S-Curves Landslide Investigation and Stabilization
Cannon Beach, Oregon

Geotech Solutions Inc.

May 12, 2003

GSI Project: Cannon-02-01-gi

May 12, 2003

Cannon-02-01-gi

City of Cannon Beach
PO Box 368
Cannon Beach, Oregon 97110
e: jega@pacifier.com

Attention: Joy Gannon

GEOTECHNICAL ENGINEERING SERVICES
S-Curves Landslide Investigation and Stabilization

As authorized, we appreciate the opportunity to present this Geotechnical Engineering Report summarizing our evaluation of the S-Curves landslide on Hemlock Street in Cannon Beach, Oregon. The ultimate objective of our work is to provide qualitative solutions for reducing the translation of the slide to acceptable levels. In order to achieve this, it was necessary to acquire subsurface and slide movement data, analyze the slide, and provide stabilization options. Our explorations were completed in October of 2002 with seven subsequent visits to monitor slope movement and water level variations.

Our report includes a description of the subsurface conditions, inclinometer data (which indicates the magnitude and depth(s) of slope movement), water level variations, survey monument movement, daily rainfall, slope stability analyses, and qualitative solutions. Our specific scope of work included the following:

- Review geologic maps, previous reports, and available subsurface information and slide data in our files.
- Complete a site reconnaissance to observe surface features relevant to the slide and to assist in planning and locating the explorations and access.
- Provide principal level project management including management of field and subcontracted services, report writing, analyses, and invoicing.
- Explore subsurface conditions by drilling eight borings to average depths of up to 70 feet using a track mounted drill rig equipped for mud rotary and rock coring methods *(Traffic Control and Utility Locates provided by City)*.
- Observe soil and ground water encountered in the explorations, obtain samples every 2.5 to 5 feet, and maintain a detailed log of the explorations.
- Determine the moisture content and dry unit weight of selected samples obtained from the explorations and conduct soil classification testing as necessary.
- Install inclinometer casing and suitable monuments and monument protection in all six borings. Use sand pack for dual use as phreatic standpipes in five of the borings.
- Use two digital water level recorders and in-line data loggers with serial pickup to evaluate water level fluctuations in two explorations.
- Complete 7 monitoring visits to take inclinometer readings and download piezometric data, providing related analyses and visit summary memos *(Traffic control by City for in-road readings)*.
- Coordinate with HLB, in their separate direct contract with the City, to install and survey 20 surface monitoring points (including the 6 boring locations) for 3 monitoring events, and to provide the locations on an existing map.

- Coordinate with HLB to complete a in-line survey of three profiles through the area and provide the information in a CAD drawing.
- Provide a summary report of slide monitoring data.
- Based on the obtained data and information provided by others, build a computer model of the slide stability and complete analyses of up to two stabilization solutions.
- Consult with a specialty contractor to assist with construction cost estimates and planned stabilization.
- Coordinate consultation with HLB regarding permitting issues associated with selected stabilization measures as well as their evaluation for suitable permitted discharge of ground water.
- Provide a summary report of the recommended stabilization option complete with initial cost estimates for implementation of stabilization measures.

SITE OBSERVATIONS AND CONDITIONS

Surface Conditions

The primary active slide, as defined by data obtained to-date and observation of surface features, encompasses an area extending south from Chena Street to the drainage located north of Nazina Street and from the beach to the Boring residence located at the top of Chena Street. The slide topography has been altered by benching to accommodate residential lots and right-of-way, but generally slopes down to the west and changes from elevation 144 near the west end of Chena Street to elevation 12 at current beach level. This primary active slide is part of larger slide terrain that is less active.

Subsurface Conditions

General **-** We explored the site by completing a total of six borings (B-1 through B-6) at the approximate locations shown on the attached ***Site Plan*** between October 8th and 18th, 2002. Borings were completed to depths of between 40 and 90 feet. Geo-Tech Explorations of Tualatin, Oregon completed the borings using both track-mounted (B-1 and B-2) and truck-mounted (B-3 through B-6) drill rigs equipped for mud-rotary drilling. Survey points by HLB are also shown on the ***Site Plan***.

Geology **-** Geology maps (DOGAMI OGI-14 and Ross, 1977) of the area indicate the site is underlain by a unit of the Astoria Formation with various mapped slide features. The Miocene aged formation consists predominantly of sedimentary micaceous siltstone, mudstone, and sandstone. The formation typically weathers to silt and clay soils near the surface.

Explorations **–** In general, the subsurface profile consisted of silt overlying siltstone to the depths explored, which is consistent with logs of borings completed by others (GeoEngineers, GeoDesign, Inc.) in the vicinity of our explorations. A 20-foot thick layer of silt containing gravel-sized siltstone fragments was encountered between these layers in B-5. The consistency of the overlying silt was generally medium stiff to stiff with moisture contents between 25 and 55 percent. Standard penetration test (SPT) blow counts ranged from 4 to 30. The siltstone was generally very soft to soft (RH0 to RH1) with blow counts of 25 to over 50 and moisture contents of 21 to 51 percent.

> **Note:** The inclinometer installed in B-3 was disturbed during repair of an adjacent utility. The installation was repaired with "new" initial readings taken on December 16, 2002. All references to B-3 refer only to data obtained after this date. Though movement between October 23 and December 16 is unlikely (based on our evaluation of all other data), it could have occurred and would not have been documented.

Groundwater – Due to the drilling methods used, it was difficult to determine the depths to groundwater during our explorations. However, groundwater depths were measured continuously in borings B-1 and B-3 and during each visit in B-2, B-4, and B-6. Plots of continuously measured groundwater depths for B-1 and B-3 have been included. The following table shows the measured groundwater depths during each of our monitoring visits.

Date	B-1	B-2	B-3	B-4[1]	B-6[1]
10/23/02	39.7	-	10.5	-	7.7
11/7/02	-	-	-	21.4	-
11/12/02	39.3	-	17.3	31.7	6.7
11/20/02	39.1	1.8	27.3	24.8	5.3
12/16/02	26.2	-	-	23.5	3.4
1/9/03	39.5	1.9	14.8	23.5	8.9
2/3/2003	32.5	1.8	13.2	27.9	5.0
3/25/2003	24.1	-	13.6	26.1	3.5

[1] – Water depths may have been influenced by infiltration into the flush roadway monument

CONCLUSIONS AND RECOMMENDATIONS

GENERAL

Intense wet season rainfall and high groundwater levels cause the primary active slide to move. Based on our observations, testing, and analyses, we recommend completing a pilot study incorporating the installation, monitoring, and evaluation of horizontal drains in the slide.

A detailed description of subsurface conditions, analyses, and proposed solutions are included in the following sections of this report.

MONITORING

Inclinometers - Based on our observations and testing, the slide moves at varying extents and magnitudes as a function of rainfall intensity and duration. Rainfall intensity and duration this winter and spring has generally been slightly below average. The resulting rainfall totals and intense events have been significant enough to initialize noticeable movement of the slide. Movement has been documented in B-1, B-2, B-4, and B-5, which are located in and below Hemlock Street. The movement observed at the top of the slide's prevalent failure surface in each of the four active installations is summarized in the table below and shown graphically on the attached plots.

Date	Movement at Top of Slide Plane (inches)					
	B-1	B-2	B-3	B-4	B-5	B-6
10/23/2002	0.0	0.0	-	0.0	0.0	0.0
11/13/2002	0.0	0.0	-	0.0	0.0	0.0
11/20/2002	0.0	0.0	-	0.0	0.0	0.0
12/16/2002	0.1	-	0.0	0.0	0.0	0.0
1/9/2003	0.8	0.4	0.0	0.2	0.1	0.0
2/3/2003	2.7	1.7	0.0	0.5	0.4	0.0
3/25/2003	NR	NR	0.0	1.1	2.1	0.0

NR - Not Readable

Survey Points - Surface survey points (by HLB) indicate that surface movement is occurring predominantly south of the drainage between tax lot 12 and 600, north of the drainage that runs approximately parallel to Nazina Avenue, and west of the overlooking residence (significant movement of the driveway has been observed). Surface movement of nearly 2.5 inches was documented in February. Subsequent readings, taken on March 26 shows the direction of movement was essentially the same with a significant increase magnitude. Between 4.0 and 6.5 inches of movement were documented in the area west of Hemlock Street (points 2, 3, 6, 7, and 8). Surface movement along Hemlock Street ranged from 0.6 to 2.5 inches (points 10, 11, 15, 16, and 24).

Movement associated with survey point 2 is likely a result of localized instability resulting from beach erosion. The large vertical displacement associated with B-6 (survey point 9) is likely an erroneous initial reading, as no deformation of the inclinometer casing has been observed which would indicate movement.

Rainfall - Rainfall information was provided by the City of Cannon Beach and is summarized in the attached plots. The monthly rainfall and associated trends (since 1987) for Cannon Beach and daily rainfall totals during out monitoring period are both included. Total rainfall for all of 2002 was the 5th lowest annual total in 16 years. With slightly less than average rainfall in the beginning of 2003, this year's rainfall is currently the 7th lowest. However, two intense events, accumulating 4 to 7 inches of rain in a three-day period, have contributed to slope movement, over 7 inches between January 29th and 31st, and over 5 inches between March 21st and 23rd.

Groundwater - Groundwater depths were measured continuously in B-1 and B-3 and during our visits in the remaining installations (B-2, B-4, and B-6). Plots showing the groundwater elevations and daily rainfall totals show the maximum groundwater elevations and the time lag associated with different rainfall events.

Groundwater levels rose to within about 26 feet of the ground surface during four separate events in boring B-1. During the two most intense aforementioned events, respectively, groundwater rose to within 22 feet of the ground surface. The groundwater levels in B-1 appear to peak slower (½ to 1 day) than those observed in B-3 (less than ½ day). The depth of the groundwater in B-3 may be influenced by damage sustained during the nearby utility repair or the proximity to the existing utility trench (which could be acting as a drain).

ANALYSES

Using topographic cross-sections (prepared by HLB) oriented through the slide mass and approximately parallel to documented movement, and in conjunction with subsurface information and monitoring data, we created a slope model using the computer program XSTABL (figures are attached). Using reasonable properties for the site soils and groundwater elevation data, we were able to back-calculate the strength of the soil within the shear zone.

After establishing the soil strengths and slide geometry, we were able to determine the necessary reduction of the water levels to achieve a suitable factor of safety. Any factors of safety greater than 1.0 indicate no movement. A factor of safety equal to 1.2 is typical for the "repair" of active slides.

SOLUTIONS

General - Slowing an active slide is accomplished by either reducing the driving force or increasing the resisting force. The driving force can by reduced by lowering groundwater levels in the slide mass (wells or drains), removing overlying soil at the top of slopes, or replacing heavier soils with light-weight fill at the top of slopes. In contrast, the resisting force can be increased by adding mass at the toe (buttress), installing reinforcing (piles, piers, or anchors), or increasing resistance along the slide surface (injection grouting, soil mixing, etc.).

Due to the geometry and topography of the slide and boundary constraints, in our opinion the most economical means by which to slow slide movement is lowering groundwater levels through improved drainage of the slide mass.

Based on observations and monitoring, slope movement is typically only observed during the wet season. Rising groundwater elevations, resulting from prolonged and intense rainfall, results in noticeable and documented movement of the slide. The goal of installing drainage elements would be to limit the rise of groundwater in the slide mass to levels shown not to induce movement.

Based on our analyses, reducing groundwater levels by 11 to 16 feet would result in a factor of safety equal to approximately 1.2. However, lower reduction of the peak groundwater levels will also result in a reduction in the magnitude of slope movement.

Drainage - Landslides are generally drained by installing cutoff trenches (french drains), horizontal drains, wells, or dewatering shafts. The depth of the slide surface and the consistency of the overlying soil is not conducive to economically installing a cutoff trench. The magnitude of slope movement, the low permeability of the overlying soils, and subsequently small capture radius would reduce the effectiveness of vertical wells. Additionally, the maintenance and operation costs of maintaining wells are higher than passive drainage, such as horizontal drains.

Due to the passive nature of such a system and the reduced maintenance required to maintain operation, for preliminary planning, we recommend completing a horizontal drain pilot study.

Monitoring - Additional monitoring of slide movement will be necessary to evaluate the effectiveness of horizontal drains installed as part of a pilot study. Existing instrumentation can be used if it remains intact through the wet season. Inclinometer casing installed in B-1 and B-2 has deflected too much this season to be functional for evaluation of the pilot study.

Risk - Investment in a pilot study may not reduce slide movement. However, based on our analyses installing horizontal drains is the most economical and feasible means to reduce slide movement to acceptable levels. The results of the pilot study will determine the viability of a full dewatering system.

PILOT STUDY

General - We have contacted Jensen Drilling Company (JDC) of Eugene, Oregon in regard to installation of horizontal drains. JDC has extensive experience installing horizontal drains along the Oregon coast, particularly in stabilization projects for slides founded in the same geologic unit as the S-Curves slide.

Procedure - The procedure would incorporate the use of a track-mounted drill positioned perpendicular to the face of the slope, with the angle above horizontal set to approximately 7 percent. An approximate 4-inch hole is advanced to the design length using rotary drilling techniques. The drill bit, (which is expendable), is attached to 3.5 inch diameter drill rods. After completion of the drilling, 1.5 inch slotted schedule #80 PVC is installed through the drill rods. The drill bit is then removed, and the drill rods are withdrawn while holding the PVC in place. This completes the installation.

Cost – Based on a quote from JDC, for projects where 1,000 feet or more of horizontal drain is installed, the resulting cost is generally about $15 per lineal foot of drain plus $3,000 in mobilization fees. For the pilot study, this would result in a sub-contract cost for installation of roughly $48,000. Additional sub-contracted services, analyses, monitoring, and documentation would be an additional $42,000 (as documented previously). Resulting project cost for a pilot study would be approximately $90,000.

DESIGN

Under the preceding separate agreement for the pilot study, we will work with HLB to provide preliminary drain locations. Consultation between HLB and the City will be necessary for determining permitting and drain discharge requirements. Present plans include installing approximately 3,000 lineal feet of drains installed from four positions during the fall of 2003.

LIMITATIONS AND OBSERVATION DURING CONSTRUCTION

We have prepared this report for use by the City of Cannon Beach and their design and construction teams for this project only. The information herein could be used for bidding or estimating purposes but should not be construed as a warranty of subsurface conditions. We have made observations only at the aforementioned locations and only to the stated depths. These observations do not reflect soil types, strata thicknesses, water levels or seepage that may exist between observations. We should be consulted to observe all construction activities related to slide stabilization, including horizontal drain installation, groundwater collection/management, and continued monitoring. We should be consulted to review final design and specifications in order to see that our recommendations are suitably followed. If any changes are made to the recommended design, we should be consulted. The preceding recommendations should be considered preliminary, as actual soil conditions may vary. In order for our recommendations to be final, we must be retained to observe actual subsurface conditions encountered. Our observations will allow us to interpret actual conditions and adapt our recommendations if needed.

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

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We appreciate the opportunity to work with you on this project and look forward to our continued involvement. If you have any questions, please do not hesitate to call.

Sincerely,

Ryan White, PE
Project Engineer

REGISTERED PROFESSIONAL ENGINEER
53860PE
OREGON
JANUARY 15, 2002
RYAN WHITE

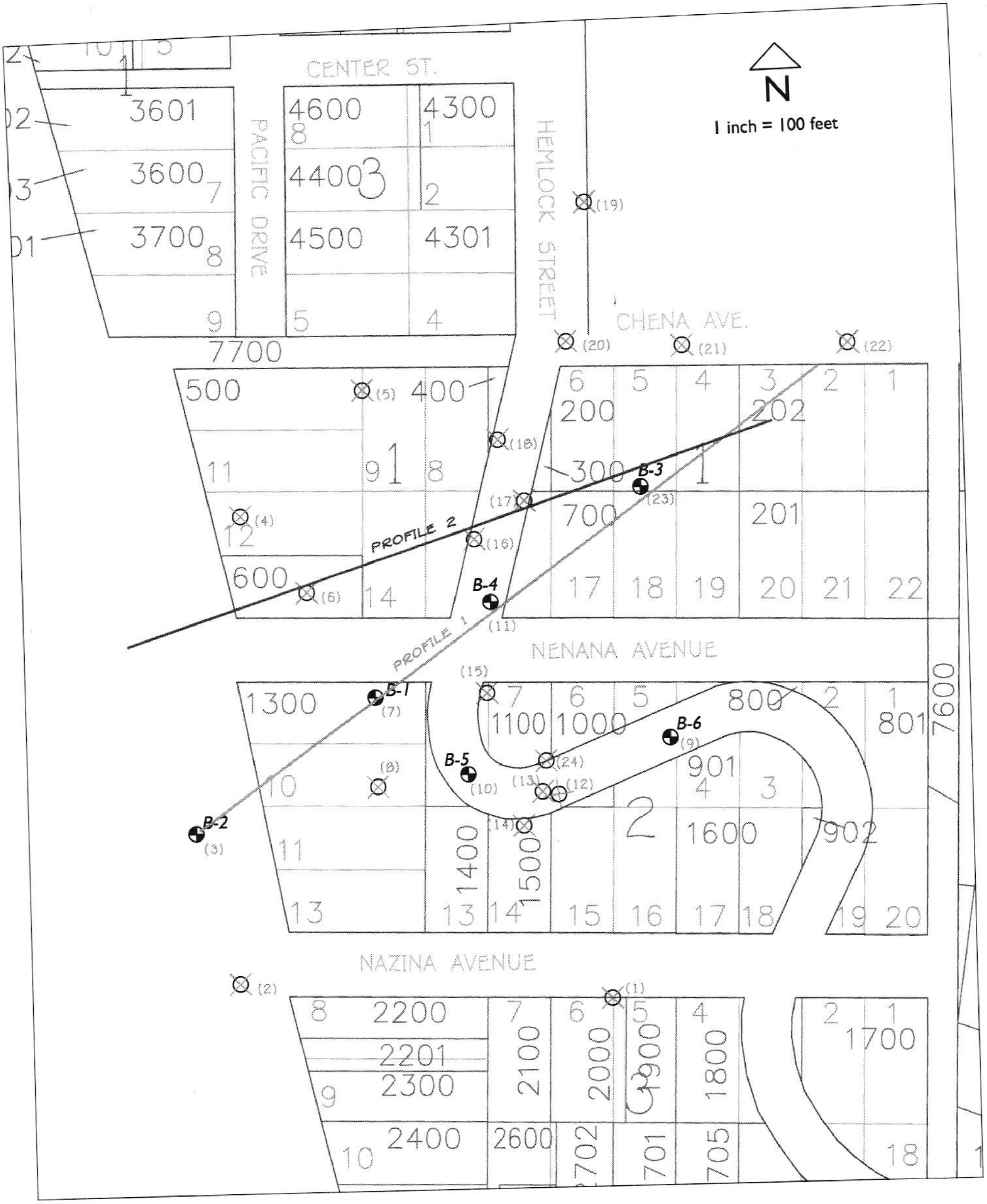
RENEWAL DATE: 6/2004

Don Rondema, MS, PE
Principal

Attachments

REGISTERED PROFESSIONAL ENGINEER
16473
OREGON
JULY 21, 1992
DON RONDEMA

Expires 12/31/04



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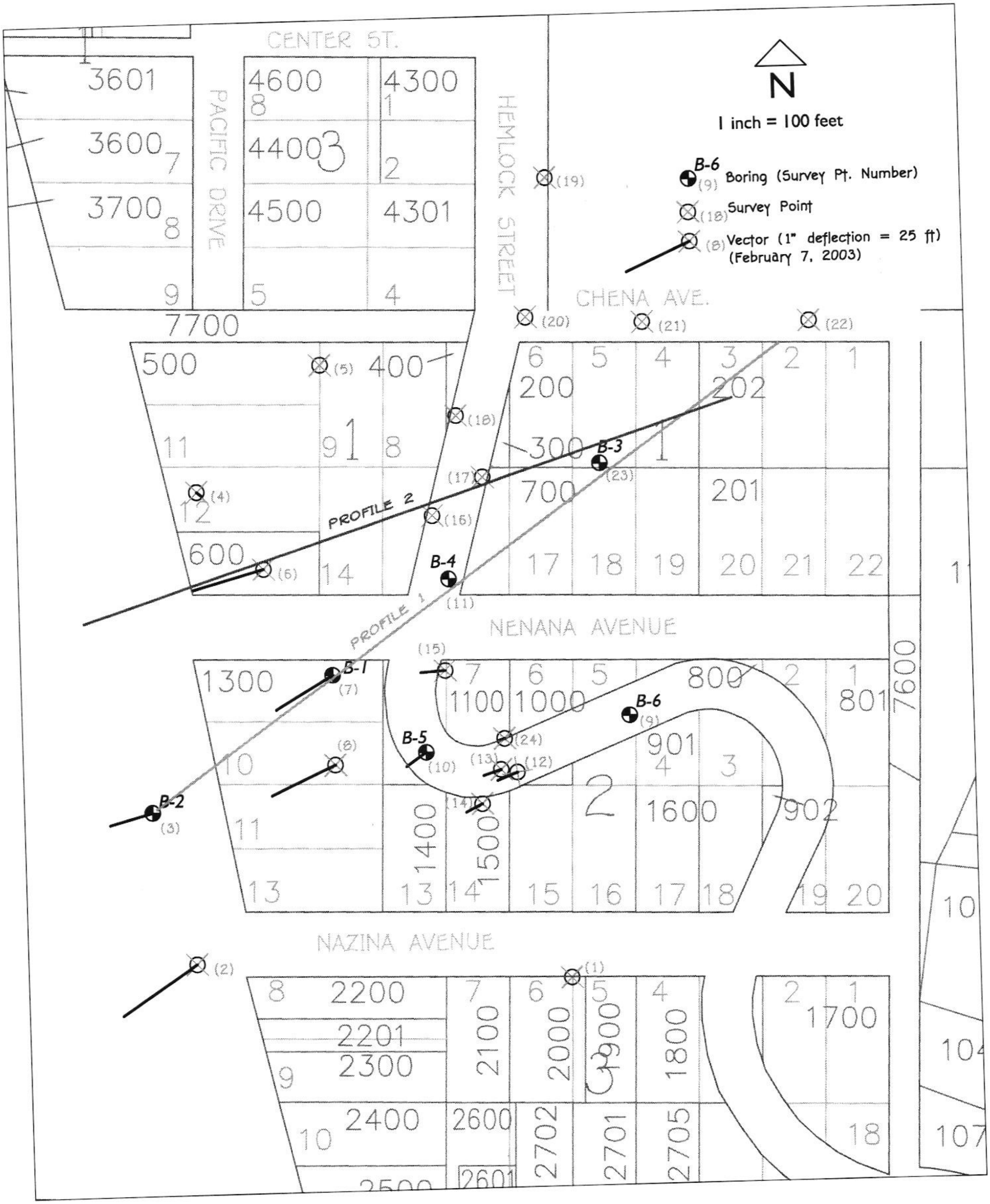
SITE PLAN

Cannon-02-01-gi



SURVEY MONITORING (2/7/03)

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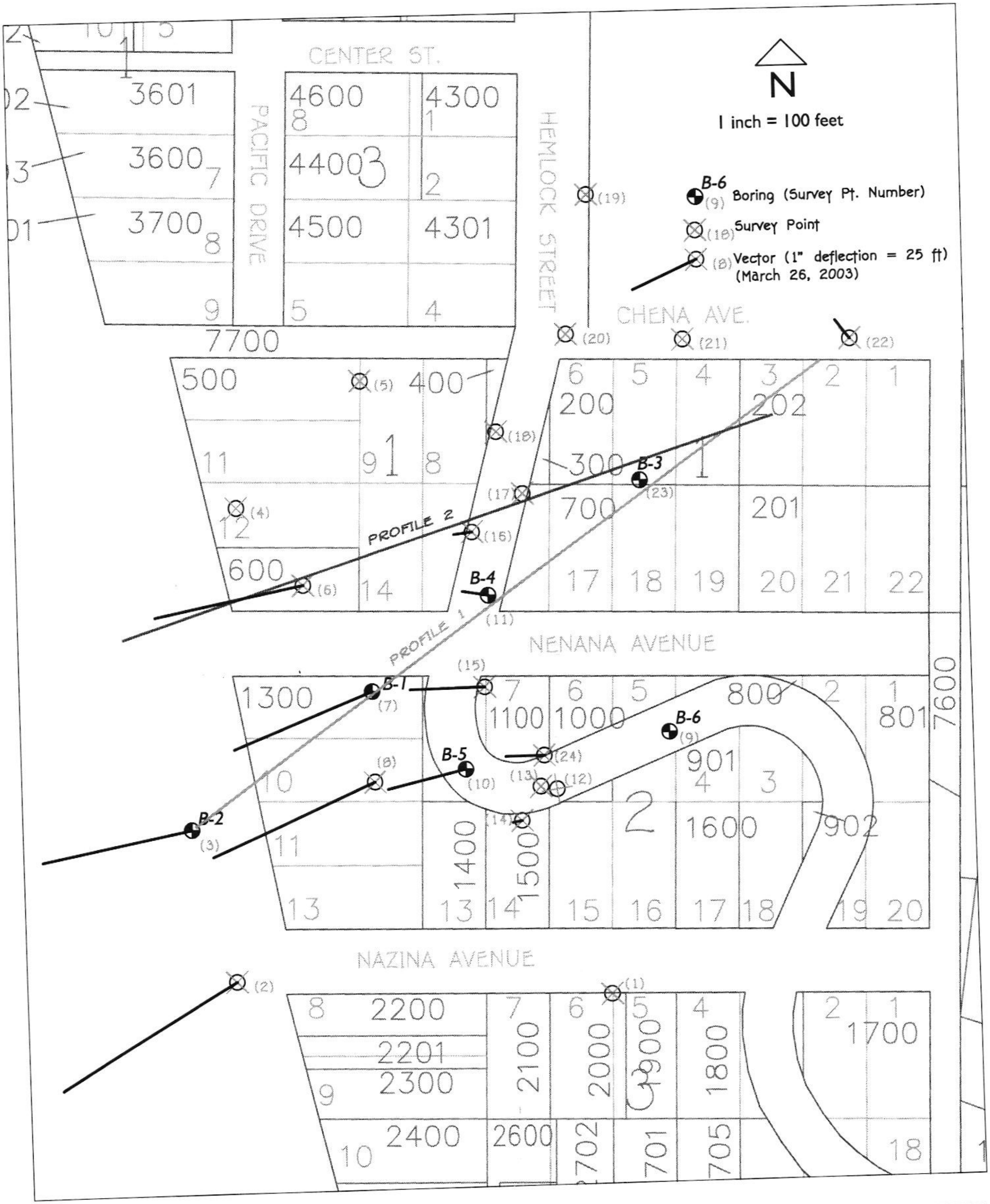
Cannon-02-01-gi



SURVEY MONITORING (3/26/03)

Cannon-02-01-gi

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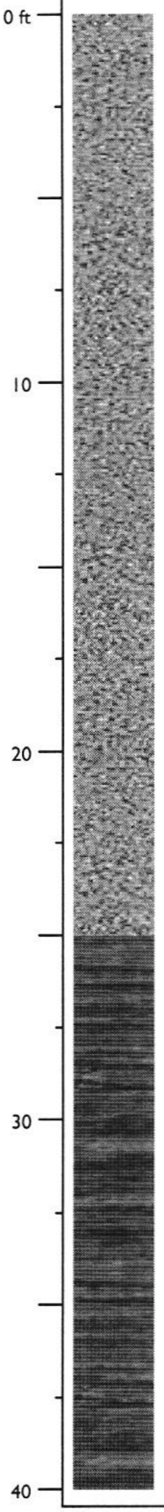
w = moisture content
N = SPT blowcount
* = No recovery

Soil and Rock Description



Medium stiff to stiff, orange, brown, and gray SILT (severely weathered siltstone); moist

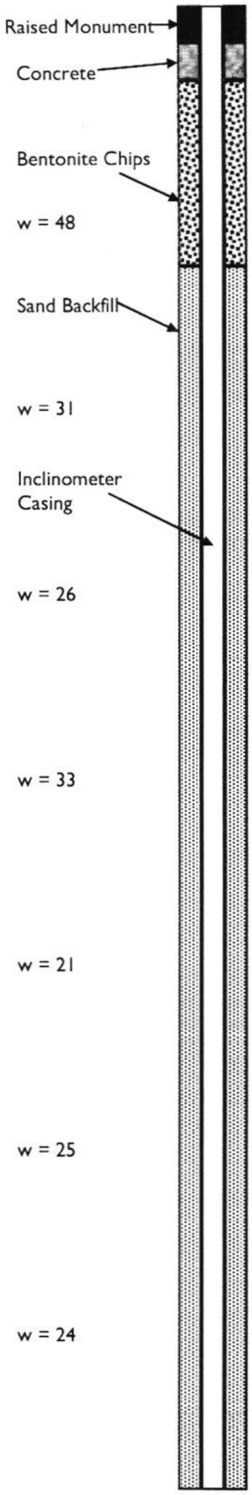
becomes gray at 15 feet.

Lose circulation of drilling mud between 15 and 17.5 feet.

Lose circulation of drilling mud 24 feet.

Very soft to soft (RH0 - RH1), little weathered, little fractured, gray SILTSTONE.

Samples and Data



6	w = 48
9	w = 31
5	w = 26
30	w = 33
59	w = 21
48	w = 25
41	w = 24

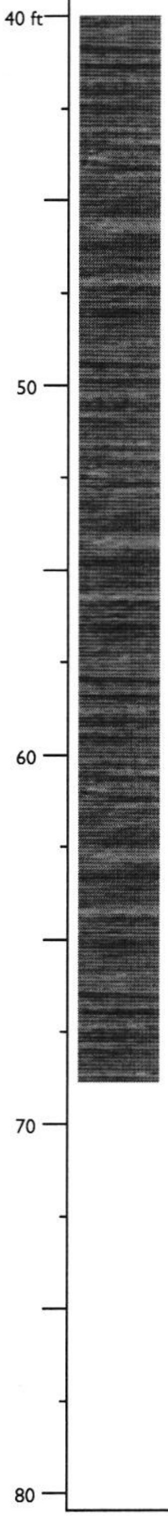
Geotech Solutions Inc.

BORING B-1

Cannon-02-01-gi

w = moisture content
N = SPT blowcount
* = No recovery

Soil and Rock Description



Very soft to soft (RH0 - RH1), little weathered, little fractured, gray SILTSTONE.

Samples and Data

30 w = 26

51 w = 23

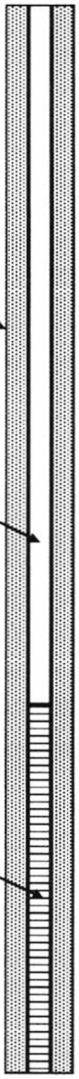
50+ w = 25
N = 42-50/6 in.

50+ w = 23
N = 35-50/5.5 in.

50+ w = 22
N = 38-50/5 in.

50+ w = 23
N = 50-50/4.5 in.

50+ w = 22
N = 50/4.5 in.



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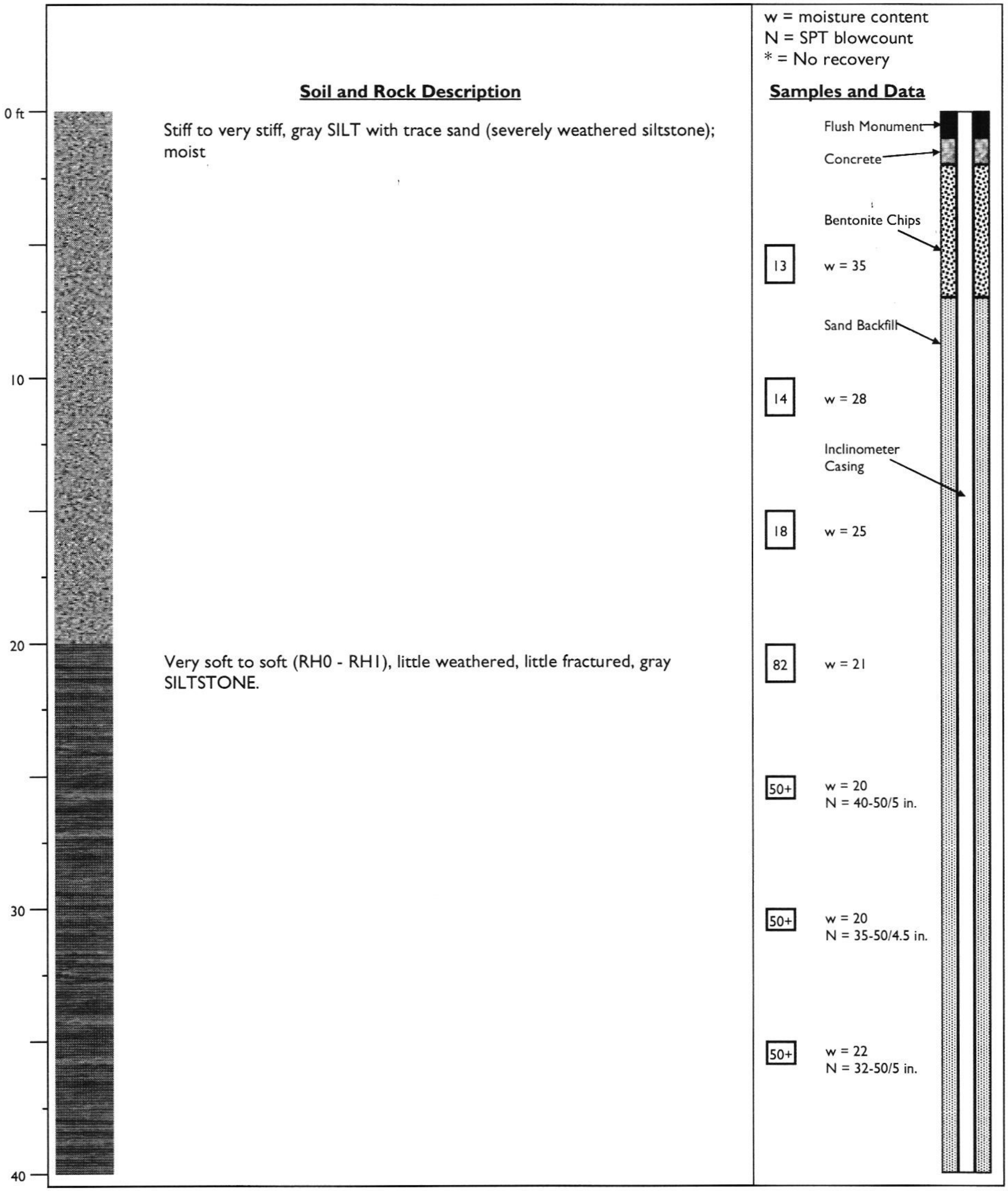
BORING B-1 cont.
Cannon-02-01-gi

w = moisture content
N = SPT blowcount
* = No recovery



Soil and Rock Description

Stiff to very stiff, gray SILT with trace sand (severely weathered siltstone); moist

Very soft to soft (RH0 - RH1), little weathered, little fractured, gray SILTSTONE.

Samples and Data

13	w = 35
14	w = 28
18	w = 25
82	w = 21
50+	w = 20 N = 40-50/5 in.
50+	w = 20 N = 35-50/4.5 in.
50+	w = 22 N = 32-50/5 in.

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BORING B-2

Cannon-02-01-gi

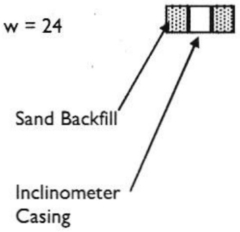
w = moisture content
N = SPT blowcount
* = No recovery

Soil and Rock Description

Samples and Data

40 ft ▨ Very soft to soft (RH0 - RH1), little weathered, little fractured, gray SILTSTONE.

50+ w = 24



50

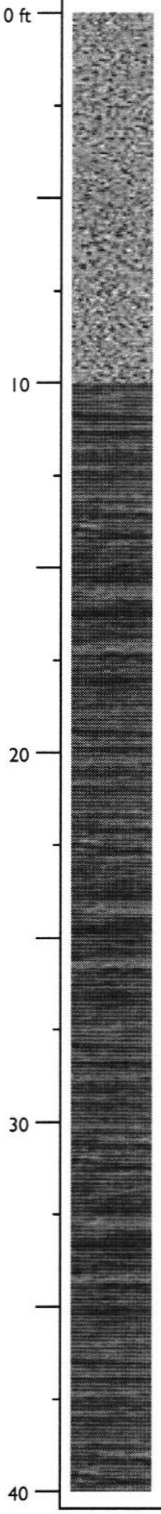
60

70

80

w = moisture content
N = SPT blowcount
* = No recovery

Soil and Rock Description



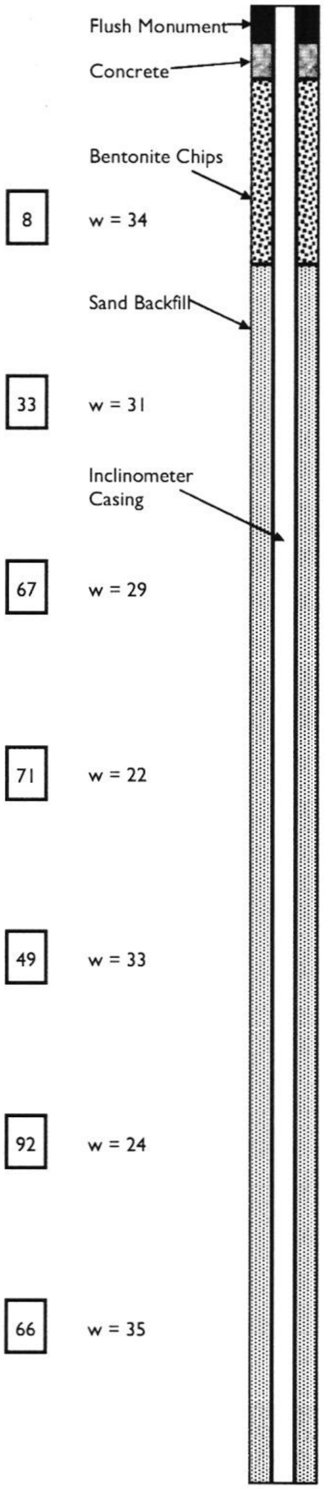
Medium stiff to stiff, orange, brown, and gray SILT (severely weathered siltstone); moist

Lose drilling mud at 8.5 feet.

Very soft to soft (RH0 - RH1), little weathered, little fractured, gray SILTSTONE.

Lose drilling circulation of drilling mud at approximately 12 feet.

Samples and Data



8	w = 34
33	w = 31
67	w = 29
71	w = 22
49	w = 33
92	w = 24
66	w = 35

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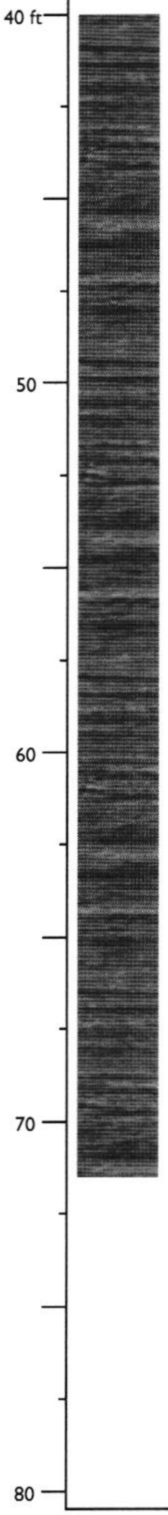
BORING B-3
Cannon-02-01-gi

w = moisture content
N = SPT blowcount
* = No recovery

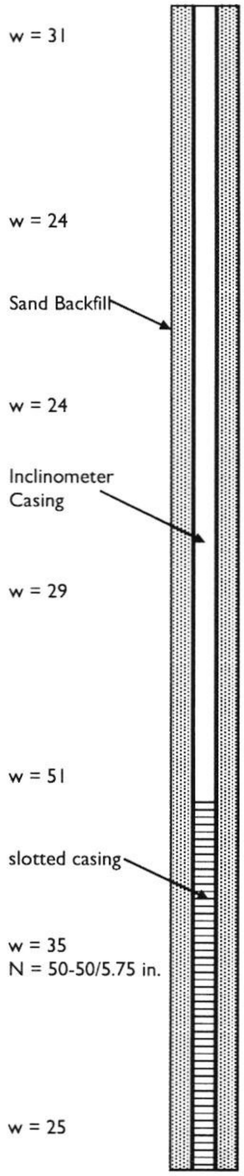


Soil and Rock Description

Very soft to soft (RH0 - RH1), little weathered, little fractured, gray SILTSTONE.

Samples and Data

75	w = 31
62	w = 24
65	w = 24
67	w = 29
52	w = 51
50+	w = 35 N = 50-50/5.75 in.
95	w = 25



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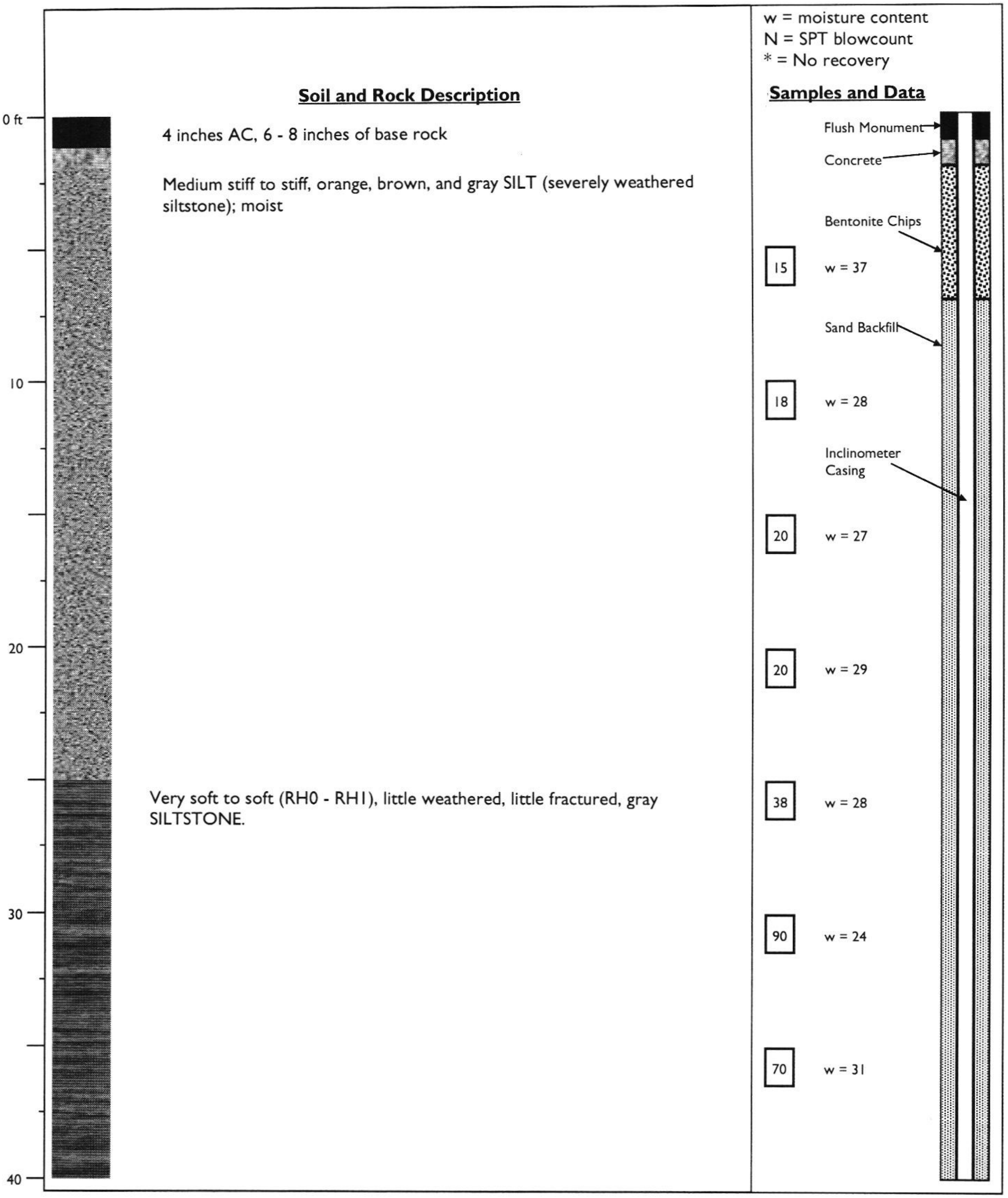
BORING B-3 cont.
Cannon-02-01-gi



w = moisture content
N = SPT blowcount
* = No recovery

Soil and Rock Description

4 inches AC, 6 - 8 inches of base rock

Medium stiff to stiff, orange, brown, and gray SILT (severely weathered siltstone); moist

Very soft to soft (RH0 - RH1), little weathered, little fractured, gray SILTSTONE.

Samples and Data

15	w = 37
18	w = 28
20	w = 27
20	w = 29
38	w = 28
90	w = 24
70	w = 31

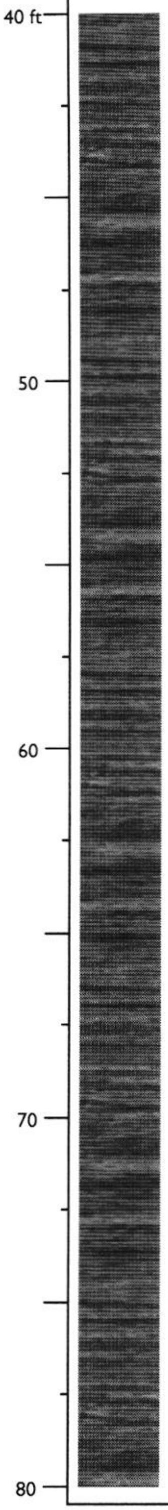
Geotech Solutions Inc.

BORING B-4

Cannon-02-01-gi

w = moisture content
N = SPT blowcount
* = No recovery

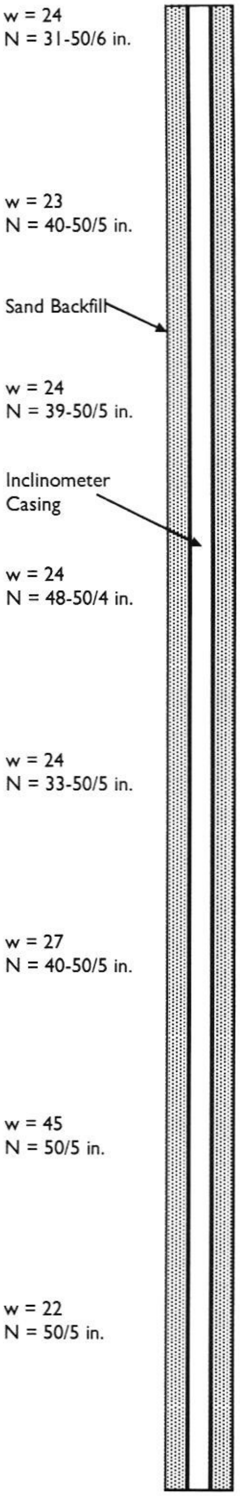
Soil and Rock Description



Very soft to soft (RH0 - RH1), little weathered, little fractured, gray SILTSTONE.

Samples and Data

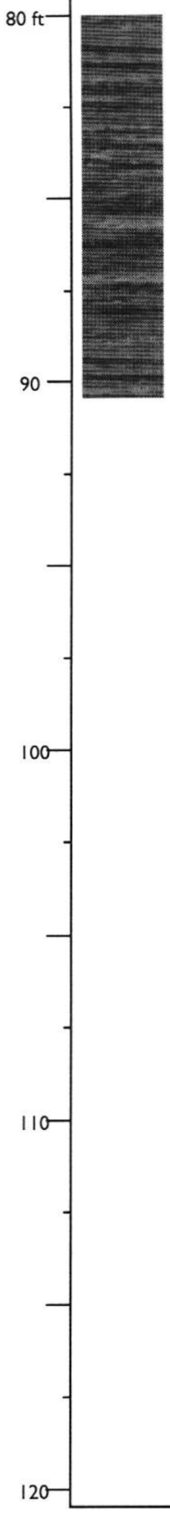
50+	w = 24	N = 31-50/6 in.
50+	w = 23	N = 40-50/5 in.
50+	w = 24	N = 39-50/5 in.
50+	w = 24	N = 48-50/4 in.
50+	w = 24	N = 33-50/5 in.
50+	w = 27	N = 40-50/5 in.
50+	w = 45	N = 50/5 in.
50+	w = 22	N = 50/5 in.



