as discussed in Section 3.3 below. Note that the shear force that the pile needs to resist at depth includes the lateral rotational earth forces; i.e., the pile must be able to resist lateral earth forces of the exposed wall section and the shear force of the lower earth movement, but not both simultaneously.

For design of the piles, the following soil parameters may be used:

Soil Type	Unit Weight (pcf)	Angle of Internal Friction	Cohesion (psf)
Sandy Backfill	120	35	0
Loose Sand	120	30	100
Hard Clay	120	0	4000

For a pile designed to withstand the force given above, the safety factor against slope movement for the slope increase to about 1.5 under static conditions. 1.5 is generally considered an adequate safety factor for static slope stability. For a pile designed as above, the slope will have a safety factor equal to or greater than one for seismic (pseudo-static) conditions with a horizontal seismic coefficient of up to 0.19. This corresponds to a design peak ground acceleration of between 0.29 and 0.38.

3.3 Lateral Earth Pressures

Lateral pressures are 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 a free standing soldier pile wall that is free to rotate away from the retained soil, lateral earth pressures should be estimated for an "active" condition. For walls that are compressing the retained soil, lateral earth pressures should be estimated for a "passive" condition.

For the proposed case of a free-standing soldier pile wall with backfill placed above the wall, the active earth pressure coefficient (K_a) will be 0.27 and the equivalent fluid unit weight will be 33 pounds per cubic foot (pcf). This assumes an angle of internal friction of 35 degrees and a unit weight of 120 pcf for the backfill and no slope above the wall. For the loose sand below the exposed portion of the wall, the passive earth pressure coefficient (K_p) will be 0.56 and the equivalent fluid unit weight will be 67 pcf. This assumes an angle of internal friction of 30 degrees, a unit weight of 120 pcf and a 2H:1V slope below the wall. A passive pressure of 2,000 pcf may be used for the hard clay encountered below the sands. Note that as the below grade portion of the wall consists of piles without lagging, the passive pressure acts over a distance equal to 2.5 times the pile diameter; i.e., if the pile spacing is greater than 2.5 times the pile diameter, the passive pressure exerted by the soil in front of the wall will be less than the maximum possible. The recommended equivalent fluid unit weights do not include hydrostatic pressure due to groundwater accumulated behind walls, seismic loading, or any surcharge due to nearby loading from structures, equipment or traffic. The "passive" pressure was reduced by a factor of 2 to limit wall translation.

Note that if the wall design includes the passive pressure provided by the loose sand downslope from the wall location, it is possible that a future landslide occurring entirely downslope of the wall may cause damage to the wall. This potential damage could be mitigated by ignoring the passive pressure provided from the loose sand. This would likely require the addition of tiebacks to the soldier piles. Alternatively, the wall could be retrofitted with tiebacks after the occurrence of such a landslide. Some movement at the top of the wall would likely occur as the soldier piles bend in response to the loss of passive pressure. In our opinion, the most reasonable alternative is to use the loose sand passive pressure, while allowing for the future installation of tiebacks should a lower slope slide occur.

The potential seismic force on the wall can be modeled as a uniform pressure on the back of the wall equal to 11 times the exposed height of the wall (in feet). The units for this pressure are pounds per square foot (psf).

Weep holes or some other provision for water to flow out from behind the wall should be installed at the base of the wall to prevent the buildup of hydrostatic pressure behind the structure. Based on the presence of water flowing out of the slope after the slide, provisions for collecting and safely draining any water from the weep hole areas should be made to avoid erosion occurring on the slope below the wall.

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Limitations

Work for this project was performed and this report prepared in accordance with generally accepted professional practices for the nature and conditions of work completed in the same or similar localities, at the time the work was performed. It is intended for the exclusive use of Mr. Seiichi Adachi for specific application to the referenced property. This report does not represent a legal opinion. No other warranty, expressed or implied, is made.

The conclusions and recommendations presented in this report are based on the geologic reconnaissance and three borings completed for this project. Because of the nature of exploratory work below ground, extrapolation of subsurface conditions between field explorations is necessary. Please note that differing subsurface conditions may be present due to the random nature of deposition and the alteration of topography by past grading and/or filling. The nature and extent of variations between the field explorations may not become fully evident until construction. If variations are observed at that time, it may be necessary to re-evaluate specific recommendations in this report and make appropriate changes.