Geotech Solutions Inc.

BORING B-4 cont.
Cannon-02-01-gi

w = moisture content
N = SPT blowcount
* = No recovery

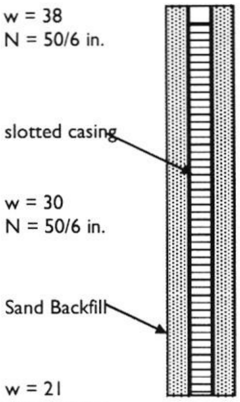


Soil and Rock Description

Very soft to soft (RH0 - RH1), little weathered, little fractured, gray SILTSTONE.

Samples and Data

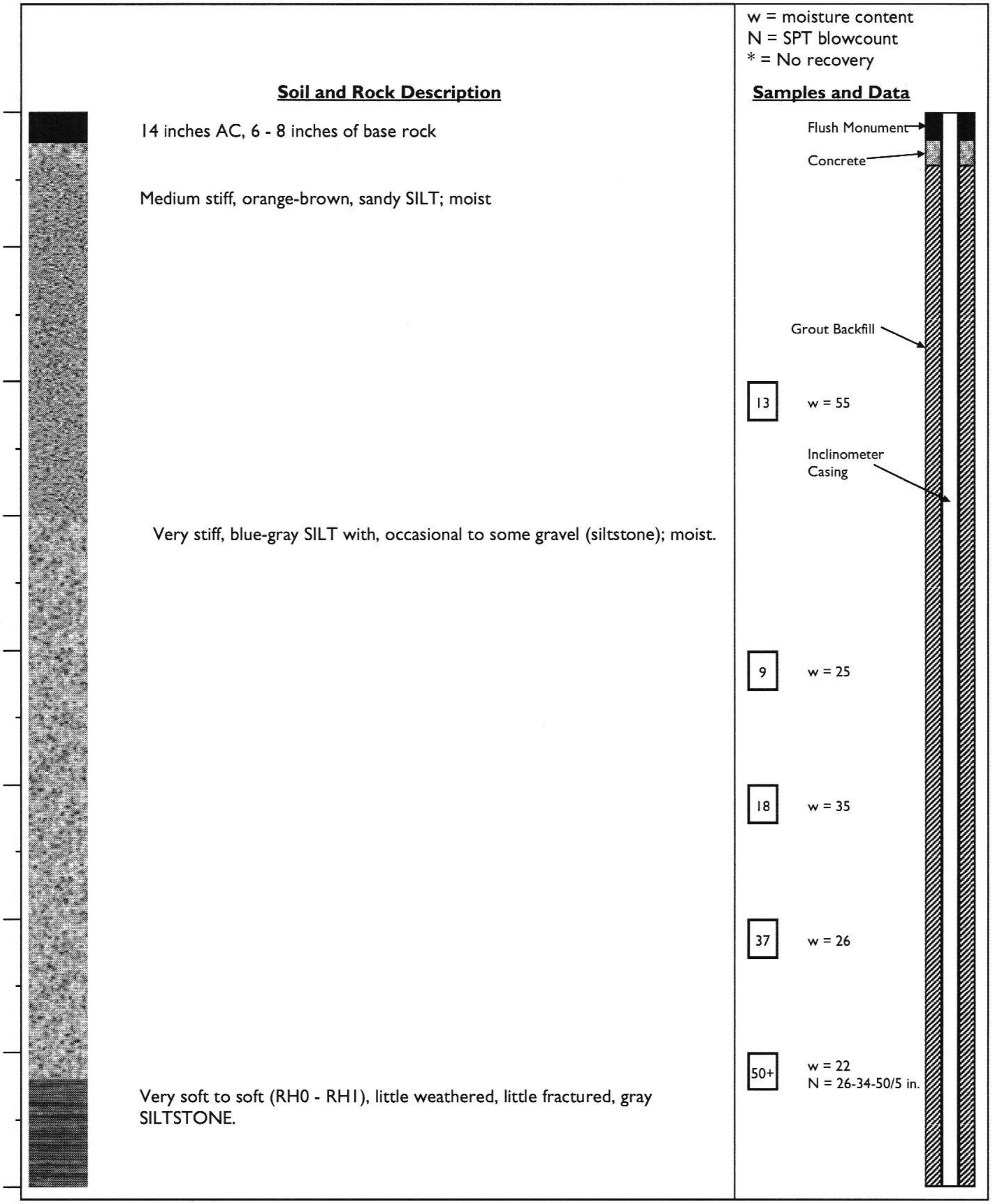
▭ 50+	w = 38 N = 50/6 in.
▭ 50+	w = 30 N = 50/6 in.
▭ 50+	w = 21 N = 50/5.5 in.



Geotech Solutions Inc.

BORING B-4 cont.

Cannon-02-01-gi



w = moisture content
N = SPT blowcount
* = No recovery

Soil and Rock Description

14 inches AC, 6 - 8 inches of base rock

Medium stiff, orange-brown, sandy SILT; moist

Very stiff, blue-gray SILT with, occasional to some gravel (siltstone); moist.

Very soft to soft (RH0 - RH1), little weathered, little fractured, gray SILTSTONE.

Samples and Data

13 w = 55

9 w = 25

18 w = 35

37 w = 26

50+ w = 22
N = 26-34-50/5 in.

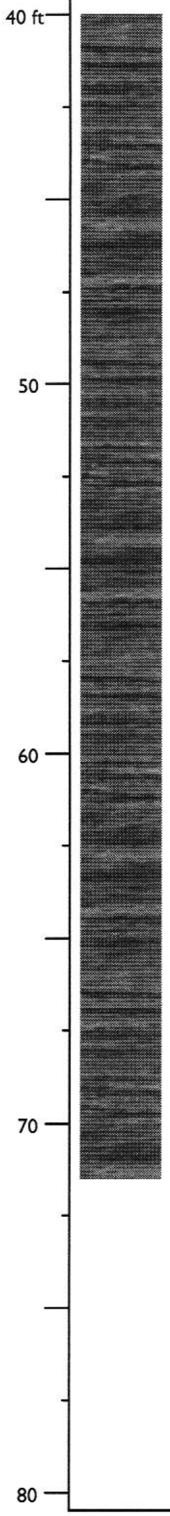
Geotech Solutions Inc.

BORING B-5

Cannon-02-01-gi

w = moisture content
N = SPT blowcount
* = No recovery

Soil and Rock Description



Very soft to soft (RH0 - RH1), little weathered, little fractured, gray SILTSTONE.

Samples and Data

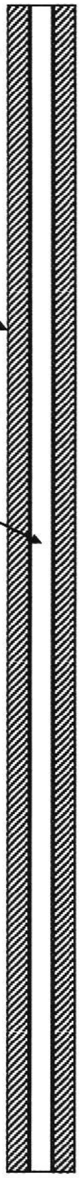
50+ w = 23
N = 17-40-50/6 in.

50 w = 25

50+ w = 22
N = 28-35-50/4 in.

50+ w = 21
N = 28-50/5 in.

50+ w = 23
N = 35-50/5 in.



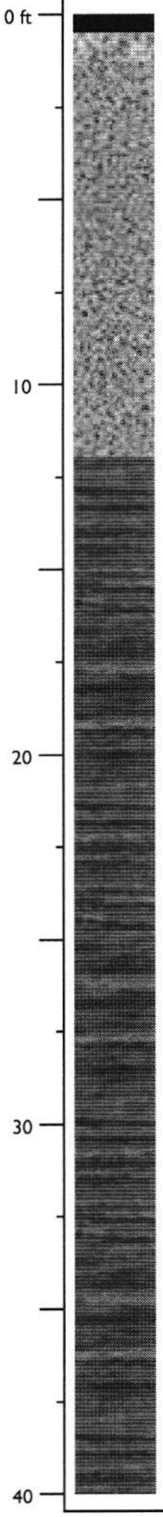
Geotech Solutions Inc.

BORING B-5 cont.

Cannon-02-01-gi

w = moisture content
N = SPT blowcount
* = No recovery

Soil and Rock Description



2 inches AC, 6 - 8 inches of base rock

Soft to medium Stiff, orange mottled gray-brown SILT with trace clay; moist.

Very soft to soft (RH0 - RH1), little weathered, little fractured, gray SILTSTONE.

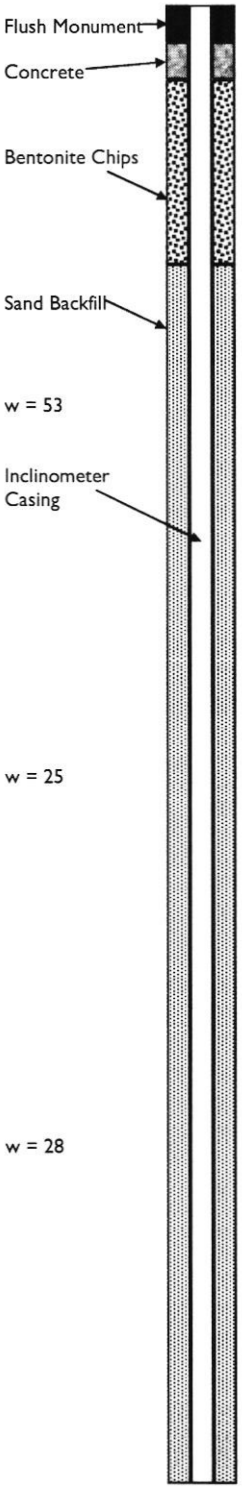
Lose drilling mud and circulation of drilling mud between 20 and 22 feet.

Samples and Data

4 w = 53

61 w = 25

25 w = 28



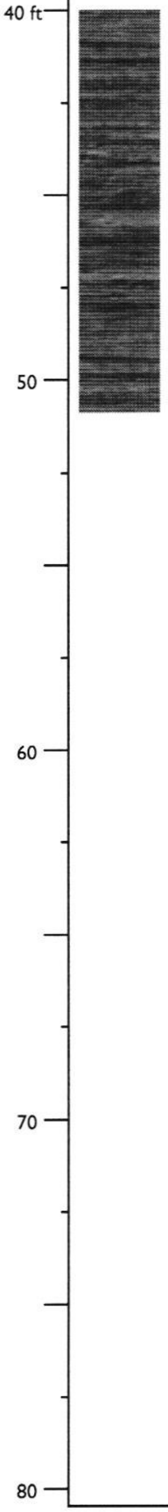
Geotech Solutions Inc.

BORING B-6

Cannon-02-01-gi

w = moisture content
N = SPT blowcount
* = No recovery

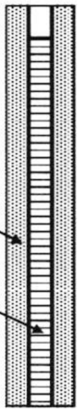
Soil and Rock Description



Very soft to soft (RH0 - RH1), little weathered, little fractured, gray SILTSTONE.

Samples and Data

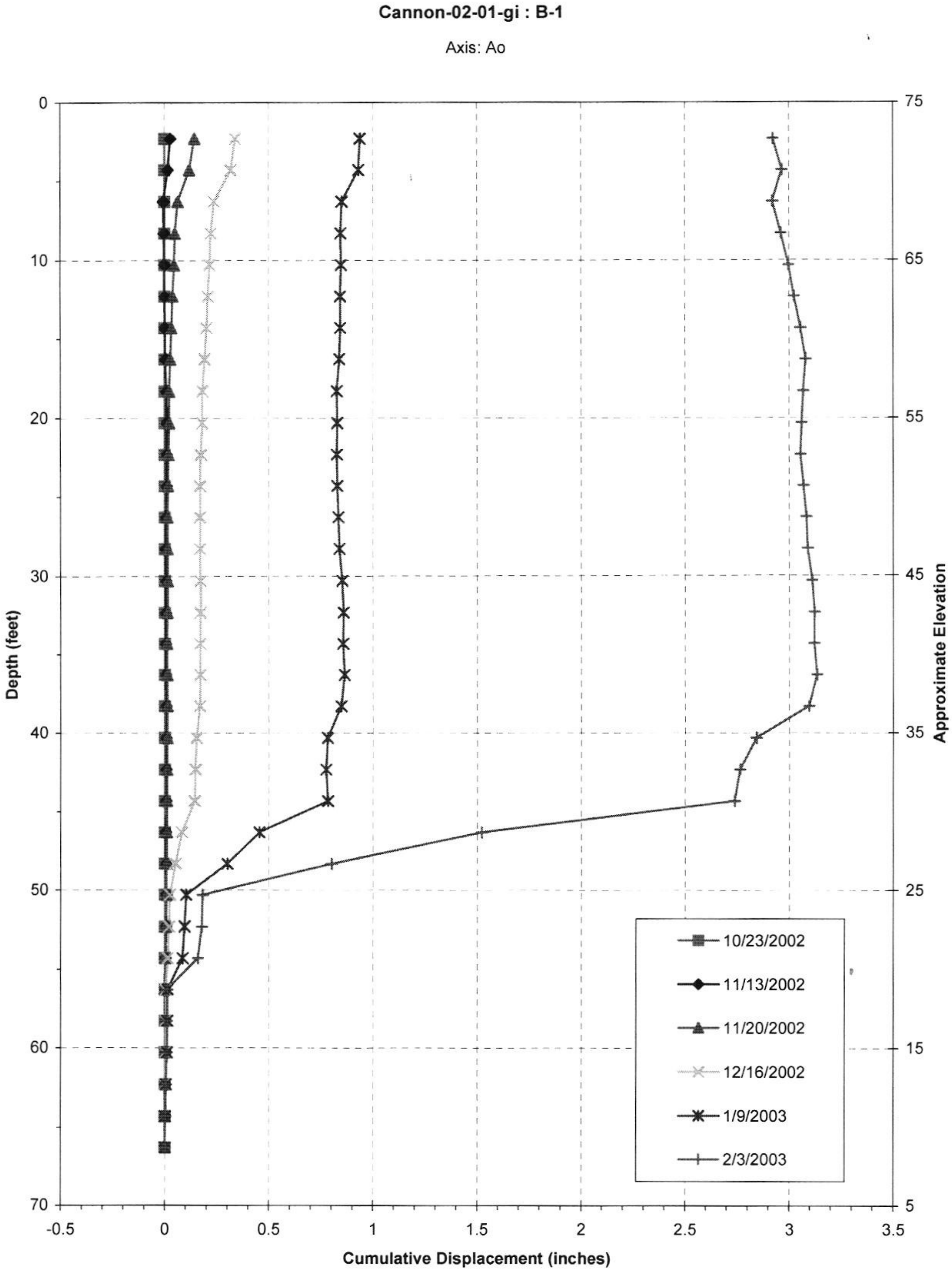
50+
w = 23
N = 19-40-50/4 in.

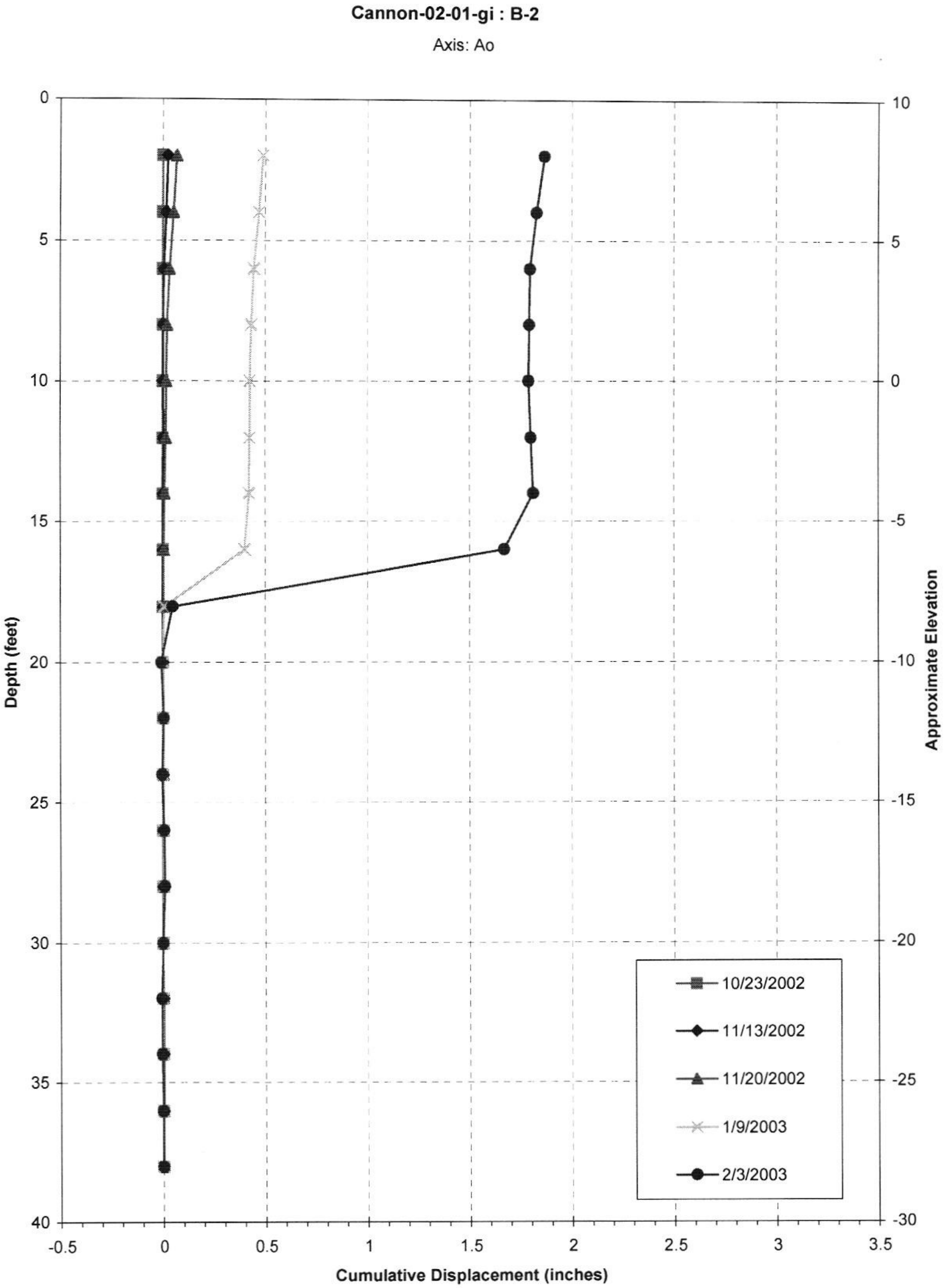


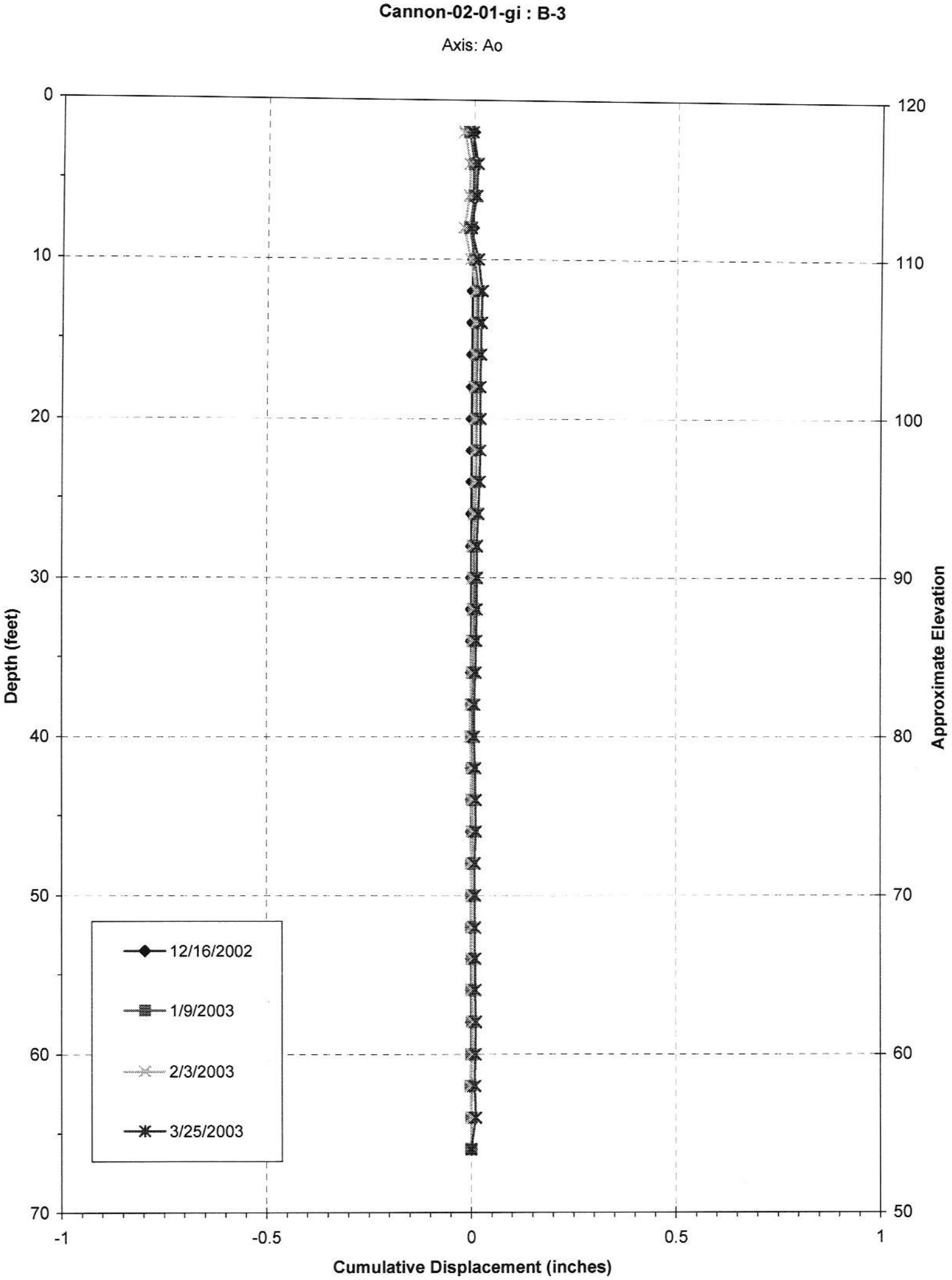
50+
w = 22
N = 47-50/4.5 in.

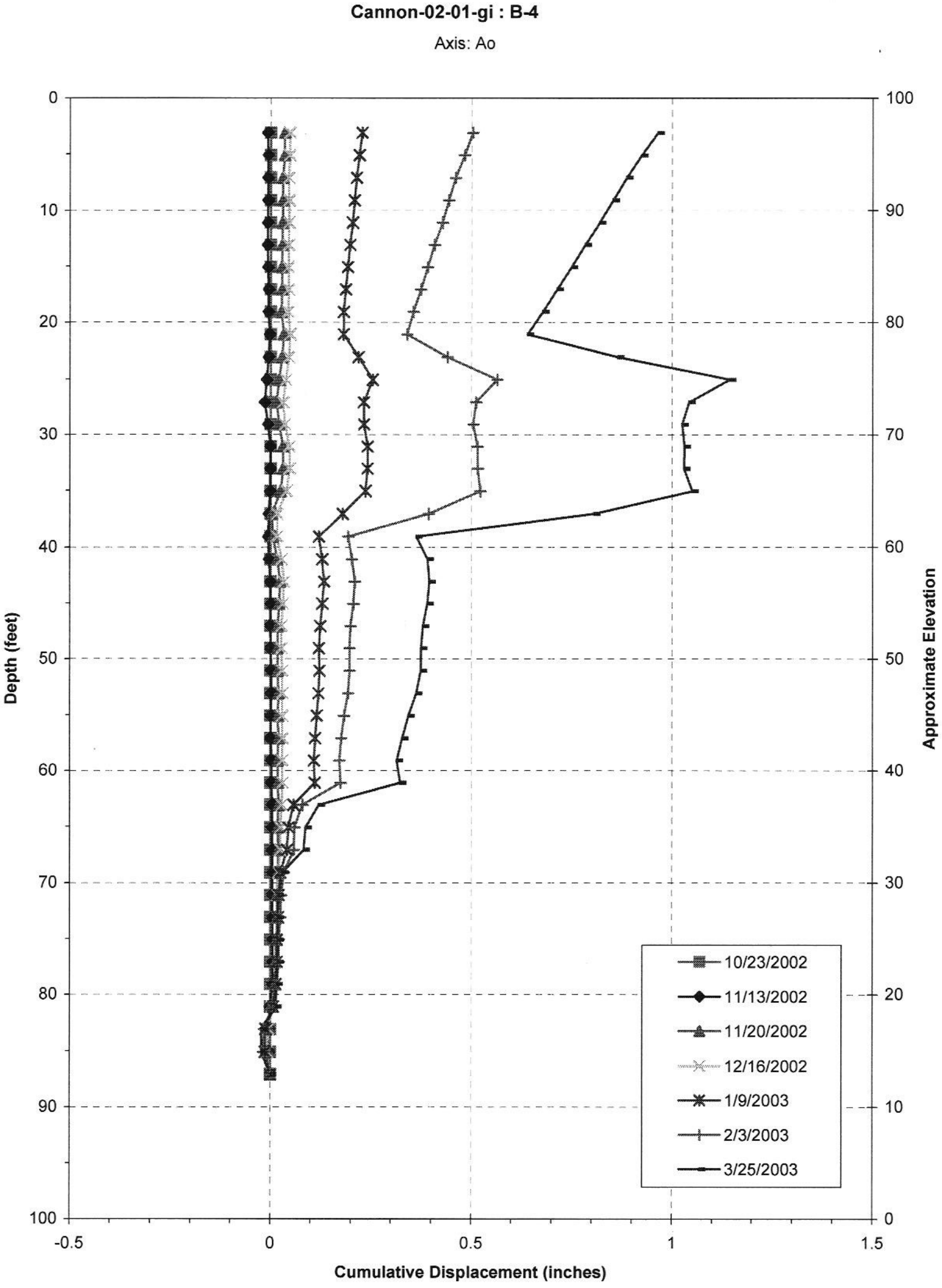
Geotech Solutions Inc.

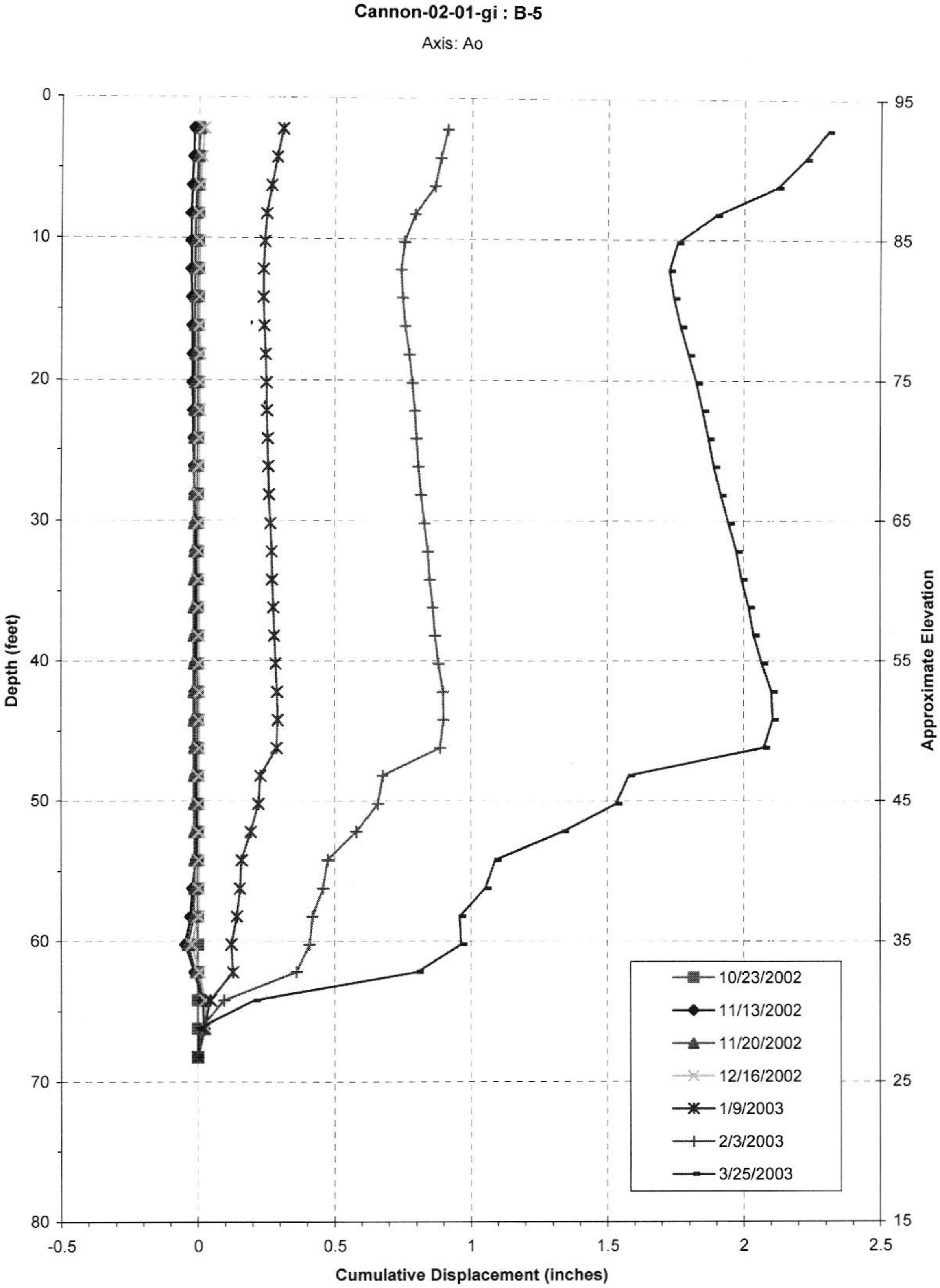
BORING B-6 cont.
Cannon-02-01-gi

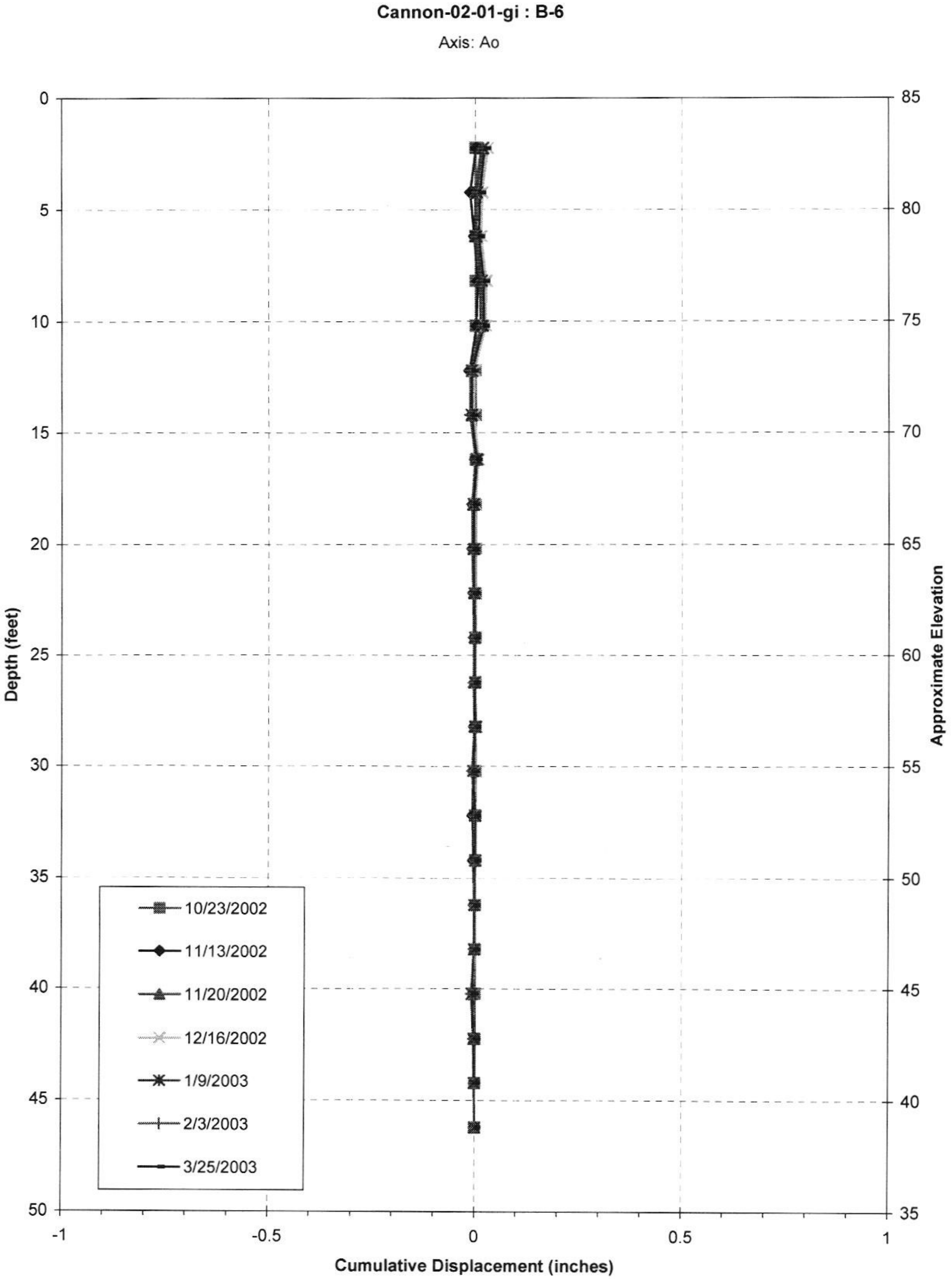

Cannon-02-01-gi : B-1
Axis: Ao
Depth (feet)
Approximate Elevation
Cumulative Displacement (inches)
0
10
20
30
40
50
60
70
75
65
55
45
35
25
15
5
-0.5
0
0.5
1
1.5
2
2.5
3
3.5
10/23/2002
11/13/2002
11/20/2002
12/16/2002
1/9/2003
2/3/2003


Cannon-02-01-gi : B-2
Axis: Ao
Depth (feet)
Approximate Elevation
Cumulative Displacement (inches)
0
5
10
15
20
25
30
35
40
10
5
0
-5
-10
-15
-20
-25
-30
-0.5
0
0.5
1
1.5
2
2.5
3
3.5
10/23/2002
11/13/2002
11/20/2002
1/9/2003
2/3/2003


Cannon-02-01-gi : B-3
Axis: Ao
Depth (feet)
0
10
20
30
40
50
60
70
Approximate Elevation
120
110
100
90
80
70
60
50
-1
-0.5
0
0.5
1
Cumulative Displacement (inches)
12/16/2002
1/9/2003
2/3/2003
3/25/2003

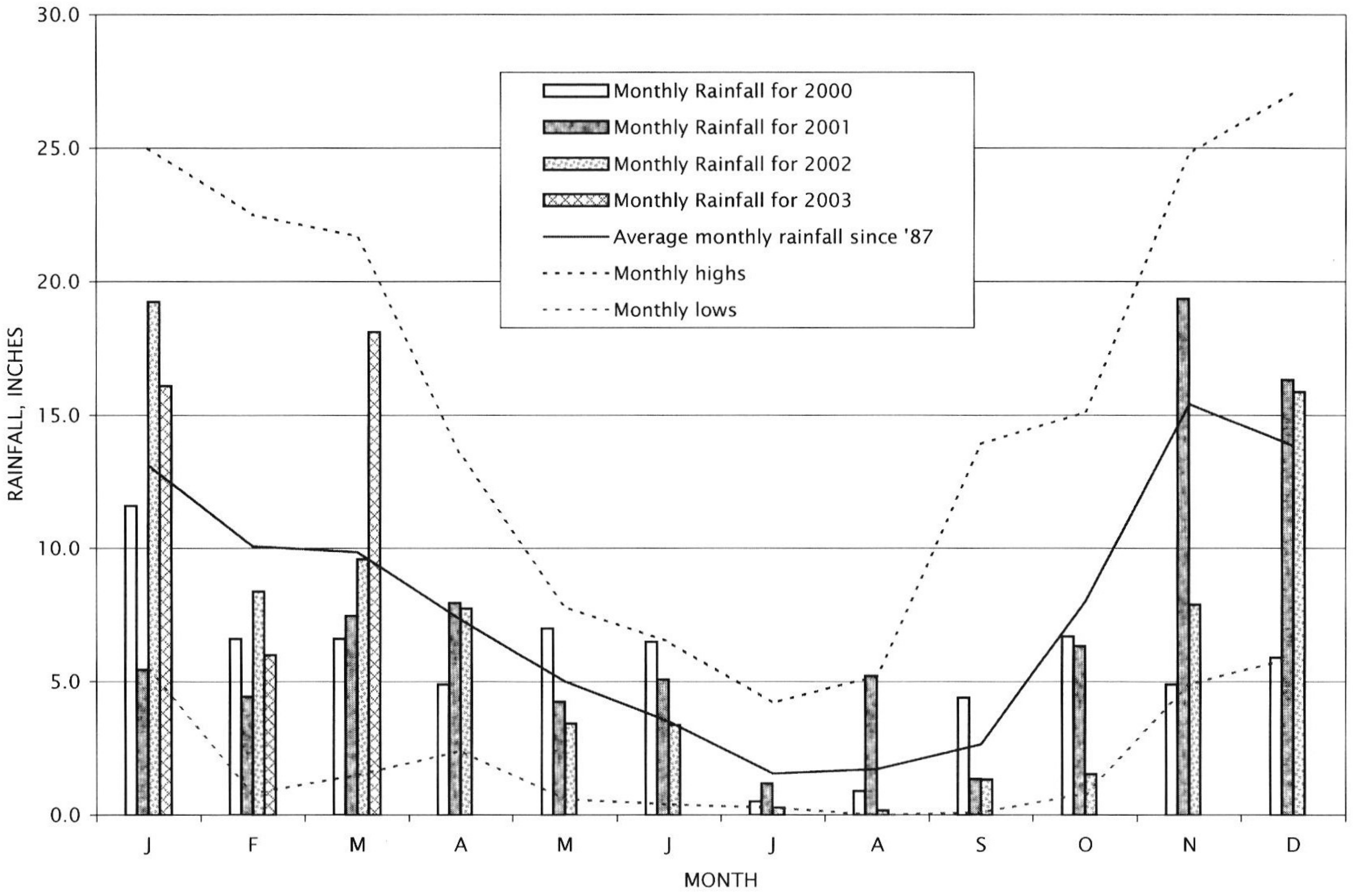

Cannon-02-01-gi : B-4
Axis: Ao
Depth (feet)
0
10
20
30
40
50
60
70
80
90
100
Approximate Elevation
100
90
80
70
60
50
40
30
20
10
0
-0.5
0
0.5
1
1.5
Cumulative Displacement (inches)
10/23/2002
11/13/2002
11/20/2002
12/16/2002
1/9/2003
2/3/2003
3/25/2003

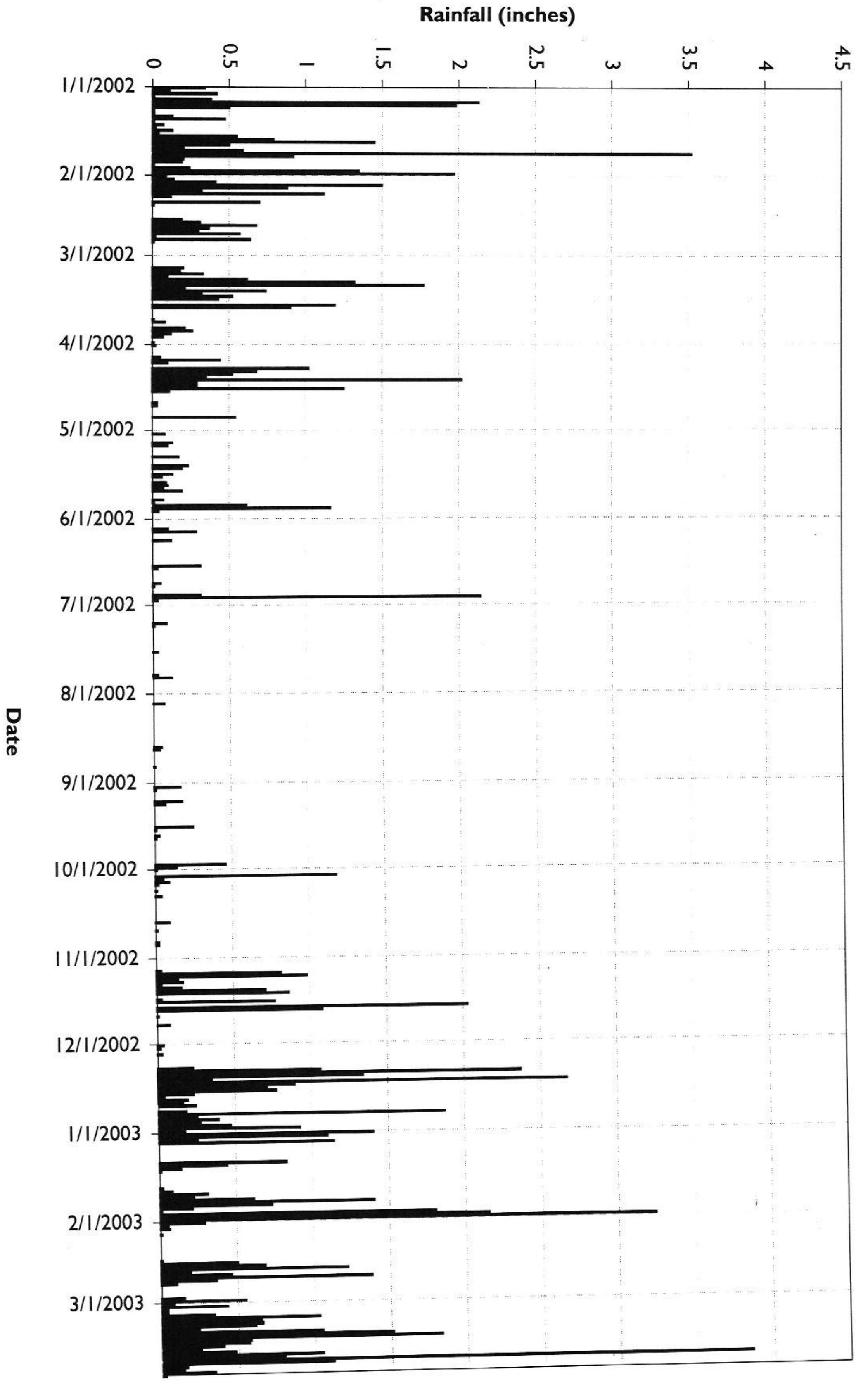

Cannon-02-01-gi : B-5
Axis: Ao
Depth (feet)
0
10
20
30
40
50
60
70
80
Approximate Elevation
95
85
75
65
55
45
35
25
15
-0.5
0
0.5
1
1.5
2
2.5
Cumulative Displacement (inches)
10/23/2002
11/13/2002
11/20/2002
12/16/2002
1/9/2003
2/3/2003
3/25/2003


Cannon-02-01-gi : B-6
Axis: Ao
Depth (feet)
Approximate Elevation
Cumulative Displacement (inches)
0
5
10
15
20
25
30
35
40
45
50
85
80
75
70
65
60
55
50
45
40
35
-1
-0.5
0
0.5
1
10/23/2002
11/13/2002
11/20/2002
12/16/2002
1/9/2003
2/3/2003
3/25/2003

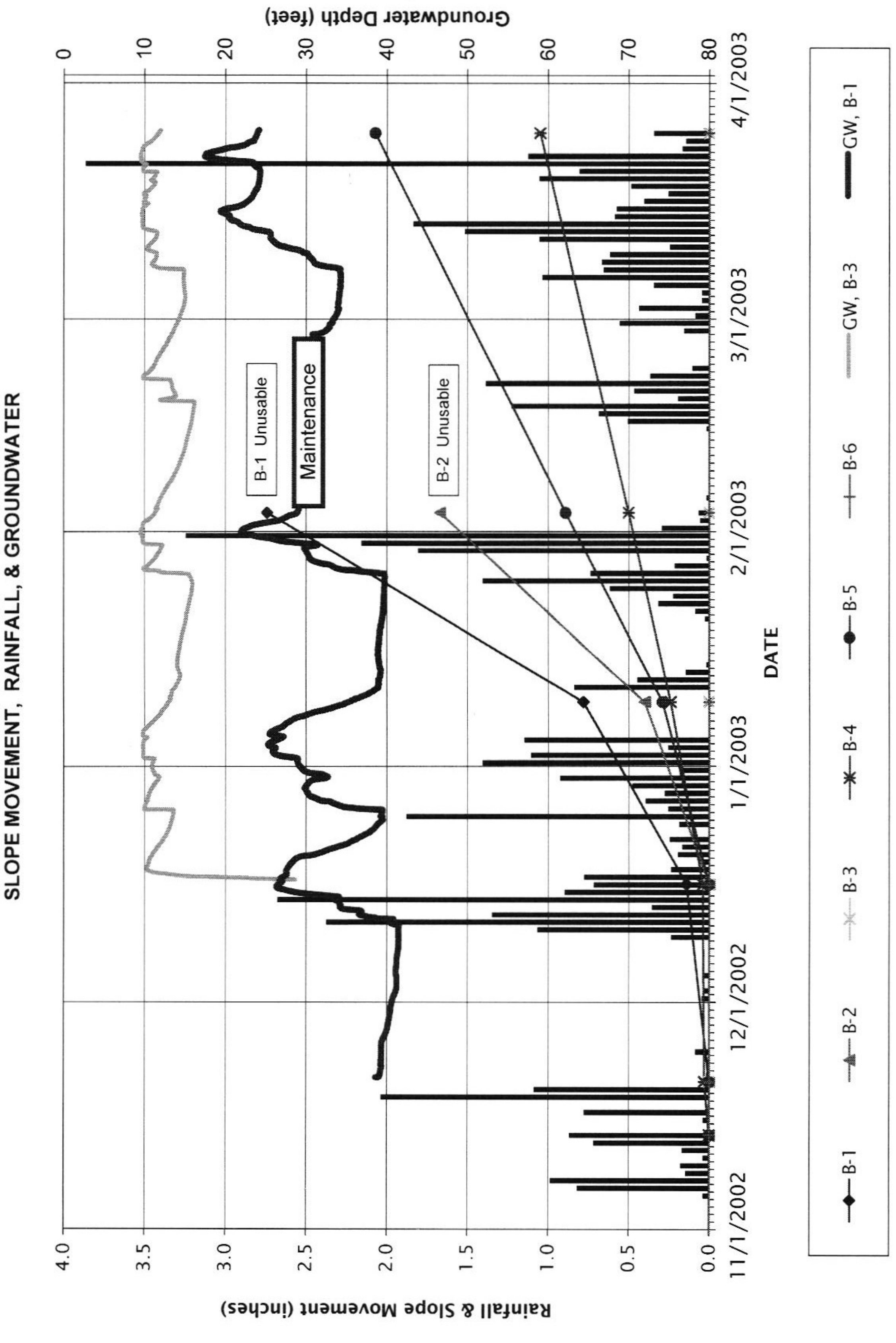
B	Point	10/24/2002			11/14/2002			2/7/2003			3/26/2003		
		Readings			Movement (inches)			Movement (inches)			Movement (inches)		
		North	East	Elevation	North	East	Elevation	North	East	Elevation	North	East	Elevation
	1	9337.1	10027.0	51.2	0.161	-0.228	0.181	-	-	-	0.300	-0.095	-0.081
	2	9355.0	9731.3	15.8	-0.255	0.226	0.616	-1.574	-2.369	0.641	-3.290	-5.579	0.843
2	3	9475.4	9699.2	12.1	-0.311	0.136	0.591	-0.366	-1.344	1.170	-0.905	-4.737	1.702
	4	9727.0	9740.8	45.0	-0.186	0.232	0.249	-0.117	0.146	0.365	0.122	-0.216	0.070
	5	9824.9	9840.7	66.7	0.202	-0.377	-0.027	0.094	0.023	0.079	-0.076	-0.377	-0.148
	6	9665.1	9792.1	61.0	0.025	-0.125	-0.020	-0.613	-2.243	-0.599	-0.914	-4.709	-1.727
1	7	9580.5	9844.3	75.8	-0.276	0.068	0.053	-1.073	-1.812	-0.706	-1.742	-4.404	-2.006
	8	9509.2	9844.6	60.0	-0.244	0.080	0.138	-0.917	-2.031	0.016	-2.248	-5.161	-0.910
6	9	9542.4	10078.3	83.8	-0.097	0.425	11.695	0.008	0.227	11.684	0.153	0.363	11.441
5	10	9517.3	9916.7	95.1	-	-	-	-0.433	-0.632	-0.730	-0.570	-2.470	-2.385
4	11	9653.7	9938.0	100.2	-0.267	0.144	0.437	-0.246	-0.026	-0.083	0.142	-0.820	-0.698
	12	9499.9	9988.3	91.9	-0.218	0.481	0.416	-0.246	0.630	0.350	0.177	0.135	0.049
	13	9502.3	9975.5	93.0	-0.179	0.513	0.435	-0.178	0.559	0.245	0.132	0.061	0.023
	14	9475.8	9960.0	95.4	-0.287	0.006	0.230	-0.258	0.522	0.197	-0.054	0.291	-0.102
	15	9581.7	9933.4	96.3	-	-	-	-0.045	-0.773	-0.493	0.035	-2.356	-1.602
	16	9704.0	9926.4	101.3	0.167	0.160	0.443	-0.176	0.084	0.096	0.062	-0.578	-0.278
	17	9733.2	9967.0	103.1	-0.130	0.315	0.281	-0.095	0.014	0.183	0.190	-0.318	-0.040
	18	9782.2	9947.3	104.0	0.295	0.146	0.160	0.105	0.072	0.136	0.103	-0.126	-0.166
	19	9967.7	10022.6	109.5	0.289	-0.302	0.290	0.100	-0.151	0.171	0.249	-0.058	0.122
	20	9858.1	10004.3	107.1	-0.033	-0.222	0.082	0.189	0.028	0.073	0.297	0.128	-0.122
	21	9852.1	10096.5	125.6	0.158	-0.410	0.269	0.248	-0.033	0.395	0.346	0.201	0.328
	22	9849.9	10228.3	144.4	0.442	-0.147	0.231	0.306	0.364	0.096	0.583	0.464	0.178
3	23	9741.7	10059.9	122.2	5.075	6.332	-1.944	-2.201	1.969	-1.205	-2.018	1.827	-1.195
	24	9526.5	9978.8	113.3	-0.172	-0.150	0.334	-0.061	-0.199	-0.197	-0.005	-1.211	-0.963

ANNUAL RAINFALL FOR CANNON BEACH, OREGON

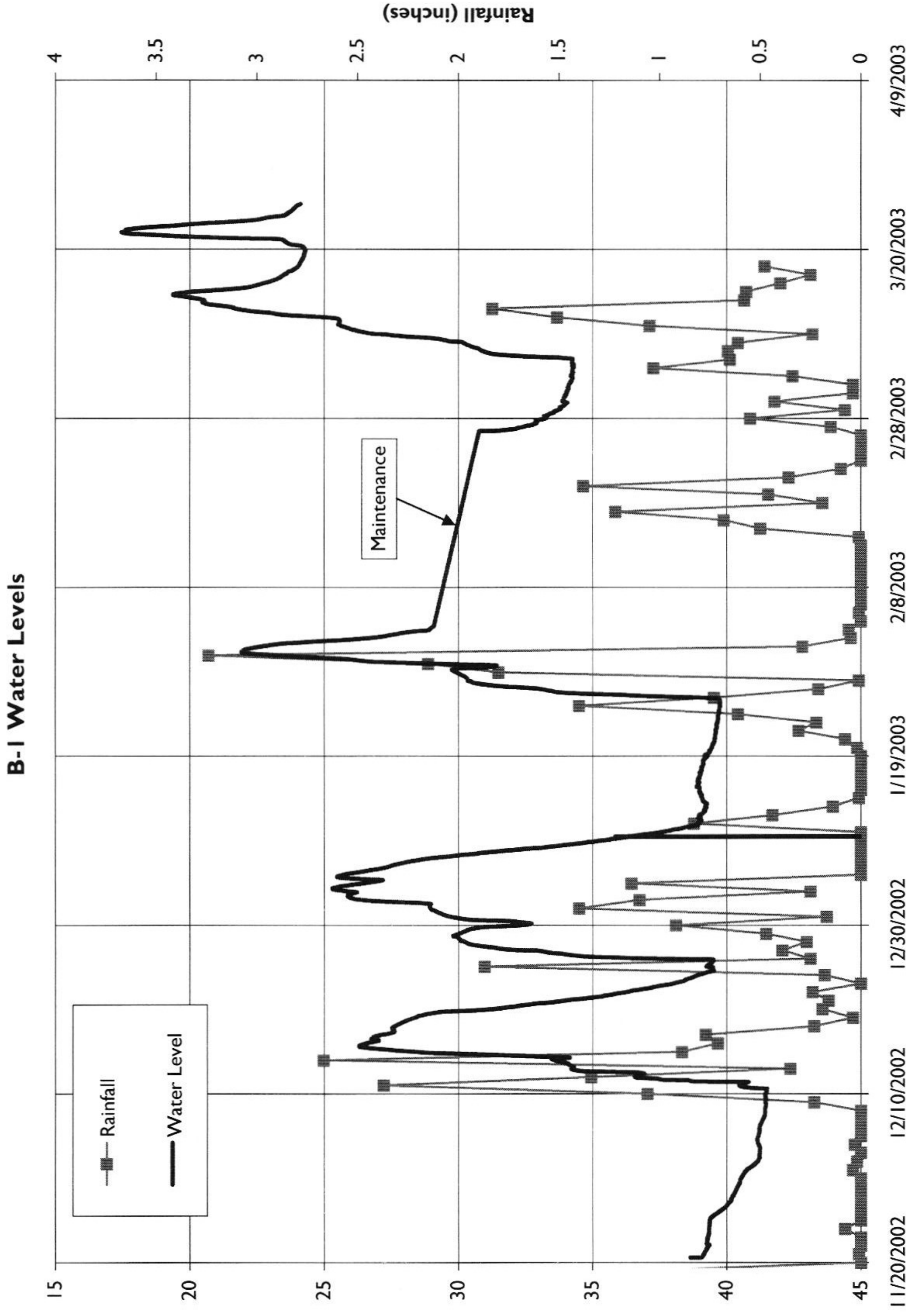


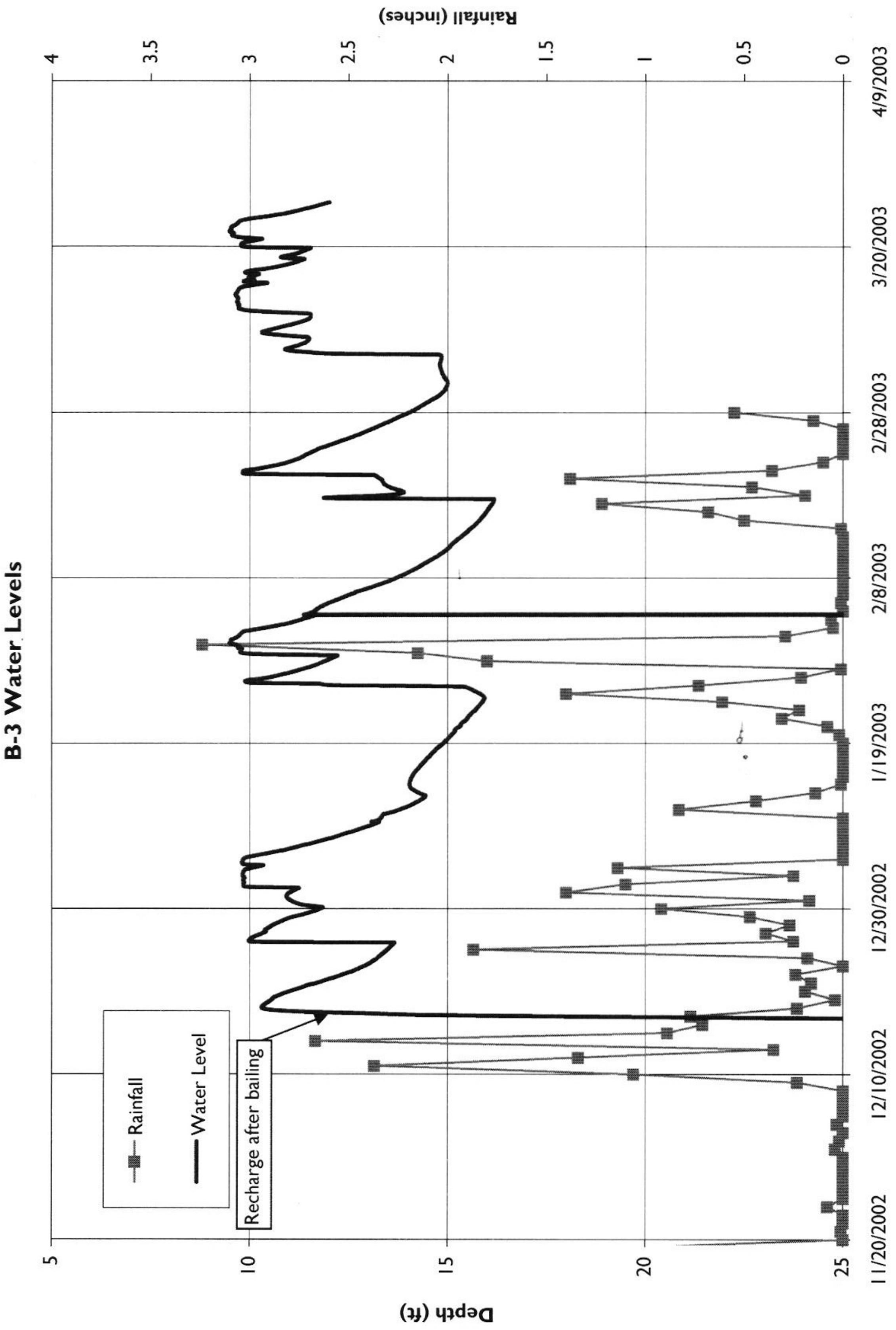


Cannon Beach: Daily Rainfall



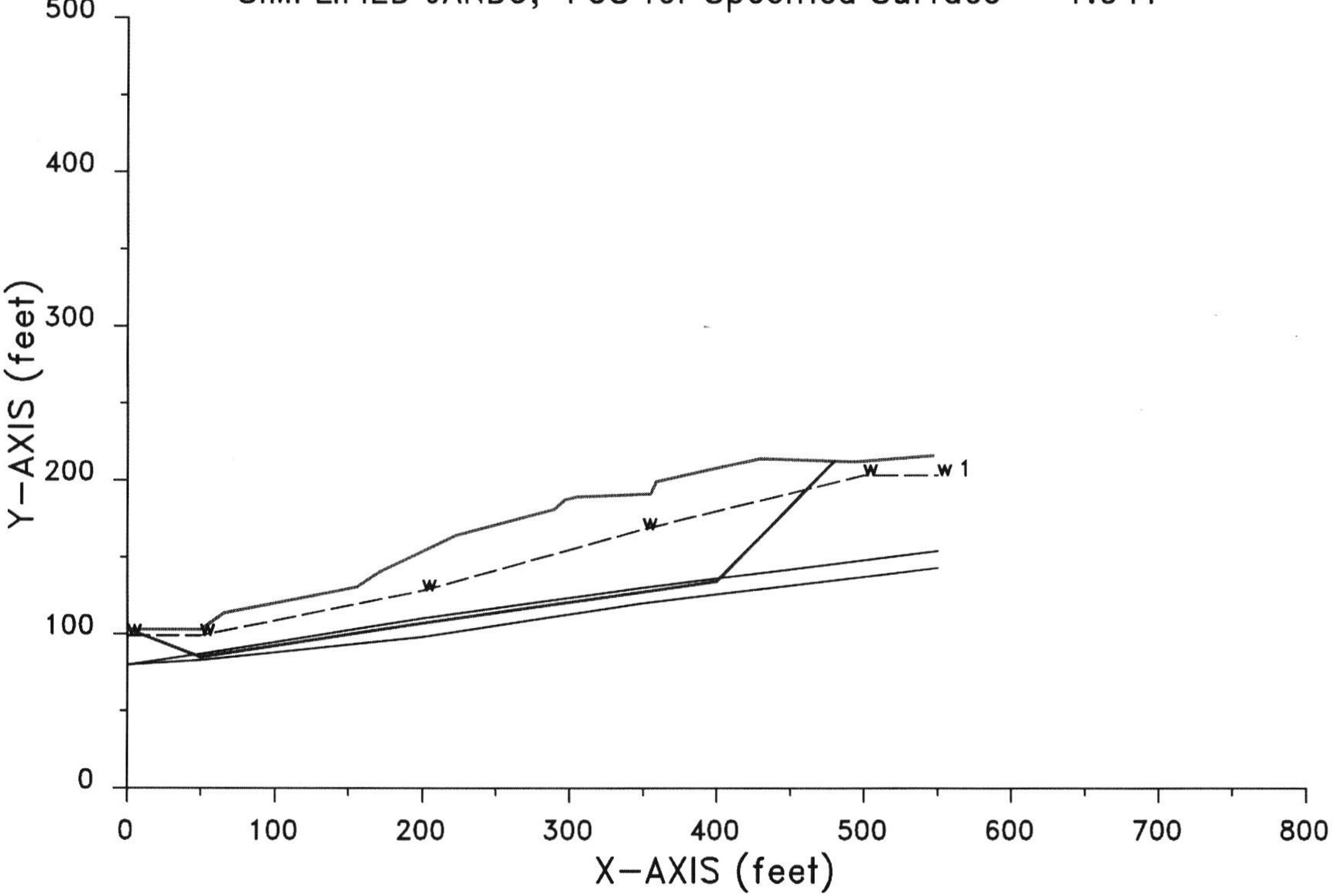
SLOPE MOVEMENT, RAINFALL, & GROUNDWATER


B-1 Water Levels
Rainfall
Water Level
Maintenance
Depth (ft)
15
20
25
30
35
40
45
Rainfall (inches)
4
3.5
3
2.5
2
1.5
1
0.5
0
11/20/2002
12/10/2002
12/30/2002
1/19/2003
2/8/2003
2/28/2003
3/20/2003
4/9/2003


B-3 Water Levels
Depth (ft)
Rainfall (inches)
5
10
15
20
25
4
3.5
3
2.5
2
1.5
1
0.5
0
11/20/2002
12/10/2002
12/30/2002
1/19/2003
2/8/2003
2/28/2003
3/20/2003
4/9/2003
Rainfall
Water Level
Recharge after bailing

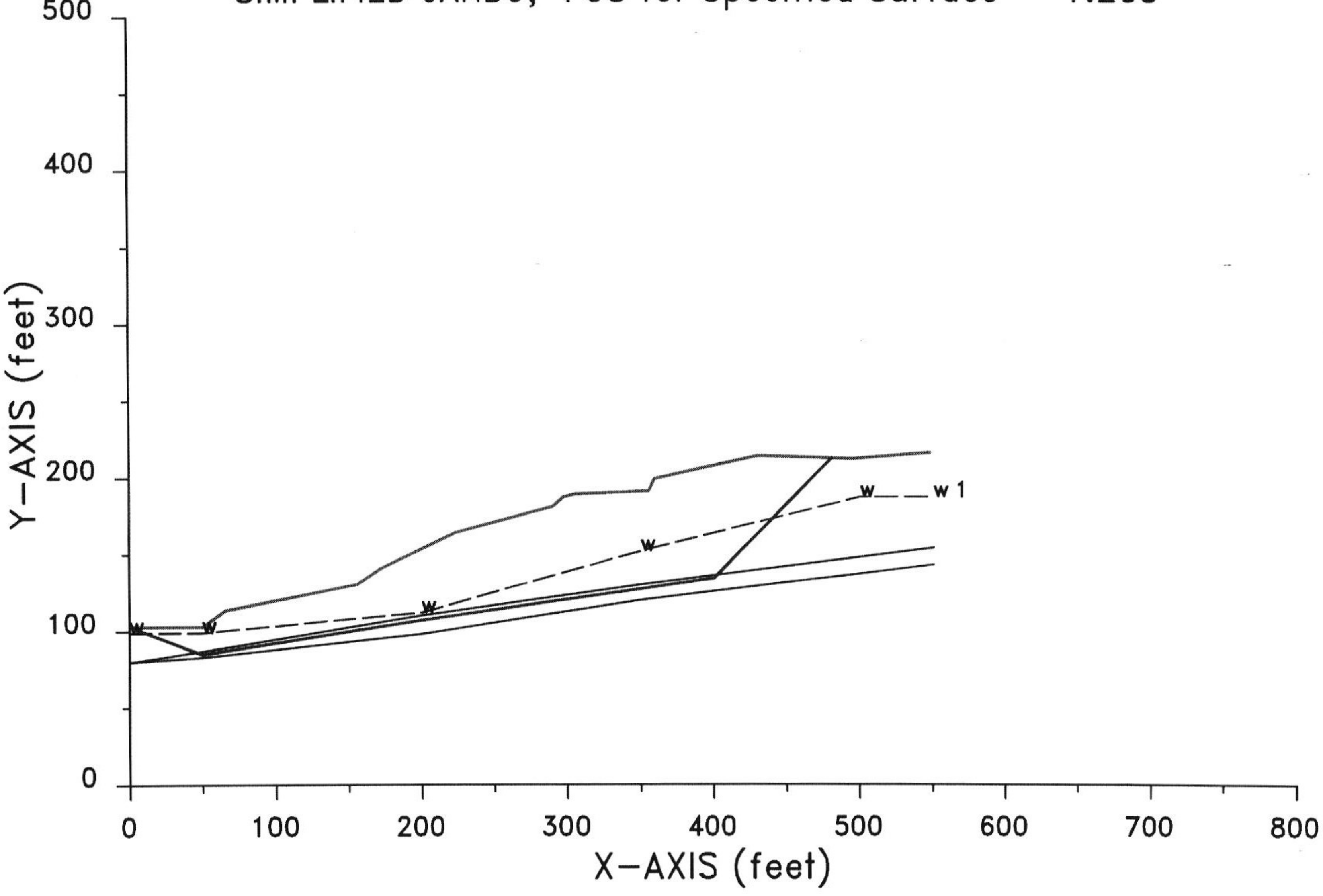
CB1 3-18-03 10:28

S-curves: Profile 1
SIMPLIFIED JANBU, FOS for Specified Surface = 1.041



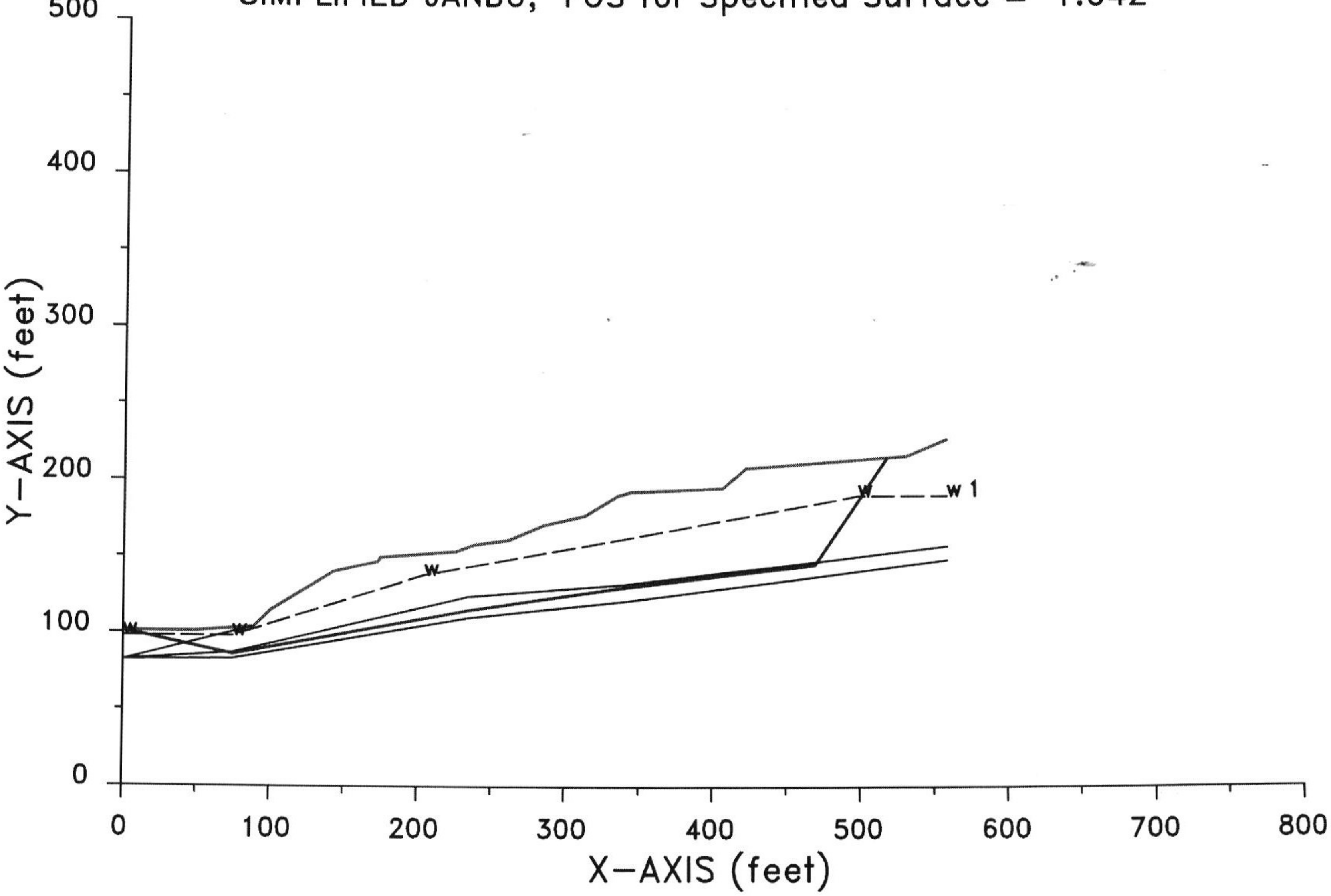
CB1LOW 3-18-03 10:29

S-curves: Profile 1, low water
SIMPLIFIED JANBU, FOS for Specified Surface = 1.203



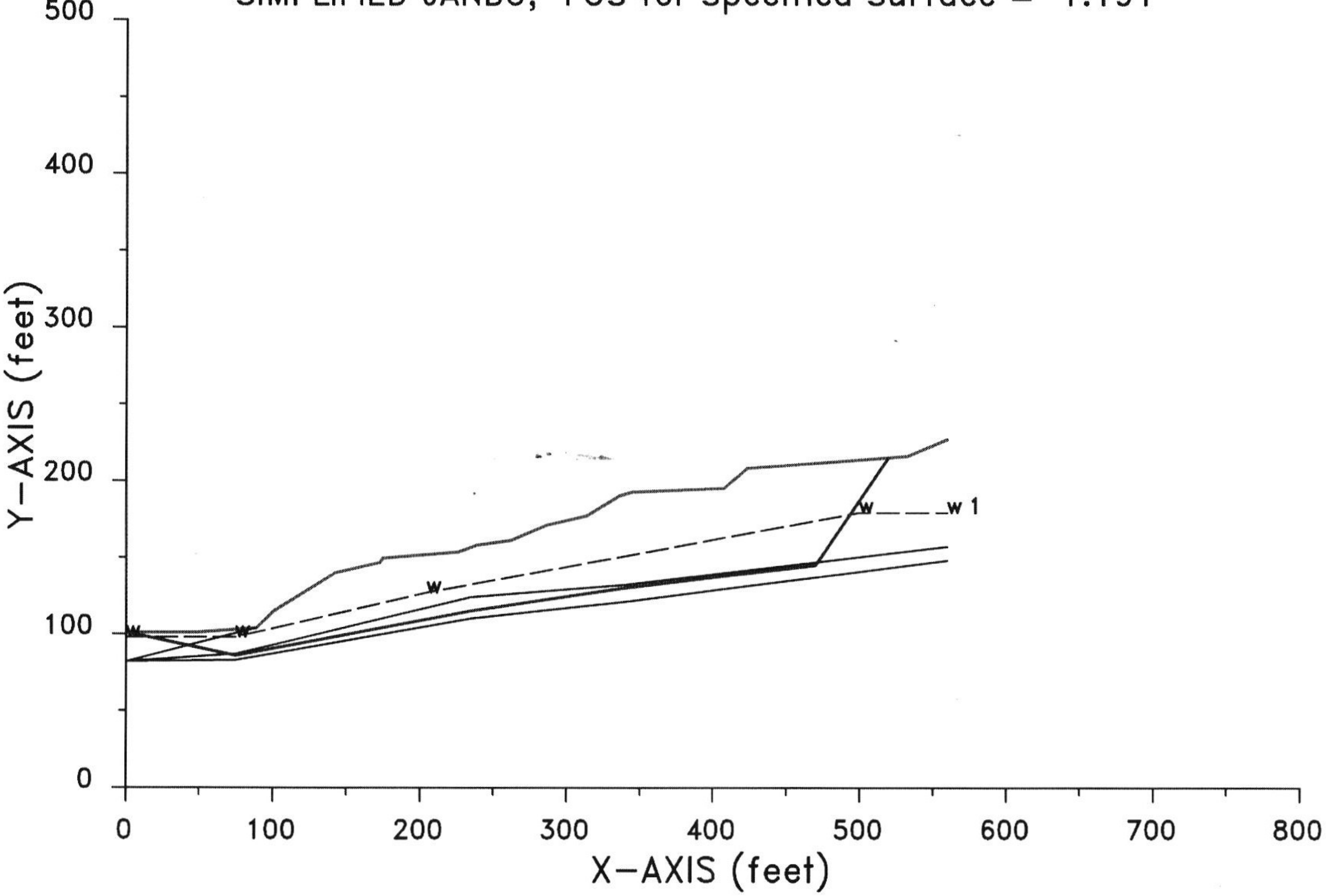
CB2 3-18-03 10:39

S-Curves: Profile 2
SIMPLIFIED JANBU, FOS for Specified Surface = 1.042



CB2LOW 3-18-03 10:39

S-Curves: Profile 2, low water
SIMPLIFIED JANBU, FOS for Specified Surface = 1.191



Geotech
Solutions Inc.

MEMORANDUM *cannon-18-1-consult*

To: Karen LaBonte, Public Works Director, City of Cannon Beach; labonte@ci.cannon-beach.or.us

Date: June 26, 2018

Subject: Hemlock Street S-Curves Slide: Status Update

Introduction and Background

This memorandum provides an update to the status of the inclinometer data from the S-curves slide as read on June 23, 2018. The previous last reading was in 2015. The reason for this reading was a centerline crack appearing in the last month or so near the apex of the curve above and slightly south of the B-1r instrument. This crack is roughly 10-15 feet in length, and open up to roughly ¼" with perhaps a slight vertical offset down to the west. In addition, and perhaps relevant to tangential slide restraint and equilibrium, slope cuts and net mass removal has occurred on an adjacent project over roughly the past year. That project abuts previous lateral shear zones observed at the southern portion of the active slide.

The water levels in the slide are no longer being recorded as the instruments have expired, and new winter storm rainfall levels had not exceeded those previously recorded. The data attached are inclinometer readings for only one instrument near the center of the slide (B-1r) which has been shown over many years to correlate well with rainfall response and water levels and other previous movement in other, now irrevocably damaged, casings. It should be understood that this correlation is in the context of the general beach/slide toe elevations and erosion conditions experienced since 2008.

Conclusions and Recommendations

Roughly 0.2 inches of movement has occurred above/near the primary shear surface since the last reading roughly 3 years ago. The previous 3 years had roughly 0.1 inches of movement. Overall readings show a total of roughly 2.5 inches of movement on this replacement casing. A plot is attached. This movement is not out of recorded context movement rates for the slide.

Based on our site observations, in our opinion the surface cracking is not discernible from an aging panel joint or related thermal separation crack. It is possible that the crack was caused by accumulated underlying movement of the slide and is exhibiting at the previously placed grid overlap joint, but it does not coincide with previous slide induced crack locations which trended southwesterly with vertical offsets greater than horizontal, and at locations north and south of this crack location.

Although B-1r is approaching its deflection life, it is still functional and in our opinion does not need replacement at this time. Replacement/redrilling for a new casing (including initial baseline readings) is estimated at roughly $10,000 as access is difficult. If additional cracking occurs that is more indicative of slide movement, then a new water level logger is recommended for the paired B-1 standpipe (P-1).

Based on our current monitoring, we still expect movement of the S-Curves to be ongoing. However, the reduction in ground water levels and movement in large rainfall events has been greatly reduced by the functional horizontal drains compared to historical observations. No measures in addition to frequent roadway surface observation and annual drain cleaning are recommended at this time.

Provided the existing drains are maintained and cleaned annually and are functional, it is our opinion that they are sufficient to continue to slow the slide for the rainfall event intensities experienced since drain installation. Exceptions would be from earthquake ground motions or significant beach toe erosion. Any significant beach level erosion (such as exposure of siltstone below the sand similar to the El Nino cycle of 1999), or toe slumping, would be cause to take inclinometer readings, as would experiencing a new threshold rainfall event. These would be anything in excess of the storm events recorded since drain installation which are 4.37”-1 day, 6.26”-2day, 6.29”-3day, or 10.21”-5day. Please alert us if any of these thresholds are met.

The Limitations of our report apply, and that report and a few predrain install crack photos are attached here for background.

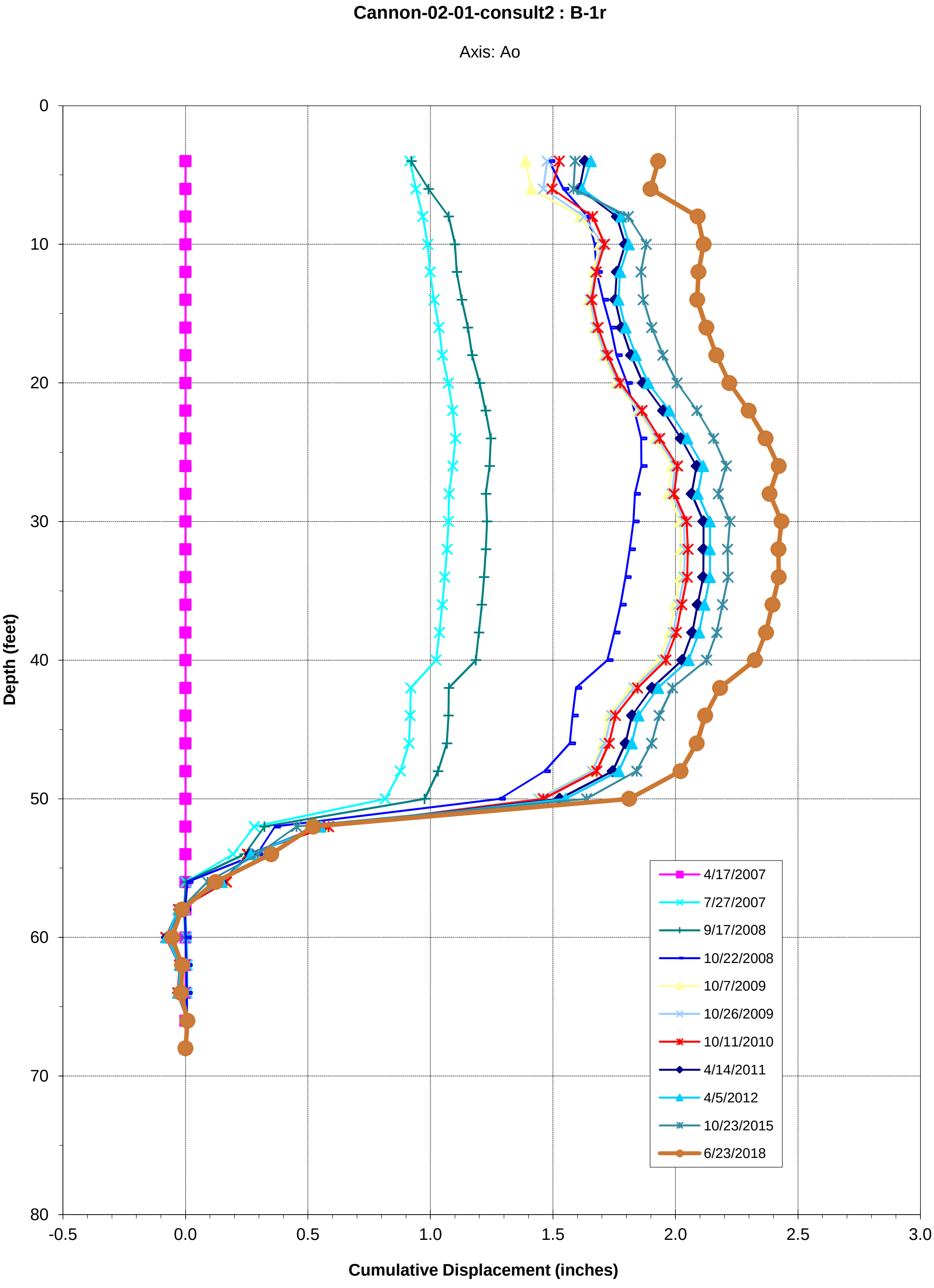
* * *

We appreciate the opportunity to work with you on this project and look forward to our continued involvement. Please call if you have questions.

Sincerely,

Don Rondema, MS, PE, GE
Principal

DON RONDEMA
Expires 12/31/18


Cannon-02-01-consult2 : B-1r
Axis: Ao
Depth (feet)
Cumulative Displacement (inches)
0
10
20
30
40
50
60
70
80
-0.5
0.0
0.5
1.0
1.5
2.0
2.5
3.0
4/17/2007
7/27/2007
9/17/2008
10/22/2008
10/7/2009
10/26/2009
10/11/2010
4/14/2011
4/5/2012
10/23/2015
6/23/2018

Geotech Solutions Inc.

August 15, 2018 *cannon-18-2-consult*

City of Cannon Beach
labonte@ci.cannon-beach.or.us; barrett@ci.cannon-beach.or.us

cc: kyle.ayers@otak.com

GEOTECHNICAL ENGINEERING CONSULTATION
S-Curves Water Line, Cannon Beach, Oregon

As authorized this letter summarizes our consultations regarding the proposed S-Curves water line. We understand the roughly 250 foot line was to be trenched or drilled along the S-curve's western extent on Hemlock, connecting at the north and south ends, to add static (non-earthquake) redundancy to the main transite line near Highway 101. The purpose of our services was to provide consultation for alignment and risk evaluation based on our S-curves landslide experience. Our scope of work included the following:

- Provide principal level project management including client communications, management of field and subcontracted services, report writing, analyses, and review of invoicing.
- Review previous reports, geologic maps and vicinity geotechnical information available in our files as indicators of subsurface conditions.
- Complete a site reconnaissance evaluating the accessible, visible surface features of the slide and attend a site meeting.
- Summarize our recommendations in a stamped letter report.
- Provide consultations as requested.

The proposed alignment is in an area of known active landslide movement, in the southern extent of a broader less active slide. Both are shown on the attached aerial photo. Our previous work for the City on the S-curves slide began in 2002 and included 6 borings to up to 90- feet deep with subsurface instruments and analyses, as well as survey monitoring for movement of this active portion of the slide. Single event deformations were up to one foot vertically and horizontally in a west-southwest direction in response to high winter rainfall events in eroded toe conditions. In 2007 and 2008 horizontal drains were installed to reduce peak ground water levels during high rainfall events. This was successful in slowing slide movement. The drains have been cleaned each early fall since installation, and drains flow during rainfall events. Current slide movement has been measured (at the replaced remaining central inclinometer location) at 0.3 inches in the primary shear zone in the last 6 years, in several increments.

Based on the preceding movement areas, and the ground cracks observed historically as more pronounced in the western shoulder, we do not recommend an alignment on the western side of Hemlock. We recommend two possible alternatives. These include an alignment near the eastern fog line or a directional bore east of the active ends of the slide under the hill. In either case, the risk of damage and rupture in a Cascadia interface earthquake is high. This water line should not be considered a redundant feature for water distribution in an earthquake and should have shut offs at each end.

A trenched alignment near the eastern fog line would likely represent the least cost, with more risk. This alignment would require dry season installation to lessen instability impacts, which is roughly from

July to mid-October. We understand trenching would be roughly 3-4 feet deep. We recommend no more than 30 feet of trench remain open or un-backfilled at any time, with backfill consisting of crushed rock fill overlain by a 15 mil fluid barrier below the top foot of pavement base rock, and extend at least 6 inches past each side of the trench (to reduce infiltration into the trench, and impacts to subsurface water levels and stability). Alternatively, above the pipe zone, and below the top 12 inches of pavement base rock, the trench could be filled with fine grained (native silt) soils dried and compacted as structural fill. With this alignment we recommend flexible connections, particularly near the north and south margins of the active slide.

A directionally bored alignment could also be used, at greater cost and much less risk of non-earthquake slide damage. Flexible connections would not be required. This alignment could start east of the active slide extent and drill under the hill to extend past the other eastern slide extent. This could be done in any season as stability impacts of construction are low. Subterranean easements would likely be required, similar to those obtained for the S-curves horizontal drains.

Limitations

We have prepared this report for use by the City of Cannon Beach and members of the design and construction team for this project only. The preceding recommendations should be considered preliminary, as actual soil conditions may vary. The information herein could be used for planning purposes but should not be construed as a warranty of surface or subsurface conditions. We have made observations only from the aforementioned information. These observations do not reflect soil types, strata thicknesses, water levels or seepage that may exist between observations, or at the time of construction. We must be consulted to observe actual conditions encountered during construction in order for our recommendations to be final. Our observations will allow us to interpret actual conditions and adapt our recommendations if needed. Within the limitations of scope, schedule and budget, our services have been executed in accordance with the generally accepted practices in this area at the time this report was prepared. No warranty, expressed or implied, is given.

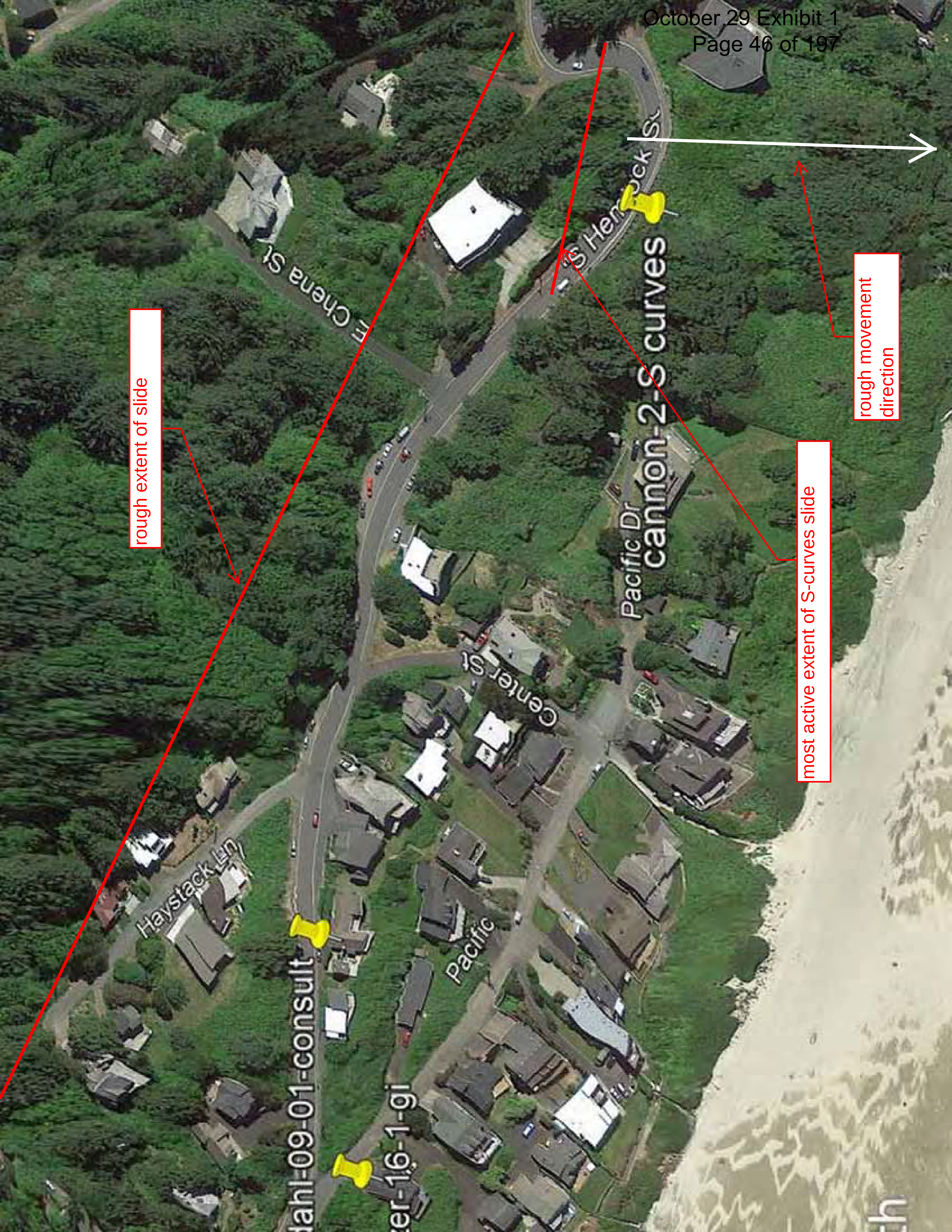
< >

We appreciate the opportunity to work with you on this project and look forward to our continued involvement. Please call if you have questions.

Sincerely,

Don Rondema, MS, PE, GE
Principal

JULY 21, 1998
DON RONDEMA
Expires 12/31/18



rough extent of slide
rough movement direction
most active extent of S-curves slide
S Hemlock St
E Chena St
cannon-2-S curves
Pacific Dr
Center St
Haystack Ln
Pacific
dahl-09-01-consult
ter-16-1-gi

Geotech Solutions Inc.

July 2, 2019 *robertscannon-18-1-consult*

Stanley and Rebecca Roberts
Stan.milliman@gmail.com

Cc:
jay@jayraskinarchitect.com
rec@opusnet.com
kevin@objectiveadvisorsllc.com
plandevelopment@msn.com

GEOTECHNICAL ENGINEERING CONSULTATION
Planning Phase
Tax Lot 600, Nenana Avenue Oceanfront Lot - Cannon Beach, Oregon

Purpose and Scope

As authorized this report summarizes our geotechnical engineering consultation for the planning phase of the subject oceanfront lot located immediately north of the (unimproved) Nenana Avenue easement west of Hemlock Street in Cannon Beach, Oregon. We understand the feasibility of developing the site is to be evaluated, and our purpose was to assist in the geotechnical aspects of planning. This did not include actual foundation design recommendations and detailed stability analyses which are required for the design phase. Our specific scope of services included the following:

- Review vicinity geological and geotechnical information available in our files including recent summaries of landslide movement and our 2018 water line study.
- Review our work on the S-Curves slide to evaluate relative stability of the site and impact of stabilization efforts at the S-Curves, including movement rates and water level impacts.
- Attend up to 2 meetings as requested by the owner or architect.
- Provide a qualitative opinion on current stability condition and provide preliminary recommendations to reduce impacts to stability such as earthwork limitations and drainage requirements.
- Provide a qualitative discussion of preliminary foundation options and related considerations such as relative costs, risks and constructability.
- Provide a letter report summarizing our review, opinion of geotechnical feasibility, and preliminary options for foundation types.

Site Stability Background

The site is located within an active portion of an ancient landslide and is mapped in a geologic hazard area as mapped by the City of Cannon Beach (mapping excerpt attached). The site is part of a "down-dropped" area of the slide that is subject to storm surge wave attack. We have completed previous work on this property and adjacent properties, and have extensive work for the City of Cannon Beach in efforts to slow movement of the active portion of the slide at and above the site. That active portion has ruptured pavements on the S-curves and caused ground movement of several properties, including tax lot 600 and movement below the beach.

Mr. Rondema's involvement on this slide goes back to 1999, and Geotech Solutions previous work for the City on the S-curves slide began in 2002. That has included 6 borings up to 90- feet deep with

subsurface instruments and analyses, as well as survey monitoring for movement and acquisition of water level fluctuation data. Single event deformations were up to one foot vertically and horizontally in a west-southwest direction in response to high winter rainfall events in eroded toe conditions. In 2007 and 2008 horizontal drains were installed to reduce peak ground water levels during high rainfall events. This has significantly slowed, but not stopped, slide movement. The drains have been cleaned by the City each fall since installation, and drains flow during and after rainfall events with seasonal increases. Current slide movement has been measured near the active center at 0.3 inches in the primary shear zone in the last 6 years. Movement has been in response to high groundwater events induced by heavy rainfall storms. Most recently in 2018 we issued the attached slide movement update to the City, and in 2019 we completed work for a new water line in Hemlock Street. That water line in Hemlock is of a type of pipe and layout that can withstand some small slide movements, but is assumed to be ruptured in a CSZ earthquake event as is the sewer force main.

Risk

As stated, the site is part of an active landslide. Although movement has been slowed by horizontal drains reducing groundwater peaks in high rainfall events, this slowing is tenuous. Events that could accelerate movement include beach erosion, slope and toe erosion, new threshold rainfall events, and changes in slope loading such as cuts and fills, and site drainage. In addition, large movement is likely in earthquake ground motions from a CSZ interface earthquake (which has roughly a 30% chance of occurring in the next 50 years). Any of these issues, or a combination, could cause movement of the site that is structurally damaging. Damage could range from cracking and settlement to extensive movement and damage that requires rebuilding. The seismic motions of a CSZ interface earthquake (not to mention the subsequent tsunami impacts) would certainly result in extensive site damage and likely a loss of occupancy condition, and may render the site unusable. Because of these circumstances, in our opinion designing a structure for safe egress is the highest reasonable long term goal.

Localized ocean front slope regression is another risk, as the high bank erodes eastward to impact the building envelope. In this area of the coast regression averages roughly one foot per year, but is episodic, and may regress 10 or more feet in one year. Regression is typically more prevalent during strong southwestern storm surges and high sea level El Nino events which can coincide with total sand removal to siltstone on the beach (we observed this condition below the site in 1999, when the passive shear wedge of the slide was also visible on the beach).

Foundation Support

If the preceding risks are understood by the owner and the design team, and can be tolerated, foundation support is achievable. The types of approaches are likely limited by site access with equipment as well as high costs. We believe two approaches should be considered. A rigid reinforced structural mat supported by fixed deep foundations would be the lower risk - higher cost approach. Another approach could be a rigid mat designed for re-levelling. This has more risk of overall movement but lower initial cost, and also more risk of slope regression and utility impacts.

In any case drilling and underground work must be done when ground water levels are low with better stability, typically May through September.

Deep Foundation Supported Structural Mat - Within the site slide mass there are several rupture and movement zones at varying depths. These zones have been observed in adjacent inclinometer readings (below and next to the site), and were plotted 3 dimensionally from “communication” during

pressurized drilling/installation of horizontal drains. For foundation support to reduce overall movement these zones must be fully penetrated and the deep foundation elements designed to resist the resulting forces. The deep foundations would likely be large, heavily reinforced drilled shafts due to the high bending moments near the rupture zone interfaces. Shaft reinforcement may include W-shape beams (if they can be delivered to the site), or substantial rebar cages. Shaft size is likely limited to equipment access size and cost. Drilling will be difficult to adequately penetrate hard underlying siltstone. Special tooling as well as casing and dewatering will likely needed. The mat would need to structurally span between shafts, using grade support only for forming during construction.