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Site Location Map

Adachi Landslide
Toandos Peninsula, Jefferson County, Washington

DATE: October 2008	PROJECT NO. 080029
DESIGNED BY: TSP	
DRAWN BY: PMB	FIGURE NO. 1
REVISED BY: -	

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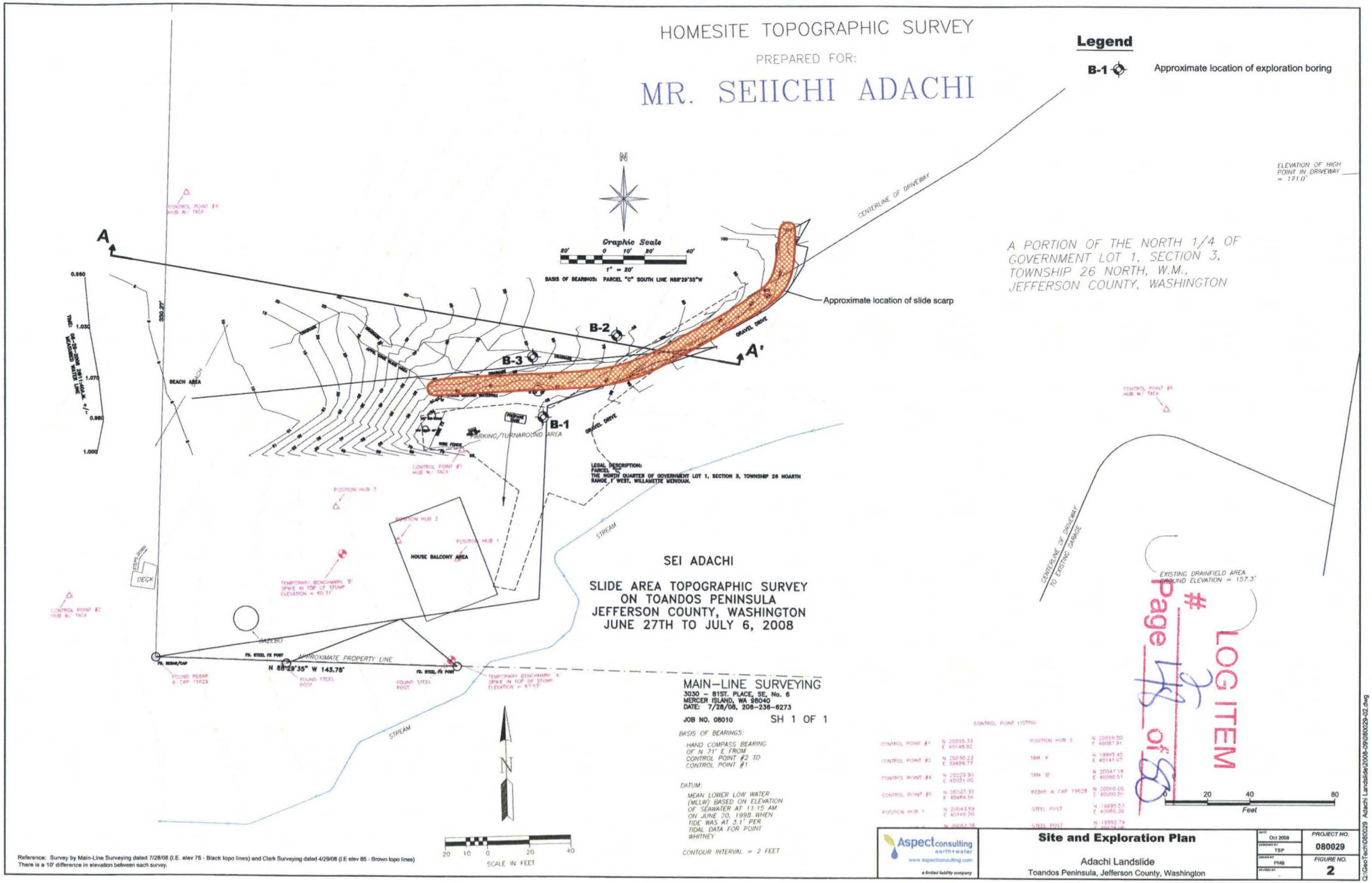
HOMESITE TOPOGRAPHIC SURVEY
PREPARED FOR:
MR. SEIICHI ADACHI
Legend
B-1 Approximate location of exploration boring
A PORTION OF THE NORTH 1/4 OF GOVERNMENT LOT 1, SECTION 3, TOWNSHIP 26 NORTH, W.M., JEFFERSON COUNTY, WASHINGTON
Approximate location of slide scarp
CENTERLINE OF DRIVEWAY
ELEVATION OF HIGH POINT IN DRIVEWAY = 171.0'
Graphic Scale
1" = 20'
BASIS OF BEARINGS: PARCEL "C" SOUTH LINE N88°29'35"W
A
A'
B-1
B-2
B-3
BEACH AREA
PARKING/TURNAROUND AREA
GRAVEL DRIVE
LEGAL DESCRIPTION:
PARCEL "C"
THE NORTH QUARTER OF GOVERNMENT LOT 1, SECTION 3, TOWNSHIP 26 NORTH RANGE 1 WEST, WILLAMETTE MERIDIAN.
HOUSE BALCONY AREA
DECK
GAZEBO
STREAM
APPROXIMATE PROPERTY LINE
N 88°29'35" W 143.78'
CENTERLINE OF DRIVEWAY TO EXISTING GARAGE
EXISTING DRAINFIELD AREA GROUND ELEVATION = 157.3'
SEI ADACHI
SLIDE AREA TOPOGRAPHIC SURVEY
ON TOANDOS PENINSULA
JEFFERSON COUNTY, WASHINGTON
JUNE 27TH TO JULY 6, 2008
MAIN-LINE SURVEYING
3030 - 81ST. PLACE, SE, No. 6
MERCER ISLAND, WA 98040
DATE: 7/28/08, 206-236-6273
JOB NO. 08010
SH 1 OF 1
BASIS OF BEARINGS:
HAND COMPASS BEARING OF N 71° E FROM CONTROL POINT #2 TO CONTROL POINT #1
DATUM:
MEAN LOWER LOW WATER (MLLW) BASED ON ELEVATION OF SEAWATER AT 11:15 AM ON JUNE 20, 1998 WHEN TIDE WAS AT 3.1' PER TIDAL DATA FOR POINT WHITNEY
CONTOUR INTERVAL = 2 FEET
SCALE IN FEET
CONTROL POINT LISTING
Feet
LOG ITEM # 2
Page 48 of 80
Reference: Survey by Main-Line Surveying dated 7/28/08 (I.E. elev 75 - Black topo lines) and Clark Surveying dated 4/29/08 (I.E elev 85 - Brown topo lines)
There is a 10' difference in elevation between each survey.
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Site and Exploration Plan
Adachi Landslide
Toandos Peninsula, Jefferson County, Washington
Oct 2008
TSP
PMB
PROJECT NO. 080029
FIGURE NO. 2
Q:\GeoTech\080029 Adachi Landslide\2008-09\080029-02.dwg

HOMESITE TOPOGRAPHIC SURVEY

PREPARED FOR:

MR. SEIICHI ADACHI

Legend

B-1 Approximate location of exploration boring

LOG ITEM
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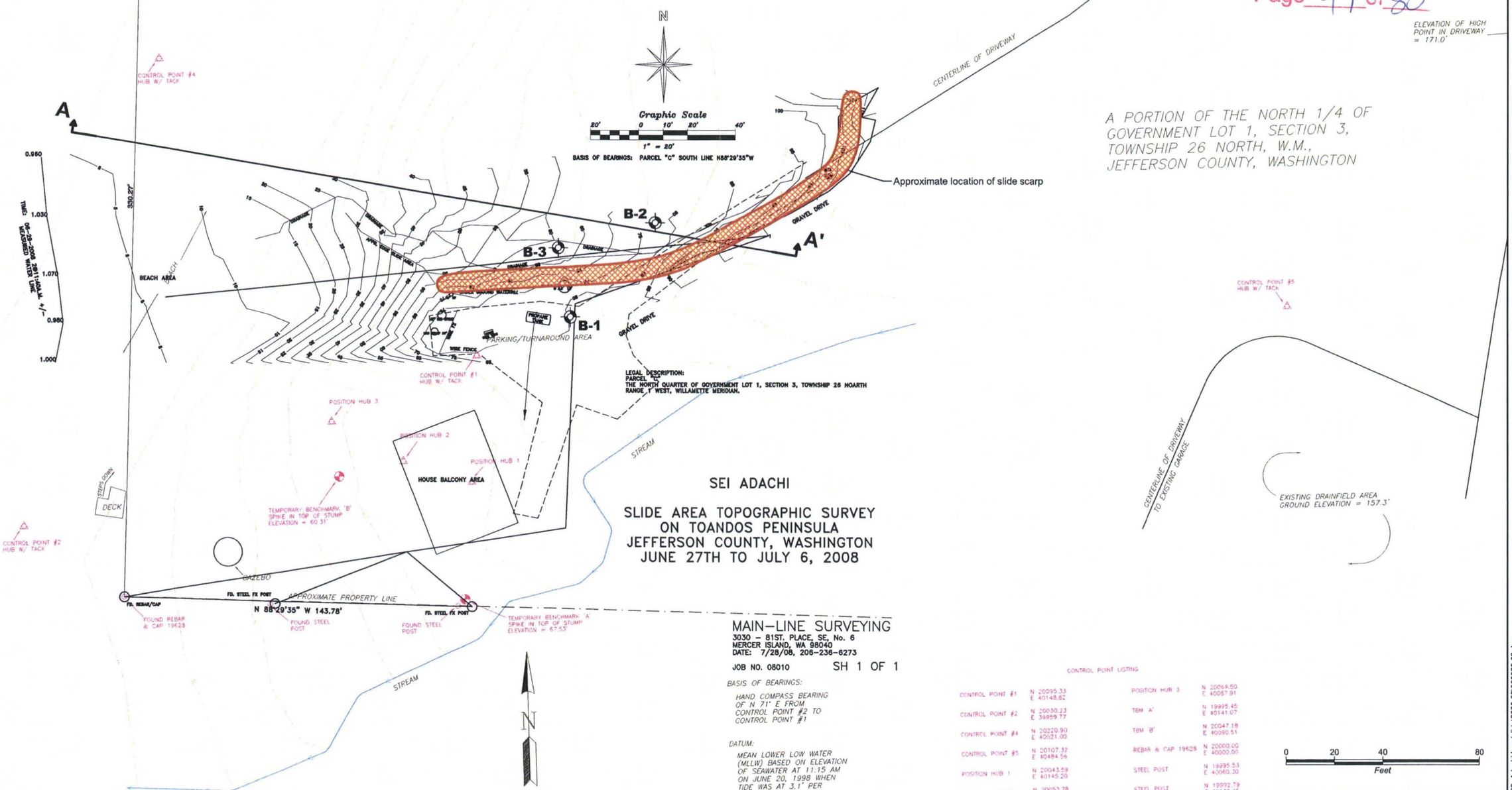