Rigid mat designed for future Relevelling – A rigid mat designed to be stiff enough to accommodate relevelling is another possible option, but carries more risk. Increased risk is from distortion related damage to utilities and hardscaping, and exposure to undermining from shoreline regression. The structural engineer would need to design for significant free spans to accommodate slide grabens, as well as perimeter uplift and bending forces for relevelling. Relevelling could be done with push piers (hydraulically/reaction drive pipe piles) that are in place as part of the original construction. Reduction in regression risk could be accommodated by adding reinforced drilled shafts to the oceanfront side.

Access

The civil engineer must be consulted to design access at suitable inclinations and turning/egress. On-grade access will be difficult due to the very steep narrow roadway transition at Hemlock and the restraints to cutting and filling that may otherwise destabilize the slide. An initial estimate is that cuts must not be made in the slopes more than 2 feet deep and must be limited horizontally, and no cuts are allowed on the slope abutting Hemlock (just west of Hemlock, south of the existing “entry”). Likewise, fills would likely need to be limited to the equivalent weight of 2 feet of soil or rock. Detailed stability analyses of alternative grading sections would need to be done to better quantify these limits. For on-grade approaches a potential solution would be a near grade and pile restrained lightweight fill option on the downslope side of the entry drive. This could employ horizontally seated and connected EPS blocks shaped to desired grades. Shaped EPS for these approach inclinations may be difficult and costly, and may require a reinforced raked concrete wearing course depending on the final inclination. A viable alternative may be a pile supported structural approach and/or platform.

Drainage

Maintaining low ground water levels and limiting erosion are critical to stability. The mid-slope horizontal drain discharges for slide improvement abutting the east side of the lot complicate drainage as they will need to be accessible and maintained, with discharge collected to hard pipe. All runoff from structures and hard scaping must be collected and routed to suitable erosion protected discharge, preferably to the swale to the north if permissible.

Utility Connections

Utility connections that are designed to allow movement without damage are recommended. Such pipe connections are present in Hemlock for the sewer force main along the S-curves. Pipe with some flexibility in curved alignments can also help, such as the new water line in Hemlock. Again the civil engineer should be consulted on these options.

Limitations

We have prepared this report for use by Stanley Roberts and members of the planning team for this project only. The preceding recommendations should be considered preliminary, as actual soil conditions may vary. The information herein could be used for planning purposes but should not be construed as a warranty of surface or subsurface conditions. We have made observations only from the aforementioned information. These observations do not reflect soil types, strata thicknesses, water levels, seepage or stability conditions that may exist between observations, or after the present time. We must be consulted to complete stability and foundation support analyses design for any structures, as well as observe actual conditions encountered during construction in order for our recommendations to be final. Our observations will allow us to interpret actual conditions and adapt our recommendations if needed. Within the limitations of scope, schedule and budget, our services have been executed in accordance with the generally accepted practices in this area at the time this report was prepared. No warranty, expressed or implied, is given.

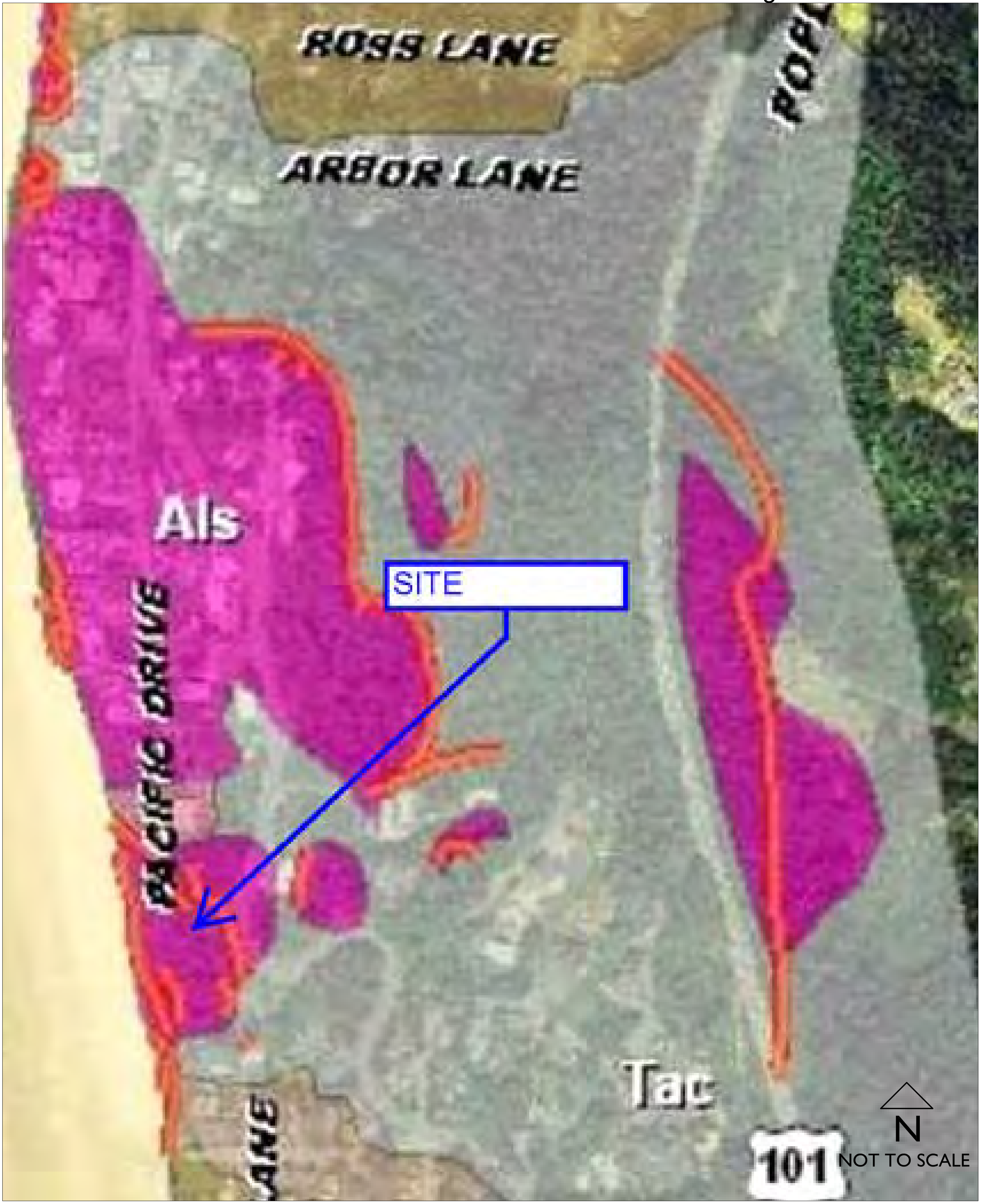
* * *

We appreciate the opportunity to work with you on this project and look forward to our continued involvement. Please call if you have questions.

Sincerely,

Don Rondema, MS, PE, GE
Principal

JULY 21, 1992
DON RONDEMA
Expires 12/31/20

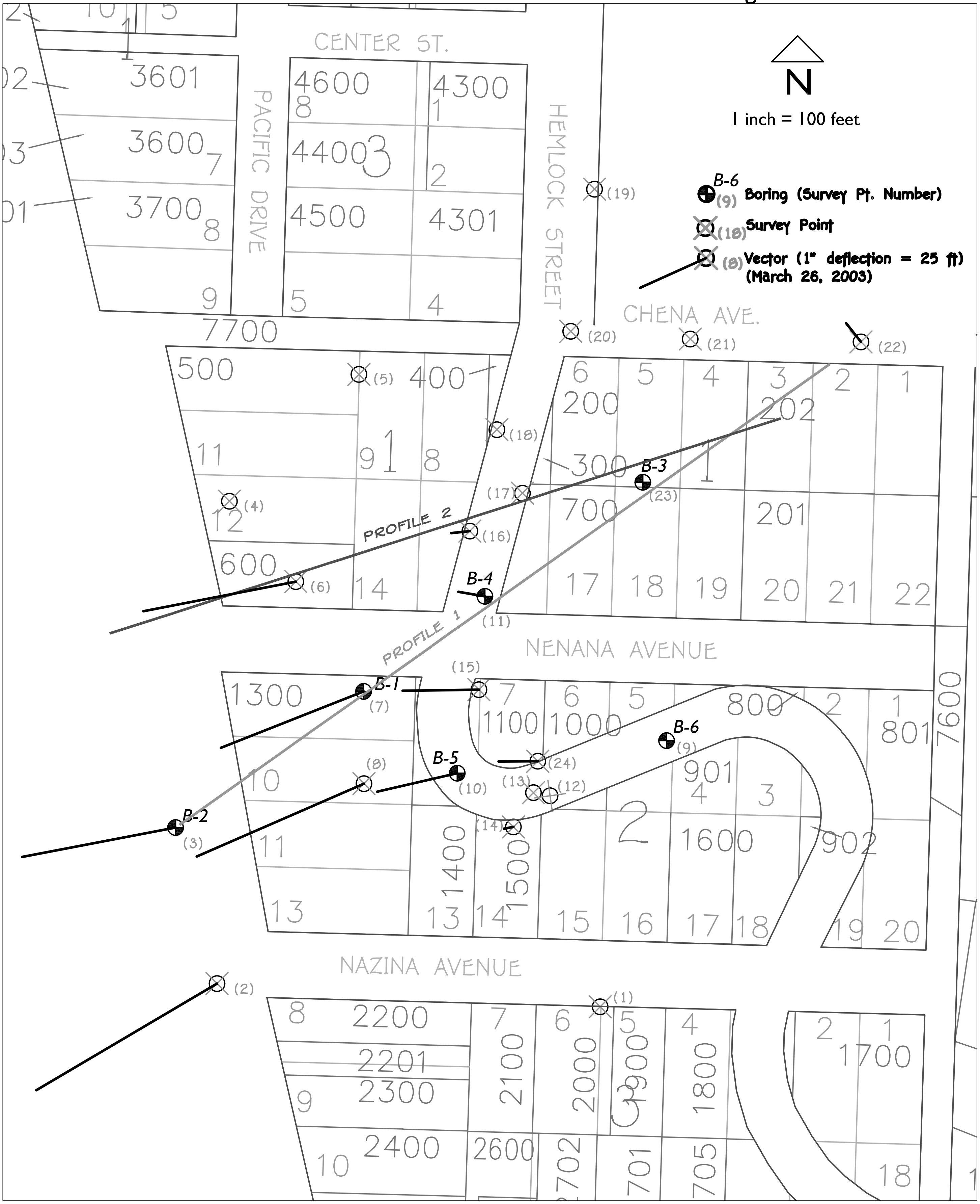


base from DOGAMI O-90-06

Geotech Solutions Inc.

SITE GEOLOGICAL MAPPING

robertscannon-18-1-consult



Geotech Solutions Inc.

SITE PLAN

Cannon-02-01-gi

Geotech
Solutions Inc.

MEMORANDUM

cannon-18-1-consult

To: Karen LaBonte, Public Works Director, City of Cannon Beach; labonte@ci.cannon-beach.or.us

Date: June 26, 2018

Subject: **Hemlock Street S-Curves Slide: Status Update**

Introduction and Background

This memorandum provides an update to the status of the inclinometer data from the S-curves slide as read on June 23, 2018. The previous last reading was in 2015. The reason for this reading was a centerline crack appearing in the last month or so near the apex of the curve above and slightly south of the B-1r instrument. This crack is roughly 10-15 feet in length, and open up to roughly ¼" with perhaps a slight vertical offset down to the west. In addition, and perhaps relevant to tangential slide restraint and equilibrium, slope cuts and net mass removal has occurred on an adjacent project over roughly the past year. That project abuts previous lateral shear zones observed at the southern portion of the active slide.

The water levels in the slide are no longer being recorded as the instruments have expired, and new winter storm rainfall levels had not exceeded those previously recorded. The data attached are inclinometer readings for only one instrument near the center of the slide (B-1r) which has been shown over many years to correlate well with rainfall response and water levels and other previous movement in other, now irrevocably damaged, casings. It should be understood that this correlation is in the context of the general beach/slide toe elevations and erosion conditions experienced since 2008.

Conclusions and Recommendations

Roughly 0.2 inches of movement has occurred above/near the primary shear surface since the last reading roughly 3 years ago. The previous 3 years had roughly 0.1 inches of movement. Overall readings show a total of roughly 2.5 inches of movement on this replacement casing. A plot is attached. This movement is not out of recorded context movement rates for the slide.

Based on our site observations, in our opinion the surface cracking is not discernible from an aging panel joint or related thermal separation crack. It is possible that the crack was caused by accumulated underlying movement of the slide and is exhibiting at the previously placed grid overlap joint, but it does not coincide with previous slide induced crack locations which trended southwesterly with vertical offsets greater than horizontal, and at locations north and south of this crack location.

Although B-1r is approaching its deflection life, it is still functional and in our opinion does not need replacement at this time. Replacement/redrilling for a new casing (including initial baseline readings) is estimated at roughly $10,000 as access is difficult. If additional cracking occurs that is more indicative of slide movement, then a new water level logger is recommended for the paired B-1 standpipe (P-1).

Based on our current monitoring, we still expect movement of the S-Curves to be ongoing. However, the reduction in ground water levels and movement in large rainfall events has been greatly reduced by the functional horizontal drains compared to historical observations. No measures in addition to frequent roadway surface observation and annual drain cleaning are recommended at this time.

Provided the existing drains are maintained and cleaned annually and are functional, it is our opinion that they are sufficient to continue to slow the slide for the rainfall event intensities experienced since drain installation. Exceptions would be from earthquake ground motions or significant beach toe erosion. Any significant beach level erosion (such as exposure of siltstone below the sand similar to the El Nino cycle of 1999), or toe slumping, would be cause to take inclinometer readings, as would experiencing a new threshold rainfall event. These would be anything in excess of the storm events recorded since drain installation which are 4.37"-1 day, 6.26"-2day, 6.29"-3day, or 10.21"-5day. Please alert us if any of these thresholds are met.

The Limitations of our report apply, and that report and a few predrain install crack photos are attached here for background.

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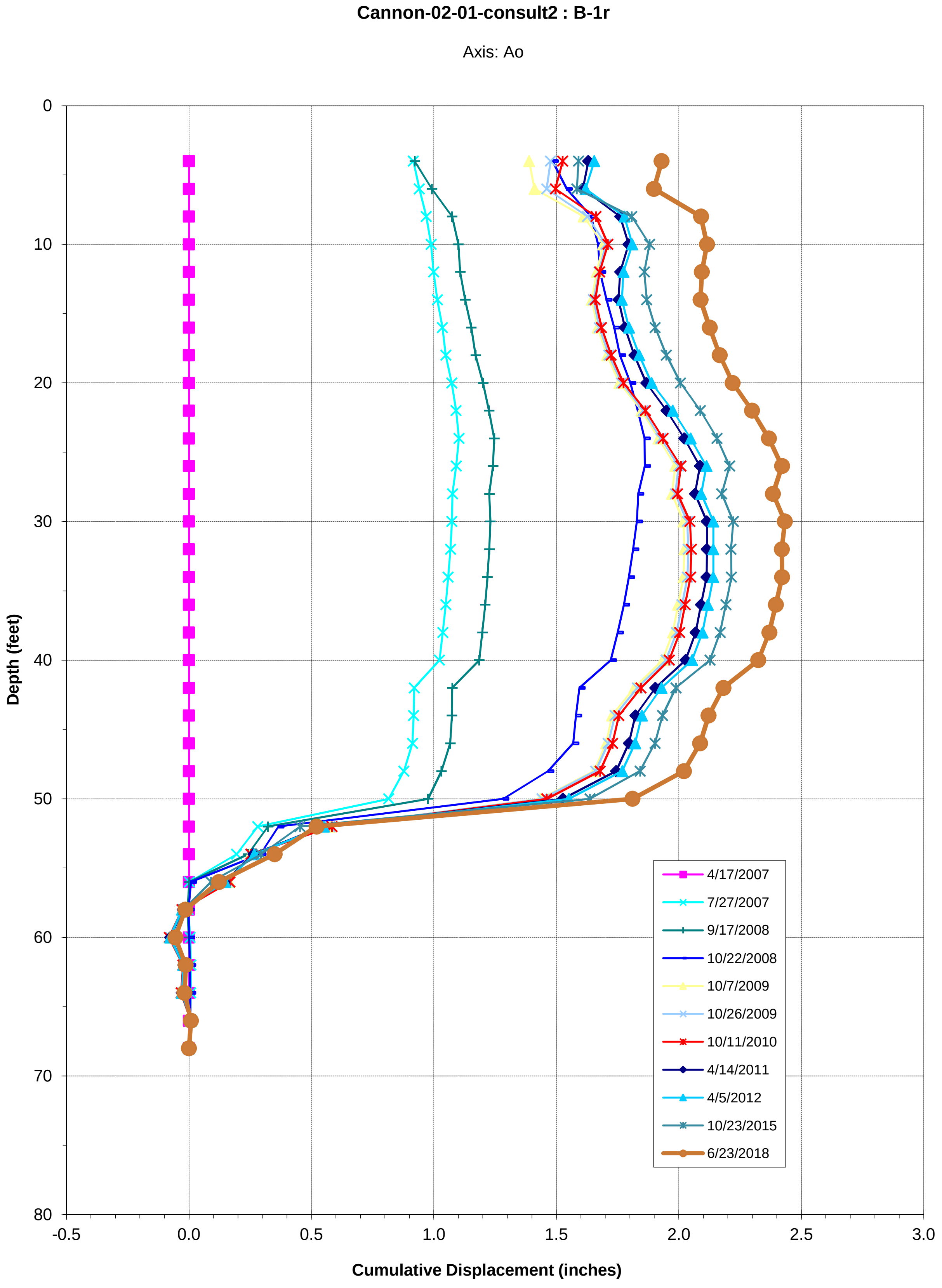
We appreciate the opportunity to work with you on this project and look forward to our continued involvement. Please call if you have questions.

Sincerely,

Don Rondema, MS, PE, GE
Principal

DON RONDEMA
Expires 12/31/18


Cannon-02-01-consult2 : B-1r
Axis: Ao
Depth (feet)
0
10
20
30
40
50
60
70
80
-0.5
0.0
0.5
1.0
1.5
2.0
2.5
3.0
Cumulative Displacement (inches)
4/17/2007
7/27/2007
9/17/2008
10/22/2008
10/7/2009
10/26/2009
10/11/2010
4/14/2011
4/5/2012
10/23/2015
6/23/2018

REPORT OF GEOTECHNICAL ENGINEERING SERVICES

Proposed Residence at Tax Lot 600 - North of Nenana ROW
Cannon Beach, Oregon

Geotech Solutions Inc.

June 6, 2020

GSI Project: robertscannon-18-1-consult3

Geotech Solutions Inc.

June 6, 2020 *robertscannon-18-1-consult3*

Stanley and Rebecca Roberts
stan.milliman@gmail.com

Cc: Kevin Patrick; kevin@objectiveadvisorsllc.com

GEOTECHNICAL ENGINEERING CONSULTATION
House Foundation Support and Stability Analyses
Tax Lot 600, Nenana Avenue Oceanfront Lot - Cannon Beach, Oregon

As authorized, this report summarizes our geotechnical engineering consultation for the subject site's house foundation support and stability analyses. Mr. Rondema has studied the lot since 1995 for several owners and has extensive involvement in the S-curves slide evaluation and stability improvements for the City of Cannon Beach. The purpose of our work was to provide geotechnical engineering analyses and consultation to the design and planning team as requested for the house design. Our scope of services included the following:

- Provide principal level geotechnical project management, including review of analyses, report writing, and invoicing, as well as client communications.
- Construct a computerized stability model of the site and proposed house pad area from above the eastern fog line of Hemlock to past the western slope toe on the beach using 2-ft on ground surveyed topography as provided by others.
- Back calculate the existing tenuous stability condition using the model by reviewing previous explorations and slide data from our files, and complete analyses of up to 2 cross sections that incorporate various cut and fill scenarios and foundation elements compatible with provided plans.
- Complete sensitivity analyses of the preceding scenarios using a scoured slope toe condition. (A seismic condition will be unstable so will not be analyzed).
- Summarize our work in a letter report stamped by a PE/GE.
- Provide up to one site meeting and 4 hours of follow-up office consultation to the team after this report is issued.

LITERATURE REVIEW

We reviewed the following geologic information and geotechnical reports available in our files and as read from others as part of our study.

- DOGAMI Bulletin 74, 1972.
- 'Field Investigation of Geologic Hazards in Cannon Beach, Oregon', Martin E. Ross, June 3, 1977.
- DOGAMI O-09-06
- 'Geotechnical Engineering Report, Tax Lot 600 – Nenana Avenue Oceanfront Lot, Cannon Beach, Oregon', GeoEngineers, December 7, 1995.
- 'Phase 1 Report of Geotechnical Engineering Services, Residential Site Development, Lots 9-12, Block 2, Tolovana Park, Cannon Beach, Oregon', GeoEngineers, May 13, 1998.

- 'Geotechnical Feasibility Study – Phase 1, Nenana Avenue Lot, Cannon Beach, Oregon', GeoDesign, Inc., November 18, 1999.
- 'Geotechnical Engineering Report, Geotechnical Investigation and Monitoring – Phase II, Tax Lot 600 – Nenana Avenue, Cannon Beach, Oregon', GeoDesign, Inc., May 25, 2001.
- S-Curves Landslide Investigation, Stabilization, and Monitoring, Geotech Solutions, 2002 to present. Original report dated May 12, 2003.
- S-Curves water line, Geotech Solutions, Inc., August 15, 2018.
- Geotechnical feasibility for planning phase, Geotech Solutions, Inc., July 2, 2019.
- Borings from Earth Engineers for Nenana ROW, 2020.
- Updated Stability Analyses of S-curves slide for Nenana Roadway improvements – in process, April 2020.

These references contain geologic and geotechnical information in the immediate vicinity and on the site itself. On-site studies included three borings and installation of inclinometer casings for measurement of ground movements and ground water levels. Off-site work is also extensive and includes borings and inclinometer casings, ground water instrumentation from the beach level to above Hemlock Street, slope stability modeling, and installation of horizontal drains for slide stabilization improvements coupled with over 12 years of monitoring. Locations of explorations are shown on the attached plans and feasibility report.

HISTORY AND BACKGROUND

The site is located near the southern end of an ancient landslide mass as mapped by Ross, 1977. A copy of this map is provided in the report attachment by others. The slide extends past the northern ends of Pacific Street and Haystack Lane north of the site. The central and northern portions of the ancient landslide are developed with several roadways and numerous residences. Small incremental movement of these areas of the slide mass is likely ongoing, especially during wet season heavy rainfall events, but we are not aware of recent large displacements in those areas that damaged structures or roadways.

The southern end of the ancient slide, including the site and areas to the east and south, has been more active for many decades. This portion of the slide is commonly referred to as the 'S-Curves slide'. Interviews completed by others (1998) describe downward movement of approximately 6 to 8 feet in Hemlock Street during the winter of 1972. This report also indicates that a previous residence was removed from the subject site in 1972. The report does not specifically indicate whether the residence was damaged by the ground movements, although cracks and displacement of the remnant slab of up to 5 inches were noted. Remnants of the residence are still present at the site including at least portions of the concrete slab, rubble fill, and evidence of previous grading.

A geotechnical investigation was done by others in 1998 for a group of tax lots located immediately south of the Nenana Avenue easement, property since purchased by the City as "Inspiration Point". This report describes evidence of shallow landslides and the investigation included two borings with slope inclinometers that measured active movements associated with a deeper landslide surface.

Ground movement in the S-Curves slide area has occurred many times, and in 1999 resulted in ground and pavement ruptures in Hemlock Street and abutting sites of 4- 6 inches vertically and several inches horizontally, and rupture of the south end City sewer force main. These movements and related events also historically deformed utilities and the roadway in Chena Street (which has since been repaired/improved) and damaged several houses. Geotech Solutions began a landslide investigation of the S-Curves slide in 2002 and groups of horizontal drains were installed under our consultation in 2007 and 2008 by the City of Cannon Beach. The goal was to decrease storm rainfall related spikes in groundwater levels and therefore reduce movement. Geotech Solutions' investigation included numerous borings and piezometers and related measurements of ground movement and ground water levels correlated to storm event rainfall. Monitoring of rainfall, drain discharge, water levels and movement has been ongoing since installation, with the latest readings in June 2018 prior to roadway water line improvements. A memo summarizing the most recent readings is attached.

Geotechnical investigations regarding private development on the subject site were done by GeoEngineers (1995) and GeoDesign (1999 and 2001) with Mr. Rondema's involvement. These investigations included three borings on site and installation of slope inclinometer casings for measurement of ground movements. In summary the inclinometers on site indicated movement at depths of 35 to 45 feet below the ground surface on the eastern portion of the lot from 2000 to 2001, with massive siltstone below that to depths of over 70 feet (roughly elevation -5 ft, 25 feet below current beach levels at the slope toe). In 2001 these casings were deformed to the point they could not be read. Logs of these borings and an inclinometer plot are attached.

S-CURVES STABILITY IMPROVEMENTS

Monitoring completed by Geotech Solutions in the S-Curves slide area has indicated that groundwater levels and slide movements have decreased, but not stopped, since installation of the network of 19 horizontal drains in areas immediately south and east of the subject site (as shown in the attached figure). No cracking or deformations of the Hemlock Street pavement have been observed since drain installation. However, our instruments indicate that small movements less than 0.2 inches have occurred at depth following at least three significant rainfall events in the last 12 years. The City of Cannon Beach annually has cleaned these drains, and one drain can no longer allow passage of the drain cleaning head for the last several years. A few of the 19 drains possibly being partially blocked is not expected to impact the S-curves or site (as that drain may still fully function anyway) due to redundancy in the drain system. Annual maintenance of the drains is required to maintain the current S-curves condition.

SITE OBSERVATIONS

We visited the site for Dave Roberts on February 18, 2010 to observe existing site conditions. At that time the existing concrete slab at the site was moderately to severely cracked with horizontal separations up to approximately 5 inches. Crack orientation was variable but larger cracks were roughly oriented north-south, parallel to the crest of the oceanfront slope.

Evidence of shallow landslide scarps and sloughing along the crest of the oceanfront slope is present (also as mapped by Ross '77). The crest of the slope is currently located approximately as close as 45 feet west of the east property line. These features can also be interpreted from the recent topographical survey. Surface water is present at the ground surface in wet conditions near the southeast property corner and is likely associated with the group of three horizontal drains located off-site to the east.

CONCLUSIONS AND RECOMMENDATIONS

General

Previous reports concluded that improved stability of the overall S-curves slide could allow for development, albeit still with some risk of damage from slide movement. As discussed in the preceding, this stability improvement in the overall S-curves slide has occurred with the installation of horizontal drains that have now been in place and monitored for over 12 years. Groundwater in the S-curves responds quickly to rainfall infiltration, with peaks occurring within hours during the wet season. Monitoring data shows that the network of horizontal drains has decreased these peaks in ground water levels, as well as baseline levels, and increased stability of the S-Curves slide. However, it should be noted that the slide is still moving fractions of an inch on deep shear surfaces in high intensity wet winter rainfall events. The measures herein are not intended to arrest overall S-curves slide movements, as such measures are not feasible on this small lot. Rather, the measures are to improve localized lot stability relative to the oceanfront slope.

The analyses done for this lot does indicate that overall S-curves slide stability conditions will not be reduced, and that sections through the lot will be slightly improved, if the recommendations herein are followed.

"Setback"/Active Instability Margin

The critical slide issue for house foundation support design is failure of the oceanfront slope in an eastward progression into the building pad. In general, the existing ocean front slopes are unstable west of the "setback" /active instability margin on the attached figure. This is not a "setback" for conventional foundations. It is rather a margin of active instability. Building west of this margin is not feasible and may further destabilize the lower slopes. The instability margin is generally above the 61 ft elevation to the south, and the 64 ft elevation to the north. Development east of the proposed margin is only suitable if the stability improvements and deep foundation recommendations of this report are followed. On-grade settlement sensitive hardscaping features (such as concrete patios and sidewalks) west of the proposed margin are not recommended. Although foundation support must be derived east of this margin, cantilevered features may be feasible west of the margin per a structural engineers' design.

It should be understood the recommendations herein are to improve stability conditions for localized stability in static conditions (no earthquake). A CSZ interface earthquake will result in failures of the oceanfront slopes, the S-curves, and likely the overall slide that extends far to the north. The measures herein for localized stability and house support improvements are intended to allow the structural engineer to design for egress during such an earthquake. House damage will still occur and will likely be irreparable following tsunami impacts to the slope and stability. Re-occupancy or even the feasibility of rebuilding is unlikely. This seismic instability condition is similar to adjacent developed properties. Our specific analyses and recommendations are detailed in the following sections.

Slope Stability Analyses

Over the last several decades we have evaluated the stability of the S-curves. This included the explorations, data acquisition, surveying, and observations described earlier in this report. From this information and the provided site topographic survey, we developed stability models for the site using the program SLIDE and limit equilibrium methods. The critical section through the site is shown on the attached stability figure. Factors of safety within the western slope were as low as 0.80 (failure is less than 1.0) and increased to just over 1.0 east of the instability margin, and near 1.1 at the east side of the lot. These factors of safety are consistent with site observations in the current S-curves "dewatered" slide context.

The preceding existing factor of safety of 1.0 to 1.1 on the east portion of the lot is unsuitable for building without improvement. Typically for active slide areas and owner accepted damage risk, a factor of safety of 1.3 is used. Therefore, measures to improve stability to this level were evaluated. This did not include buttressing or armoring the oceanfront slope as it was assumed to not be permittable and would still only be part of a solution. This also did not include new horizontal drains, as installation of such drains could exacerbate the localized oceanfront slope instability (phase 1 drain installation for the overall slide caused slight temporary mobilization in the Nenana ROW B-1 inclinometer casing).

Sensitivity analyses were performed on many variables. For example, embedding a basement would decrease stability upslope, and adding significant fill to the site would increase instability of the oceanfront slope. Extreme beach front toe slope scour, such as observed in the 1999 El Nino and winter storm surge events, could also decrease stability. An eroded toe condition is addressed with the lot stabilization measures herein but would reduce the overall S-curves stability by roughly 5%.

Stabilization systems in the form of deep foundations and "shear piles" were evaluated with various configurations, sizes, and frequency to achieve a relative factor of safety for localized stability of 1.3 (up to a 30% increase over the existing condition). Detailed descriptions of these systems are included in following sections of this report.

Erosion Protection

Erosion protection of the slopes is vital to maintain some resistance to ongoing sloughing which may impact surface features and stability upslope. The existing slope vegetation is well developed and thick and should not be disturbed. Root intensive salt tolerant plantings such as hooker willows would aid in toe stabilization if any exposed soils are present. If needed, we recommend a local expert on oceanfront erosion control plantings be consulted to provide recommended planting details and address possible permitting issues.

Earthwork

Site Preparation - Site preparation for earthwork will require removal of vegetation, existing debris and slabs, and other unsuitable materials within proposed foundation support and building footprint areas. Existing bollards and casings should be removed, and the casings filled with grout. Root balls from trees or shrubs may extend several feet and grubbing operations can cause considerable subgrade disturbance. All disturbed material should be removed to undisturbed subgrade and backfilled with structural fill. In general, roots greater than one inch in diameter should be removed.

Temporary Cut Slopes - Temporary and permanent cut slopes should be no more than 2 feet high.

Fill Height Limitation – Site stability modeling indicates an average applied load of 200 to 250 psf to the lot does not significantly impact instability. Thererfore, fills must be limited to an average of less than 2 feet above existing grades, including that needed around grade beams and pile caps. Likewise, landscape fills must not increase site elevations on average more than two feet.

Stabilization and Soft Areas - After stripping, we must be contacted to evaluate the exposed subgrade in any on-grade structure areas such as flatwork, etc. Soft areas will require over-excavation and backfilling with well graded, clean angular gravel compacted as structural fill. A separation geosynthetic will also be required, such as a Propex Geotex 801 or equivalent.

Working Blankets and Haul Roads - Construction equipment should not operate directly on the subgrade when wet, as it is susceptible to disturbance and softening. Rock working blankets and haul roads placed over the preceding geosynthetic can be used to protect subgrades. We recommend that sound, angular, pit run or crushed basalt with no more than 6 percent passing a #200 sieve be used to construct haul roads and working blankets. Working blankets should be at least 12 inches thick, and haul roads at least 20 inches thick. The preceding rock thicknesses are the minimum recommended. Subgrade protection is the responsibility of the contractor and thicker sections may be required based on subgrade conditions and type and frequency of construction equipment.

Imported Granular Fill - Imported granular fill, such as clean sand or rock, should have a maximum particle size of 6-inches, be well graded, and have less than 5 percent passing the #200 sieve. This material should be compacted to 95 percent relative to ASTM D 1557.

Trenches - Utility trenches may encounter groundwater seepage and caving should be expected where seepage is present and in soft and/or loose soils. Shoring of utility trenches will be required for depths greater than 4 feet. We recommend that the type and design of the shoring system be the responsibility of the contractor, who is in the best position to choose a system that fits the overall plan of operation. At building connections, tolerance of deflection should be part of the design, as the building is expected to move less than areas off site. No infiltration of collected storm water is allowed.

Pipe bedding should be installed in accordance with the pipe manufacturers' recommendations. If groundwater seepage is present in the base of the utility trench excavation, we recommend over-excavating the trench by 12 inches and placing trench stabilization material in the base. Trench stabilization material should consist of well-graded, crushed rock or crushed gravel with a maximum particle size of 4 inches and be free of deleterious materials. The percent passing the U.S. Standard #200 Sieve shall be less than 5 percent by weight when tested in accordance with ASTM C 117.

Trench backfill above the pipe zone should consist of well graded, angular crushed rock or sand fill with no more than 7 percent passing a #200 sieve. Trench backfill should be compacted to 92 percent relative to ASTM D 1557, and construction of hard surfaces, such as sidewalks or pavement, should not occur within two weeks of backfilling.

Stability and Foundations – Grouted Micropiles

Localized oceanfront slope stability is a high risk that can be decreased by improved resistance across the slide surface(s) as well as by providing a relatively rigid house foundation system. The risk cannot be made zero, but the intent is to improve conditions enough to prolong movement damage within current static (non-earthquake) conditions. An actual CSZ interface earthquake will induce S-curves slide movement regardless of what is done on this site, as the site is a very small part of the slide. In that scenario, the design goal is again to provide a rigid enough system that structural collapse will not occur and that egress prior to tsunami arrival is accommodated. Although technically above the inundation elevation, tsunamis may runup the slope and may cause immediate irreparable damage on its own, and certainly long-term slope damage.

Western Pile Stability Improvement System - As the overall slide is relatively deep and within hard siltstone, drilled grouted micropiles are the recommended approach to penetrate through this zone to massive siltstone. A westerly location of a stabilization micropile system at or just east of the instability margin is required to limit failures up into the building pad. To this end a westerly grade beam with paired battered piles is recommended. These have significant lateral shear and bending resistance. FHWA based micropile slide stabilization "up-down" coupled moment analyses procedures were used in conjunction with SLIDE slope stability analyses to evaluate stability improvements and pile types and sizes.

We recommend paired (one battered down to the west, one down to the east) 7-inch diameter, 0.45 wall thickness API N80 casing enclosed in a corrosion protection grout column (and with a grout filled interior). These piles will need to be inclined at 30 degrees from vertical to allow for mobilization of axial strength and reduction in bending. These pairings must be spaced no greater than 6 feet on center for the full N-S width of the property (as movement direction is not orthogonal E-W). The heads can be two feet apart, with the piles down to the east set west of the opposing piles (a staggered overlap). The encompassing western grade beam must be designed to be free-standing. It must be noted that overall stability is dependent on the lower water level conditions maintained by the system of horizontal drains employed and cleaned by the City.

Forces generated by pile strength mobilization resisting the slide are shown in the attached sketch, which includes a conceptual layout.

Based on previous observation of the on-site inclinometer casing (the SE bollard/casing on site) movement occurred as deep as 45 feet, roughly elevation 20 feet. Piles will need to penetrate at least 10 feet past this depth into hard siltstone (estimated near elevation 10 feet) to provide enough bond to resist lateral slide forces and their corresponding moments.

The preceding piles must not be included in the structural engineer's house support or lateral resistance calculations (but can be used for wind loading) as they are fully engaged in slide resistance. However, due to physical constraints, house support piles can be included in this grade beam.

Vertical House Support Piles – Grouted micro-piles are also recommended for house foundation support. As vertical house loads are modest, 6-inch diameter grouted Titan 40/16 micropiles are recommended. Embedment must again reach the required 10 feet past the shear zone and be at or below elevation 10 feet. For the preceding pile an allowable capacity of 53 kips may be used for design. This accounts for some reduction from the shear zone. The structural engineer should determine the appropriate layout and spacing to optimize design. These piles also slightly increase the factor of safety for stability if spaced no more than 10 feet apart.

No isolated pier caps are allowed, and all piles must be connected with grade beams in the east-west direction roughly perpendicular to the slope. For resistance to lateral loads, 5 kips can be used for these vertical piles. Other battered piles for the house loading may be required, and the horizontal vector of the preceding pile load can be used with batters up to 30 degrees. Grade beams are not to be used for lateral design due to ground settlement and must be designed as self-supporting.

Capacities for additional pile sizes and inclinations can be provided upon request. We must be retained to review pile support design and called to the site to observe installation of piles.

Seismic Design

In accordance with the International Building Code (IBC) as adopted by SOSSC, the subject project should be evaluated using the parameters associated with Site Class D. Tsunami hazard maps (TIM-Clat-09) indicate that the western portions of the site may be inundated by the largest expected CSZ interface earthquake event of Mw=9.1. We recommend the occupants have an evacuation plan. Instability and tsunami damage are expected to the oceanfront slope as described herein.

Ground Moisture and Drainage

***General* -** The perimeter ground surface and hard-scaping must be sloped to drain away from all structures, and rain drains must be routed to suitable erosion protected discharge near the base of the oceanfront slope. This includes collection and routing of the horizontal drain outlets east of the site. Gutters must be tight-lined to a suitable discharge and maintained as free-flowing. All crawl spaces must be adequately ventilated.

Slope stability, settlement, and foundation support can be reduced by increased surface infiltration and erosion. Therefore, we recommend that all surface runoff from hard surfaces, including downspouts, be collected and routed by tight line to suitable erosion protected discharge at the base of the western oceanfront slope. Gutters must be maintained as free flowing. Ground surface slopes must be inclined away from the structure and be graded to prevent ponding. Periodic grading may be required to maintain proper slopes due to ground distortion or settlement.

***Perimeter Drain* -** A perimeter foundation drain is required at the base of the exterior grade beams. The drain should consist of a one-foot wide zone of drain rock encompassing a 4-inch diameter perforated pipe, all enclosed with a nonwoven geosynthetic. The drain rock should have no more than 2 percent passing a #200 sieve and should extend to within one foot of the ground surface. The geosynthetic should have an AOS of a #70 sieve, a minimum permittivity of 1.0 sec^{-1}, and a minimum puncture resistance of 80 pounds (such as a Propex Geotex 601 or equivalent). As an alternative, a composite drain board (such as an Amerdrain 500/520 or equivalent) can be used above and encompassing the perimeter drain pipe. One foot of low permeability soil (such as the on-site silt) should be placed over the fabric at the top of the drain to isolate the drain from surface runoff.

***Vapor Flow Retardant* -** Some flooring manufacturers require specific slab moisture levels and/or vapor barriers to validate the warranties on their products. A properly installed and protected vapor flow retardant can reduce slab moistures. If moisture sensitive floor coverings or operations are planned, we recommend a vapor barrier be used. Typically, a reinforced product or thick product (such as a 15 mil STEGO wrap or equivalent) can be used. Experienced contractors using appropriate concrete mix designs and placement commonly place concrete directly over the vapor barrier which overlies the base rock/underslab rock. This avoids the issue of water trapped in the rock between the slab and vapor barrier, which otherwise requires removal. In either case, slab moisture must be tested until it meets floor covering manufacturer's recommendations.

Limitations and Observation During Construction

We have prepared the preceding information for use by Stan and Rebecca Roberts and members of their design and construction team for this lot and project only. The information herein can be used for bidding or estimating purposes but must not be construed as a warranty of subsurface conditions. We have made observations only at the aforementioned locations, and only at the stated depths. These observations do not reflect soil types, strata thicknesses, water levels or seepage that may exist between observations or at other areas of the site. We must be consulted to review final design and specifications in order to see that our recommendations are suitably followed. If any changes are made to the anticipated locations, loads, configurations, or construction timing, our recommendations may not be applicable, and we should be consulted. The preceding recommendations must be considered preliminary, as actual soil conditions may vary. In order for our recommendations to be final, we must be retained to review final plans, to observe actual subsurface conditions encountered, and to observe underpinning installation. Our observations will allow us to adapt to actual conditions and to update our recommendations if needed.

◆

We appreciate the opportunity to work with you on this project and look forward to our continued involvement. Please contact us if you have any questions.