SEI ADACHI

SLIDE AREA TOPOGRAPHIC SURVEY ON TOANDOS PENINSULA JEFFERSON COUNTY, WASHINGTON JUNE 27TH TO JULY 6, 2008

MAIN-LINE SURVEYING
3030 – 81ST. PLACE, SE, No. 6
MERCER ISLAND, WA 98040
DATE: 7/28/08, 206-236-6273

JOB NO. 08010 SH 1 OF 1

BASIS OF BEARINGS:

HAND COMPASS BEARING OF N 71° E FROM CONTROL POINT #2 TO CONTROL POINT #1

DATUM:

MEAN LOWER LOW WATER (MLLW) BASED ON ELEVATION OF SEAWATER AT 11:15 AM ON JUNE 20, 1998 WHEN TIDE WAS AT 3.1' PER TIDAL DATA FOR POINT WHITNEY

CONTOUR INTERVAL = 2 FEET

CONTROL POINT LISTING

CONTROL POINT #1	N 20095.33 E 40148.82	POSITION HUB 3	N 20069.50 E 40087.91
CONTROL POINT #2	N 20030.23 E 39959.77	TBM 'A'	N 19995.45 E 40141.07
CONTROL POINT #4	N 20220.90 E 40021.00	TBM 'B'	N 20047.18 E 40090.51
CONTROL POINT #5	N 20107.32 E 40484.56	REBAR & CAP 19628	N 20000.00 E 40000.00
POSITION HUB 1	N 20043.59 E 40145.20	STEEL POST	N 19995.53 E 40060.30
[illegible]	N 20052.78 [illegible]	STEEL POST	N 19992.79 [illegible]

Reference: Survey by Main-Line Surveying dated 7/28/08 (I.E. elev 75 - Black topo lines) and Clark Surveying dated 4/29/08 (I.E elev 85 - Brown topo lines)
There is a 10' difference in elevation between each survey.

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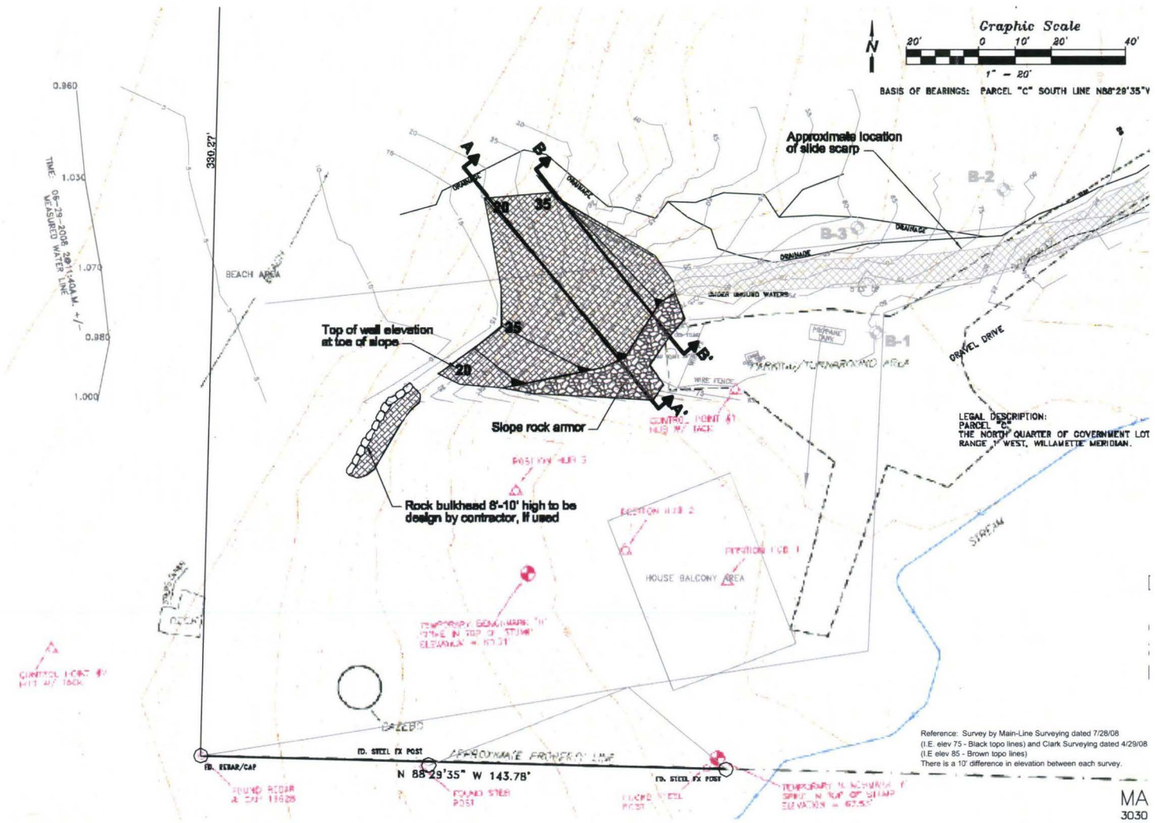
Site and Exploration Plan

Adachi Landslide
Toandos Peninsula, Jefferson County, Washington

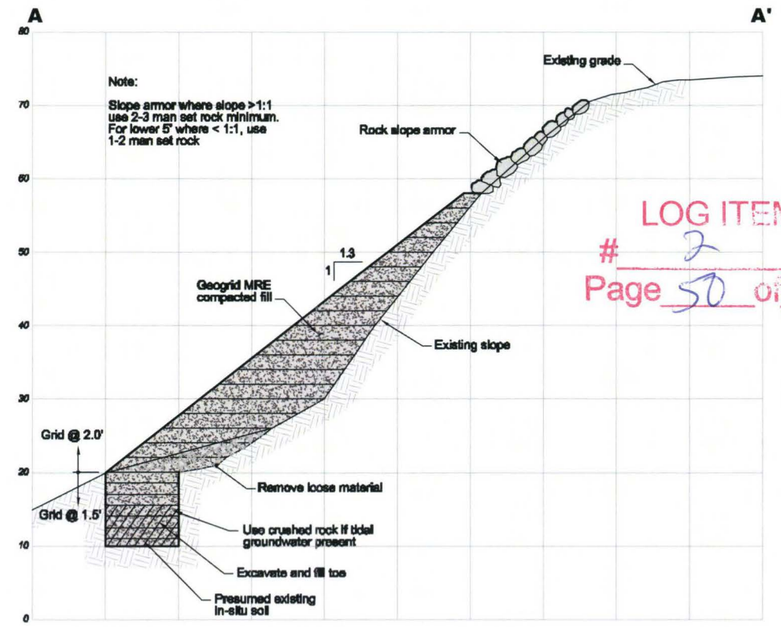
DATE: Oct 2008	PROJECT NO. 080029
DESIGNED BY: TSP	
DRAWN BY: PMB	FIGURE NO. 2
REVISED BY: .	

Q:\GeoTech\080029 Adachi Landslide\2008-09\080029-02.dwg



SITE PLAN

Scale: 1" = 20'-0"



CROSS SECTION A-A'

Scale: 1" = 10'-0" Vertical
1" = 10'-0" Horizontal

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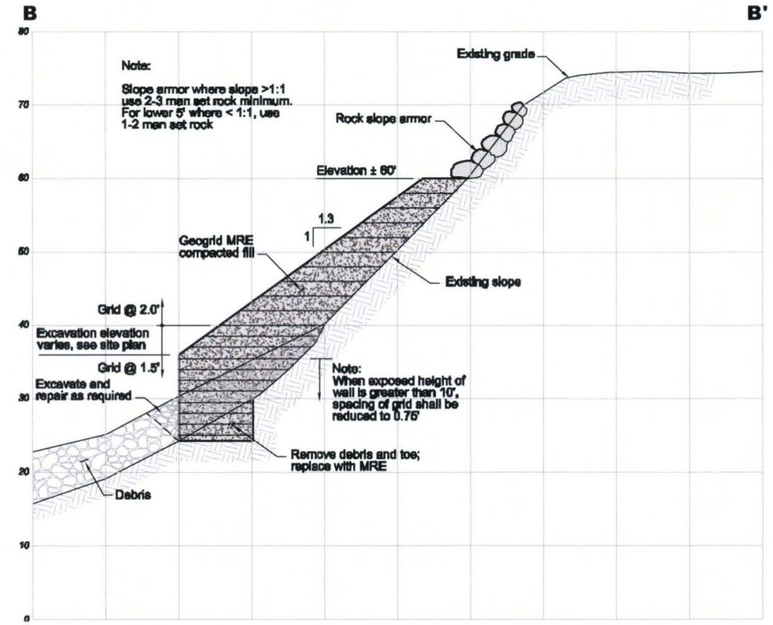
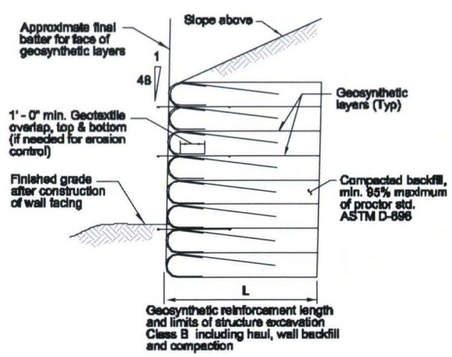


CROSS SECTION B-B'

Scale: 1" = 10'-0" Vertical
1" = 10'-0" Horizontal

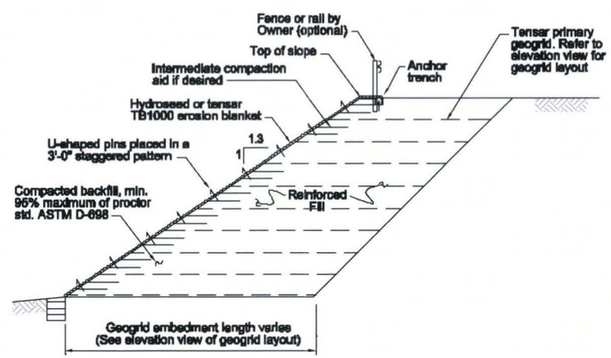
GENERAL NOTES

1. Length of toe block is 10' minimum.
2. Grid should extend from face to contact with in-situ slope.
3. Grid strength is 400 lbs/linear foot.
4. Tensor - BX1200 will meet grid requirements.



MRE WALL SECTION

Not to scale



REINFORCED EARTH BUTTRESS

Not to scale

JOHN L. PETERSON
STATE OF WASHINGTON
18802
REGISTERED
PROFESSIONAL ENGINEER

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Site Plan, Cross Sections, Details and Notes

Adachi Slope Protection
Toandos Peninsula, Jefferson County, Washington

DATE: Oct 2009	PROJECT NO. 080029
DESIGNED: JLP	
DRAWN: PMB	FIGURE NO. A
REVIEWED: -	

Q:\GeoTech\080029 Adachi Landslide\2009-10\080029-01.dwg

LOG ITEM
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APPENDIX A

Boring Logs

Coarse-Grained Soils - More than 50%[1] Retained on No. 200 Sieve	Gravels - More than 50%[1] of Coarse Fraction Retained on No. 4 Sieve	≤5% Fines[5]	GW	Well-graded gravel and gravel with sand, little to no fines
			GP	Poorly-graded gravel and gravel with sand, little to no fines
		≥15% Fines[5]	GM	Silty gravel and silty gravel with sand
			GC	Clayey gravel and clayey gravel with sand
	Sands - 50%[1] or More of Coarse Fraction Passes No. 4 Sieve	≤5% Fines[5]	SW	Well-graded sand and sand with gravel, little to no fines
			SP	Poorly-graded sand and sand with gravel, little to no fines
		≥15% Fines[5]	SM	Silty sand and silty sand with gravel
			SC	Clayey sand and clayey sand with gravel
Fine-Grained Soils - 50%[1] or More Passes No. 200 Sieve	Silts and Clays Liquid Limit Less than 50		ML	Silt, sandy silt, gravelly silt, silt with sand or gravel
			CL	Clay of low to medium plasticity; silty, sandy, or gravelly clay, lean clay
			OL	Organic clay or silt of low plasticity
	Silts and Clays Liquid Limit 50 or More		MH	Elastic silt, clayey silt, silt with micaceous or diatomaceous fine sand or silt
			CH	Clay of high plasticity, sandy or gravelly clay, fat clay with sand or gravel
			OH	Organic clay or silt of medium to high plasticity
Highly Organic Soils			PT	Peat, muck and other highly organic soils

Terms Describing Relative Density and Consistency

Coarse-Grained Soils

Density	SPT[2] blows/foot
Very Loose	0 to 4
Loose	4 to 10
Medium Dense	10 to 30
Dense	30 to 50
Very Dense	>50

Fine-Grained Soils

Consistency	SPT[2] blows/foot
Very Soft	0 to 2
Soft	2 to 4
Medium Stiff	4 to 8
Stiff	8 to 15
Very Stiff	15 to 30
Hard	>30

Test Symbols

G = Grain Size
M = Moisture Content
A = Atterberg Limits
C = Chemical
DD = Dry Density
K = Permeability

Component Definitions

Descriptive Term	Size Range and Sieve Number
Boulders	Larger than 12"
Cobbles	3" to 12"
Gravel	3" to No. 4 (4.75 mm)
Coarse Gravel	3" to 3/4"
Fine Gravel	3/4" to No. 4 (4.75 mm)
Sand	No. 4 (4.75 mm) to No. 200 (0.075 mm)
Coarse Sand	No. 4 (4.75 mm) to No. 10 (2.00 mm)
Medium Sand	No. 10 (2.00 mm) to No. 40 (0.425 mm)
Fine Sand	No. 40 (0.425 mm) to No. 200 (0.075 mm)
Silt and Clay	Smaller than No. 200 (0.075 mm)

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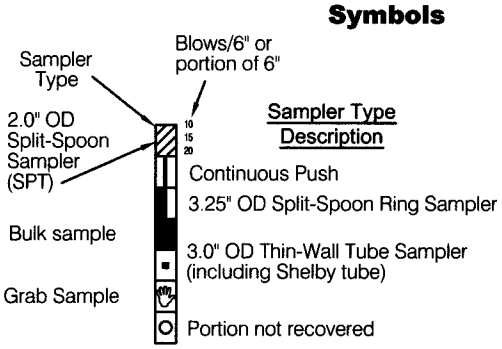
[3] Estimated Percentage

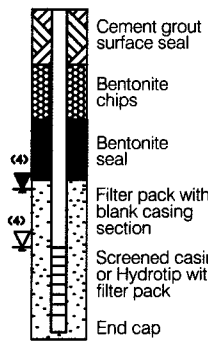
Percentage by Weight	Modifier
<5	Trace
5 to 15	Slightly (sandy, silty, clayey, gravelly)
15 to 30	Sandy, silty, clayey, gravelly)
30 to 49	Very (sandy, silty, clayey, gravelly)

Moisture Content

Dry - Absence of moisture, dusty, dry to the touch
Slightly Moist - Perceptible moisture
Moist - Damp but no visible water
Very Moist - Water visible but not free draining
Wet - Visible free water, usually from below water table

Symbols





(1) Percentage by dry weight
(2) (SPT) Standard Penetration Test (ASTM D-1586)
(3) In General Accordance with Standard Practice for Description and Identification of Soils (ASTM D-2488)
(4) Depth of groundwater
(5) Combined USCS symbols used for fines between 5% and 15% as estimated in General Accordance with Standard Practice for Description and Identification of Soils (ASTM D-2488)

ATD = At time of drilling
Static water level (date)

Classifications of soils in this report are based on visual field and/or laboratory observations, which include density/consistency, moisture condition, grain size, and plasticity estimates and should not be construed to imply field or laboratory testing unless presented herein. Visual-manual and/or laboratory classification methods of ASTM D-2487 and D-2488 were used as an identification guide for the Unified Soil Classification System.