Sincerely,

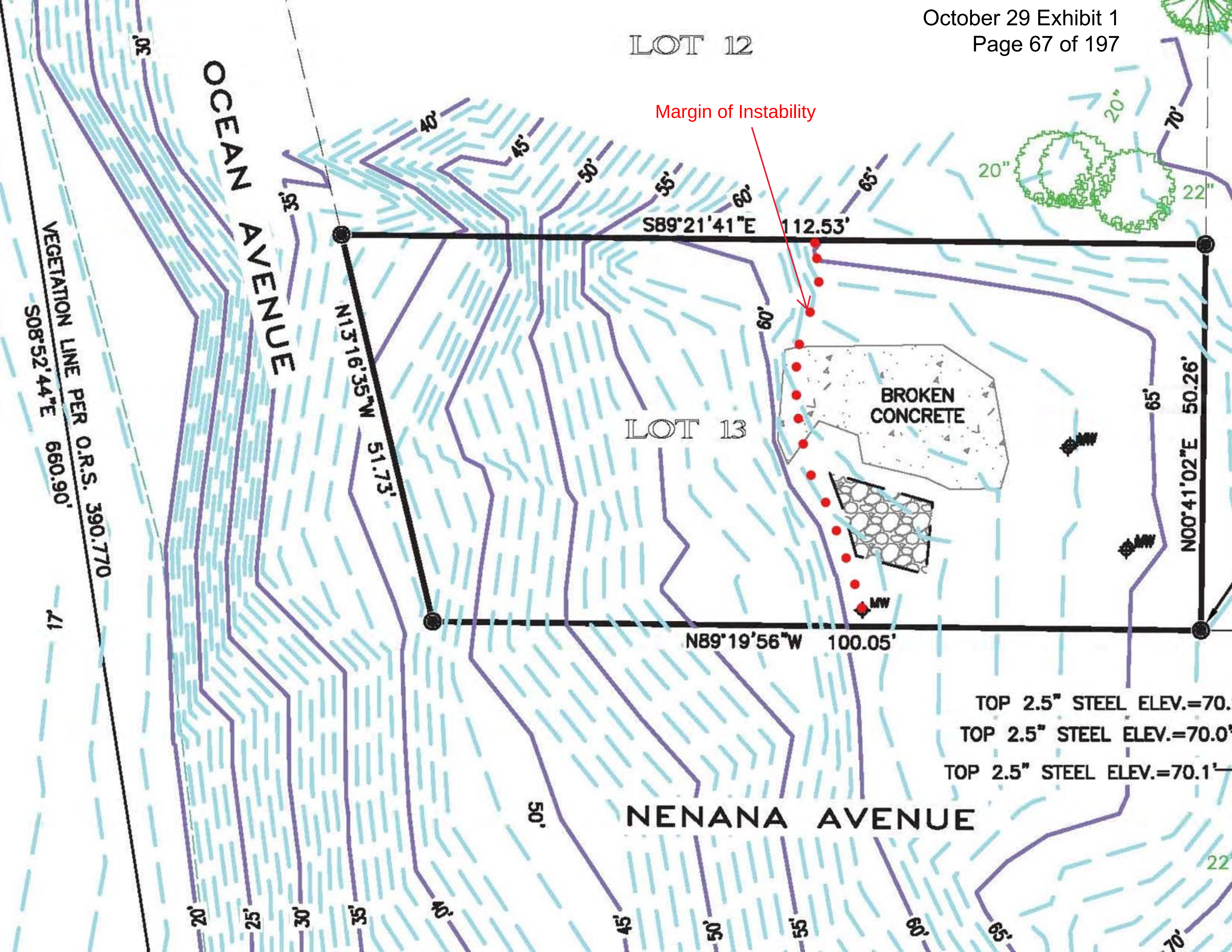
Don Rondema, MS, PE, GE
Principal

DON RONDEMA
Expires 12/31/20

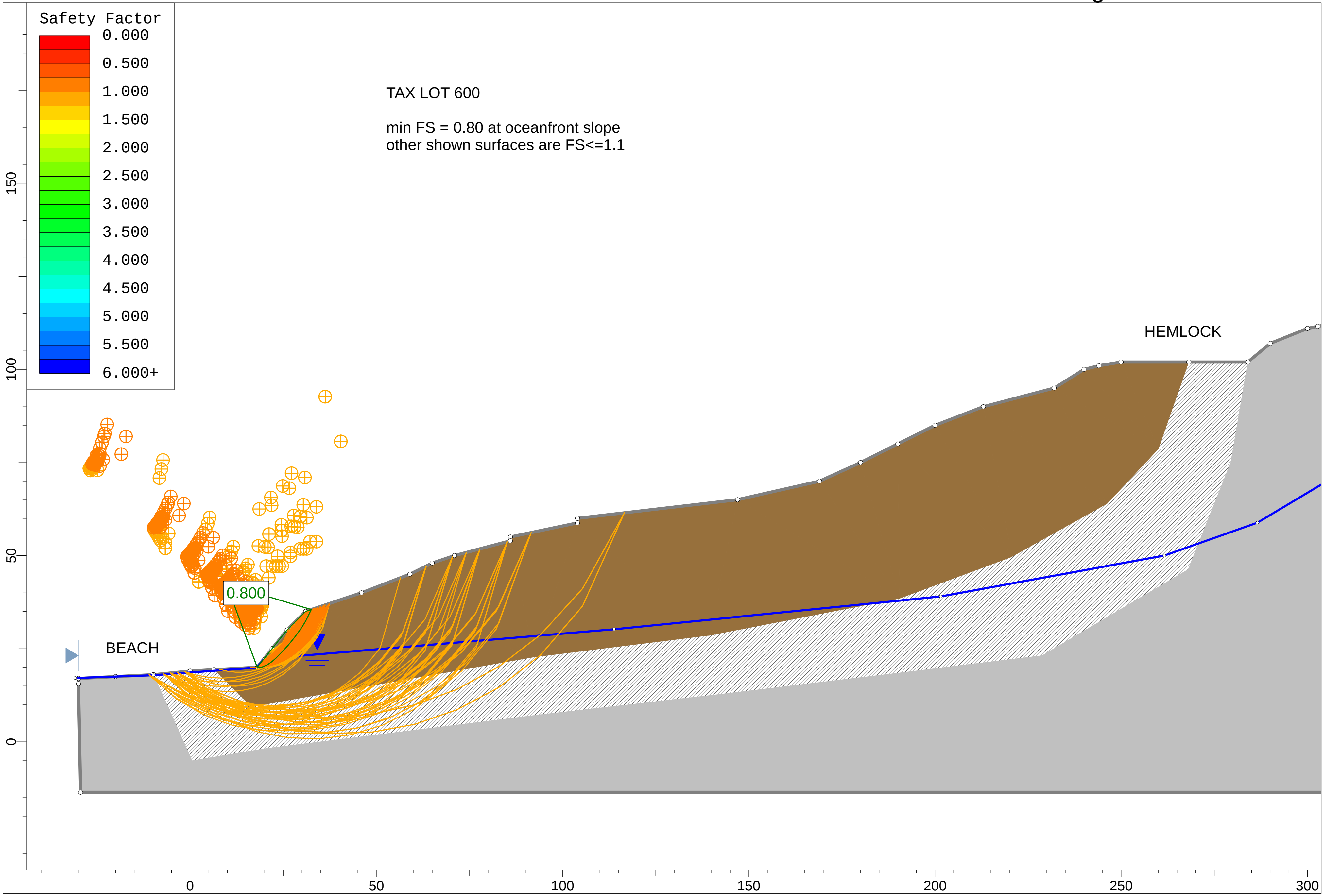
Attachments:

Site Aerial Photo with stability sections
Instability margin sketch on topo
Stability Analyses (4)
Pile Force and Concept Sketch
Geotech Solutions feasibility report
S-Curves Slide update memo
Horizontal drain layout
previous explorations by others


Pacific Dr
E Chena
SW cross section TL 600
robertscannon-9,18
critical slide section for TL600
cannon-2-S curves
Google Earth
© 2020 Google
N
200 ft

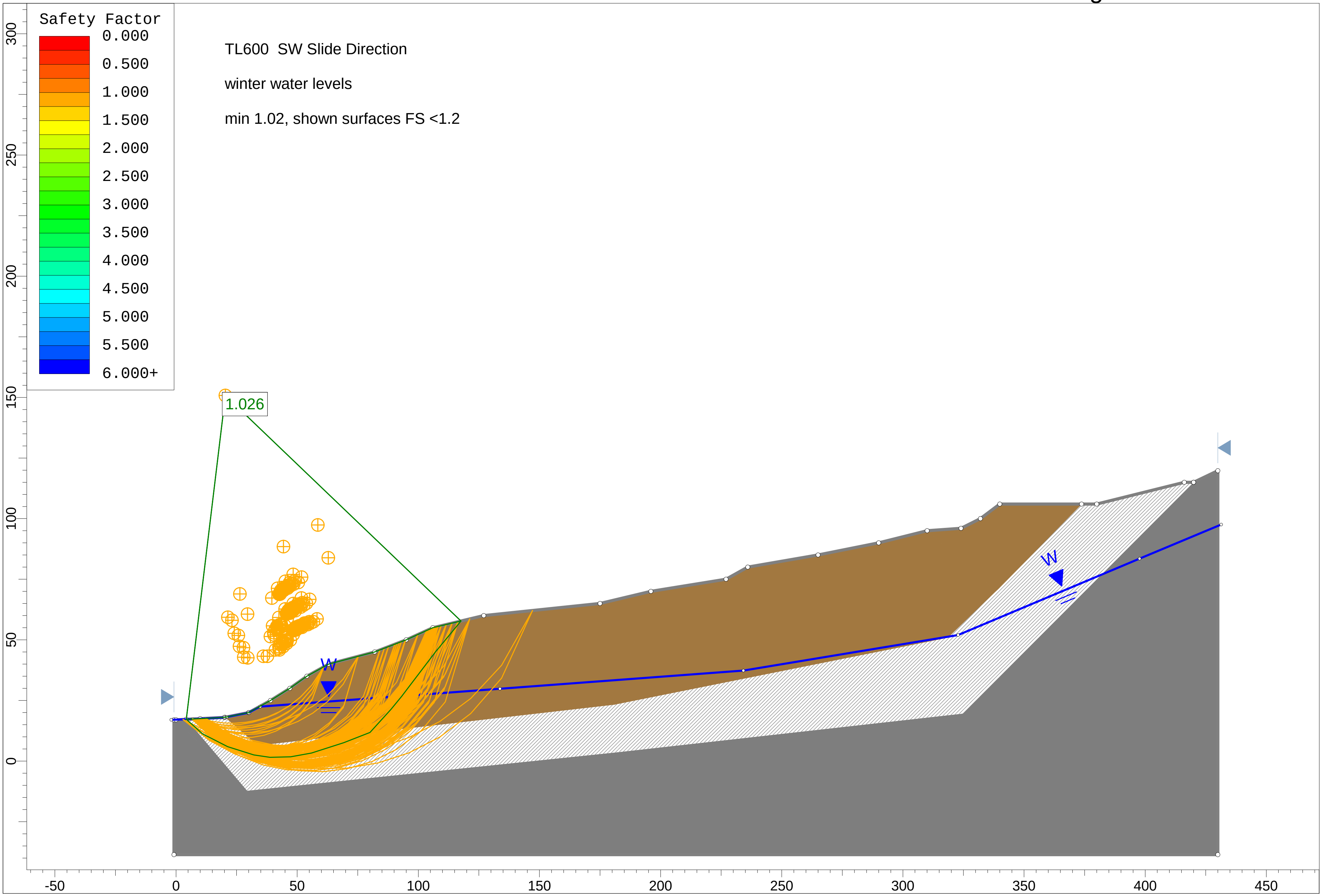


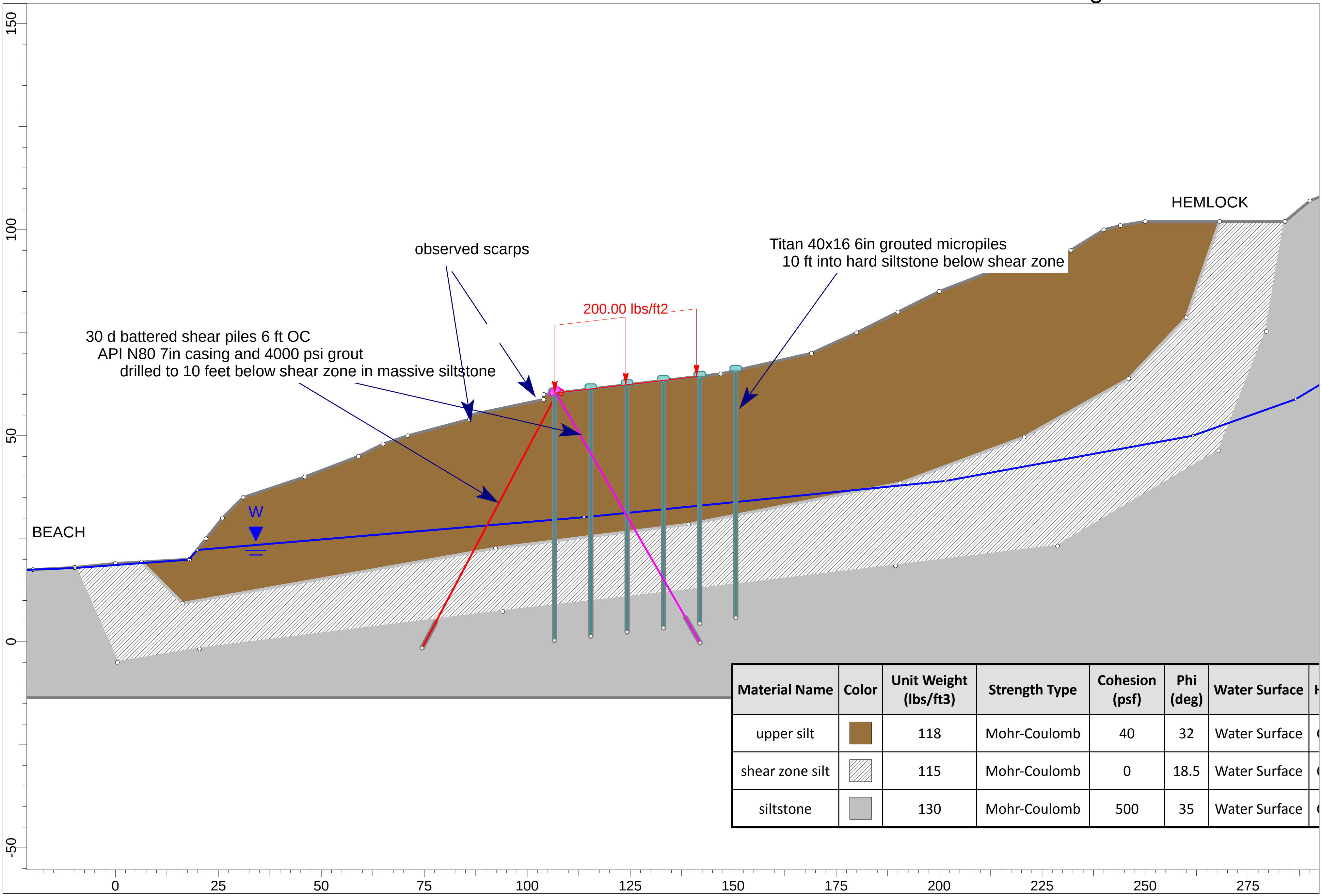
Margin of Instability



tl600-winter drained crit slope dxn.slim

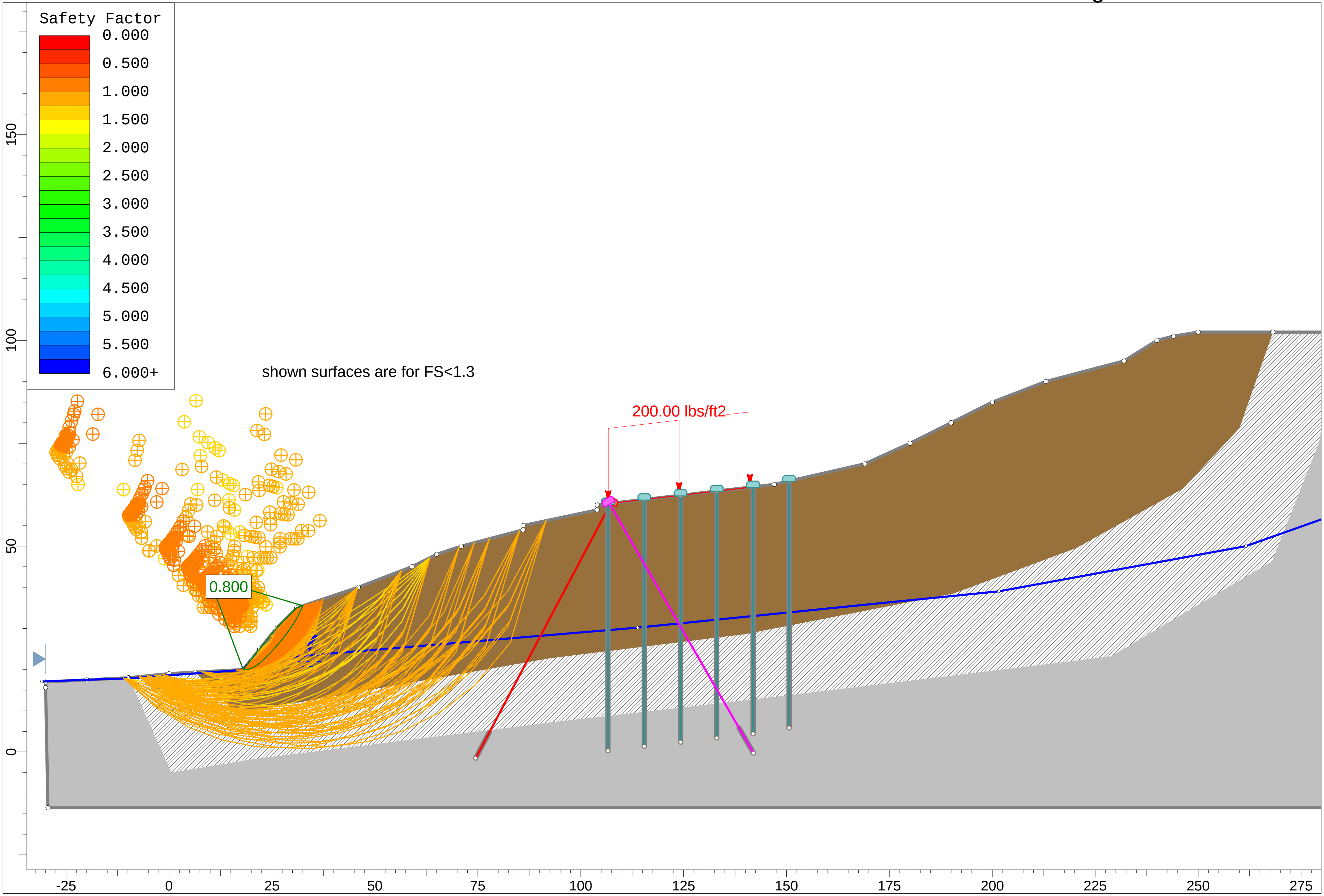
Geotech Solutions, Inc.


Safety Factor
0.000
0.500
1.000
1.500
2.000
2.500
3.000
3.500
4.000
4.500
5.000
5.500
6.000+
TL600 SW Slide Direction
winter water levels
min 1.02, shown surfaces FS <1.2
1.026
W
W
300
250
200
150
100
50
0
-50
0
50
100
150
200
250
300
350
400
450



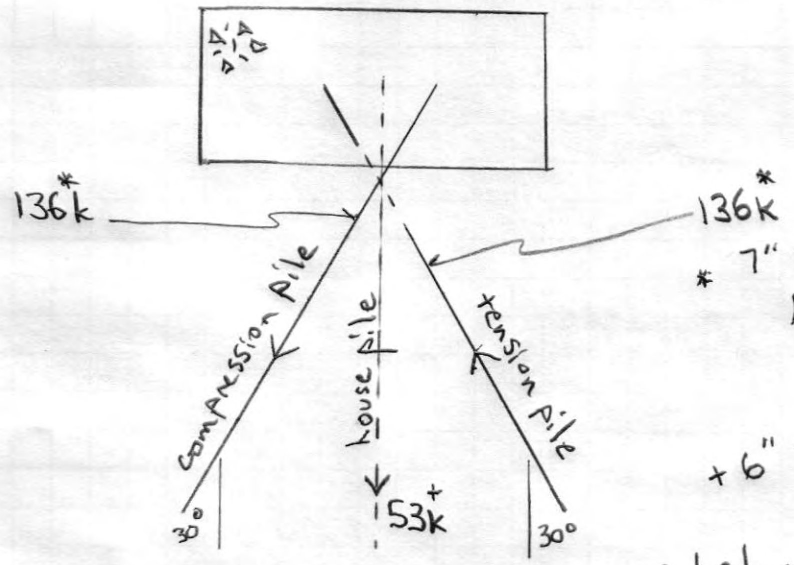
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface
upper silt	🟫	118	Mohr-Coulomb	40	32	Water Surface
shear zone silt	▨	115	Mohr-Coulomb	0	18.5	Water Surface
siltstone	⬜	130	Mohr-Coulomb	500	35	Water Surface

tl600-winter drained crit slope dxn - w 200 psf load and battered shear piles - FOR REPORT.slim


Safety Factor
0.000
0.500
1.000
1.500
2.000
2.500
3.000
3.500
4.000
4.500
5.000
5.500
6.000+
shown surfaces are for FS<1.3
200.00 lbs/ft2
0.800
150
100
50
0
-25
0
25
50
75
100
125
150
175
200
225
250
275

West Grade Beam Pile Forces

NTS

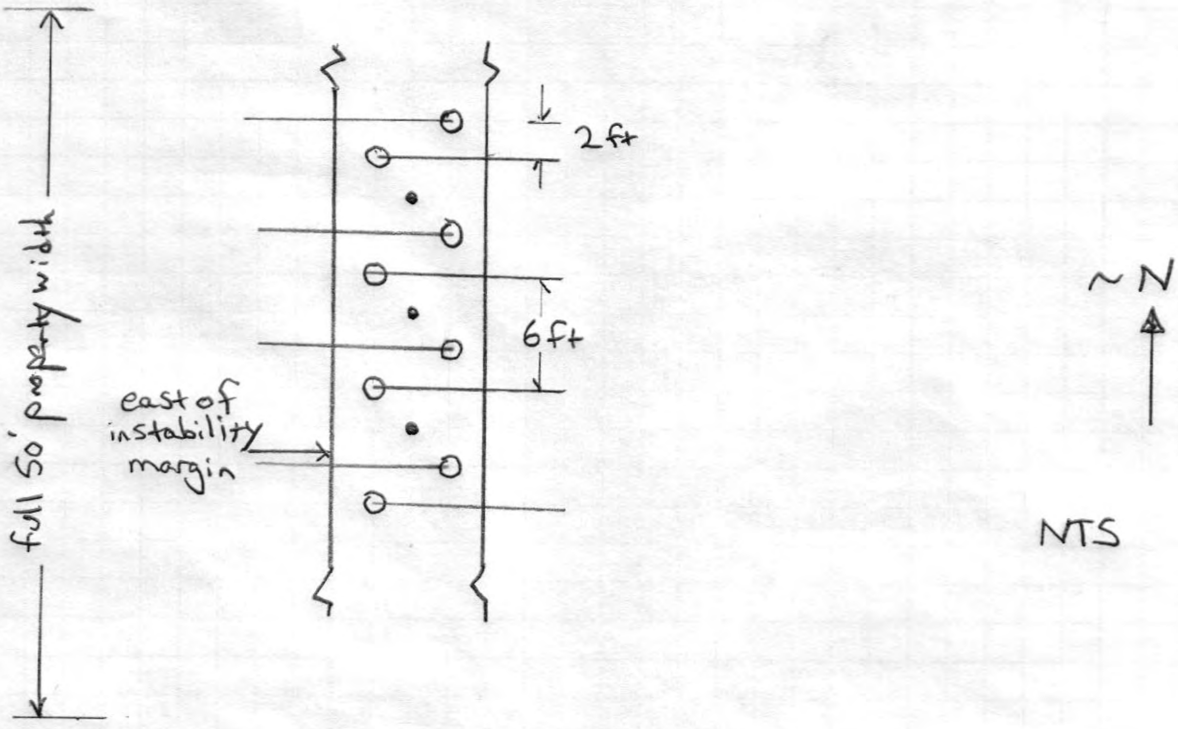


* 7" 0.45" wall grouted (4ksi) API N80 casing micropile

+ 6" titan 40/16 grouted (4ksi) micropile

embedment per report

Conceptual Layout



NTS

robertscannon-18-1-consult3

Geotech Solutions, Inc.

Geotech Solutions Inc.

July 2, 2019 *robertscannon-18-1-consult*

Stanley and Rebecca Roberts
Stan.milliman@gmail.com

Cc:
jay@jayraskinarchitect.com
rec@opusnet.com
kevin@objectiveadvisorsllc.com
plandevelopment@msn.com

GEOTECHNICAL ENGINEERING CONSULTATION
Planning Phase
Tax Lot 600, Nenana Avenue Oceanfront Lot - Cannon Beach, Oregon

Purpose and Scope

As authorized this report summarizes our geotechnical engineering consultation for the planning phase of the subject oceanfront lot located immediately north of the (unimproved) Nenana Avenue easement west of Hemlock Street in Cannon Beach, Oregon. We understand the feasibility of developing the site is to be evaluated, and our purpose was to assist in the geotechnical aspects of planning. This did not include actual foundation design recommendations and detailed stability analyses which are required for the design phase. Our specific scope of services included the following:

- Review vicinity geological and geotechnical information available in our files including recent summaries of landslide movement and our 2018 water line study.
- Review our work on the S-Curves slide to evaluate relative stability of the site and impact of stabilization efforts at the S-Curves, including movement rates and water level impacts.
- Attend up to 2 meetings as requested by the owner or architect.
- Provide a qualitative opinion on current stability condition and provide preliminary recommendations to reduce impacts to stability such as earthwork limitations and drainage requirements.
- Provide a qualitative discussion of preliminary foundation options and related considerations such as relative costs, risks and constructability.
- Provide a letter report summarizing our review, opinion of geotechnical feasibility, and preliminary options for foundation types.

Site Stability Background

The site is located within an active portion of an ancient landslide and is mapped in a geologic hazard area as mapped by the City of Cannon Beach (mapping excerpt attached). The site is part of a "down-dropped" area of the slide that is subject to storm surge wave attack. We have completed previous work on this property and adjacent properties, and have extensive work for the City of Cannon Beach in efforts to slow movement of the active portion of the slide at and above the site. That active portion has ruptured pavements on the S-curves and caused ground movement of several properties, including tax lot 600 and movement below the beach.

Mr. Rondema's involvement on this slide goes back to 1999, and Geotech Solutions previous work for the City on the S-curves slide began in 2002. That has included 6 borings up to 90- feet deep with

subsurface instruments and analyses, as well as survey monitoring for movement and acquisition of water level fluctuation data. Single event deformations were up to one foot vertically and horizontally in a west-southwest direction in response to high winter rainfall events in eroded toe conditions. In 2007 and 2008 horizontal drains were installed to reduce peak ground water levels during high rainfall events. This has significantly slowed, but not stopped, slide movement. The drains have been cleaned by the City each fall since installation, and drains flow during and after rainfall events with seasonal increases. Current slide movement has been measured near the active center at 0.3 inches in the primary shear zone in the last 6 years. Movement has been in response to high groundwater events induced by heavy rainfall storms. Most recently in 2018 we issued the attached slide movement update to the City, and in 2019 we completed work for a new water line in Hemlock Street. That water line in Hemlock is of a type of pipe and layout that can withstand some small slide movements, but is assumed to be ruptured in a CSZ earthquake event as is the sewer force main.

Risk

As stated, the site is part of an active landslide. Although movement has been slowed by horizontal drains reducing groundwater peaks in high rainfall events, this slowing is tenuous. Events that could accelerate movement include beach erosion, slope and toe erosion, new threshold rainfall events, and changes in slope loading such as cuts and fills, and site drainage. In addition, large movement is likely in earthquake ground motions from a CSZ interface earthquake (which has roughly a 30% chance of occurring in the next 50 years). Any of these issues, or a combination, could cause movement of the site that is structurally damaging. Damage could range from cracking and settlement to extensive movement and damage that requires rebuilding. The seismic motions of a CSZ interface earthquake (not to mention the subsequent tsunami impacts) would certainly result in extensive site damage and likely a loss of occupancy condition, and may render the site unusable. Because of these circumstances, in our opinion designing a structure for safe egress is the highest reasonable long term goal.

Localized ocean front slope regression is another risk, as the high bank erodes eastward to impact the building envelope. In this area of the coast regression averages roughly one foot per year, but is episodic, and may regress 10 or more feet in one year. Regression is typically more prevalent during strong southwestern storm surges and high sea level El Nino events which can coincide with total sand removal to siltstone on the beach (we observed this condition below the site in 1999, when the passive shear wedge of the slide was also visible on the beach).

Foundation Support

If the preceding risks are understood by the owner and the design team, and can be tolerated, foundation support is achievable. The types of approaches are likely limited by site access with equipment as well as high costs. We believe two approaches should be considered. A rigid reinforced structural mat supported by fixed deep foundations would be the lower risk - higher cost approach. Another approach could be a rigid mat designed for re-levelling. This has more risk of overall movement but lower initial cost, and also more risk of slope regression and utility impacts.

In any case drilling and underground work must be done when ground water levels are low with better stability, typically May through September.

Deep Foundation Supported Structural Mat - Within the site slide mass there are several rupture and movement zones at varying depths. These zones have been observed in adjacent inclinometer readings (below and next to the site), and were plotted 3 dimensionally from “communication” during

pressurized drilling/installation of horizontal drains. For foundation support to reduce overall movement these zones must be fully penetrated and the deep foundation elements designed to resist the resulting forces. The deep foundations would likely be large, heavily reinforced drilled shafts due to the high bending moments near the rupture zone interfaces. Shaft reinforcement may include W-shape beams (if they can be delivered to the site), or substantial rebar cages. Shaft size is likely limited to equipment access size and cost. Drilling will be difficult to adequately penetrate hard underlying siltstone. Special tooling as well as casing and dewatering will likely needed. The mat would need to structurally span between shafts, using grade support only for forming during construction.

Rigid mat designed for future Relevelling – A rigid mat designed to be stiff enough to accommodate relevelling is another possible option, but carries more risk. Increased risk is from distortion related damage to utilities and hardscaping, and exposure to undermining from shoreline regression. The structural engineer would need to design for significant free spans to accommodate slide grabens, as well as perimeter uplift and bending forces for relevelling. Relevelling could be done with push piers (hydraulically/reaction drive pipe piles) that are in place as part of the original construction. Reduction in regression risk could be accommodated by adding reinforced drilled shafts to the oceanfront side.

Access

The civil engineer must be consulted to design access at suitable inclinations and turning/egress. On-grade access will be difficult due to the very steep narrow roadway transition at Hemlock and the restraints to cutting and filling that may otherwise destabilize the slide. An initial estimate is that cuts must not be made in the slopes more than 2 feet deep and must be limited horizontally, and no cuts are allowed on the slope abutting Hemlock (just west of Hemlock, south of the existing "entry"). Likewise, fills would likely need to be limited to the equivalent weight of 2 feet of soil or rock. Detailed stability analyses of alternative grading sections would need to be done to better quantify these limits. For on-grade approaches a potential solution would be a near grade and pile restrained lightweight fill option on the downslope side of the entry drive. This could employ horizontally seated and connected EPS blocks shaped to desired grades. Shaped EPS for these approach inclinations may be difficult and costly, and may require a reinforced raked concrete wearing course depending on the final inclination. A viable alternative may be a pile supported structural approach and/or platform.

Drainage

Maintaining low ground water levels and limiting erosion are critical to stability. The mid-slope horizontal drain discharges for slide improvement abutting the east side of the lot complicate drainage as they will need to be accessible and maintained, with discharge collected to hard pipe. All runoff from structures and hard scaping must be collected and routed to suitable erosion protected discharge, preferably to the swale to the north if permissible.

Utility Connections

Utility connections that are designed to allow movement without damage are recommended. Such pipe connections are present in Hemlock for the sewer force main along the S-curves. Pipe with some flexibility in curved alignments can also help, such as the new water line in Hemlock. Again the civil engineer should be consulted on these options.

Limitations

We have prepared this report for use by Stanley Roberts and members of the planning team for this project only. The preceding recommendations should be considered preliminary, as actual soil conditions may vary. The information herein could be used for planning purposes but should not be construed as a warranty of surface or subsurface conditions. We have made observations only from the aforementioned information. These observations do not reflect soil types, strata thicknesses, water levels, seepage or stability conditions that may exist between observations, or after the present time. We must be consulted to complete stability and foundation support analyses design for any structures, as well as observe actual conditions encountered during construction in order for our recommendations to be final. Our observations will allow us to interpret actual conditions and adapt our recommendations if needed. Within the limitations of scope, schedule and budget, our services have been executed in accordance with the generally accepted practices in this area at the time this report was prepared. No warranty, expressed or implied, is given.

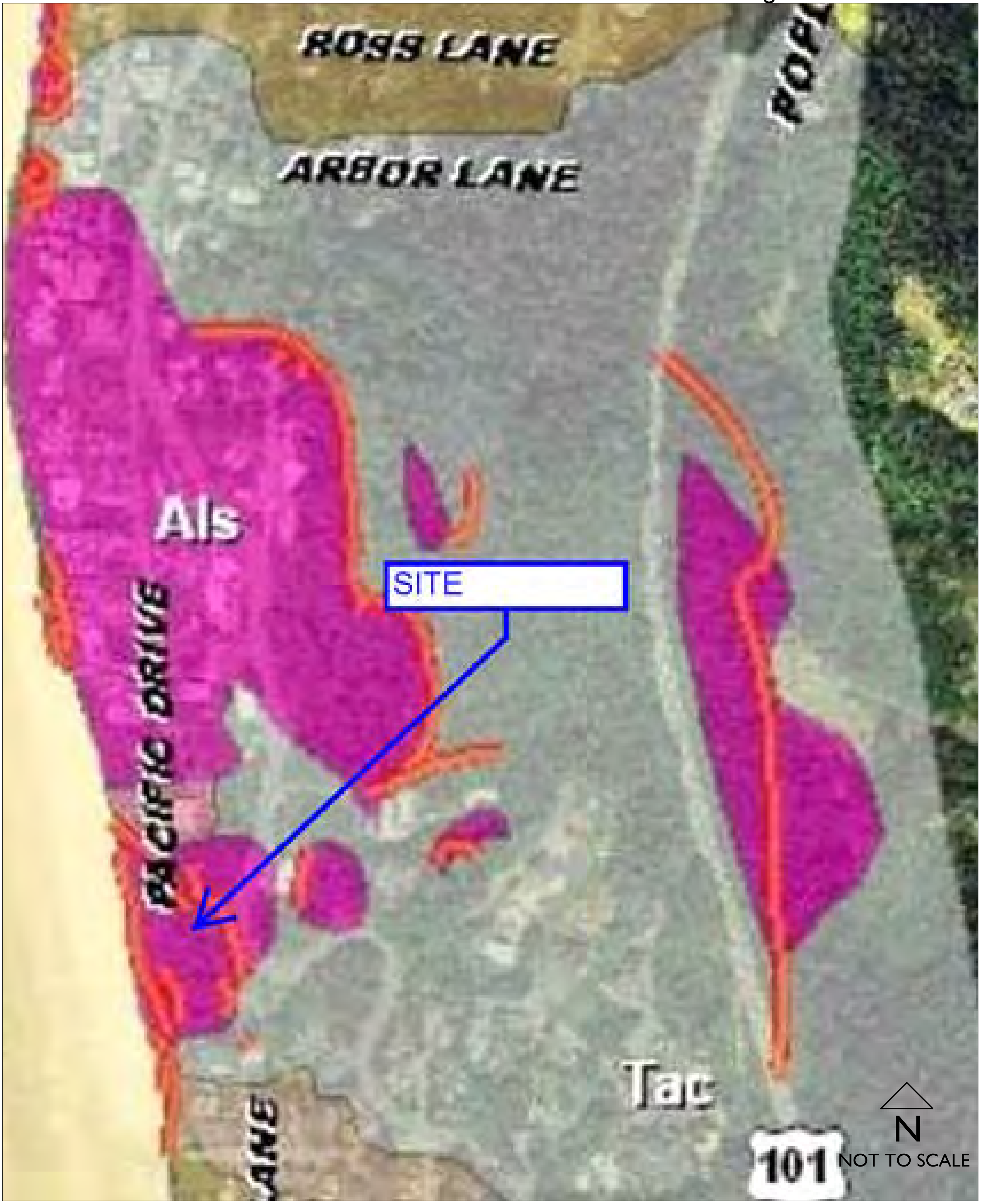
◆

We appreciate the opportunity to work with you on this project and look forward to our continued involvement. Please call if you have questions.

Sincerely,

Don Rondema, MS, PE, GE
Principal

REGISTERED PROFESSIONAL ENGINEER
OREGON
JULY 21, 1992
DON RONDEMA
Expires 12/31/20

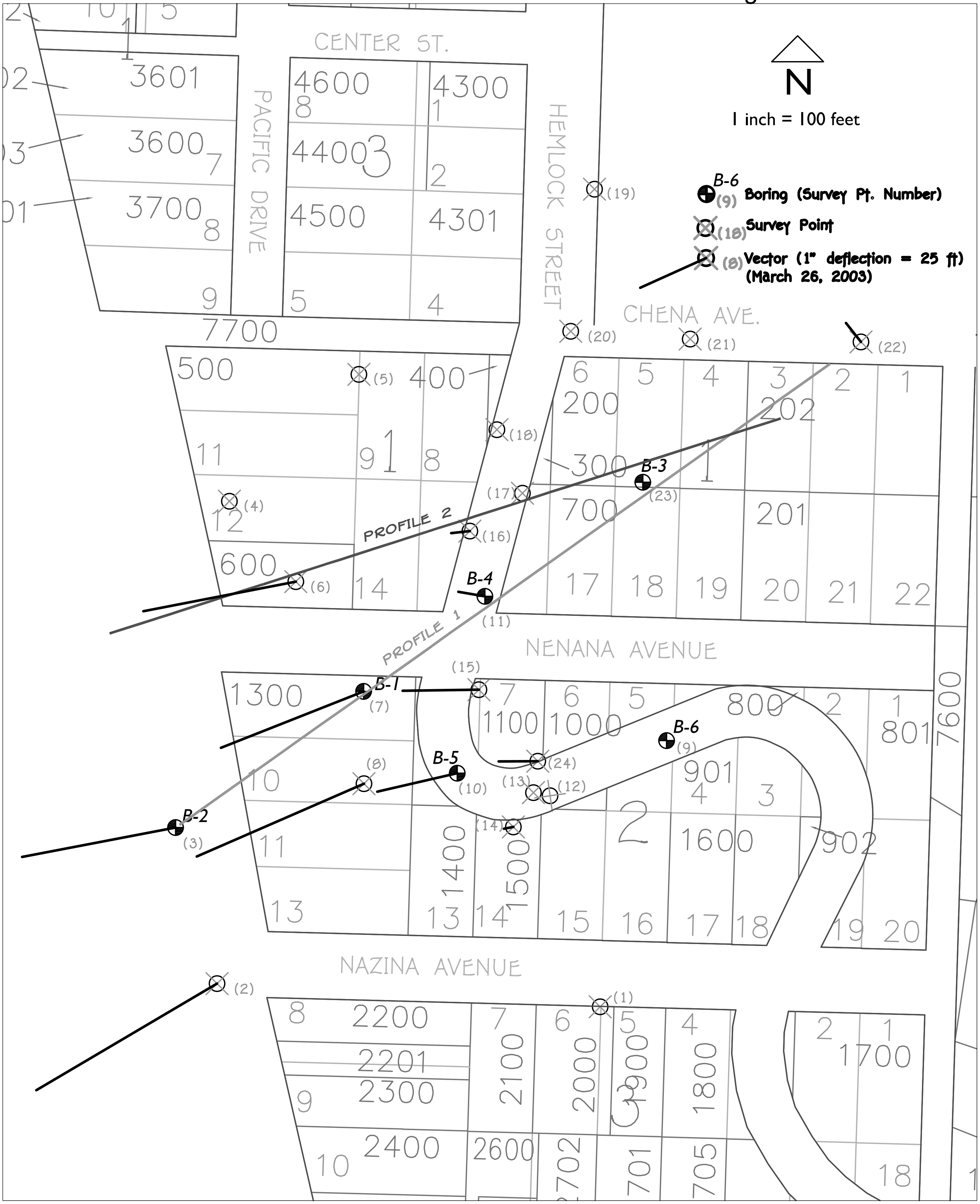


base from DOGAMI O-90-06

Geotech Solutions Inc.

SITE GEOLOGICAL MAPPING

robertscannon-18-1-consult



Geotech Solutions Inc.

SITE PLAN

Cannon-02-01-gi

Geotech Solutions Inc.

MEMORANDUM

cannon-18-1-consult

To: Karen LaBonte, Public Works Director, City of Cannon Beach; labonte@ci.cannon-beach.or.us

Date: June 26, 2018

Subject: **Hemlock Street S-Curves Slide: Status Update**

Introduction and Background

This memorandum provides an update to the status of the inclinometer data from the S-curves slide as read on June 23, 2018. The previous last reading was in 2015. The reason for this reading was a centerline crack appearing in the last month or so near the apex of the curve above and slightly south of the B-1r instrument. This crack is roughly 10-15 feet in length, and open up to roughly ¼" with perhaps a slight vertical offset down to the west. In addition, and perhaps relevant to tangential slide restraint and equilibrium, slope cuts and net mass removal has occurred on an adjacent project over roughly the past year. That project abuts previous lateral shear zones observed at the southern portion of the active slide.

The water levels in the slide are no longer being recorded as the instruments have expired, and new winter storm rainfall levels had not exceeded those previously recorded. The data attached are inclinometer readings for only one instrument near the center of the slide (B-1r) which has been shown over many years to correlate well with rainfall response and water levels and other previous movement in other, now irrevocably damaged, casings. It should be understood that this correlation is in the context of the general beach/slide toe elevations and erosion conditions experienced since 2008.

Conclusions and Recommendations

Roughly 0.2 inches of movement has occurred above/near the primary shear surface since the last reading roughly 3 years ago. The previous 3 years had roughly 0.1 inches of movement. Overall readings show a total of roughly 2.5 inches of movement on this replacement casing. A plot is attached. This movement is not out of recorded context movement rates for the slide.

Based on our site observations, in our opinion the surface cracking is not discernible from an aging panel joint or related thermal separation crack. It is possible that the crack was caused by accumulated underlying movement of the slide and is exhibiting at the previously placed grid overlap joint, but it does not coincide with previous slide induced crack locations which trended southwesterly with vertical offsets greater than horizontal, and at locations north and south of this crack location.

Although B-1r is approaching its deflection life, it is still functional and in our opinion does not need replacement at this time. Replacement/redrilling for a new casing (including initial baseline readings) is estimated at roughly $10,000 as access is difficult. If additional cracking occurs that is more indicative of slide movement, then a new water level logger is recommended for the paired B-1 standpipe (P-1).

Based on our current monitoring, we still expect movement of the S-Curves to be ongoing. However, the reduction in ground water levels and movement in large rainfall events has been greatly reduced by the functional horizontal drains compared to historical observations. No measures in addition to frequent roadway surface observation and annual drain cleaning are recommended at this time.

Provided the existing drains are maintained and cleaned annually and are functional, it is our opinion that they are sufficient to continue to slow the slide for the rainfall event intensities experienced since drain installation. Exceptions would be from earthquake ground motions or significant beach toe erosion. Any significant beach level erosion (such as exposure of siltstone below the sand similar to the El Nino cycle of 1999), or toe slumping, would be cause to take inclinometer readings, as would experiencing a new threshold rainfall event. These would be anything in excess of the storm events recorded since drain installation which are 4.37”-1 day, 6.26”-2day, 6.29”-3day, or 10.21”-5day. Please alert us if any of these thresholds are met.

The Limitations of our report apply, and that report and a few predrain install crack photos are attached here for background.

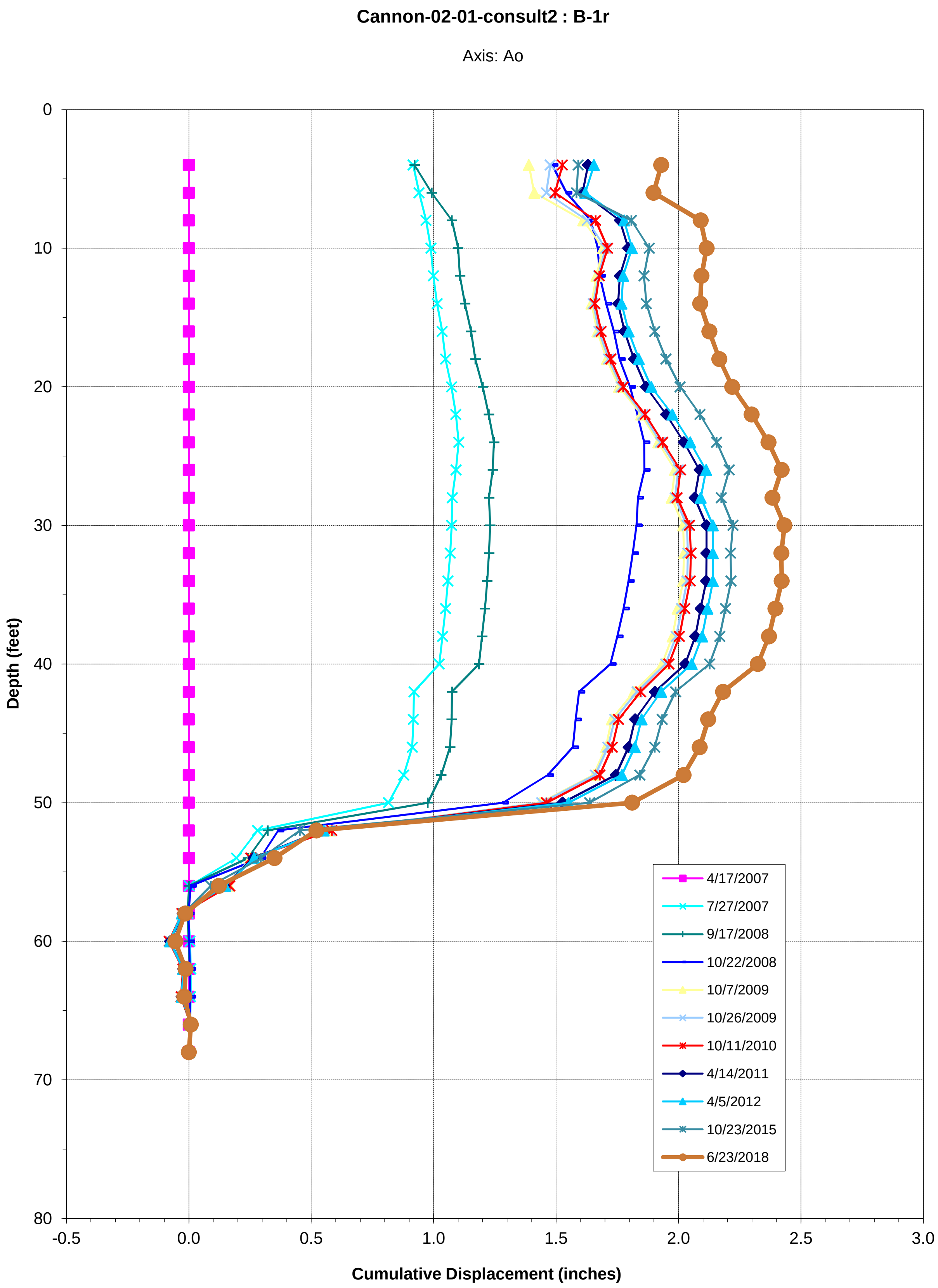
◆

We appreciate the opportunity to work with you on this project and look forward to our continued involvement. Please call if you have questions.

Sincerely,

Don Rondema, MS, PE, GE
Principal

DON RONDEMA
Expires 12/31/18


Cannon-02-01-consult2 : B-1r
Axis: Ao
Depth (feet)
0
10
20
30
40
50
60
70
80
-0.5
0.0
0.5
1.0
1.5
2.0
2.5
3.0
Cumulative Displacement (inches)
4/17/2007
7/27/2007
9/17/2008
10/22/2008
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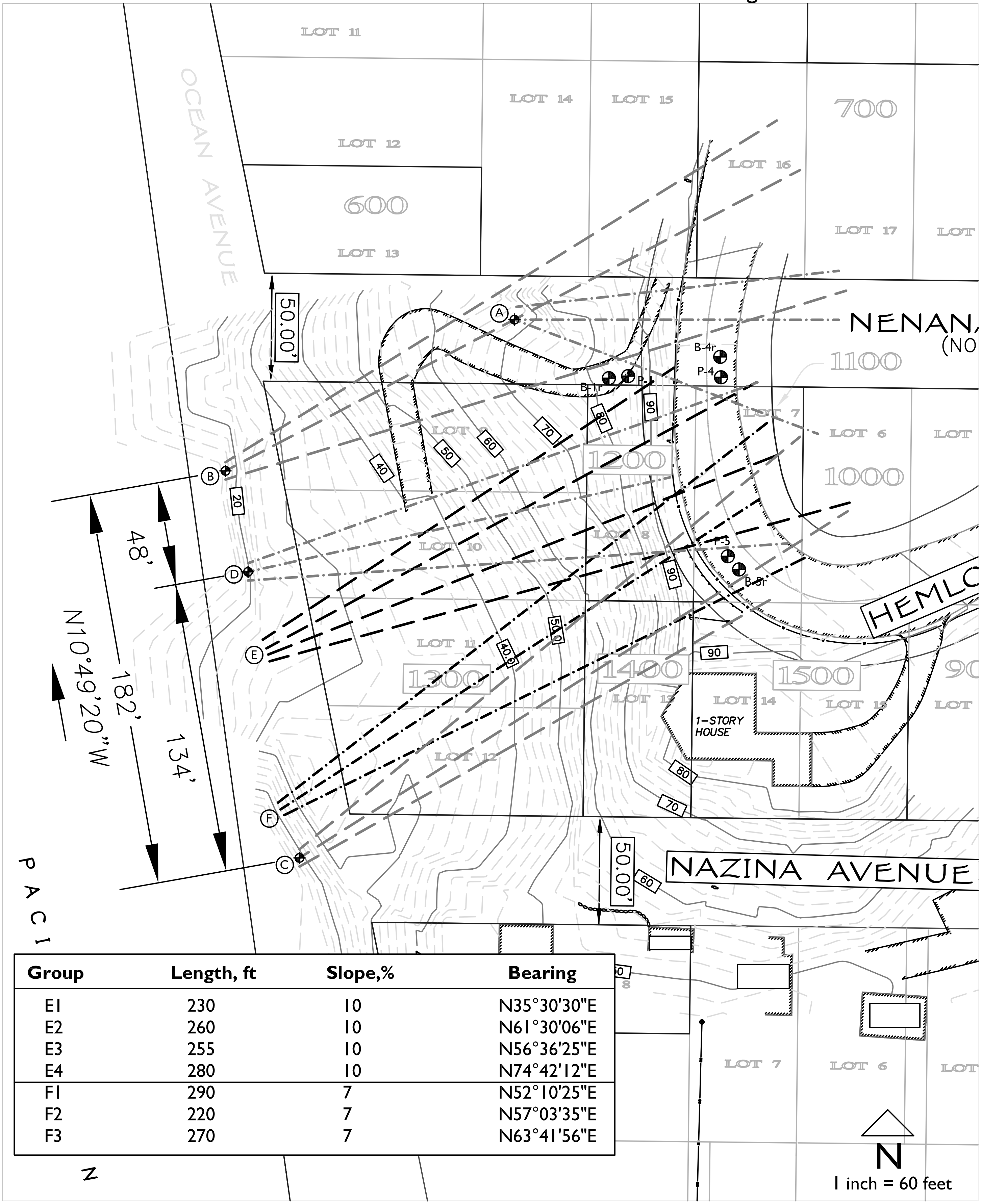
4/5/2012
10/23/2015
6/23/2018



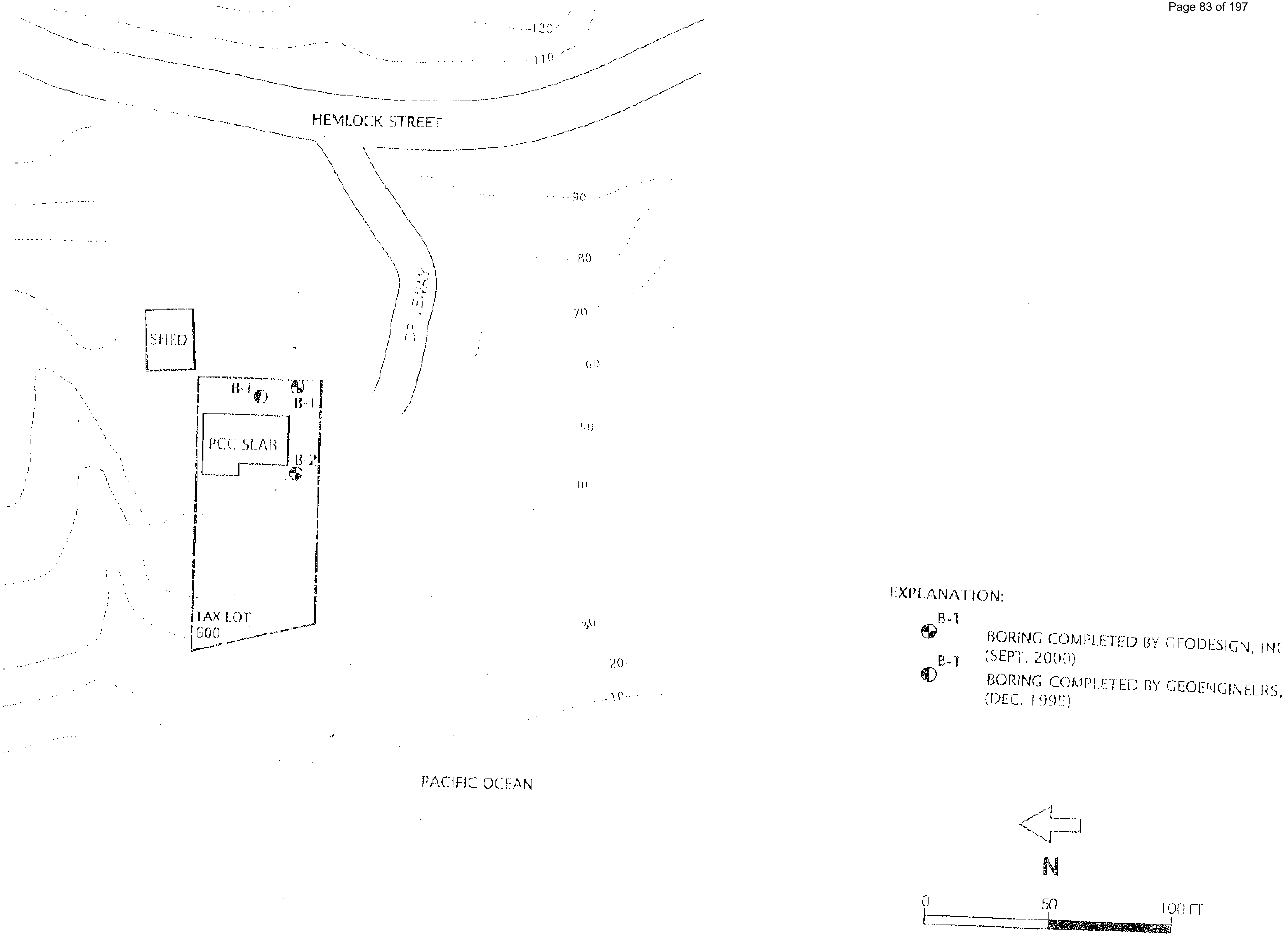
Group	Length, ft	Slope,%	Bearing
E1	230	10	N35°30'30"E
E2	260	10	N61°30'06"E
E3	255	10	N56°36'25"E
E4	280	10	N74°42'12"E
F1	290	7	N52°10'25"E
F2	220	7	N57°03'35"E
F3	270	7	N63°41'56"E

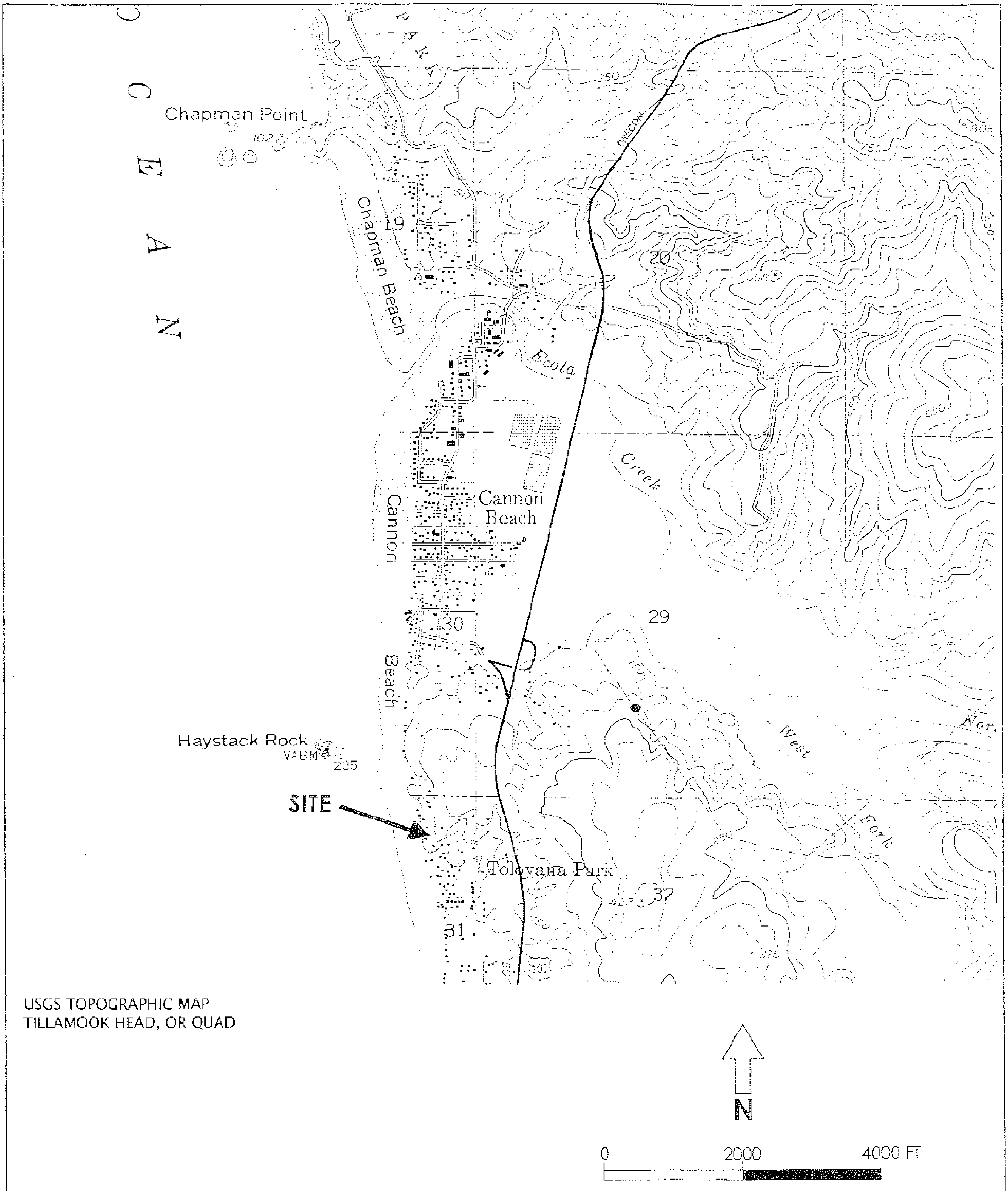
BASE DRAWING PROVIDED BY HLB-OTAK, IINC.