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Exploration Log Key

DATE:
DESIGNED BY:
DRAWN BY:
REVISED BY:

PROJECT NO.

FIGURE NO.
A-1

Q:_ACAD Standards\Standard Details\Exploration Log Key A1.dwg

Boring Log

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Project Number	Boring Number	Sheet
080029-001-02	B-1	1 of 2

Project Name: Adachi Residence- Landslide
Ground Surface Elev: 90+/-
Location:
Driller/Equipment: Geologic Drill / Track mounted rig
Depth to Water:
Drilling Method: Hollow Stem Auger
Start/Finish Date: 3/19/2008

Depth / Elevation (feet)	Sample Type/ID	Tests	Blows/ 6"	Material Type	Description	Depth (ft)
1, 2					Loose, slightly moist, silty, very sandy GRAVEL (GM), trace organics.	1, 2
3			4 4 6		Stiff, moist, very sandy SILT (ML), fine sand, trace organics.	3
4						4
5		m=24%	5 5 7			5
6					Grades to very moist.	6
7						7
8			8 10 12		Grades to very stiff.	8
9						9
10		m=29%, -200=63%	5 6 9		Grades to wet and stiff.	10
11						11
12						12
13			5 6 7			13
14						14
15		m=30%	6 6 7			15
16						16
17						17
18			3 3 5			18
19						19
20		m=34%	11 19 27			20
21					Hard, wet, dark gray CLAY (CL).	21
22						22
23			10 18 29		Grades to very moist.	23
24						24

N-value / Water Content % (0 10 20 30 40 50)

LOG ITEM # 2
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Sampler Type:
- No Recovery
- Split Spoon

PID - Photoionization Detector (Headspace Measurement)
- Static Water Level
- Water Level (ATD)

Logged by: JTL
Approved by: TSP
Figure No. A- 2

GEOTECH BORING LOG ADACHI LANDSLIDE.GPJ October 21, 2008

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Boring Log

Project Number	Boring Number	Sheet
080029-001-02	B-1	2 of 2

Project Name: Adachi Residence- Landslide
Ground Surface Elev: 90+/-
Location:
Driller/Equipment: Geologic Drill / Track mounted rig
Depth to Water:
Drilling Method: Hollow Stem Auger
Start/Finish Date: 3/19/2008

Depth / Elevation (feet)	Borehole Completion	Sample Type/ID	Tests	Blows/ 6"	N-value ▲ Water Content % ★ (0 10 20 30 40 50)	Material Type	Description	Depth (ft)
			m=24%	9 18 30				
26								26
27							LOG ITEM # 2 Page 54 of 80	27
28								28
29								29
30			m=22%	11 18 30				30
31								31
32							Bottom of borehole at 31.5' bgs. Borehole backfilled with bentonite chips.	32
33								33
34								34
35								35
36								36
37								37
38								38
39								39
40								40
41								41
42								42
43								43
44								44
45								45
46								46
47								47
48								48
49								49

Sampler Type:
- No Recovery
- Split Spoon

PID - Photoionization Detector (Headspace Measurement)
- Static Water Level
- Water Level (ATD)

Logged by: JTL
Approved by: TSP
Figure No. A- 2

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Boring Log

Project Number	Boring Number	Sheet
080029-001-02	B-2	1 of 2

Project Name: Adachi Residence- Landslide
Ground Surface Elev: 82+/-
Location:
Driller/Equipment: Geologic Drill / Track mounted rig
Depth to Water:
Drilling Method: Hollow Stem Auger
Start/Finish Date: 3/19/2008

Depth / Elevation (feet)	Sample Type/ID	Tests	Blows/ 6"	Material Type	Description	Depth (ft)
1					Loose, wet, brown, slightly silty SAND (SP-SM), trace organics, fine sand.	1
2						2
3	X		2 2 2			3
4						4
5	X	m=35%, -200=9%	2 2 2			5
6						6
7						7
8	X		4 3 2			8
9						9
10	X	m=26%	13 20 25		Hard, wet, dark gray CLAY (CL).	10
11					Slow drilling.	11
12						12
13	X		8 15 20		Grades to very moist.	13
14						14
15	X	m=28%	10 18 18			15
16						16
17						17
18	X		12 20 20			18
19						19
20	X	m=24%	10 10 25			20
21						21
22						22
23						23
24						24

N-value / Water Content % ★: 0 10 20 30 40 50

LOG ITEM
2
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Sampler Type:
No Recovery
Split Spoon

PID - Photoionization Detector (Headspace Measurement)
Static Water Level
Water Level (ATD)

Logged by: JTL
Approved by: TSP
Figure No. A- 3

GEOTECH BORING LOG ADACHI LANDSLIDE.GPJ October 21, 2008

Boring Log

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Project Number	Boring Number	Sheet
080029-001-02	B-2	2 of 2

Project Name: Adachi Residence- Landslide
Ground Surface Elev: 82+/-
Location:
Driller/Equipment: Geologic Drill / Track mounted rig
Depth to Water:
Drilling Method: Hollow Stem Auger
Start/Finish Date: 3/19/2008

Depth / Elevation (feet)	Borehole Completion	Sample Type/ID	Tests	Blows/ 6"	N-value / Water Content % (0 10 20 30 40 50)	Material Type	Description	Depth (ft)
26			m=26%	13 18 21				26
27							LOG ITEM # 2 Page 56 of 50	27
28								28
29								29
30			m=23%	10 12 18				30
31							6" of wet SILT (ML).	31
32								32
33							Driller indicates gravel layer.	33
34								34
35			m=22%	8 13 19				35
36								36
37								37
38								38
39								39
40			m=26%	13 16 19				40
41								41
42							Bottom of borehole at 41.5' bgs. Borehole backfilled with bentonite chips.	42
43								43
44								44
45								45
46								46
47								47
48								48
49								49

Sampler Type:
- No Recovery
- Split Spoon

PID - Photoionization Detector (Headspace Measurement)
- Static Water Level
- Water Level (ATD)

Logged by: JTL
Approved by: TSP
Figure No. A- 3

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Boring Log

Project Number	Boring Number	Sheet
080029-001-02	B-3	1 of 2

Project Name: Adachi Residence- Landslide
Ground Surface Elev: 60+/-
Location:
Driller/Equipment: Geologic Drill / Track mounted rig
Depth to Water:
Drilling Method: Hollow Stem Auger
Start/Finish Date: 3/19/2008

Depth / Elevation (feet)	Sample Type/ID	Tests	Blows/ 6"	N-value / Water Content % (0 10 20 30 40 50)	Material Type	Description	Depth (ft)
1, 2						Loose, wet, brown, slightly gravelly, very silty SAND (SM), trace organics, trace wood.	1, 2
3	Split Spoon	m=27%, -200=99%	2 2 6			Stiff, wet, dark gray SILT (ML), trace sand, trace organics.	3
4	No Recovery						4
5	Split Spoon		15 13 7			Grades to very stiff.	5
6							6
7						Grades to hard.	7
8	Split Spoon		7 14 23			6" of sandy SILT (ML).	8
9							9
10	Split Spoon	m=24%	13 24 30	50+			10
11						Trace fine sand.	11
12							12
13	Split Spoon		14 22 27				13
14						Trace fine sand.	14
15	Split Spoon	m=24%	14 23 26				15
16							16
17							17
18							18
19							19
20	Split Spoon		9 18 26				20
21							21
22							22
23							23
24							24

LOG ITEM # 2
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Sampler Type:
- No Recovery
- Split Spoon

PID - Photoionization Detector (Headspace Measurement)
- Static Water Level
- Water Level (ATD)

Logged by: JTL
Approved by: TSP
Figure No. A- 4

GEOTECH BORING LOG ADACHI LANDSLIDE.GPJ October 21, 2008

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Boring Log

Project Number	Boring Number	Sheet
080029-001-02	B-3	2 of 2

Project Name: Adachi Residence- Landslide
Ground Surface Elev: 60+/-
Location:
Driller/Equipment: Geologic Drill / Track mounted rig
Depth to Water:
Drilling Method: Hollow Stem Auger
Start/Finish Date: 3/19/2008

Depth / Elevation (feet)	Borehole Completion	Sample Type/ID	Tests	Blows/ 6"	N-value ▲ / Water Content % ★ (0 10 20 30 40 50)	Material Type	Description	Depth (ft)
			m=29%, -200=100%	18 31 47	50+			
26							Tip of sampler is wet.	26
27							LOG # 2 Page 58 of 80	27
28								28
29								29
30			m=21%, -200=100%	11 24 38	50+		N-value collected again at 31.5; no sample retained. N-value (blow counts) possibly over-estimated due to sampler flexing.	30
31								31
32				11 20 31	50+			32
33								33
34							Bottom of borehole at 33' bgs. Borehole backfilled with bentonite chips.	34
35								35
36								36
37								37
38								38
39								39
40								40
41								41
42								42
43								43
44								44
45								45
46								46
47								47
48								48
49								49

Sampler Type:
- ○ No Recovery
- ⊠ Split Spoon

PID - Photoionization Detector (Headspace Measurement)
- ▼ Static Water Level
- ▽ Water Level (ATD)

Logged by: JTL
Approved by: TSP
Figure No. A- 4

LOG ITEM
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APPENDIX B

Laboratory Test Results

LOG ITEM
2
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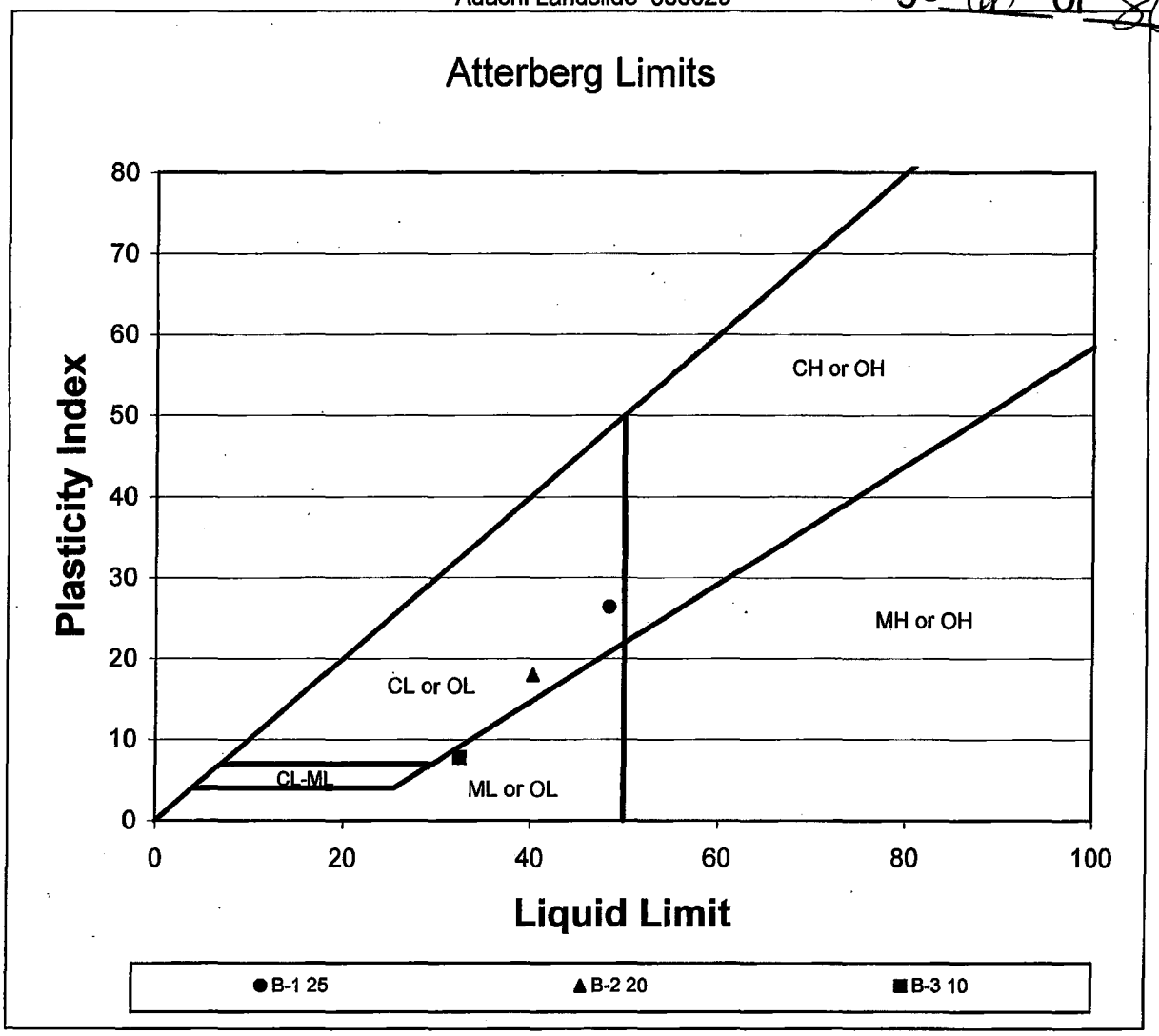
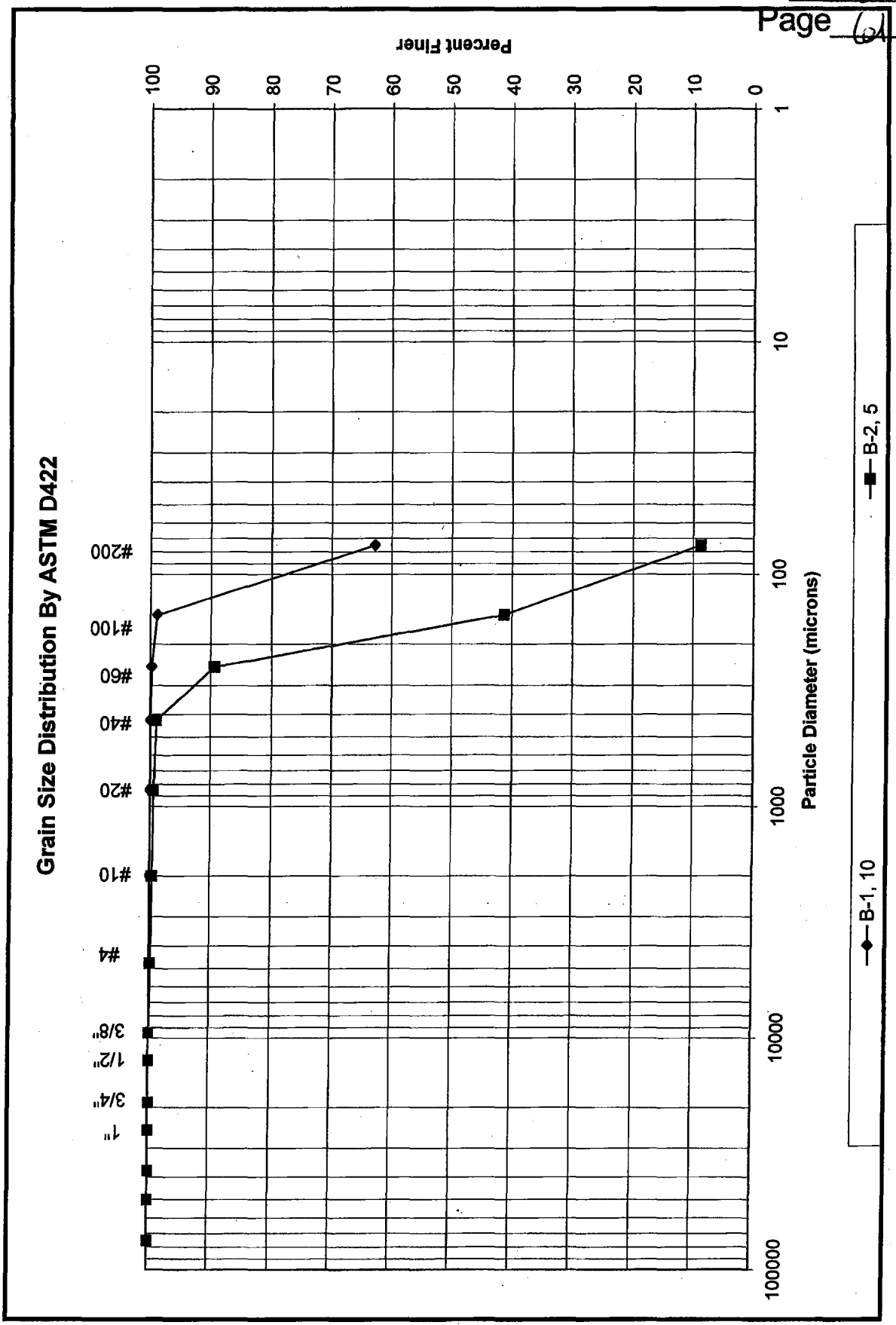
Adachi Landslide 080029