Geotech Solutions Inc.

SITE PLAN

cannon-02-01-consult2


120
110
HEMLOCK STREET
90
80
70
60
50
40
30
20
10
SHED
B-1
B-1
PCC SLAB
B-2
TAX LOT 600
PACIFIC OCEAN
EXPLANATION:
B-1
BORING COMPLETED BY GEODESIGN, INC. (SEPT. 2000)
B-1
BORING COMPLETED BY GEOENGINEERS, (DEC. 1995)
N
0
50
100 FT



GEODESIGN INC

VICINITY MAP

BERTMAN-2	FEBRUARY 2001	FIGURE 1

APPENDIX A

FIELD EXPLORATIONS

We explored subsurface conditions at the site by advancing two borings (B-1 and B-2) at the approximate locations shown in Figure 2. Geo-Tech Explorations of Tualatin, Oregon, drilled the borings using a track-mounted drill rig equipped with mud rotary methods to depths of up 70.0 feet in on September 20 and 21, 2000.

We determined the exploration locations in the field from existing site features. The locations shown on Figure 2 should be considered approximate. A qualified member of GeoDesign's staff observed and documented all field activities.

We obtained representative samples of the various soils encountered for geotechnical laboratory testing. Classifications and sampling intervals are shown on the logs included in this appendix.

We classified the materials present in the samplers in the field in accordance with the "Key to Test Pit and Boring Log Symbols," "Soil Classification System and Guidelines," and "Rock Classification Guidelines," copies of which are included in this appendix. The boring logs indicate the depths at which the soils or their characteristics change, although the change actually may be gradual. If the change occurred between sample locations, the depth was interpreted.

LABORATORY TESTING

We classified soil samples in the laboratory to confirm field classifications. The laboratory classifications are included in the boring logs if those classifications differed from the field classifications.

We tested the natural moisture content of selected soil samples in general accordance with guidelines presented in ASTM D 2216. The moisture contents are included in the boring logs in this appendix.

We also completed unconfined compression testing of plaster capped siltstone cores of select samples. Results of this testing are attached.

KEY TO TEST PIT AND BORING LOG SYMBOLS

SYMBOL	SOIL DESCRIPTION
▮	Location of sample obtained in general accordance with ASTM D 1586 Standard Penetration Test
▯	Location of SPT sampling attempt with no sample recovery
◪	Location of sample obtained using thin wall, shelby tube, or Geoprobe® sampler in general accordance with ASTM D 1587
⧄	Location of thin wall, shelby tube, or Geoprobe® sampling attempt with no sample recovery
▦	Location of sample obtained using Dames and Moore sampler and 300 pound hammer or pushed
▤	Location of Dames and Moore sampling attempt (300 pound hammer or pushed) with no sample recovery
☒	Location of grab sample
▓	Rock Coring Interval
▽	Water level

GEOTECHNICAL TESTING EXPLANATIONS

PP	Pocket Penetrometer	LL	Liquid Limit
TOR	Torvane	PI	Plasticity Index
CONSOL	Consolidation	PCF	Pounds Per Cubic Foot
DS	Direct Shear	PSF	Pounds Per Square Foot
P200	Percent Passing U.S. No. 200 Sieve	TSF	Tons Per Square Foot
W	Moisture Content	P	Pushed Sample
DD	Dry Density	OC	Organic Content

ENVIRONMENTAL TESTING EXPLANATIONS

CA	Sample Submitted for Chemical Analysis	ND	Not Detected
PID	Photoionization Detector Headspace Analysis	NS	No Visible Sheen
PPM	Parts Per Million	SS	Slight Sheen
MG/KG	Milligrams Per Kilogram	MS	Moderate Sheen
P	Pushed Sample	HS	Heavy Sheen

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KEY TO TEST PIT AND
BORING LOG SYMBOLS

TABLE A-1

SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			SYMBOL	NAME
Coarse Grained Soils More than 50% retained on No. 200 Sieve	Gravel More than 50% of coarse fraction retained on No. 4 Sieve	Clean Gravel	GW	Well graded, fine to coarse gravel
			GP	Poorly graded gravel
		Gravel with Fines	GM	Silty gravel
			GC	Clayey gravel
	Sand More than 50% of coarse fraction passes No. 4 Sieve	Clean Sand	SW	Well graded, fine to coarse sand
			SP	Poorly graded sand
		Sand with Fines	SM	Silty sand
			SC	Clayey sand
Fine Grained Soils More than 50% passes No. 200 Sieve	Silt and Clay Liquid Limit less than 50%	Inorganic	ML	Low plasticity silt
			CL	Low plasticity clay
		Organic	OL	Organic silt, organic clay
	Silt and Clay Liquid Limit greater than 50%	Inorganic	MH	High plasticity silt
			CH	High plasticity clay, fat clay
		Organic	OH	Organic clay, organic silt
Highly Organic Soils			PT	Peat

SOIL CLASSIFICATION GUIDELINES

GRANULAR SOILS	
Relative Density	**Standard Penetration Resistance**
Very Loose	0 - 4
Loose	4 - 10
Medium Dense	10 - 30
Dense	30 - 50
Very Dense	More than 50

COHESIVE SOILS		
Consistency	**Standard Penetration Resistance**	**Unconfined Compressive Strength (tsf)**
Very Soft	Less than 2	Less than 0.25
Soft	2 - 4	0.25 - 0.50
Medium Stiff	4 - 8	0.50 - 1.0
Stiff	8 - 15	1.0 - 2.0
Very Stiff	15 - 30	2.0 - 4.0
Hard	More than 30	More than 4.0

GRAIN SIZE CLASSIFICATION

Boulders	12 - 36 inches
Cobbles	3 - 12 inches
Gravel	¾ - 3 inches (coarse) ¼ - ¾ inches (fine)
Sand	No. 10 - No. 4 Sieve (coarse) No. 10 - No. 40 Sieve (medium) No. 40 - No. 200 Sieve (fine)

Subclassifications	
	Percentage of other material in sample
Clean	0 - 2
Trace	2 - 10
Some	10 - 30
Sandy, Silty, Clayey, etc.	30 - 50

Dry = very low moisture, dry to the touch; Moist = damp, without visible moisture; Wet = saturated, with visible free water.

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SOIL CLASSIFICATION SYSTEM
AND GUIDELINES

TABLE A-2

ROCK CLASSIFICATION GUIDELINES

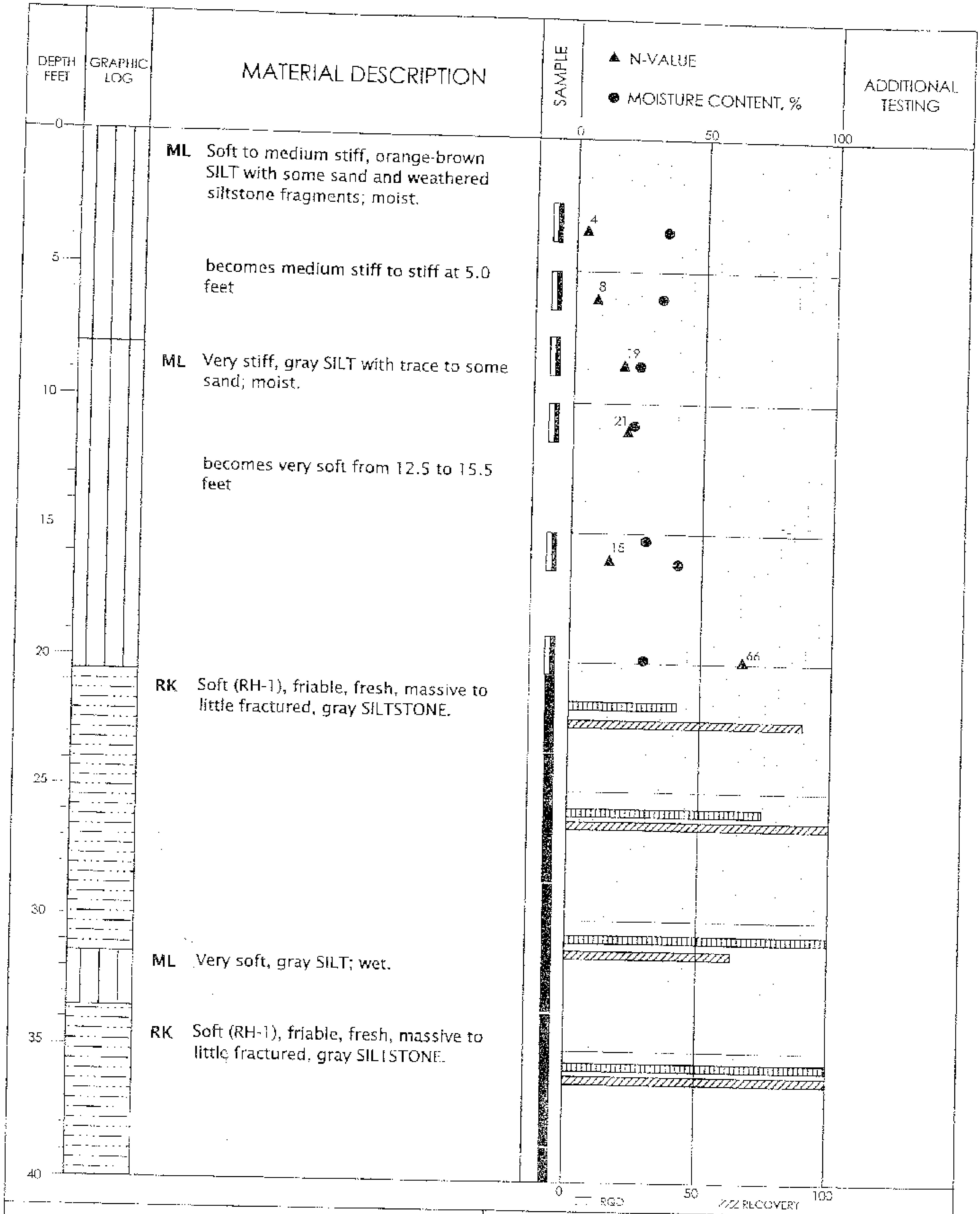
HARDNESS	DESCRIPTION
Very soft (RH-0)	For plastic material only
Soft (RH-1)	Carved or gouged with a knife
Moderate (RH-2)	Scratched with a knife
Hard (RH-3)	Difficult to scratch with a knife
Very hard (RH-4)	Rock scratches metal; rock cannot be scratched with a knife

STRENGTH	DESCRIPTION
Plastic	Easily deformable with finger pressure
Friable	Crumbles by rubbing with fingers
Weak	Crumbles only under light hammer blows
Moderately Strong	Few heavy hammer blows before breaking
Strong	Withstands few heavy hammer blows and yields large fragments
Very Strong	Withstands many heavy hammer blows, yields dust and small fragments

WEATHERING	DESCRIPTION
Severe	Rock decomposed; thorough discoloration; all fractures extensively coated with clay, oxides, or carbonates
Moderate	Intense localized discoloration of rock; fracture surfaces coated with weathering minerals
Little	Slight and intermittent discoloration of rock; few stains on fracture surfaces
Fresh	Rock unaffected by weathering

FRACTURING	FRACTURE SPACING
Crushed	Less than 5/8 inch to contains clay
Highly Fractured	5/8 inch to 2 inches
Closely Fractured	2 inches to 6 inches
Moderately fractured	6 inches to 1 foot
Little Fractured	1 foot to 4 feet
Massive	Greater than 4 feet

JOINT SPACING	DESCRIPTION
Papery	Less than 1/8 inch
Shaley or Platey	1/8 inch to 5/8 inch
Very Close	5/8 inch to 3 inches
Close	3 inches to 2 feet
Blocky	2 to 4 feet
Massive	Greater than 4 feet



ML Soft to medium stiff, orange-brown SILT with some sand and weathered siltstone fragments; moist.

becomes medium stiff to stiff at 5.0 feet

ML Very stiff, gray SILT with trace to some sand; moist.

becomes very soft from 12.5 to 15.5 feet

RK Soft (RH-1), friable, fresh, massive to little fractured, gray SILTSTONE.

ML Very soft, gray SILT; wet.

RK Soft (RH-1), friable, fresh, massive to little fractured, gray SILTSTONE.

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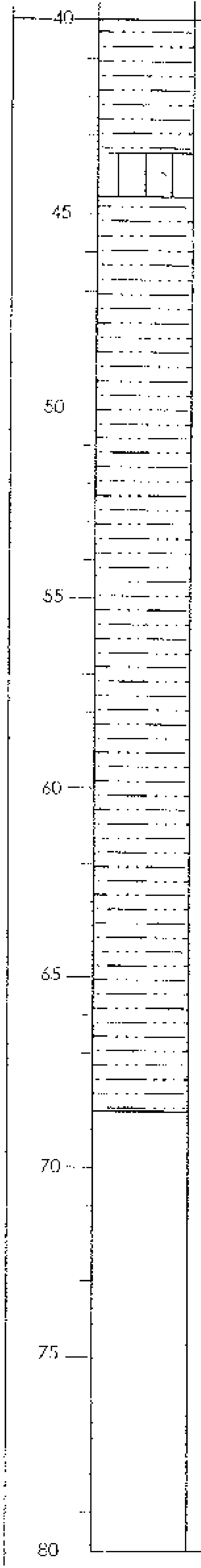
BORING B-1

BERTMAN-2	FEBRUARY 2001	FIGURE A-1

DEPTH FEET | GRAPHIC LOG | MATERIAL DESCRIPTION | SAMPLE | ▲ N-VALUE | ● MOISTURE CONTENT, % | ADDITIONAL TESTING

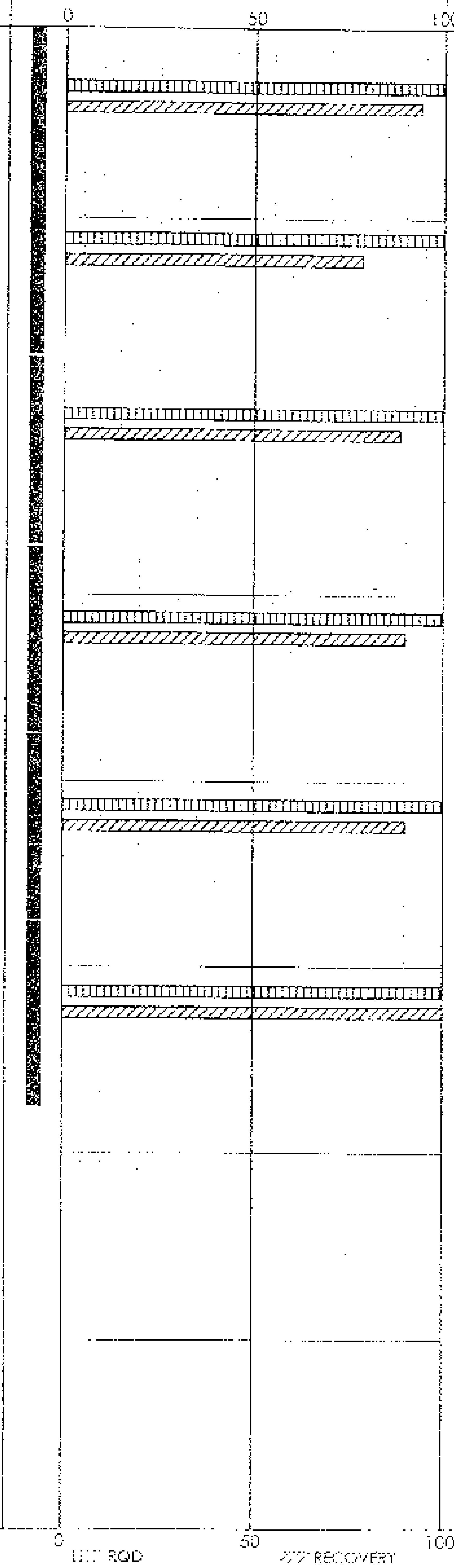


ML Very soft, gray SILT; wet.

RK Soft (RH-1), friable, fresh, massive to little fractured, gray SILTSTONE.

Boring completed at 68.5 feet on September 20, 2000.

Inclinometer casing installed to full depth with above ground monument.



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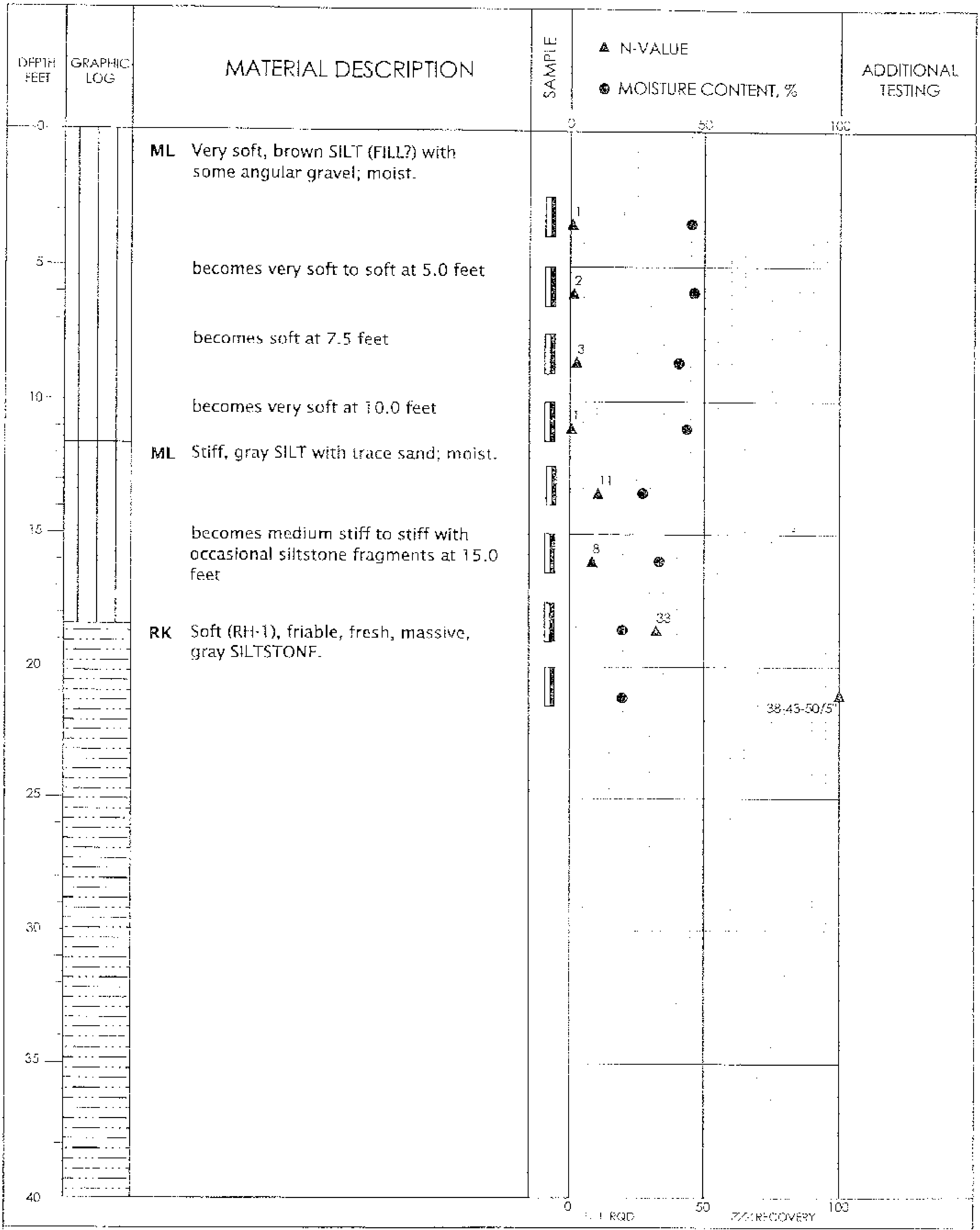
BORING B-1 (CONT.)

BERTMAN-2

FEBRUARY 2001

FIGURE A-1



ML Very soft, brown SILT (FILL?) with some angular gravel; moist.

becomes very soft to soft at 5.0 feet

becomes soft at 7.5 feet

becomes very soft at 10.0 feet

ML Stiff, gray SILT with trace sand; moist.

becomes medium stiff to stiff with occasional siltstone fragments at 15.0 feet

RK Soft (RH-1), friable, fresh, massive, gray SILTSTONE.

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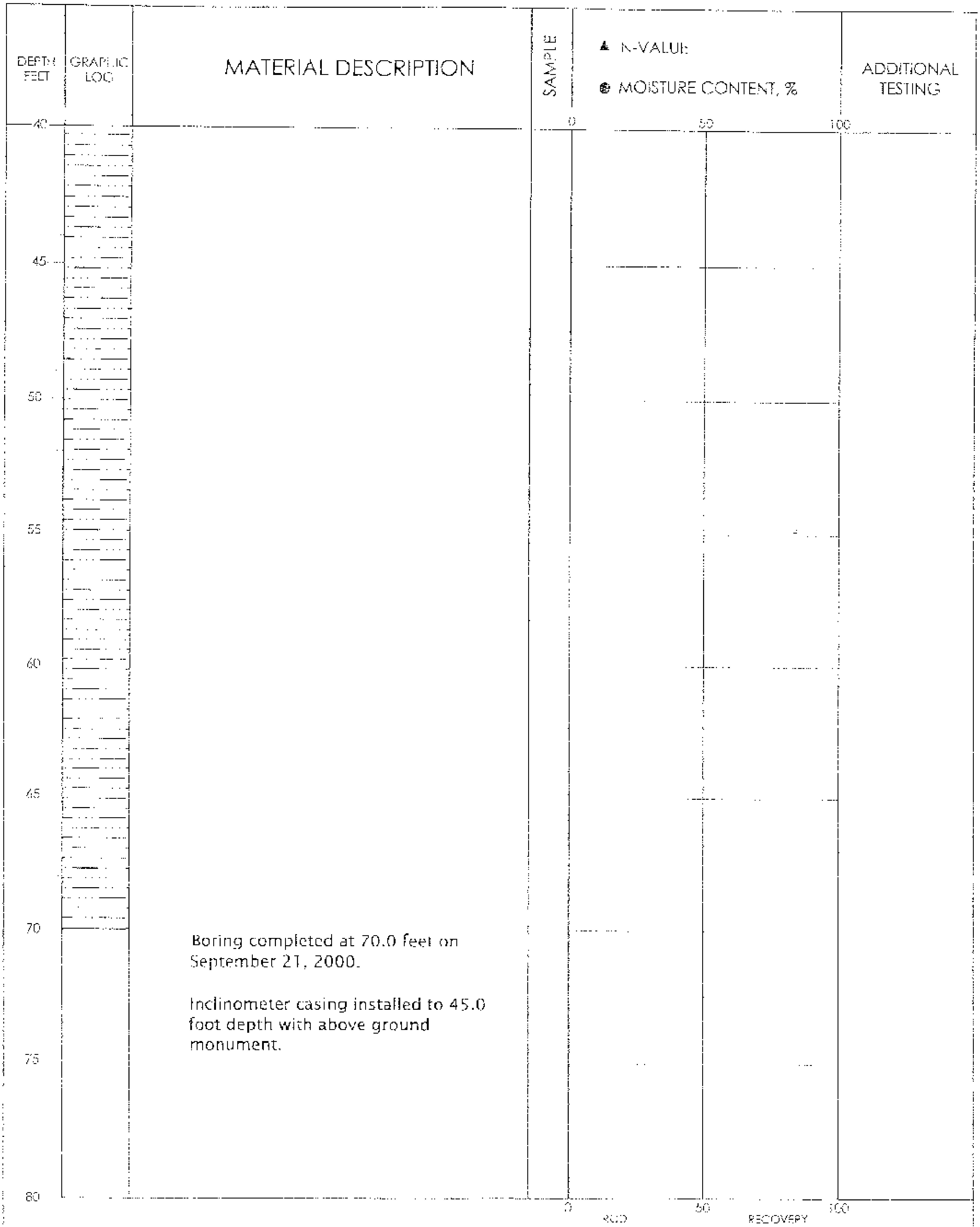
BORING B-2

BERTMAN-2

FEBRUARY 2001

FIGURE A 2



Boring completed at 70.0 feet on September 21, 2000.

Inclinometer casing installed to 45.0 foot depth with above ground monument.

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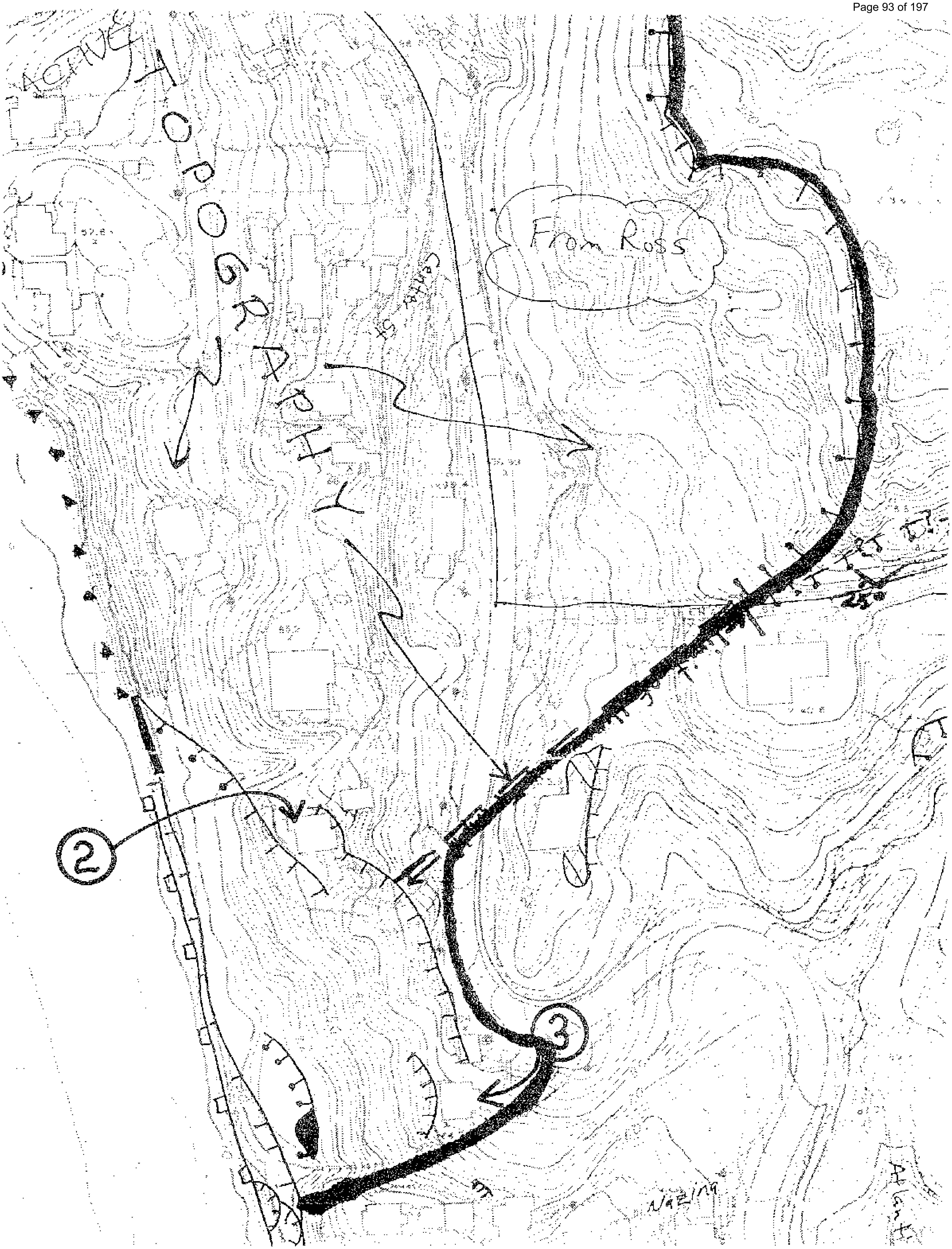
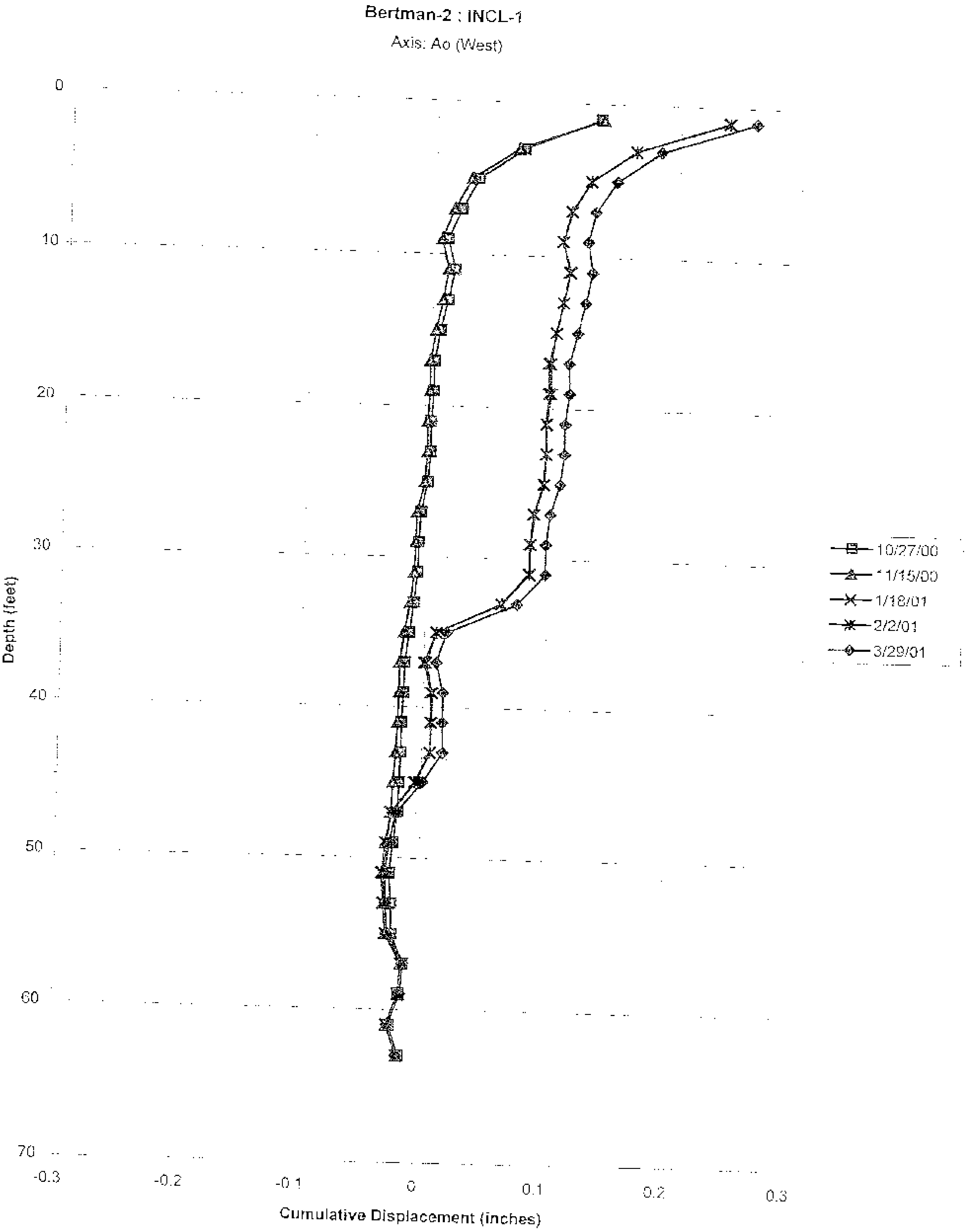
BORING B-2 (CONT.)

BERTMAN-2	FEBRUARY 2001	FIGURE A-2


ACTIVE
TOPOGRAPHY
From Ross
Cntr 18+45
2
3
1/42/19
Alan F.


Bertman-2 : INCL-1
Axis: Ao (West)
Depth (feet)
0
10
20
30
40
50
60
70
-0.3
-0.2
-0.1
0
0.1
0.2
0.3
Cumulative Displacement (inches)
10/27/00
11/15/00
1/18/01
2/2/01
3/29/01

Earth Engineers, Inc.
2411 Southeast 8th Avenue ● Camas ● WA 98607
Phone: 360-567-1806 ● Fax: 360-253-8624
www.earth-engineers.com

June 22, 2020
Revised June 30, 2020

Stanley Roberts
925 Lake Street South
Apartment No. 201
Kirkland, Washington 98033

Phone: 206-465-4220
E-mail: stan.milliman@gmail.com

Subject: **Geotechnical Investigation Report**
Proposed Nenana Avenue and Tax Lot 600 Private Driveway Construction
Cannon Beach, Clatsop County, Oregon
EEI Report No. 20-014-1-R1

Dear Mr. Roberts:

Earth Engineers, Inc. (EEI) is pleased to transmit our revised Geotechnical Investigation Report for the above referenced project. The attached report includes the results of field investigation and laboratory testing, an evaluation of geotechnical factors that may influence the proposed construction, recommendations for roadway structure design, as well as recommendations for general site development.

We appreciate the opportunity to perform this geotechnical study and look forward to continued participation during the design and construction phases of this project. If you have any questions pertaining to this report, or if we may be of further service, please contact our office.

Respectfully submitted,
Earth Engineers, Inc.

Anita Bauer

Troy Hull, P.E., G.E.
Principal Geotechnical Engineer

Anita Bauer
Geologic Associate

Attachment: Geotechnical Investigation Report

Distribution (electronic copy only):
Addressee
Kevin Patrick (Kevin@objectiveadvisorsllc.com)
Sabrina Pearson, Plan Development LLC (plandevelopment@msn.com)
Jason Morgan, Morgan Civil Engineering (jason@morgancivil.com)
Eric Watson, Miller Consulting Engineers (eric@miller-se.com)
Rich Elstrom, Rich Elstrom Construction (rec@opusnet.com)
Don Rondema, Geotech Solutions (don@geotechsolutionsinc.com)
Jorge Castaneda, PLi Systems (jorge@plisystems.com)

Earth
Engineers,
Inc.

GEOTECHNICAL INVESTIGATION REPORT

for the

Proposed Nenana Avenue and Tax Lot 600 Private Driveway Construction
Cannon Beach, Clatsop County, Oregon

Prepared for

Stan Roberts
925 Lake Street South
Apartment Number 201
Kirkland, Washington 98033

Prepared by

Earth Engineers, Inc.
2411 Southeast 8th Avenue
Camas, Washington 98607
Telephone (360) 567-1806
Fax (360) 253-8624

EEI Report No. 20-014-1-R1

June 22, 2020
Revised June 30, 2020

19206PE
JULY 15, 1997
TROY M. HULL
EXPIRES: 6/30 21

Troy Hull, P.E., G.E.
Principal Geotechnical Engineer

Anita Bauer

Anita Bauer
Geologic Associate

TABLE OF CONTENTS

0.0 EXECUTIVE SUMMARY

Earth Engineers, Inc. has completed a geotechnical investigation report for the proposed construction of Nenana Avenue, as well as a private driveway that will access the Roberts residence (Tax Lot 600) from Nenana Avenue. We completed a subsurface investigation, which consisted of 2 drilled Standard Penetration Test (SPT) borings to depths of 36 ½ feet and 2 drive probe tests completed to depths of 7 ½ to 9 ½ feet. This subsurface data, along with previous geotechnical data shared by Don Rondema with Geotech Solutions, was evaluated and it was determined that an elevated bridge structure would be an appropriate solution, when compared to constructing a road on grade. The primary concern with ruling out the road on grade was that adding weight to the slope by placing fill should be avoided given that the Nenana Avenue right-of-way and Tax Lot 600 are both located in a known, very slowly moving landslide area.

One factor in constructing the new elevated road is that it cannot interrupt the existing horizontal drains that were previously designed by Geotech Solutions and installed. Our report recommends monitoring the condition of the horizontal drains before and after construction of the new Nenana Avenue roadway, and repairing/replacing any drains that are damaged.

With regard to restrictions on the construction schedule, we recommend all below ground work be conducted during the dry season, which we generally consider to be June through October.

We are recommending the elevated public roadway and private driveway be supported on drilled piers and tiebacks. We are not recommending driven piles due to concerns over ground vibration caused by pile driving.

With regard to geologic hazards, as required by the City, the proposed public road and private driveway construction will preserve the natural slope, follow the slope contours, reduce the need for grading and filling, minimize vegetation removal, not alter drainage patterns, or block stream drainage ways. This will be accomplished by using an elevated roadway system supported on bridge bents that does not require any significant amounts of structural fill.

1.0 PROJECT INFORMATION

1.1 Project Authorization

Earth Engineers, Inc. (EEI) has completed a Geotechnical Investigation Report for the proposed Nenana Avenue and Tax Lot 600 private driveway construction located immediately west of South Hemlock Street, in Cannon Beach, Clatsop County, Oregon. Our services were authorized by Stanley Roberts on February 3, 2020 by signing EEI Proposal No. 20-P033 dated February 1, 2020.

1.2 Project Description

Our current understanding of the project is based on the information provided to EEI Principal Geotechnical Engineer Troy Hull through various e-mails and telephone conversations with our client Stanley Roberts, Don Rondema at Geotech Solutions (the Geotechnical Engineer of Record for the house project on the adjacent Tax Lot 600), and other project team members.

We have been provided the following documents:

- May 12, 2003 report by Geotech Solutions to the City of Cannon Beach titled “S-Curves Landslide Investigation and Stabilization, Cannon Beach, Oregon.”
- November 13, 2008 report by Geotech Solutions to the City of Cannon Beach titled “Geotechnical Report, Phase II Horizontal Drain Installation, S-Curves – Cannon Beach.”
- June 26, 2018 memorandum by Geotech Solutions to the City of Cannon Beach Public Works titled “Hemlock Street S-Curves Slide: Status Update.”
- July 2, 2019 report by Geotech Solutions to yourself titled “Geotechnical Engineering Consultation, Planning Phase, Tax Lot 600, Nenana Avenue Oceanfront Lot – Cannon Beach, Oregon.”
- May 19, 2020 civil drawing set (sheets 1 through 10) by Morgan Civil Engineering titled “Stanley Roberts, Tax Lot 600 - Nenana Avenue.”
- June 6, 2020 “Report of Geotechnical Engineering Services, Proposed Residence at Tax Lot 600 – North of Nenana ROW, Cannon Beach, Oregon.” This report was written by Mr. Don Rondema of Geotech Solutions for our client, Mr. Roberts. Given Mr. Rondema’s extensive past history working in this area, the information in this report is invaluable and much of the design approach and recommendations in our report is based upon the background information in Mr. Rondema’s report. Key points from the report include:

- The proposed residence and Nenana Avenue are located in the active “S-Curves Slide.”
- Ground movement of the S-Curves Slide has occurred many times over the years.
- The S-Curves Slide was reported to have moved 6 to 8 feet in Hemlock Street in the winter of 1972.
- In 1999 the slide moved 4 to 6 inches in Hemlock Street. Based on past inclinometer data, the failure plane is roughly 35 to 45 feet below grade at the eastern edge of Lot 600 (the Roberts lot). The failure plane is roughly 50 feet deep further up the slope on the Nenana Avenue right-of-way. Below the failure plane is massive siltstone that extends to a depths of over 70 feet (i.e. well below the beach elevation).
- Long, deep horizontal drains were installed in the S-Curves slide in 2007 (starting at the beach and extending to the east). See Figures 1 and 2 below for locations. The goal of the horizontal drains was to decrease storm rainfall related spikes in groundwater levels within the landslide mass and therefore reduce landslide movement.
- Monitoring the S-Curves Slide area since the horizontal drains were installed indicates that groundwater levels and slide movements have decreased, but not stopped. Movements less than 0.2 inches have occurred following at least 3 significant rainfall events in the past 12 years.
- Geotech Solutions concluded that the slide is still moving fractions of an inch on deep shear surfaces in high intensity rainfall events.
- The geotechnical recommendations for developing within the S-Curves Slide are not intended to totally stop all future slide movement, as it is not feasible. Instead, the measures are intended to improve the localized lot stability relative to the oceanfront slope (i.e. the “west” slope).
- The west slope instability margin is generally above elevation 61 feet.
- The S-Curves Slide cannot be fully mitigated from moving due to an earthquake induced slide (i.e. Cascadia Subduction Zone earthquake).
- Cut and fills are generally limited to 2 feet.
- The house is recommended to be supported on drilled and grouted micropiles that extend at least 10 feet past the slide plane and into the hard siltstone stratum.
- No isolated footings are allowed; they must all be interconnected with grade beams.
- Tsunami hazard maps indicate the site could be inundated by the largest expected CSZ earthquake (i.e. M_W=9.1).

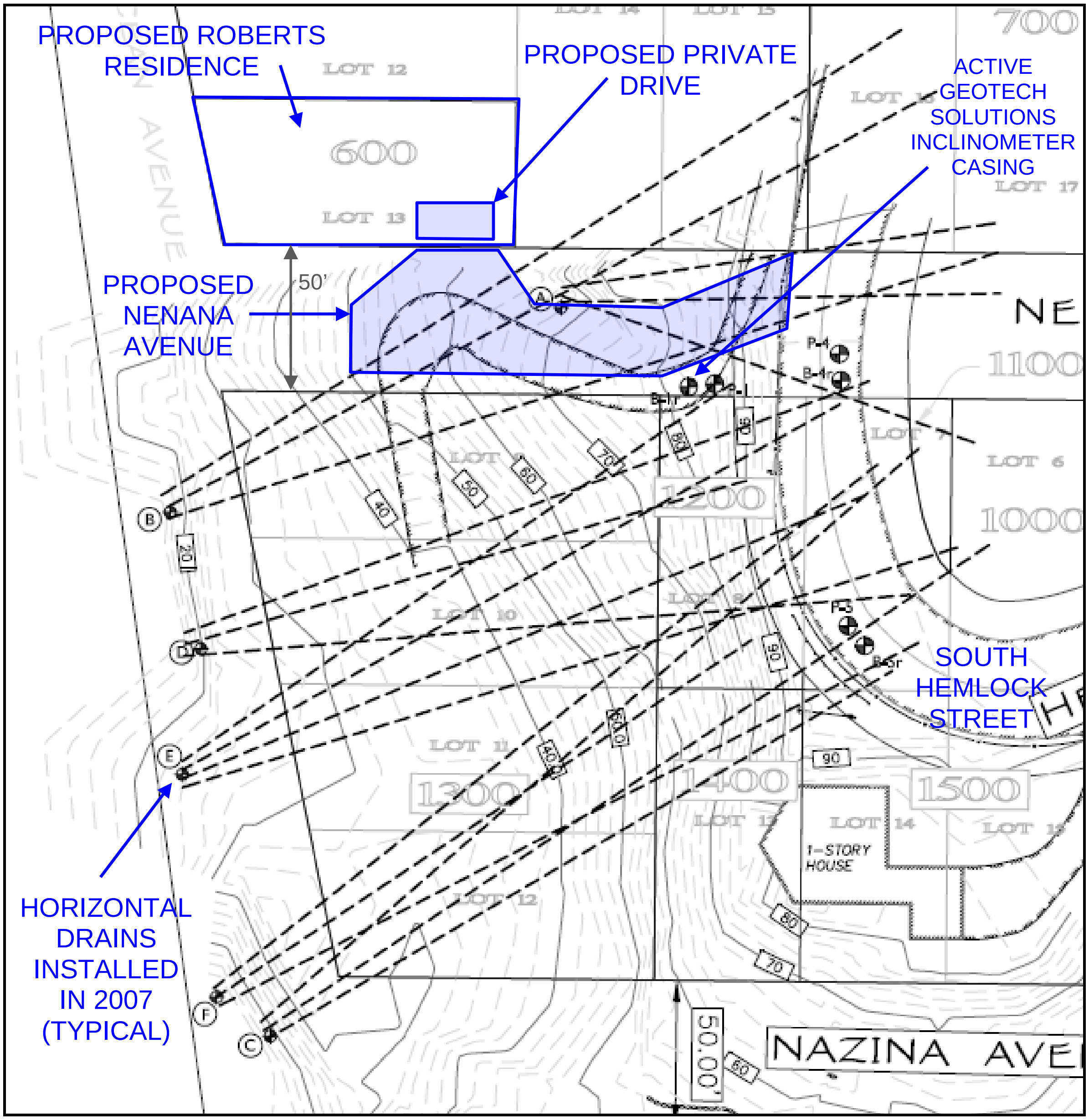


Figure 1: Horizontal drain location plan (base drawing source: Geotech Solutions, 11/13/08).