Sample Number	Depth (ft)	As-Received Moisture Content	Plasticity Index	Liquid Limit	Plastic Limit	USCS
B-1	25	23.54	26.4	48.5	22.1	CL
B-2	20	23.90	18.0	40.3	22.3	CL
B-3	10	23.83	7.7	32.5	24.7	ML

MP77


Grain Size Distribution By ASTM D422
Percent Finer
100
90
80
70
60
50
40
30
20
10
0
1
10
100
1000
10000
100000
Particle Diameter (microns)
1"
3/4"
1/2"
3/8"
#4
#10
#20
#40
#60
#100
#200
B-1, 10
B-2, 5

LOG ITEM
2
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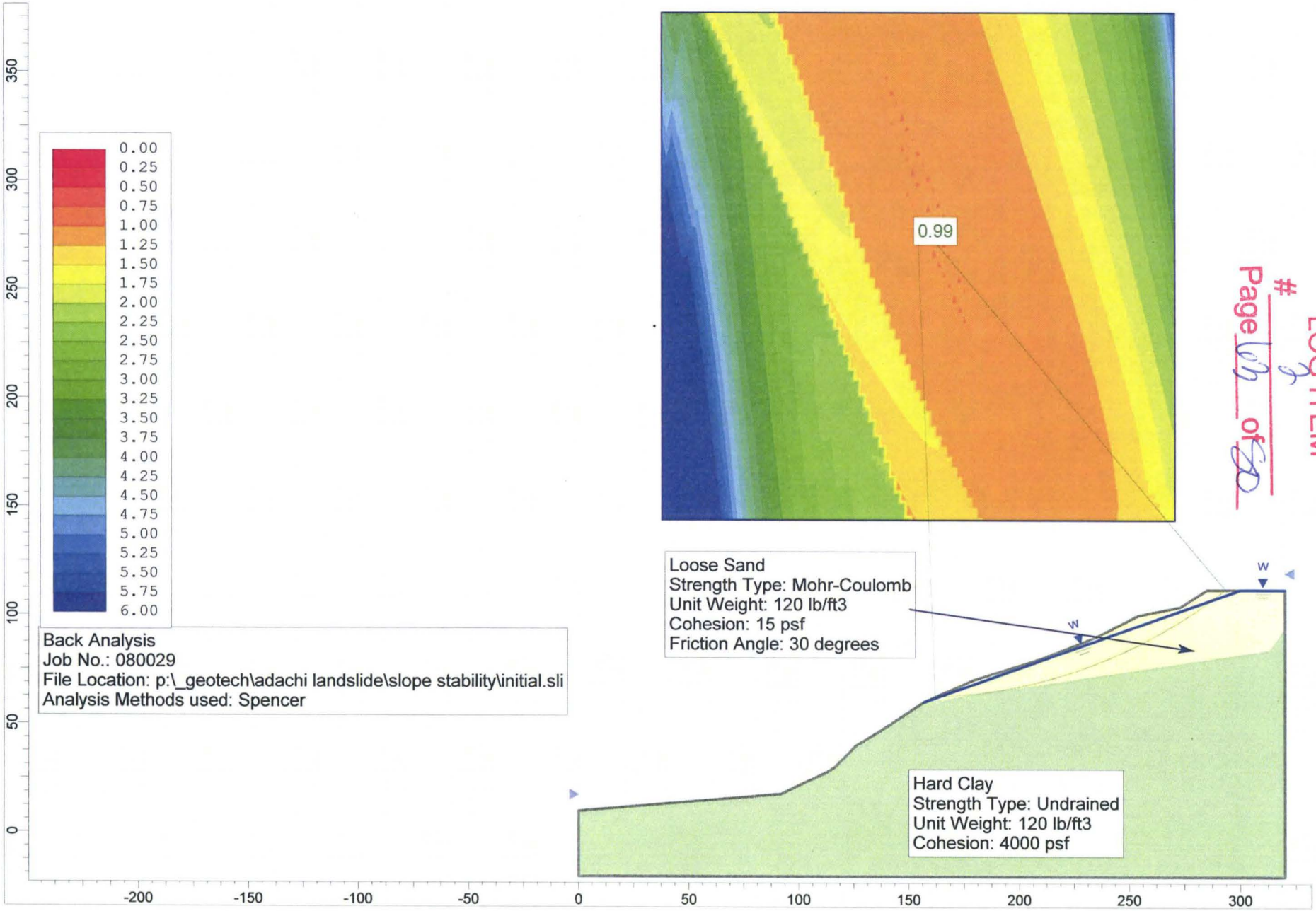
APPENDIX C

Slope Stability Analyses



LOG ITEM
3
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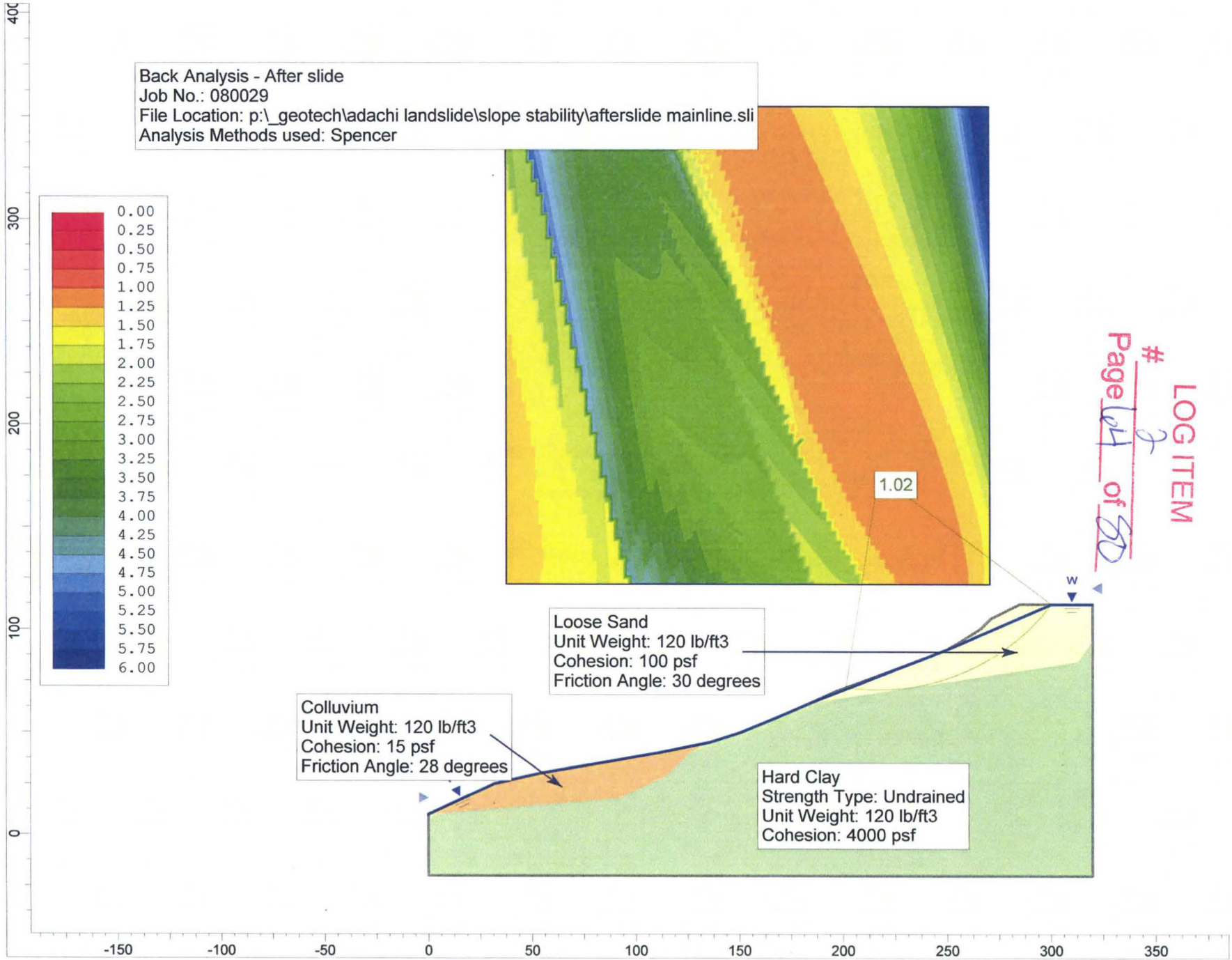

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2
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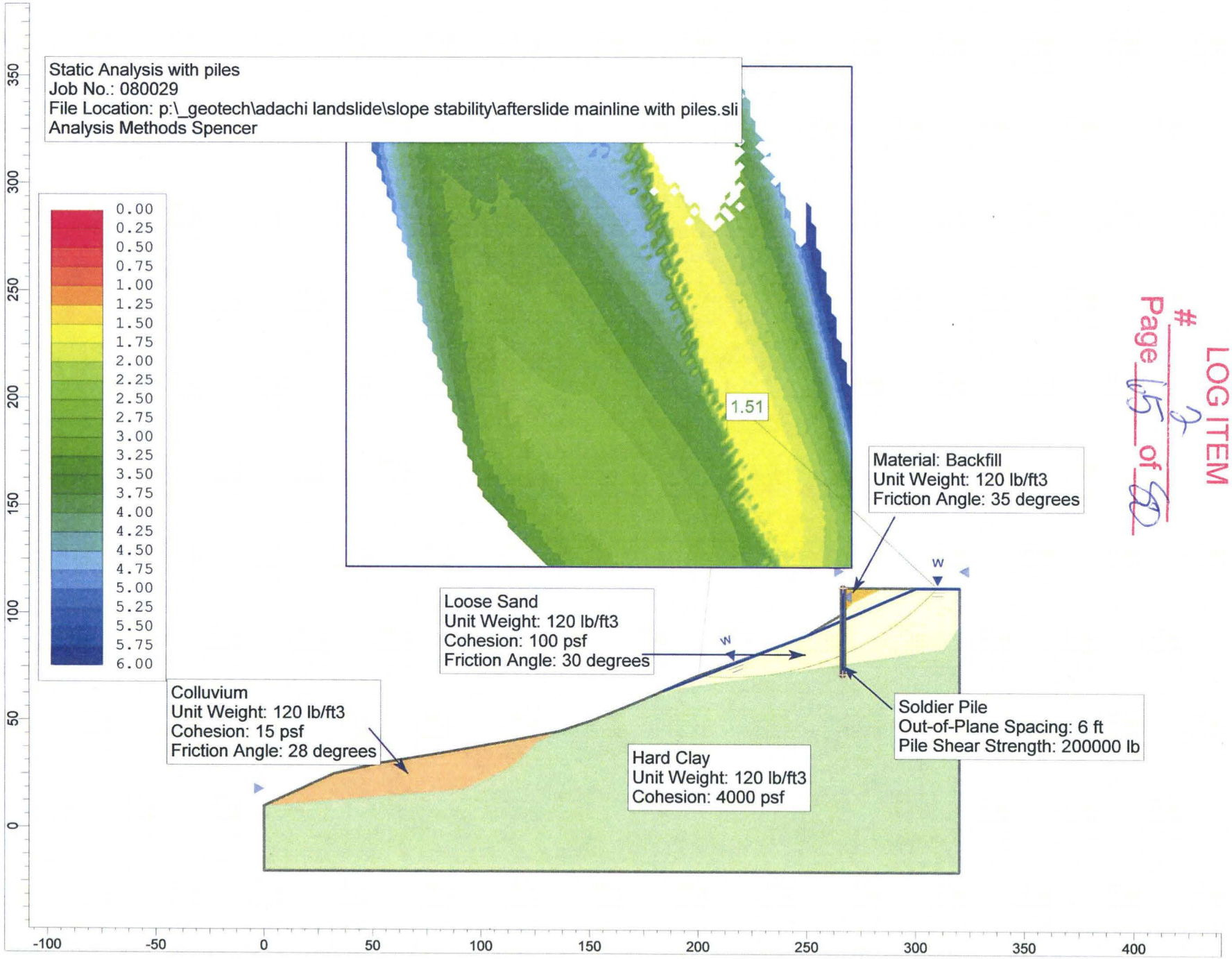

LOG ITEM
2
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