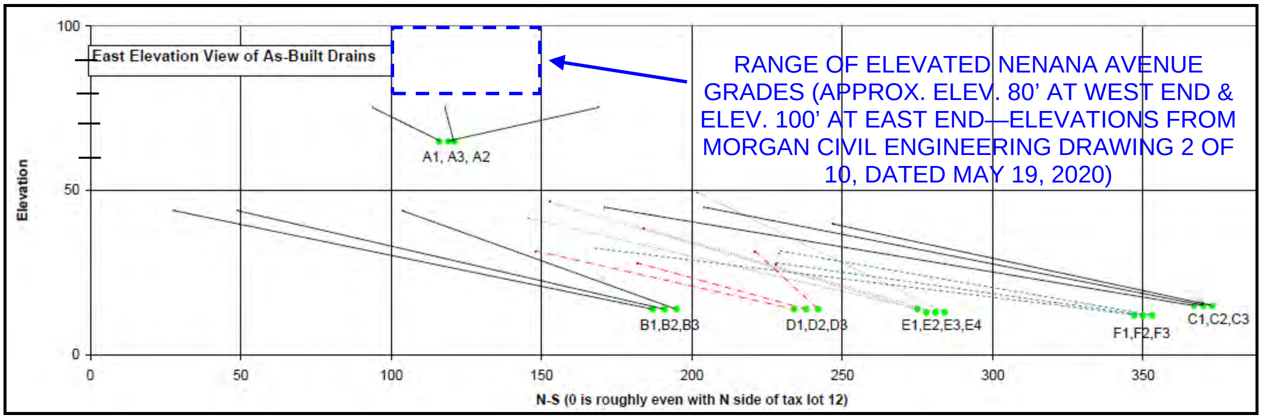


Figure 2: Horizontal drain elevations (base drawing source: Geotech Solutions, 11/13/08).

Additional project information was provided in a site meeting Mr. Hull attended with the general contractor (Rich Elstrom with Rich Elstrom Construction), civil engineer (Jason Morgan with Morgan Civil Engineering), earthwork subcontractor (Mike McEwan), and specialty geotechnical subcontractor (Jorge Castaneda with PLi Systems) on January 29, 2020 to discuss the project background and needs moving forward. The preferred design approach at the meeting was a road constructed on structural fill (i.e. foam blocks). Issues raised at the meeting included how to retain the foam blocks (i.e. would retaining walls be needed) and a need to be able to construct the roadway before the 2020-21 winter season.

Finally, Mr. Hull attended a site meeting on February 28, 2020 with Mr. Roberts, Mr. Elstrom, Mr. McEwan, Mr. Castaneda, and Mr. Eric Watson with Miller Consulting Engineers (the structural engineer for both the proposed road structure and the home) to discuss the roadway design concept. It was agreed that a bridge type structure was acceptable if it made sense from a geotechnical standpoint. Again, it was emphasized by Mr. Roberts that the roadway needed to be constructed before the 2020-21 winter season.

Briefly, we understand that the plan is to construct Nenana Avenue (a City of Cannon Beach public right of way) and a private driveway in order to provide access for the planned construction of a new home on Tax Lot 600. The proposed public roadway will lead from South Hemlock Street down slope to the west towards the Pacific Ocean and provide access to tax lot 600 (see Figure 3 below). According to the current planned configuration of Nenana Avenue (Sheet 2 of Morgan Civil Engineering's 5/19/20 drawing set), the final grade will be up to about 20 feet higher than the current surface grade (see Figure 4). There have been two design options in discussion. One is to use lightweight foam block fill to support the road on grade and not add significant weight to the slope. The other is to build a pile supported bridge (which does not include any lightweight foam block fill).

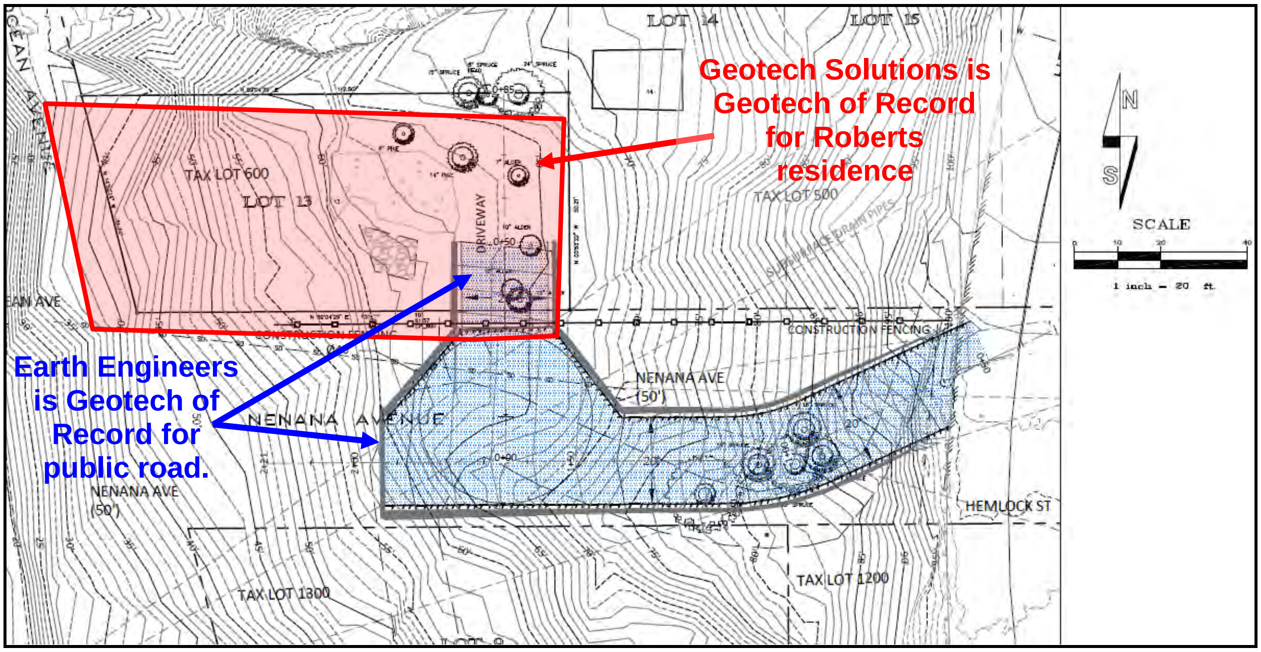


Figure 3: Site plan (base drawing source: Sheet 1 of Morgan Civil Engineering's 5/19/20 drawing set).

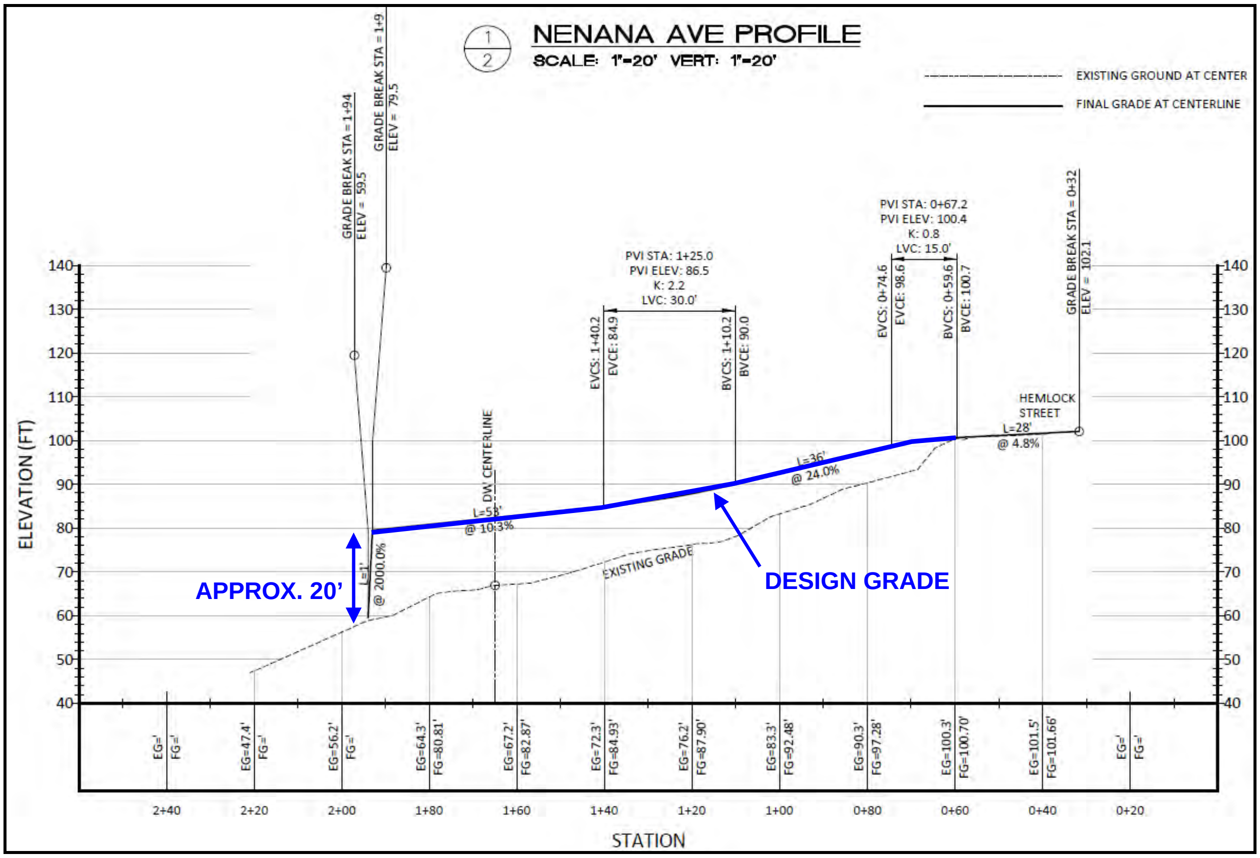


Figure 4: Proposed Nenana Avenue slope profile (base drawing source: Sheet 2 of Morgan Civil Engineering's 5/19/20 drawing set).

The Nenana Avenue public right-of-way, Tax Lot 600, and adjacent properties are all located within the boundaries of a known active landslide area (S-curves slide) as mapped by the City of Cannon Beach. Principle Geotechnical Engineer Don Rondema with Geotech Solutions Inc. has been involved with and has been working with the City on the S-curves slide since about 1999. According the Geotechnical Engineering Consultation report issued to Stanley Roberts by Geotech Solutions Inc. on July 2, 2019, horizontal drains were installed in 2007 and 2008 and the current slide movement has been reduced to 0.3 inches in the last 6 years. Additionally, the Oregon HazVu: Statewide Geohazards Viewer (http://www.oregongeology. org/hazvu/) lists the site as being within a severe to very strong earthquake shaking zone, a very high (active) coastal erosion hazard zone, and a high landslide (landslide likely) hazard area.

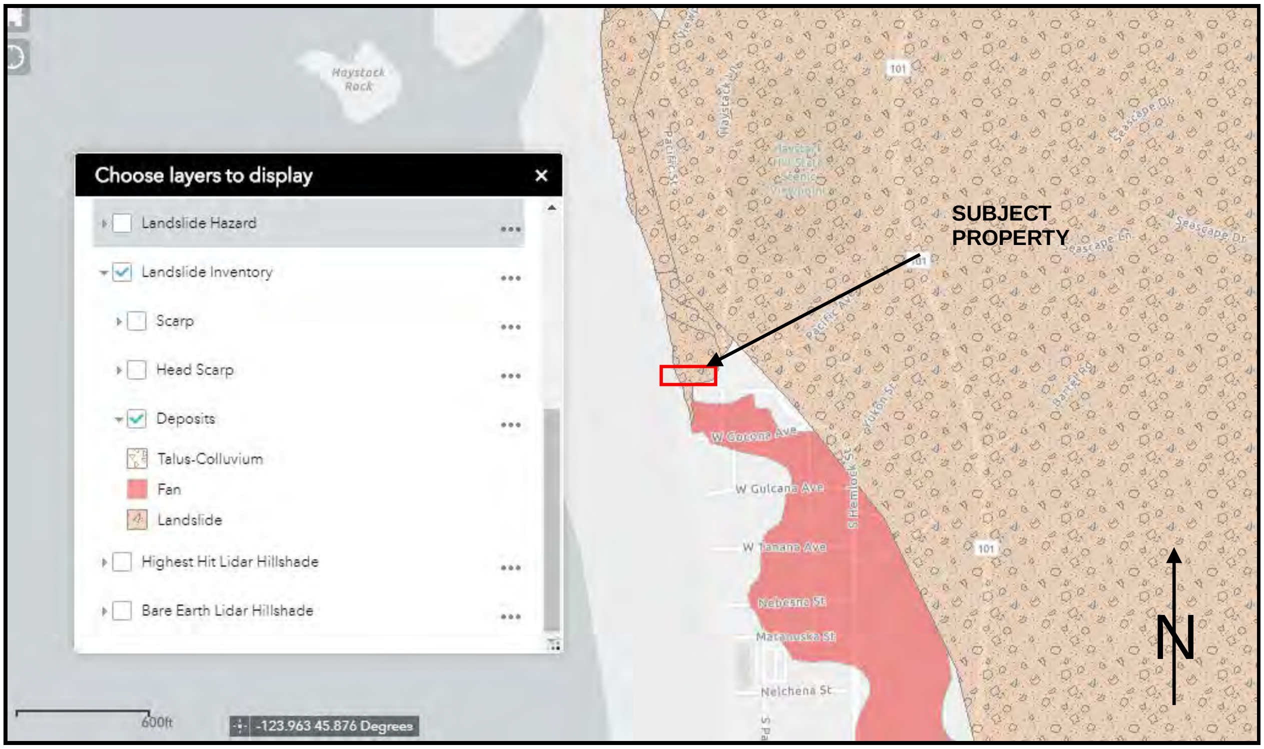


Figure 5: Property is located within a large slide mass (base map source: http://www.oregongeology.org/hazvu/).

We understand the public roadway is being structurally designed by Miller Consulting Engineers in accordance with the 7th Edition of the AASHTO LRFD Bridge Design Specification manual. The private driveway is being structurally designed in accordance with the 2019 Oregon Structural Specialty Code (OSSC) and ASCE 7-16.

1.3 Purpose and Scope of Services

The purpose of our services was to perform a geotechnical investigation in order to supplement the subsurface data already provided by Geotech Solutions, and provide geotechnical recommendations for the proposed Nenana Avenue public roadway and Tax Lot 600 private driveway construction. Our site investigation consisted of advancing 2 drilled Standard Penetration Test (SPT) borings (B-1 and B-2) using a Beretta T26 tracked drill rig subcontracted from PLi Systems of Hillsboro, Oregon. Additionally supplemental drive probe testing (DP-1 and DP-2) was performed by EEI staff to further characterize the overall site subsurface soil and groundwater conditions.

Soil samples from the drilled borings were collected at regular intervals of 2.5 feet in the upper 15 feet and at 5 foot intervals thereafter. Select samples were tested in the laboratory to determine the material properties for our evaluation. Laboratory testing was accomplished in general accordance with ASTM procedures.

This report briefly outlines the testing procedures, presents available project information, describes the site and subsurface conditions, and presents recommendations regarding the following:

- A discussion of subsurface conditions encountered including pertinent soil and groundwater conditions.
- Seismic design parameters in accordance with ASCE 7-16.
- Deep foundation recommendations, including allowable load capacities and embedment lengths.
- Retaining wall design recommendations including earth pressures, drainage and backfill.
- Structural fill recommendations, including an evaluation of whether the existing site soils can be used.
- Slab on grade support recommendations.
- General site earthwork recommendations, including temporary and permanent slopes as well as site drainage.
- Other discussion on geotechnical issues that may impact the project.

Our scope of services did not include a global slope stability analysis or a seismic site hazard analysis. The slope stability analysis is not necessary because we already know the overall slope is globally unstable.

2.0 SITE AND SUBSURFACE CONDITIONS

2.1 Site Location and Description

The subject property includes the Nenana Avenue easement (or public right of way) immediately west of South Hemlock Street, as well as a small section of Tax Lot 600 where the private driveway will be constructed. The project site is bordered by tax lots 600 and 500 to the north, tax lots 1300 and 1200 to the south, South Hemlock Street to the east and Pacific Ocean to the west. The property is currently undeveloped, with variable topography and some vegetation, which includes bushes and trees. The property slopes relatively steeply towards the Pacific Ocean to the west. See Figure 6 below for the project vicinity.

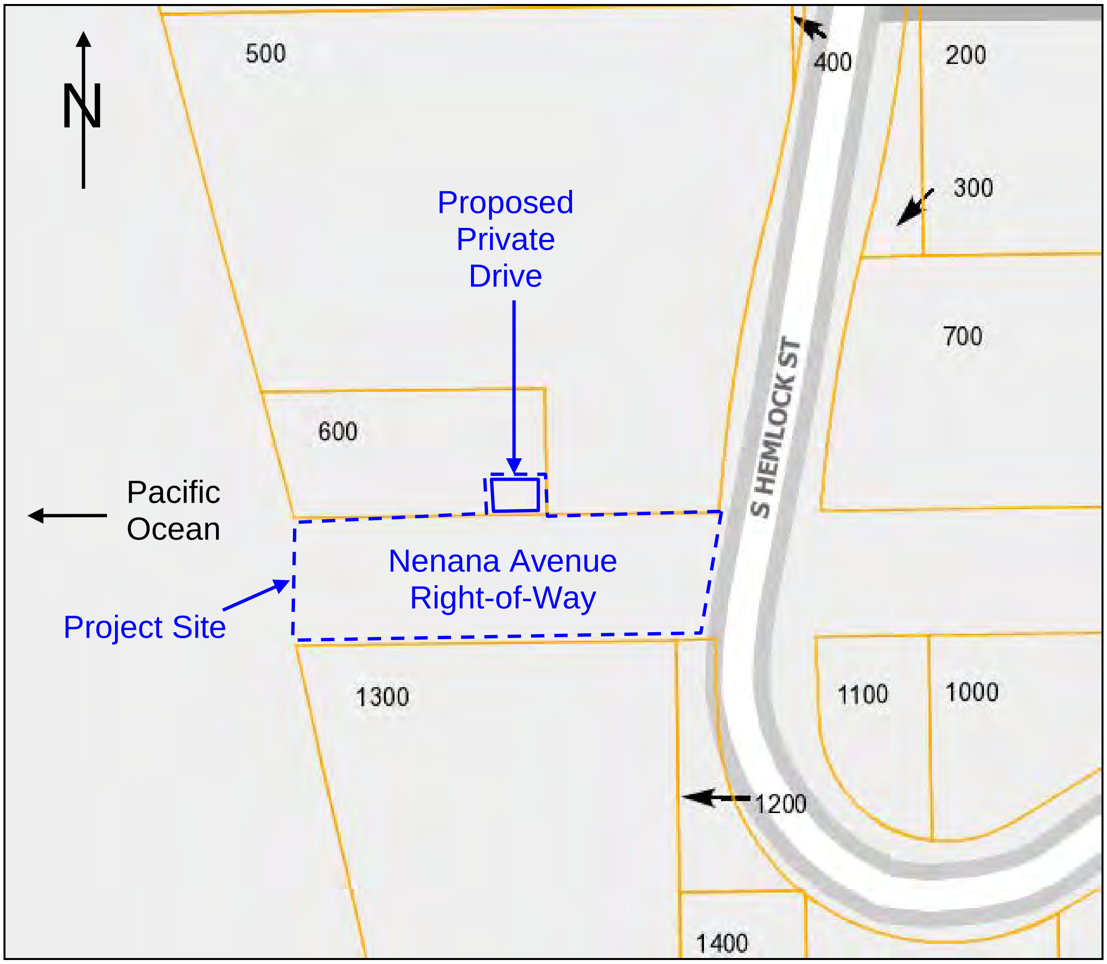


Figure 6: Project vicinity (base map source: Clatsop County Webmaps).

On the subject property there is a footpath that leads form South Hemlock Street to the west and then curves partway past tax lot 600 and continues south. The location of the proposed public roadway roughly follows this existing path and there are slopes on both the north and south sides. On the north side there is a shallow ravine that goes through the middle of the Nenana Avenue easement (see Photo 1). The current proposed construction will include part of this ravine. In terms of topography along the path, the property slopes down from South

Hemlock Street at about 1H:1V (45 degrees) for the first 25 feet, and then 4H:1V to 3H:1V (about 15 degrees) for about 85 feet. At this point the path ends and we continued to estimate the slope towards the beach. We visually estimated a 25 degree slope for about 35 feet, then about 40 degrees for another 35 feet, then about 15 degrees for about 15 feet, and finally about 50 degrees for another 15 feet down to the beach.

Photo 1: Looking south at the Nenana Avenue right-of-way from north edge of property facing south, showing the existing path (red) and shallow ravine (yellow).

Photo 2: Looking west at the middle portion of the path, sloping down to the west at approximately 15 degrees; drill rig is setup on B-1 in the background.

2.2 Mapped Soils and Geology

The project site is located on the lower west foothills of the Oregon Coast Range, specifically above Canon Beach and about 1,700 feet southeast of the iconic haystack rock. The Oregon coast range is defined by a 30 to 40 mile wide swath of moderately high mountains that spans 200 miles along the Pacific Coast. In general, the region has been uplifted as a result of plate convergence from the Cascadia subduction zone located about 150 to 200 km west of the coast range[1]. The region is underlain by a framework of Miocene aged (23 to 5 million years ago) volcanic rocks and Oligocene (33 to 23 million years ago) to Miocene aged marine sedimentary deposits that have been deposited over a basement rock of Eocene-aged (60 to 33 million years ago) volcanic arc deposits. Overlying this framework are Quaternary–aged (1.8 million years ago to present) marine terrace deposits, beach and dune deposits and landslide deposits.

The project area was mapped by Alan R. Niem and Wendy A. Niem, of the U.S. Geological Survey from 1972 to 1984. Within the project vicinity the underlying geologic unit is mapped as the Cannon Beach member of the Astoria formation (Tac). This unit consists of well-bedded, fine-grained marine sandstone, siltstone, and mudstone from the middle to lower Miocene. Haystack Rock is mapped as Wanapum Basalt and specially Frenchman Springs Member of pillow palagonite complexes (Tfsp). This unit is from the middle Miocene and is composed of isolated pillow breccia associated with autointrusive sills and dikes (igneous intrusions).

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[1] Kelsey, H.M., and J.G. Bockheim, Coastal landscape evolution as a function of eustasy and surface uplift rate, Cascadia margin, southern Oregon, Geol. Soc. Am. Bull., 106, 840-854, 1994.

Quaternary alluvium (unconsolidated flood plain deposits) and beach sand from the Holocene (the past 11,000 years) have also been mapped within the vicinity of the project site[2].

The United States Department of Agriculture (USDA) Soil Survey provides geographical information of the soils in Clatsop County as well as summarizing various properties of the soils. The USDA shows the native soils on the site mostly mapped as 71C-Walluski Medial silt loam on 7 to 15 percent slopes and 61E-Templeton-Ecola silt loam on 30 to 60 percent slopes.[3] The Walluski Medial silt loam is moderately well-drained, forms stream terraces and consists of alluvium or fluviomarine deposits derived sedimentary rock. The other soil type (Templeton-Ecola) is well drained, occurs on hillslopes and mountain slopes, and includes colluvium deposits derived from sedimentary rock.

2.3 Subsurface Materials

The site was explored with 2 SPT borings (B-1 and B-2). For the approximate exploration locations, see Appendix B. The two SPT borings were advanced using a portable T26 Beretta tracked drill rig using mud-rotary drilling techniques and equipped with an automatic SPT hammer. In March of 2018, the hammer was calibrated by GeoDesign, Inc. and found to have an energy efficiency of 83.3 percent; standard N_{60} values assume a hammer efficiency of 60 percent. Therefore, our SPT values have been multiplied by a factor of 1.388 (83.3/60 = 1.388) to more accurately reflect the strength conditions of the subsurface soils we encountered. Both borings B-1 and B-2 were advanced to depths of 36.5 feet below existing ground surface (bgs). Both borings were terminated after drilling at least 10 feet into the harder siltstone stratum. SPT samples were taken at intervals of 2.5 feet in the upper 15 feet and 5 feet thereafter to the terminal depths of the borings.

Two supplemental drive probe tests (DP-1 and DP-2) were also performed. The drive probe tests were advanced to depths of 7.5 (DP-1) and 9.5 (DP-2) feet bgs. The drive probe test is based on a "relative density" exploration device used to determine the distribution and to estimate strength of the subsurface soil and decomposed rock units. The resistance to penetration is measured in blows-per-1/2 foot of an 11-pound hammer which free falls roughly 3.5 feet driving a 1-inch diameter pipe into the ground. This measure of resistance to penetration can be used to estimate relative density of soils. For a more detailed description of this geotechnical exploration method, please refer to the Slope Stability Reference Guide for National Forests in the United States, Volume I, United States Department of Agriculture, EM-7170-13, August 1994, P 317-321. The results are included in the drive probe logs attached to this report.

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[2] Niem, A.R., and Niem, W., 1985, Geologic map of the Astoria Basin, Clatsop and northernmost Tillamook Counties, northwest Oregon: Portland, Oreg., Oregon Dept. of Geology and Mineral Industries Oil and Gas Investigation Map OGI-14, Plate 1, scale 1:100,0

[3] Soil Survey Staff, Natural Resources Conservation Service, United States Department of Agriculture. Web Soil Survey. Available online at http://websoilsurvey.nrcs.usda.gov/ accessed March 3, 2020.

Select soil samples were tested in the laboratory to determine material properties for our evaluation. Laboratory testing was accomplished in general accordance with ASTM procedures. The testing performed included moisture content tests (ASTM D2216), the amount of material in the soils finer than the #200 sieve (ASTM D1140), and Atterberg limits tests (ASTM D4318). The test results have been included on the Exploration Logs in Appendix C.

In general, we encountered native soils that consisted of severely weathered siltstone overlying gray siltstone. Note that the soils encountered in our borings were generally similar to the soil conditions encountered in the nearby Geotech Solutions borings. Each of the strata we encountered in our explorations are described below:

FILL

While onsite we observed the earthwork subcontractor (Mike McEwan) use a small excavator to place temporary fill to form a level pad for the Berretta T26 drill rig at both boring locations. At boring location B-1, Mr. McEwan used about 2 feet of native brown sandy silt. At boring location B-2, about 1 foot of imported 3/4 inch minus crushed rock gravel was used.

NATIVE SOILS

Beneath the temporary fill described above, we encountered native fine-grained soils of severely weathered siltstone in both borings. This stratum consisted of medium stiff to hard, brown and gray fat clay (CH) with red and orange mottling. At a depth of 15 feet in boring B-1 and 12.5 feet in boring B-2 the material became gray with red staining and all gray in both borings at a depth of 20 feet bgs. In both borings this stratum extended to a depth of 25 feet bgs. Laboratory moisture content testing on samples obtained within this stratum ranged from 26 to 46 percent, indicating a moist to wet condition. Fines content laboratory testing for samples obtained within this stratum ranged from 61 to 95 percent passing the #200 sieve. An Atterberg limits test was conducted on the most cohesive appearing sample and had a liquid limit of 56, a plastic limit of 23, and a calculated plasticity index of 33.

SILTSTONE

Beneath the native fine-grained soils described above, we encountered sedimentary rock at a depth of 25 feet and continued to the terminated depth of both borings at 36.5 bgs. This rock stratum consisted of gray, medium- to fine-grained, soft to very soft siltstone. The measured moisture contents in this stratum ranged from 21 to 23 percent.

The classifications noted above were made in accordance with the Unified Soil Classification System (USCS) as shown in Appendix D. The above subsurface description is of a generalized nature to highlight the major subsurface stratification features and material characteristics. The exploration logs included in Appendix C should be reviewed for specific information at specific locations. These records include soil descriptions, stratifications, and locations of the samples. The stratifications shown on the logs represent the conditions only at the actual exploration locations. The stratifications represent the approximate boundary between subsurface materials and the actual transition may be gradual. Water level information obtained during field operations is also shown on these logs. The samples that were not altered by laboratory testing will be retained for 90 days from the date of this report and then will be discarded.

2.4 Groundwater Information

Groundwater was encountered at a depth of 16.2 feet (approximate elevation 52 feet) in boring B-1 on February 19, 2020, after being left open overnight. Groundwater was not measured in B-2. It should be noted that the groundwater elevation can fluctuate seasonally and annually, especially during periods of extended wet or dry weather or from changes in land use.

2.5 Seismicity

In accordance with of ASCE 7-16, we recommend a Site Class D (stiff soil profile with an average standard penetration resistance of between 15 and 50 blows per foot) when considering the average of the upper 100 feet of soil. Inputting our recommended Site Class as well as the site latitude and longitude into the Seismic Design Maps (SEAOC/OSHPD) website[4], we obtained the seismic design parameters shown in Table 1 below. The return interval for these ground motions is 2 percent probability of exceedance in 50 years.

Table 1: Seismic Design Parameter Recommendations (ASCE 7-16)

PARAMETER	RECOMMENDATION
S_S	1.317g
S_1	0.691g
F_a	1.000
F_v	Null – See ASCE 7-16 Section 11.4.8
S_{MS} (=S_s x F_a)	1.317g
S_{M1} (=S_1 x F_v)	Null – See ASCE 7-16 Section 11.4.8
S_{DS} (=2/3 x S_s x F_a)	0.878g
S_{D1} (=2/3 x S_1 x F_v)	Null – See ASCE 7-16 Section 11.4.8
Design PGA (=S_{DS}/2.5)	0.351g
MCE_G PGA	0.664g
F_{PGA}	1.100
PGA_M=(MCE_G PGA x F_{PGA})	0.730g

Note: Site latitude = 45.88116915, longitude = -123.96260365

Per Section 11.4.8 of ASCE 7-16 a site-specific seismic site response analysis (i.e. SHAKE software or equivalent) is required for structures on Site Class D and E sites with S_1 greater than or equal to 0.2g. The S_1 value for this site is greater than 0.2g as shown in Table 1 above. Therefore a site response analysis is required as part of the design phase. However, Section 11.4.8 does provide an exception for not requiring a site response analysis (reference Sections 11.4.8.1, 11.4.8.2 and 11.4.8.3). The project Structural Engineer should determine if the

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[4] SEAOC/OSHPD, Seismic Design Maps, http://seismicmaps.org

proposed structure will meet any of the exceptions – if the structure does not meet the exception requirements then EEI should be retained to perform a site-specific site response analysis.

We understand a Supplement 1 dated December 12, 2018 has been issued for ASCE 7-16 to correct some issues in the original publication. One of the corrections in the Supplement pertains to Table 11.4-2 (see table below) for determining the value of the Long-Period Site Coefficient, F_V, which is then used to calculate the value of T_S. The T_S value is needed for one of the exceptions in Section 11.4.8. Without the correction in Supplement 1, it would not be possible to determine F_V and calculate T_s. Based on Supplement 1, the F_V value may be determined from the following corrected table.

Table 2: Long-Period Site Coefficient, F_V (corrected Table 11.4-2 for ASCE 7-16).

	Mapped Risk-Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameter at 1-s Period					
Site Class	S_1<=0.1	S_1<=0.2	S_1<=0.3	S_1<=0.4	S_1<=0.5	S_1<=0.6
A	0.8	0.8	0.8	0.8	0.8	0.8
B	0.8	0.8	0.8	0.8	0.8	0.8
C	1.5	1.5	1.5	1.5	1.5	1.4
D	2.4	**2.2**[a]	**2.0**[a]	**1.9**[a]	**1.8**[a]	**1.7**[a]
E	4.2	**3.3**[a]	**2.8**[a]	**2.4**[a]	**2.2**[a]	**2.0**[a]
F	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8

Note: use linear interpolation for intermediate values of S_1.

[a] **See requirements for site-specific ground motions in Section 11.4.8. These values of F_V shall be used only for calculation of T_S.**

2.6 Slope Stability

EEI is not performing a detailed slope stability analysis as part of our scope. Geotech Solutions has already performed an extensive evaluation of the existing landslide that covers the subject City property as well as other properties in the immediate vicinity. Geotech Solutions has shared that information with us and given us permission to utilize it.

To briefly summarize the findings by Geotech Solutions, there is a very deep slide plane under the property. Based on Geotech Solutions' historical inclinometer data, the depth of the slide plane on the City property is roughly 35 to 50 feet deep. Further north of the Nenana Avenue project, in South Hemlock Street, Geotech Solutions' inclinometer casing indicates the slide plane is roughly 70 feet deep (reference Geotech Solutions report dated May 12, 2003).

Prior to 2007-2008, single slide events were up to 1 foot vertically and horizontally, in response to high winter rainfall events in eroded toe conditions. In 2007 and 2008, horizontal drains were installed to reduced peak groundwater levels during high rainfall events. The horizontal drains have slowed, but not stopped the landslide movement. Over roughly the past 6 years, the slide mass movement has been measured at 0.3 inches—significantly less than prior to the horizontal drain installation. It appears that the horizontal drains must be making a big improvement in the landslide stability to date. However, the landslide mass is still at risk of significant movement, primarily due to loss of toe support typically caused by winter storms, and/or earthquakes. It is also reasonable to assume that heavy rainfall events could increase the landslide movement in the future.

3.0 EVALUATION AND FOUNDATION RECOMMENDATIONS

4.1 Geotechnical Discussion

Based on the information provided to us and our subsurface investigation, it is our professional opinion that the primary factors impacting the proposed development include the following:

1. **Significant elevation difference between the road grade and the soil grade.** Because South Hemlock Street is much higher in elevation than the Nenana Avenue right-of-way grade, and because the Nenana Avenue road grade cannot be constructed very steep, Nenana Avenue will be quite a bit higher than the existing grade. The new street will be as much as about 20 feet higher. This requires the street to either be constructed with up to about 20 feet of fill or constructed as a bridge structure. One major issue with the fill is that it will add weight to the slope.

2. **Slope instability.** As discussed in Section 2.6 above, the roadway will be located in an actively moving landslide mass where the failure plane is roughly 35 to 50 feet below existing grade. While it has reportedly only moved 0.3 inches over the past 6 years (0.05 inches per year average), it has the potential for a much faster rate of displacement. Developing the roadway requires the acknowledgement that this risk cannot be fully mitigated. The primary design consideration is to provide a rigid enough roadway structure such that there would be enough time to exit the roadway on foot prior to collapse (i.e. provide life-safety protection). The roadway design should assume there will could be several feet of lateral and vertical movement in a worst-case single slide event.

   The design should also aim to not significantly increase the slope instability. This means that weight should not be added to the slope and the existing horizontal drains should remain functional, or be replaced if they are damaged during the new construction.

3. **Soft, compressible near-surface soils.** We observed that roughly the upper 5 feet of fat clay soils in our subsurface explorations was generally soft and would be compressible when subjected to loads from heavy foundations or thick structural fill.

4. **The connection of Nenana Avenue to the private drive for the Roberts residence driveway.** This will be a major consideration for the design team. There could be significant differential movement between the Nenana Avenue structure and the private driveway structure. And this movement will occur roughly 15 to 20 feet above existing grade. The differential movement could be vertical or lateral and the designs on public and private property will need to take this into consideration.

When considering all of the above constraints, it is our professional opinion that the roadway should be constructed as a bridge structure, rather than a more conventional on-grade roadway. If the on-grade roadway were to be constructed, it would require up to about 20 feet of structural fill. If conventional gravel were used, it would add significant weight to the landslide, which is not allowed. If lightweight foam blocks (typically weighing about 2 to 3 pounds per cubic foot) were used, it would need a retaining structure to contain the blocks.

The bridge structure should be supported on a drilled deep foundation system for vertical support and tiebacks for lateral support. The bridge should be designed to be overly rigid (i.e. more like a fully interconnected box-type structure) to resist potential differential movement of each bridge support. More detailed discussion is provided in Section 4.4 below

To be clear, supporting the road structure on a deep foundation system with tiebacks does not fully mitigate the future risk of landsliding that could impact all or part of this property. It only decreases the risk of the road being damaged.

4.2 Site Preparation

Site preparation is anticipated to be very limited. We envision this would include limited grading in preparation for the bridge bents and grade beams. Topsoil, vegetation, roots, and any other deleterious soils will need to be stripped from beneath the new foundation areas. A representative of the Geotechnical Engineer should determine the depth of removal at the time of construction.

Any existing utilities (other than the horizontal drains) present beneath the proposed construction will need to be located and rerouted as necessary and any abandoned pipes or utility conduits should be removed to inhibit the potential for subsurface erosion. Utility trench excavations should be backfilled with properly compacted structural fill in accordance with Section 4.3 below.

4.3 Structural Fill

As stated above we recommend that minimal fill be placed to raise site grades. In general, cuts and fills should be limited to no more than about 2 feet. Any structural fill to be placed should be free of organics or other deleterious materials, have a maximum particle size less than 3 inches, be relatively well graded, and have a liquid limit less than 45 and plasticity index less than 25. In our professional opinion, the on-site fat clay soils are not appropriate for use as structural fill. We recommend fill be moisture conditioned to within 3 percentage points below and 2 percentage points above optimum moisture as determined by ASTM D1557 (Modified Proctor).

There will be a significant amount of soil spoils created by the drilled pier installation. All drilling spoils should be hauled off site. If fill is to be placed, it should be placed in relatively uniform horizontal lifts on the prepared subgrade which has been stripped of deleterious materials (i.e. topsoil and fill) and approved by the Geotechnical Engineer's representative. Each loose lift should be no greater than about 1-foot thick. The type of compaction equipment used will ultimately determine the maximum lift thickness. Structural fill should be compacted to at least

95 percent of the maximum dry density as determined by ASTM D1557. Each lift of compacted engineered fill should be tested by a representative of the Geotechnical Engineer prior to placement of subsequent lifts.

4.4 Foundation Recommendations

Based on the drawings by Miller Consulting Engineers, we understand the bridge will be supported on a series of bents. As discussed in Section 4.1 above, we recommend supporting the roadway on a deep foundation system comprised of drilled piers. Lateral support should be provided by tiebacks where possible. Where there is not room for tiebacks, the lateral capacity may be provided by the drilled piers.

There are several considerations to be made when selecting the deep foundation and tieback systems:

1. There are existing horizontal drains beneath the proposed roadway that increase the stability of the existing S-Curves Slide. These drains must remain functional during and after construction of the roadway (or be replaced with new drains if their integrity is in question). To reduce the risk of hitting a horizontal drain with a drilled pier or tieback, we recommend their diameters be as small as practical. In conversations with Jorge at PLi Systems, he recommends the drilled pier diameter not be less than 16 inches due to the typical restrictions of the drilling equipment (i.e. diameter of the Kelly bar needed to drill into hard material). Tieback diameters are much less than the drilled piers (i.e. typically 4.5 to 6 inches) so they are not as much of a concern.

2. The S-Curves Slide plane is relatively deep, ranging from about 35 to 50 feet based on old inclinometer data. We have had some internal debate whether to recommend all the deep foundation and tieback elements deeper than the slide plane to try to use them as shear resisting elements to stabilize the S-Curves Slide mass. In discussions with Mr. Rondema, we agree with him that this common approach is not practical for this site. The landslide is too deep and the forces are too large and would likely shear the piles and tiebacks at the slide plane. As such we recommend the piers and tiebacks be designed for embedment depths that will satisfactorily meet the static (non-landslide) load demands, as well as protect the public roadway structure from being undermined should the western slope (between the west end of Nenana Avenue and the beach) experience sloughing.

Our specific recommendations for drilled piers and tiebacks follow in the next sections.

4.4.1 Drilled Pier Recommendations

We recommend that all of the load carrying capacity of the drilled piers come from the slightly weathered siltstone first encountered in our borings B-1 and B-2 at a depth of 25 feet (note that the topography of the right-of-way is highly variable and we expect that the siltstone will be much shallower--and could possibly be deeper--in some locations). In designing the drilled piers, we calculated an estimated ultimate skin friction of 920 psf (305 psf allowable). For end bearing capacity, we calculated an estimated ultimate value of 38,500 psf (13,000 psf allowable). We did not include any load carrying contribution from the overlying intensely weathered and variable strength fat clay stratum.

Based on our assumed parameters, we provide axial compression and uplift capacities for 16-inch, 18-inch, 24-inch, and 30-inch diameter drilled piers provided in Tables 3 and 4 below to aid your structural engineer in selecting what is appropriate depending on the design loads. Table 3 presents allowable capacities based on ASD design (i.e. the private driveway). Table 4 presents ultimate capacities based on LRFD design (i.e. Nenana Avenue).

We can provide additional load capacity recommendations upon request, by increasing the drilled pier embedment into the slightly weathered siltstone stratum. To reduce the risk of hitting the existing horizontal drains, it appears best to use the largest pier diameter that is practical in order to reduce the overall number of piers to be installed.

It should be noted that the axial tension capacity (i.e. uplift) does not include the weight of the drilled pier concrete. It would be acceptable from a geotechnical standpoint to include the drilled pier concrete weight when determining the total uplift resistance.

Table 3: Recommended Allowable Compression and Tension Capacities for Drilled Piers with 10-foot Minimum Embed into Slightly Weathered Siltstone Stratum (ASD Design)

Drilled Pier Diameter (inches)	Allowable[1] Axial Compressive Capacity (kips)	Allowable[1] Axial Tension Capacity (kips)[2]
30	194	13.7
24	128	11.0
18	76	8.2
16	61	7.3

Notes: [1] Allowable capacities reported include a Factor of Safety of 3. If one pile load test is performed, the Factor of Safety may be reduced to 2.
[2] Does not include self-weight of drilled pier.

For LRFD design, we provide the following ultimate compression and tension capacities for 10-foot minimum embed into

Table 4: Recommended Ultimate Compression and Tension Capacities for Drilled Piers with 10-foot Minimum Embed into Slightly Weathered Siltstone Stratum (LRFD design)

Drilled Pier Diameter (inches)	Ultimate[1] Axial Compressive Capacity (kips)	Ultimate[1] Axial Tension Capacity (kips)[2]
30	291	16.4
24	192	13.2
18	114	9.8
16	91.5	8.7

Notes: [1] Ultimate capacities include an AASHTO resistance factor of 0.5 for compression and 0.4 for tension.
[2] Does not include self-weight of drilled pier.

The drilled piers should be designed for a minimum center-to-center spacing of 3 pier diameters in order to use the axial compressive load capacities shown in Table 2 (i.e. this spacing allows the piers to be considered as acting independently and not as a group). Where the private driveway abuts the public roadway, we understand the drilled piers will be located closer than 3 pier diameters. In this case, the private driveway pier allowable capacities should be reduced by 50% to account for the closer spacing.

We estimate that total and differential post-construction settlements of pier-supported elements (excluding landslide movement) will not exceed 1 inch and ½ inch, respectively.

Lateral capacity of the piers was determined using LPILE computer software. Table 5 and 6 below present our calculated lateral load capacities for both fixed head (i.e. constrained from angular deformation at the top) and pinned head condition (i.e. not constrained at the top). For our LPILE analysis, we assumed the piles have a minimum concrete compressive strength of 4,000 psi. Pile deflection, shear and moment diagrams are presented in Appendix G. We are available to provide lateral load capacity recommendations for other pile configurations upon request.

Table 5: Allowable Lateral Load Capacities for Drilled Piers (ASD Design)

Drilled Pier Diameter (inches)	Drilled Pier Axis	Drilled Pier Head Condition	Allowable Lateral Load Capacity (kips)[1]	Minimum Drilled Pier Length, i.e. Point of Fixity (feet)
30	Strong	Fixed	43.5	26
	Strong	Pinned	19	26
	Weak	Fixed	37	24
	Weak	Pinned	16	24
24	Strong	Fixed	35	24
	Strong	Pinned	15	24
	Weak	Fixed	26.5	22
	Weak	Pinned	11.5	22
18	Strong	Fixed	27	23
	Strong	Pinned	11	23
	Weak	Fixed	20	20
	Weak	Pinned	8.5	20

Note: [1] Recommended allowable lateral load includes a FOS of 2.

Table 6: Ultimate Lateral Load Capacities for Drilled Piers (LRFD Design)

Drilled Pier Diameter (inches)	Drilled Pier Axis	Drilled Pier Head Condition	Allowable Lateral Load Capacity (kips)[1]	Minimum Drilled Pier Length, i.e. Point of Fixity (feet)
30	Strong	Fixed	87	28
	Strong	Pinned	38	26
	Strong	Ultimate Moment (Mu) of 58.24 kips[2]	33	26
	Strong	Ultimate Moment (Mu) of 17.99 kips[3]	36	24
	Weak	Fixed	74	28
	Weak	Pinned	32	24
24	Strong	Fixed	70	24
	Strong	Pinned	30	24
	Weak	Fixed	53	22
	Weak	Pinned	23	22
18	Strong	Fixed	54	23
	Strong	Pinned	22	23
	Weak	Fixed	40	20
	Weak	Pinned	17	20

Notes: [1] Recommended ultimate lateral load includes an AASHTO resistance factor of 1.0.
[2] Ultimate moment value applied at pile head was provided by Miller Consulting Engineers in 6/30/20 e-mail and represents soil and traffic surcharge at Bent #1.
[3] Ultimate moment value applied at pile head was provided by Miller Consulting Engineers in 6/30/20 e-mail and represents soil surcharge at Bent #1.

In consideration of the drilled pier spacing/group effects, we recommend the following p-multiplier for center-to-center lateral pier spacings less than 6D, where D equals the pier diameter.

Table 7: P-Multipliers for Design of Laterally Loaded Drilled Pier Groups[1]

Center-to-Center Lateral Pile Spacing	p-Multiplier for Rows 1, 2 and 3+
6D	1.0, 1.0, 1.0
5D	1.0, 0.85, 0.70
4D	0.85, 0.65, 0.50
3D	0.70, 0.50, 0.35

Note: [1]Methodology is based on May 2010 Federal Highway Administration Report No. FHWA-NHI-10-016 titled “Drilled Shafts: Construction Procedures and LRFD Design Methods.”

Other drilled pier design and construction considerations include:

- Any loose materials that accumulate at the bottom of the drilled hole should be removed prior to concrete placement. No rebar or concrete should be placed before the

Geotechnical Engineer approves the drilled shaft for the embedment depth and cleanliness.

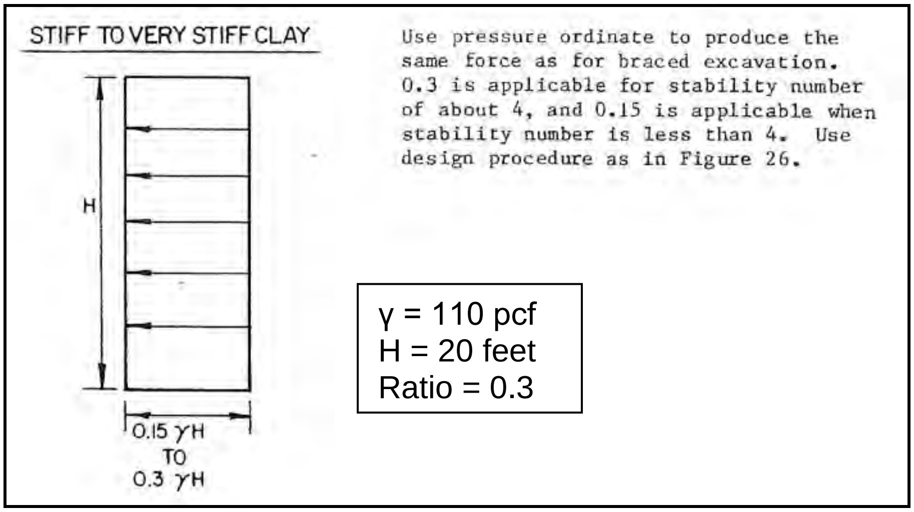
- Drilled shafts should be filled with concrete immediately following approval by the Geotechnical Engineer. **Excavations should in no case be allowed to stand open overnight.**
- Temporary casing may be needed to stabilize the excavations during drilling and until the holes are backfilled. If casing is used to drill the piers, it should be withdrawn as the concrete is poured. The fluid concrete level should be maintained above the bottom of the casing as the casing is removed. It is up to the contractor to review the boring logs and determine if casing will be needed.
- Given that groundwater was encountered in one of our borings at a depth of about 16 feet, the concrete should be placed by a tremie that is lowered to the bottom of each shaft. The concrete should not be allowed to free fall. The flow of concrete out of the tremie hose should not be allowed to hit the earth sidewalls of the shaft or the rebar cage as it could cause concrete contamination or segregation, respectively. Once placed, the concrete should be vibrated briefly to ensure there are no voids. The vibrator should not be allowed to come in contact with the rebar. If concrete with a slump of 7 inches or greater is used, then no vibration will be necessary. We anticipate a high slump concrete mix would require chemical admixtures.
- We understand reinforcement will include steel wide-flange (W-section) piles extending the full depth of the pier. If the steel cannot be inserted the full length of the shaft, then the shaft should be re-drilled until the steel reinforcement can be inserted the full length. The reinforcement of the remaining piers should be sized by the project structural engineer depending on the design loads and moments that need to be supported.
- Concrete volumes placed should be measured to confirm the volume of concrete placed in each pier is greater than the theoretical volume of the hole created by the auger. A minimum ratio of 1.1 (actual/theoretical) is recommended.
- We suggest a minimum of 24 hours elapse between the installation of adjacent drilled piers (generally within 4 pier diameters of each other, as measured from center to center). The purpose of this requirement is to prevent the intrusion of soil or contaminated concrete into a recently installed pier (one in which the concrete has not yet set). The actual delay period between adjacent piers is dependent upon the set time of the concrete.
- Ultimately, the structural engineer should determine the drilled pier concrete compressive strength. From a geotechnical standpoint, we recommend the compressive strength not be any less than 4,000 psi.

4.4.2 Tieback Recommendations

As noted above, we recommend tiebacks be utilized to provide the lateral support for the roadway and driveway (bridge) structures. The purpose of the tiebacks will be to (1) resist the normal lateral loads acting on the bridge, as well as (2) temporarily support the western-most bridge bent should the west slope (i.e. slope between the bridge and beach) slough away.

To evaluate the requirements for supporting the west end of the roadway should the west slope slide, we performed our geotechnical engineering analysis using Shoring Suite, Version 8.17a software from CivilTech. The following assumptions were made in our analysis:

- As much as **20 feet** of soil could slough away from the west side of the roadway.
- The drilled piers at the at the west end of the roadway will be spaced no closer than **6 feet** on center.
- The tiebacks at the west end of the roadway will be spaced no further than **15 feet** on center.
- We assumed an allowable bond strength of **50 psi (7.2 ksf)** between the tieback grout and slightly weathered siltstone.
- A tieback diameter of at least **6** inches.
- The pile design is based on a minimum static FOS of 1.5 against overturning. We did not evaluate a seismic loading condition as we felt it would be overly conservative to evaluate a landslide and worst case earthquake occurring at the same time, especially considering this is only a roadway and not an occupied structure. The FOS was applied to the passive earth pressure, or the resisting force.
- The earth pressure to be retained were determined from the NAVFAC method for tied-back walls. Assuming a stiff soil condition, the earth pressure were assumed to be a rectangular distribution using the input parameters shown below resulting in 660psf acting over the assumed 20 foot exposed wall face (if sliding occurs).



Use pressure ordinate to produce the same force as for braced excavation. 0.3 is applicable for stability number of about 4, and 0.15 is applicable when stability number is less than 4. Use design procedure as in Figure 26.

γ = 110 pcf
H = 20 feet
Ratio = 0.3

Figure 7: Partial excerpt from Figure 29, pg. 2.7-106, of NAVFAC.

The output files summarizing our Shoring Suite analysis are provided in Appendix F. Based on the above design parameters, we provide the following tieback recommendations for the .

- The slope may be shored using **W14x68 (or equivalent) steel piles** in accordance with Table 4 below. In order to install the piles, **2-foot** diameter boreholes should be predrilled so that the piles can be set and then the entire drilled excavation backfilled with concrete with a 28-day compressive strength (f'c) of at least **3,000 psi**.

Table 8: Summary of Permanent Tieback Anchor Recommendations

Tieback Anchor Row #	Tieback Installation Angle Below Horizontal (degrees)	Maximum Tieback Lateral Spacing (feet) [1]	Design Maximum Tieback Tension Force (kips)	Estimated Unbonded Tieback Length (feet)[2]	Estimated Tieback Grouted Length (feet)[3]	Total Estimated Tieback Length (feet)
1	35	15	158	44	14	58

Notes: [1] Tieback lateral spacing requirement only applies to the western-most bridge bent (where the slope to the west could slide away).

[2] Unbonded tieback length is the entire length prior to encountering slightly weathered siltstone, and may be variable.

[3] Grouted length based on a tieback diameter of 6 inches and an assumed allowable bond strength of 7.2 kips per square foot (9.6 ksf ultimate). Actual required grouted tieback length will be based on tieback pull testing. The tiebacks may actually be shorter or longer than reported in Table 4, depending upon tieback pull test results.

The remainder of tiebacks (supporting the other bridge bents) may designed per Table 6. Note that some tiebacks may need to be shortened so that they do not cross into private property owned by others. This may apply at the east end of the project so that tiebacks don't cross the east boundary of the South Hemlock Street right-of-way. In addition this may occur on Tax Lot 600 so that they don't cross into Tax Lot 500. Tables 9 and 10 below provide recommendations for shorter tiebacks.

Table 9: Summary of Permanent Tieback Anchor Recommendations with Maximum 35-foot Horizontal Length

Tieback Anchor Row #	Tieback Installation Angle Below Horizontal (degrees)	Maximum Tieback Lateral Spacing (feet)	Design Maximum Tieback Tension Force (kips)	Unbonded Tieback Length (feet)	Estimated Tieback Grouted Length (feet)[1]	Total Estimated Tieback Length (feet)
1	35	n/a	118	21	21	42

Notes:

[1] Grouted length based on a tieback diameter of 6 inches and an assumed allowable bond strength of 3.6 kips per square foot (4.8 ksf ultimate). Actual required grouted tieback length will be based on tieback pull testing. The tiebacks may actually be shorter or longer than reported in Table 5, depending upon tieback pull test results.

Table 10: Summary of Permanent Tieback Anchor Recommendations with Maximum 20-foot Horizontal Length

Tieback Anchor Row #	Tieback Installation Angle Below Horizontal (degrees)	Maximum Tieback Lateral Spacing (feet)	Design Maximum Tieback Tension Force (kips)	Unbonded Tieback Length (feet)	Estimated Tieback Grouted Length (feet)[1]	Total Estimated Tieback Length (feet)
1	35	n/a	67	12	12	24

Note: [1] Grouted length based on a tieback diameter of 6 inches and an assumed allowable bond strength of 3.6 kips per square foot (4.8 ksf ultimate). Actual required grouted tieback length will be based on tieback pull testing. The tiebacks may actually be shorter or longer than reported in Table 5, depending upon tieback pull test results.

Other drilled tieback design and construction considerations include:

- The tieback grout strength (f’g) should be at least **5,000 psi** at 28 days.
- All tiebacks should allow for post-grouting through the use of post-grout tubes.
- Because these are permanent tiebacks and we anticipate a corrosive environment due to the close proximity to the Pacific Ocean, tieback corrosion protection will be essential. The unbonded portion of the tiebacks should be metalized or epoxy coated.
- The tiebacks will need to be structurally connected to the piles as designed by a qualified structural engineer.
- The tiebacks should be encased within a continuous grade beam designed by the project Structural Engineer. The grade beams should be in a grid that runs in the north-south direction and east-west direction to rigidly connect all of the piers/piles together.
- All of the tiebacks should be proof tested to **133 percent** in accordance with the following load intervals: Alignment Load (no greater than 5 percent of the design load), 0.25 Design Load (DL), 0.50 DL, 0.75 DL, 1.00 DL, 1.25 DL, 1.33 DL, and the Lock off Load (0.3DL).

  Proof test readings shall be taken immediately after reading each load increment, except at 1.5 DL. At 1.5 DL, readings shall be taken at 1, 2, 3, 4, 5, 6 and 10 minutes. If the total creep movement exceeds 0.040 inches between 1 and 10, then the test load shall be maintained for an additional 50 minutes, with recordings at 20, 30, 40 50 and 60 minutes. Readings need not be taken all the way up to 60 minutes if, in the opinion of the Geotechnical Engineer, the creep movement has essentially stopped. A maximum of an additional 0.040 inches of movement is acceptable between 6 and 60 minutes. EEI’s Geotechnical Engineer should evaluate the proof test results to verify the anchors will achieve their designed capacity without excessive movement.

- A representative of the Geotechnical Engineer should continuously observe the pile and tieback installation and tieback testing to confirm that construction is proceeding according to our design recommendations and to confirm our assumptions about the subsurface conditions at the site.

5.0 CONSTRUCTION CONSIDERATIONS

EEI should be retained to provide observation and testing of construction activities involved in the foundation, earthwork, and related activities of this project. EEI cannot accept any responsibility for any conditions that deviate from those described in this report, nor for the performance of the foundations if not engaged to also provide construction observation for this project.

5.1 Moisture Sensitive Soils/Weather Related Concerns

The upper soils encountered at this site are expected to be sensitive to disturbances caused by construction traffic and to changes in moisture content. During wet weather periods, increases in the moisture content of the soil can cause significant reduction in the soil strength and support capabilities. In addition, soils that become wet may be slow to dry and thus significantly retard the progress of grading and compaction activities. It will, therefore, be advantageous to perform earthwork and foundation construction activities during dry weather.

Exposed fine grained soils can be extremely sensitive to moisture and should be protected with a layer granular fill (at least 2 inches thick) if the excavations are to be left open during periods of wet weather.

5.2 Drainage, Groundwater, and Stormwater Considerations

Water should not be allowed to collect in the foundation excavations. Positive site drainage should be maintained throughout construction activities. Undercut or excavated areas should be sloped toward one corner to facilitate removal of any collected rainwater, groundwater, or surface runoff.

Because this site is relatively steep and the subsurface soils consist of landslide debris, we strongly recommend that stormwater be hard piped to the base of the slope (i.e. the beach). Our preference would not be to dispose of stormwater on site.

5.3 Excavations

Because the project site is located within an active landslide area, we recommend that excavations be very limited. In general, we recommend that excavations be limited to no more than about 2 feet. If there are areas where excavations need to be deeper than 2 feet, we can review those isolated cases to determine if it will be safe to make the cuts without destabilizing the slope.

With regard to the drilled pier foundation excavations, we recommend that they be backfilled (with rebar and concrete) the same day due to the fact that the piles will be

drilled in an active landslide area. To be clear, pile excavations should not be left open overnight. We mention this because it is sometimes common practice for the pile contractor to drill holes for multiple days and then backfill them all at one time with concrete because it is usually more cost effective. That practice should be avoided for this project.

Based on our past experience in the area, vertical cut slopes in the slide debris may at first appear to be stable. However, over time (typically a few days), the soils may “relax” and slough. As such, the contractor should take care when excavating into these soils and we strongly recommend that they either use temporary shoring, or lay the excavated slopes back. Once the site soils are exposed, we can consult with the contractor to determine a safe layback slope angle. We can also provide temporary shoring recommendations, if requested.

In Federal Register, Volume 54, No. 209 (October 1989), the United States Department of Labor, Occupational Safety and Health Administration (OSHA) amended its "Construction Standards for Excavations, 29 CFR, part 1926, Subpart P". This document and subsequent updates were issued to better insure the safety of workmen entering trenches or excavations. It is mandated by this federal regulation that excavations, whether they be utility trenches, basement excavations or footing excavations, be constructed in accordance with the new OSHA guidelines. It is our understanding that these regulations are being strictly enforced and if they are not closely followed, the owner and the contractor could be liable for substantial penalties.

The contractor is solely responsible for designing and constructing stable, temporary excavations and should shore, slope, or bench the sides of the excavations as required to maintain stability of both the excavation sides and bottom. The contractor's "responsible person", as defined in 29 CFR Part 1926, should evaluate the soil exposed in the excavations as part of the contractor's safety procedures. In no case should slope height, slope inclination, or excavation depth, including utility trench excavation depth, exceed those specified in local, state, and federal safety regulations.

We are providing this information solely as a service to our client. EEI does not assume responsibility for construction site safety or the contractor's compliance with local, state, and federal safety or other regulations.

5.4 Slope Stability Monitoring During Construction

First off, we recommend that all below grade construction (i.e. drilled pier and tieback installation) be conducted during the dry season (i.e. generally about June through October) to reduce the risk of accelerating the S-curves Slide area.

Secondly, we recommend that the slope stability of South Hemlock Street and the construction project area be monitored regularly during the life of the construction project. Monitoring should consist of:

- Weekly monitoring for cracks in the Hemlock Street pavement.
- Weekly monitoring for ground cracks on the construction site.
- Weekly monitoring for changes in the plumbness of the installed bridge bents.

This monitoring should be documented and may be performed by either a representative of the general contractor or EEI. If the general contractor performs the monitoring, EEI should be forwarded the reports and should be notified immediately of any noticeable changes.

Finally, with regard to the existing horizontal drains that are slowing down the rate of movement of the S-curves Slide area, we recommend that all of the drains be checked prior to the start of construction. Their lengths should be documented and internal conditions recorded on video. When installation of the drilled piers and tiebacks are completed, the horizontal drain lengths should be checked again to confirm the pipes are not blocked. If any drains have been blocked, they should be replaced.

6.0 REPORT LIMITATIONS

The subject development is located on a steep bluff fronting the Pacific Ocean. This property will be subject to very dynamic forces (i.e. powerful winter storms, ocean currents, and earthquakes) over the life of the structure. The conditions of the subject property could change drastically in the future due to these forces and cannot be entirely predicted, nor can they be fully mitigated. Determining the absolute worst case geologic/geotechnical risks to the project and then designing a structure to fully eliminate every one of those risks is not reasonable nor economically feasible. Therefore, in accepting the recommendations herein, the owner must assume some risk that the reasonable worst case conditions, described earlier in this report, may be exceeded.

This project is located in a known active landslide (i.e. the S-curves slide). The City has made attempts to stop the landslide but reportedly it continues to move, albeit at a much slower rate. Given this advanced knowledge, building the street in an active landslide carries risk that there will likely be greater than normal roadway maintenance and repairs over time. And there could even be significant structural damage. The primary risk mitigation consideration is to provide a rigid enough roadway structure such that there would be enough time to exit the roadway on foot prior to collapse (i.e. provide life-safety protection).

As is standard practice in the geotechnical industry, the conclusions contained in our report are considered preliminary because they are based on assumptions made about the soil, rock, and groundwater conditions exposed at the site during our subsurface investigation. A more complete extent of the actual subsurface conditions can only be identified when they are exposed during construction. Therefore, EEI should be retained as your consultant during construction to observe the actual conditions and to provide our final conclusions. If a different geotechnical consultant is retained to perform geotechnical inspection during construction then they should be relied upon to provide final design conclusions and recommendations, and should assume the role of geotechnical engineer of record, as is the typical procedure required by the governing jurisdiction.

The geotechnical recommendations presented in this report are based on the available project information, and the subsurface materials described in this report. If there are any revisions to the plans for this project, or if deviations from the subsurface conditions noted in this report are encountered during construction, EEI should be notified immediately to determine if changes in the foundation recommendations are required. If EEI is not retained to review these changes, we will not be responsible for the impact of those conditions on the project.

The Geotechnical Engineer warrants that the findings, recommendations, specifications, or professional advice contained herein have been made in accordance with generally accepted professional geotechnical engineering practices in the local area. No other warranties are implied or expressed.

After the plans and specifications are more complete, the Geotechnical Engineer should be retained and provided the opportunity to review the final design plans and specifications to check that our engineering recommendations have been properly incorporated into the design documents. At this time, it may be necessary to submit supplementary recommendations.

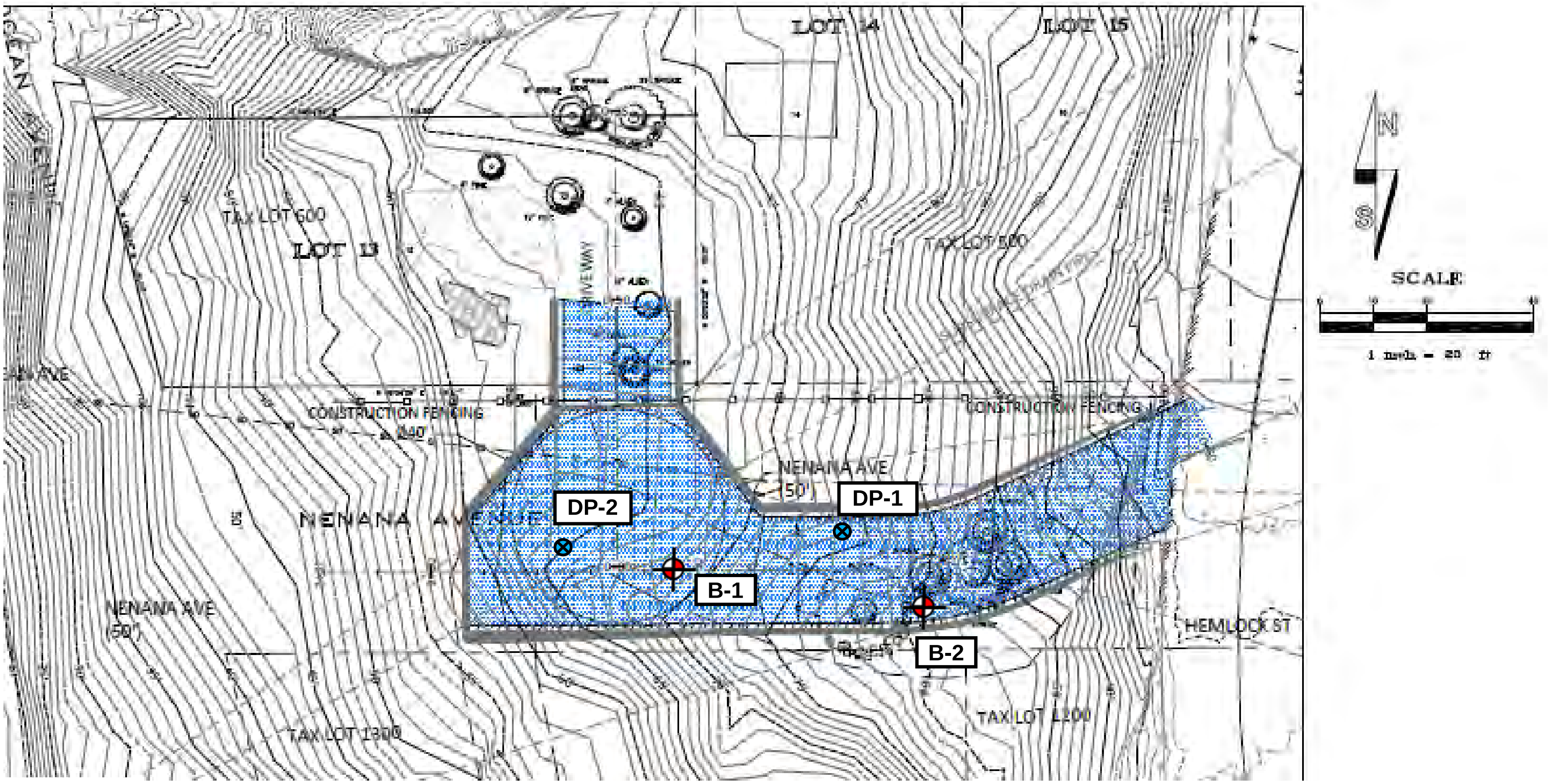
This report has been prepared for the exclusive use of Stanley Roberts for the specific application to the proposed Nenana Avenue and Tax Lot 600 private driveway development located in Cannon Beach, Clatsop County, Oregon. EEI does not authorize the use of the advice herein nor the reliance upon the report by third parties without prior written authorization by EEI.

APPENDICES

APPENDIX A – SITE LOCATION PLAN

Earth Engineers, Inc.	**Proposed Nenana Avenue and Tax Lot 600** **Private Driveway Construction** **Cannon Beach, Clatsop County, Oregon**	**Report No.** **20-014-1-R1** **June 22, 2020** **Revised June 30, 2020**

APPENDIX B – EXPLORATION LOCATION PLAN



Legend:

⊕ = **Approximate EEI Drilled Boring Location**

⊗ = **Approximate EEI Drive Probe Test Location**

Base drawing source: Sheet 1 of Morgan Civil Engineering's 5/19/20 drawing set

Earth
Engineers,
Inc.

Proposed Nenana Avenue and Tax Lot 600
Private Driveway Construction
Cannon Beach, Clatsop County, Oregon

Report No.
20-014-1-R1

June 22, 2020
Revised June 30, 2020

Earth
Engineers,
Inc.

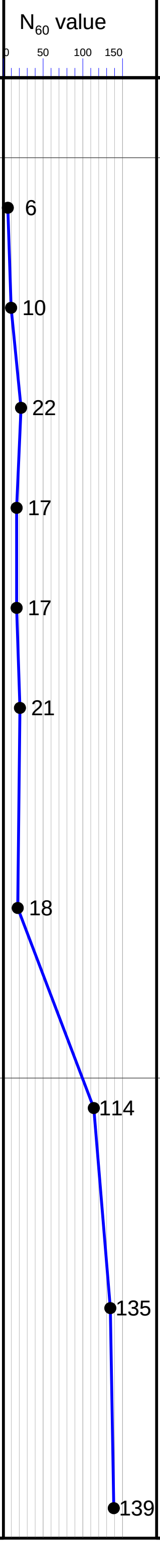
Appendix C: Boring B-1

Sheet 1 of 1

Client: Stan Roberts
Project: Nenana Avenue Construction
Site Address: Proposed Nenana Avenue, Cannon Beach, OR
Location of Borehole: See Appendix B
Date Drilled: 2/18/2020
Logged By: Anita Bauer

Report Number: 20-014-1
Drilling Contractor: PLi Systems
Drilling Method: Mud Rotary
Drilling Equipment: Beretta drill rig T26
Ground Surface Elevation (ft msl): 71

		Lithology		Sampling Data								
Depth (ft)	Water Level	Lithologic Symbol	Geologic Description of Soil and Rock Strata	Sample Number	Blows per 6 Inches	N60 value	% Passing #200 Sieve	Liquid Limit	Plastic Limit	Moisture Content (%)	Pocket Pen (tsf)	Remarks
0		▦	Fill - brown sandy silt, roots, and moist									Fill used to form a level spot for the drill rig.
2		▨	Fat Clay (CH) - Medium stiff to very stiff, brown and gray clay with red and orange mottling, intensely weathered siltstone, moist									
				SPT-1	1 2 2	6				46		
4												
6				SPT-2	2 3 4	10	88			42		
8				SPT-3	5 6 10	22				38		
10												
				SPT-4	4 5 7	17				35		
12												
				SPT-5	3 5 7	17				35		
14												
			Becomes gray with red staining									
16	▼			SPT-6	10 7 8	21				30		
18												
20			Becomes gray without staining									
				SPT-7	5 5 8	18	95	56	23	31		
22												
24												
		▤	Siltstone - Soft to very soft, gray, fine grained, slightly weathered siltstone									
26				SPT-8	19 32 50	114				22		
28												
30												
				SPT-9	24 47 50/5"	135				22		
32												
34												
36				SPT-10	30 50 50/5"	139				23		



Notes : Boring terminated at a depth of approximately 36.5 feet bgs. Groundwater was measured at a depth of 16.2 feet on 2/19/20 after being left open overnight. Boring backfilled with bentonite chips on 2/19/2020. N60 values reported are based on a SPT hammer energy correction factor of 1.388 (i.e. 83.3/60). Approximate elevations from Sheet 1 of MCE’s Stanley Roberts Tax Lot 600 - Nenana Avenue Site Layout, 2/9/20.

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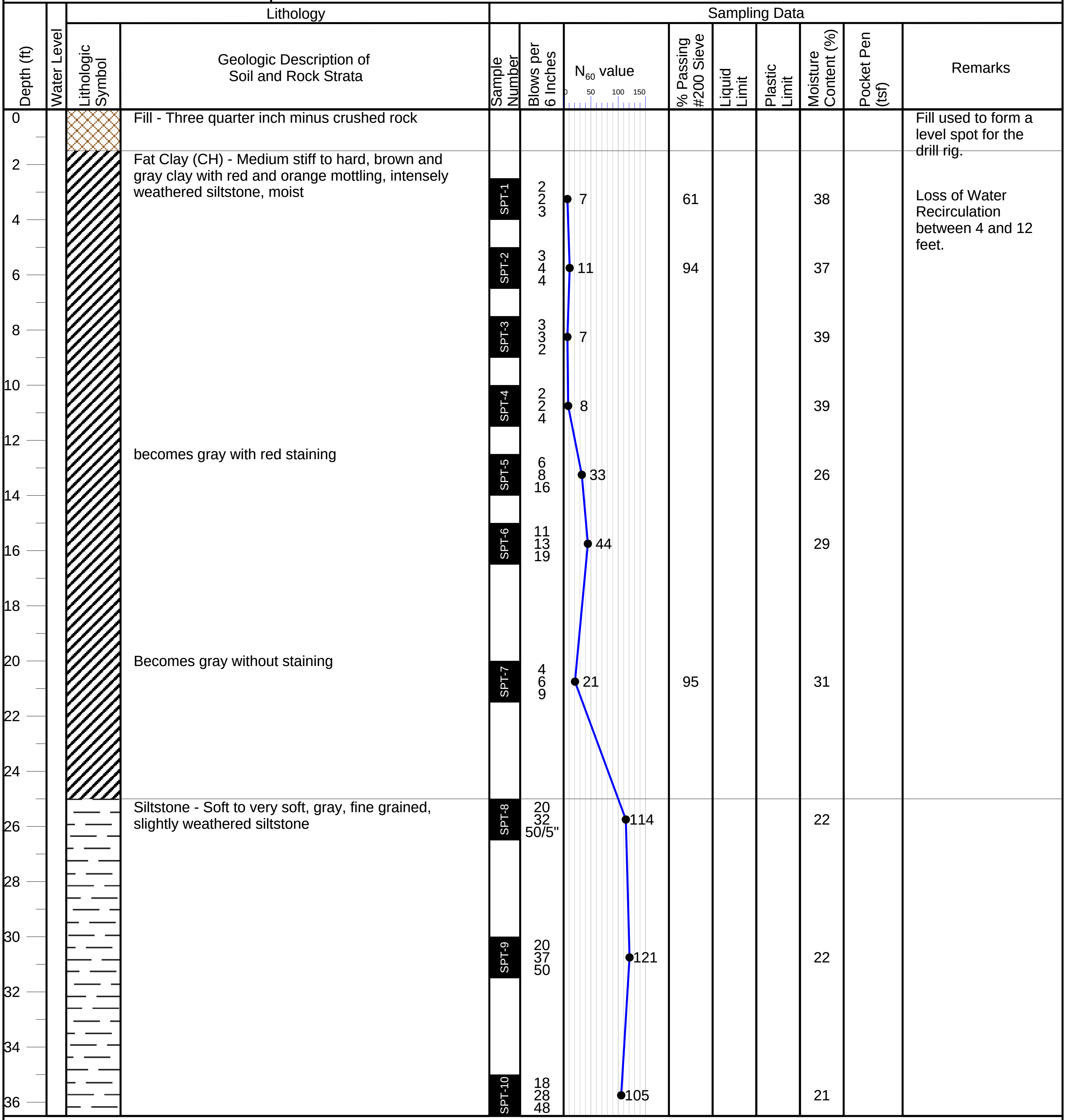
Appendix C: Boring B-2

Sheet 1 of 1

Client: Stan Roberts
Project: Nenana Avenue Construction
Site Address: Proposed Nenana Avenue, Cannon Beach, OR
Location of Borehole: See Appendix B
Date Drilled: 2/19/2020
Logged By: Anita Bauer

Report Number: 20-014-1
Drilling Contractor: PLi Systems
Drilling Method: Mud Rotary
Drilling Equipment: Beretta drill rig T26
Ground Surface Elevation (ft msl): 79



Lithology			Sampling Data								
Water Level	Lithologic Symbol	Geologic Description of Soil and Rock Strata	Sample Number	Blows per 6 Inches	N_{60} value	% Passing #200 Sieve	Liquid Limit	Plastic Limit	Moisture Content (%)	Pocket Pen (tsf)	Remarks
	▦	Fill - Three quarter inch minus crushed rock									Fill used to form a level spot for the drill rig.
	▨	Fat Clay (CH) - Medium stiff to hard, brown and gray clay with red and orange mottling, intensely weathered siltstone, moist									
	▨		SPT-1	2 2 3	7	61			38		Loss of Water Recirculation between 4 and 12 feet.
	▨		SPT-2	3 4 4	11	94			37		
	▨		SPT-3	3 3 2	7				39		
	▨		SPT-4	2 2 4	8				39		
	▨	becomes gray with red staining	SPT-5	6 8 16	33				26		
	▨		SPT-6	11 13 19	44				29		
	▨	Becomes gray without staining	SPT-7	4 6 9	21	95			31		
	☰	Siltstone - Soft to very soft, gray, fine grained, slightly weathered siltstone	SPT-8	20 32 50/5"	114				22		
	☰		SPT-9	20 37 50	121				22		
	☰		SPT-10	18 28 48	105				21		

Notes : Boring terminated at a depth of approximately 36.5 feet bgs. Groundwater was not able to be determined due to mud rotary drilling method. Boring backfilled with bentonite chips on 2/19/2020. N60 values reported are based on a SPT hammer energy correction factor of 1.388 (i.e. 83.3/60). Approximate elevations from Sheet 1 of MCE's Stanley Roberts Tax Lot 600 - Nenana Avenue Site Layout, 2/9/20.

Earth Engineers, Inc.

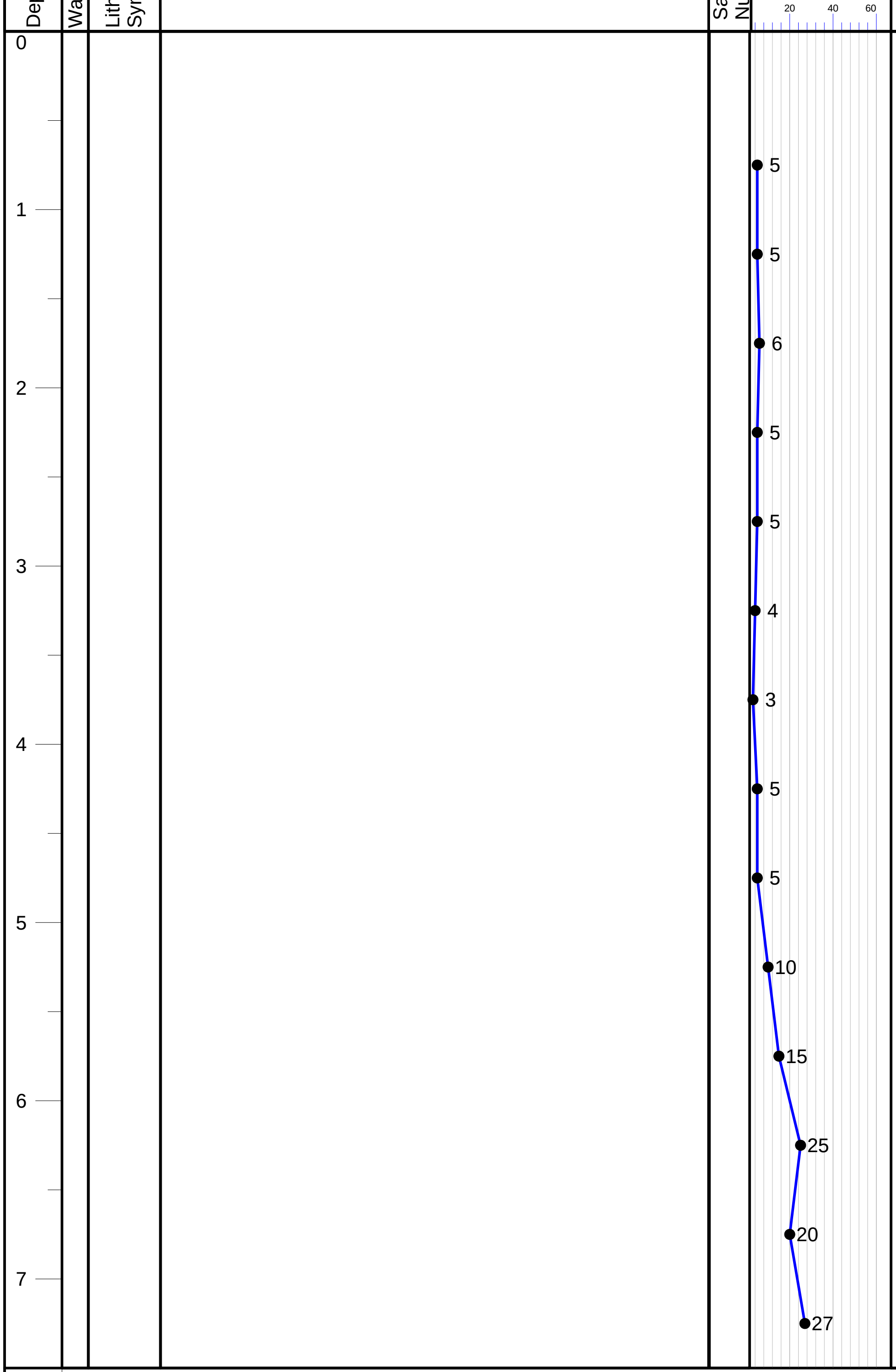
Appendix C: Drive Probe DP-1

Sheet 1 of 1

Client: Stan Roberts
Project: Nenana Avenue Construction
Site Address: Proposed Nenana Avenue, Cannon Beach, OR
Location of Borehole: See Appendix B
Date Drilled: 2/18/2020
Logged By: Troy Hull, P.E., G.E.

Report Number: 20-014-1
Drilling Contractor: N/A
Drilling Method: Hand Equipment
Drilling Equipment: Drive Probe
Ground Surface Elevation (ft msl): 74'

		Lithology		Sampling Data						
Depth (ft)	Water Level	Lithologic Symbol	Geologic Description of Soil and Rock Strata	Sample Number	Drive Probe Blows Per 6 Inches	% Passing #200 Sieve	Liquid Limit	Plastic Limit	Moisture Content (%)	Remarks



Notes : Drive probe test terminated at a depth of approximately 7.5 feet bgs. Approximate elevations from Sheet 1 of MCE’s Stanley Roberts Tax Lot 600 - Nenana Avenue Site Layout, 2/9/20.

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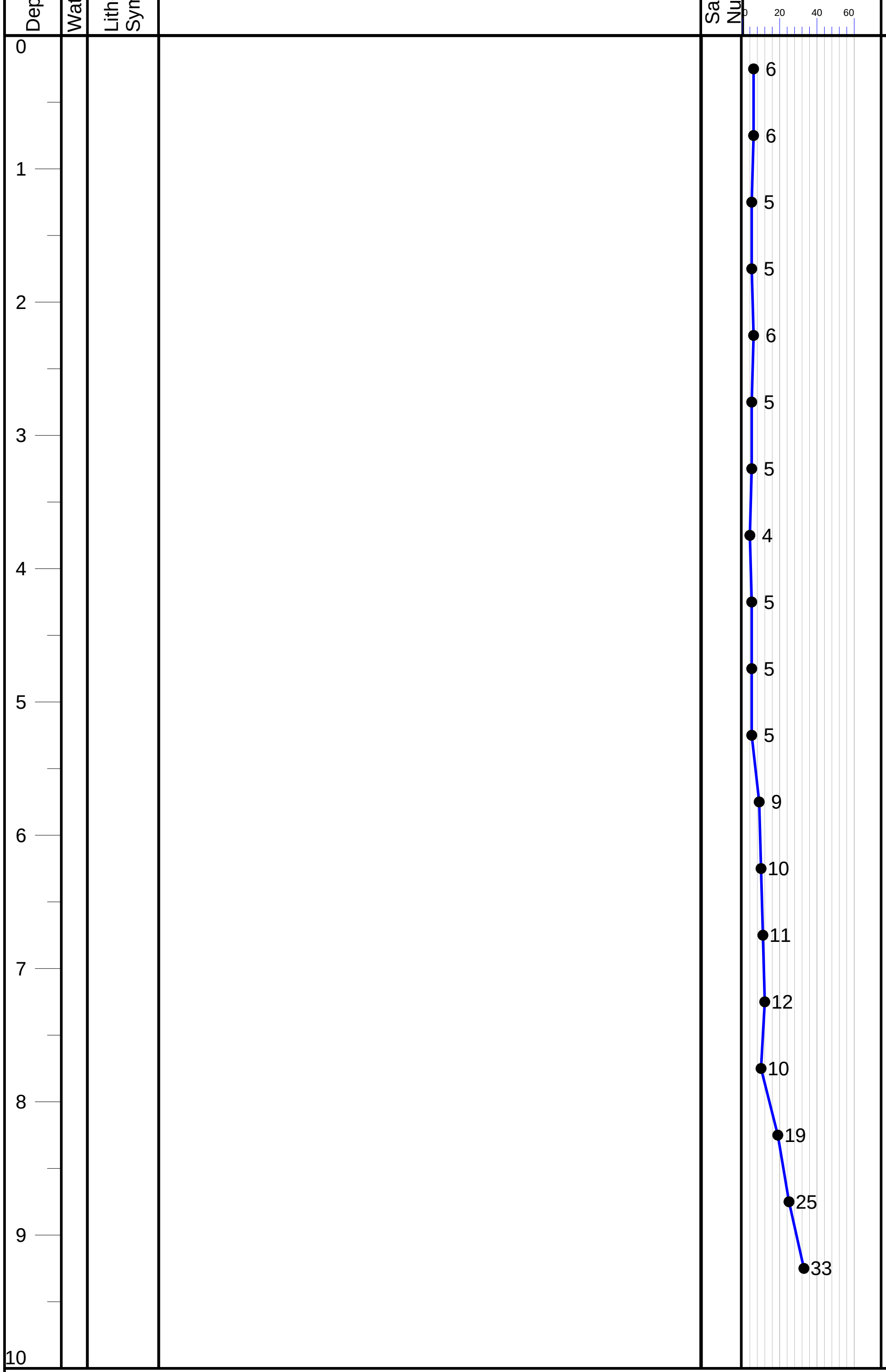
Appendix C: Drive Probe DP-2

Sheet 1 of 1

Client: Stan Roberts
Project: Nenana Avenue Construction
Site Address: Proposed Nenana Avenue, Cannon Beach, OR
Location of Borehole: See Appendix B
Date Drilled: 2/18/2020
Logged By: Troy Hull, P.E., G.E.

Report Number: 20-014-1
Drilling Contractor: N/A
Drilling Method: Hand Equipment
Drilling Equipment: Drive Probe
Ground Surface Elevation (ft msl): 62'

		Lithology		Sampling Data						
Depth (ft)	Water Level	Lithologic Symbol	Geologic Description of Soil and Rock Strata	Sample Number	Drive Probe Blows Per 6 Inches	% Passing #200 Sieve	Liquid Limit	Plastic Limit	Moisture Content (%)	Remarks



Notes : Drive probe test terminated at a depth of approximately 9.5 feet bgs. Approximate elevations from Sheet 1 of MCE’s Stanley Roberts Tax Lot 600 - Nenana Avenue Site Layout, 2/9/20.

APPENDIX D: SOIL CLASSIFICATION LEGEND

APPARENT CONSISTENCY OF COHESIVE SOILS (PECK, HANSON & THORNBURN 1974, AASHTO 1988)

Descriptor	SPT N_{60} (blows/foot)*	Pocket Penetrometer, Qp (tsf)	Torvane (tsf)	Field Approximation
Very Soft	< 2	< 0.25	< 0.12	Easily penetrated several inches by fist
Soft	2 – 4	0.25 – 0.50	0.12 – 0.25	Easily penetrated several inches by thumb
Medium Stiff	5 – 8	0.50 – 1.0	0.25 – 0.50	Penetrated several inches by thumb w/moderate effort
Stiff	9 – 15	1.0 – 2.0	0.50 – 1.0	Readily indented by thumbnail
Very Stiff	16 – 30	2.0 – 4.0	1.0 – 2.0	Indented by thumb but penetrated only with great effort
Hard	> 30	> 4.0	> 2.0	Indented by thumbnail with difficulty

* Using SPT N_{60} is considered a crude approximation for cohesive soils.

APPARENT DENSITY OF COHESIONLESS SOILS (AASHTO 1988)

Descriptor	SPT N_{60} Value (blows/foot)
Very Loose	0 – 4
Loose	5 – 10
Medium Dense	11 – 30
Dense	31 – 50
Very Dense	> 50

MOISTURE (ASTM D2488-06)

Descriptor	Criteria
Dry	Absence of moisture, dusty, dry to the touch, well below optimum moisture content (per ASTM D698 or D1557)
Moist	Damp but no visible water
Wet	Visible free water, usually soil is below water table, well above optimum moisture content (per ASTM D698 or D1557)

PERCENT OR PROPORTION OF SOILS (ASTM D2488-06)

Descriptor	Criteria
Trace	Particles are present but estimated < 5%
Few	5 – 10%
Little	15 – 25%
Some	30 – 45%
Mostly	50 – 100%

Percentages are estimated to nearest 5% in the field. Use "about" unless percentages are based on laboratory testing.

SOIL PARTICLE SIZE (ASTM D2488-06)

Descriptor	Size
Boulder	> 12 inches
Cobble	3 to 12 inches
Gravel - Coarse Fine	¾ inch to 3 inches No. 4 sieve to ¾ inch
Sand - Coarse Medium Fine	No. 10 to No. 4 sieve (4.75mm) No. 40 to No. 10 sieve (2mm) No. 200 to No. 40 sieve (.425mm)
Silt and Clay ("fines")	Passing No. 200 sieve (0.075mm)

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D2488)

Major Division			Group Symbol	Description
Coarse Grained Soils (more than 50% retained on #200 sieve)	**Gravel** (50% or more retained on No. 4 sieve)	Clean Gravel	GW	Well-graded gravels and gravel-sand mixtures, little or no fines
			GP	Poorly graded gravels and gravel-sand mixtures, little or no fines
		Gravel with fines	GM	Silty gravels and gravel-sand-silt mixtures
			GC	Clayey gravels and gravel-sand-clay mixtures
	Sand (> 50% passing No. 4 sieve)	Clean sand	SW	Well-graded sands and gravelly sands, little or no fines
			SP	Poorly-graded sands and gravelly sands, little or no fines
		Sand with fines	SM	Silty sands and sand-silt mixtures
			SC	Clayey sands and sand-clay mixtures
Fine Grained Soils (50% or more passing #200 sieve)	**Silt and Clay** (liquid limit < 50)		ML	Inorganic silts, rock flour and clayey silts
			CL	Inorganic clays of low-medium plasticity, gravelly, sandy & lean clays
			OL	Organic silts and organic silty clays of low plasticity
	Silt and Clay (liquid limit > 50)		MH	Inorganic silts and clayey silts
			CH	Inorganic clays or high plasticity, fat clays
			OH	Organic clays of medium to high plasticity
Highly Organic Soils			PT	Peat, muck and other highly organic soils

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GRAPHIC SYMBOL LEGEND

GRAB	☒	Grab sample
SPT	■	Standard Penetration Test (2" OD), ASTM D1586
ST	▨	Shelby Tube, ASTM D1587 (pushed)
DM	▤	Dames and Moore ring sampler (3.25" OD and 140-pound hammer)
CORE	▥	Rock coring

APPENDIX E: ROCK CLASSIFICATION LEGEND

WEATHERING DESCRIPTORS FOR INTACT ROCK (USBR, 2001)

Descriptor	Chemical Weathering-Discoloration-Oxidation		Mechanical Weathering and Grain Boundary Conditions	Texture and Solutioning		General Characteristics
	Body of Rock	Fracture Surfaces		Texture	Solutioning	
Fresh	No discoloration, not oxidized	No discoloration or oxidation	No separation, intact (tight)	No change	No solutioning	Hammer rings when crystalline rocks are struck
Slightly Weathered	Discoloration or oxidation limited to surface or short distance from fractures; some feldspar crystals are dull	Minor or complete discoloration or oxidation of most surfaces	No visible separation, intact (tight)	Preserved	Minor leaching of some soluble minerals may be noted	Hammer rings when crystalline rocks are struck; body of rock not weakened
Moderately Weathered	Discoloration or oxidation extends from fractures usually throughout; Fe-Mg minerals are “rusty,” feldspar crystals are “cloudy”	All fracture surfaces are discolored or oxidized	Partial separation of boundaries visible	Generally preserved	Soluble minerals may be mostly leached	Hammer does not ring when rock is struck; body of rock is slightly weakened
Intensely Weathered	Discoloration or oxidation throughout; all feldspars and Fe-Mg minerals are altered to clay to some extent or chemical alteration produces in-situ disaggregation	All fracture surfaces are discolored or oxidized; surfaces are friable	Partial separation; rock is friable; granitics are disaggregated in semi-arid conditions	Altered by chemical disaggregation such as via hydration or argillation	Leaching of soluble minerals may be complete	Dull sound when struck with hammer; usually can be broken with moderate to heavy manual pressure or by light hammer blow; rock is significantly weakened
Decomposed	Discolored or oxidized throughout, but resistant minerals such as quartz may be unaltered; all feldspars and Fe-Mg minerals are completely altered to clay		Complete separation of grain boundaries (disaggregation)	Resembles a soil; partial or complete remnant rock structure may be preserved; leaching of soluble minerals usually complete		Can be granulated by hand; resistant minerals such as quartz may be present as “stringers” or “dikes”

RELATIVE STRENGTH OF INTACT ROCK

Descriptor	Uniaxial Compressive Strength (psi)
Extremely Strong	> 30,000
Very Strong	14,500 – 30,000
Strong	7,000 – 14,500
Medium Strong	3,500 – 7,000
Weak	700 – 3,500
Very Weak	150 – 700
Extremely Weak	< 150

BEDDING SPACING (modified USBR, 2001)

Descriptor	Thickness or Spacing
Massive	> 10 feet
Very thickly bedded	3 to 10 feet
Thickly bedded	1 to 3 feet
Moderately bedded	3-5/8 inches to 1 foot
Thinly Bedded	1-1/4 inches to 3-5/8 inches
Very thinly bedded	3/8 inch to 1-1/4 inches
Laminated	< 3/8 inch

CORE RECOVERY CALCULATION (%)

$$= \frac{\text{length of recovered core pieces}}{\text{total length of core run}} \times 100\%$$

RQD CALCULATION (%)

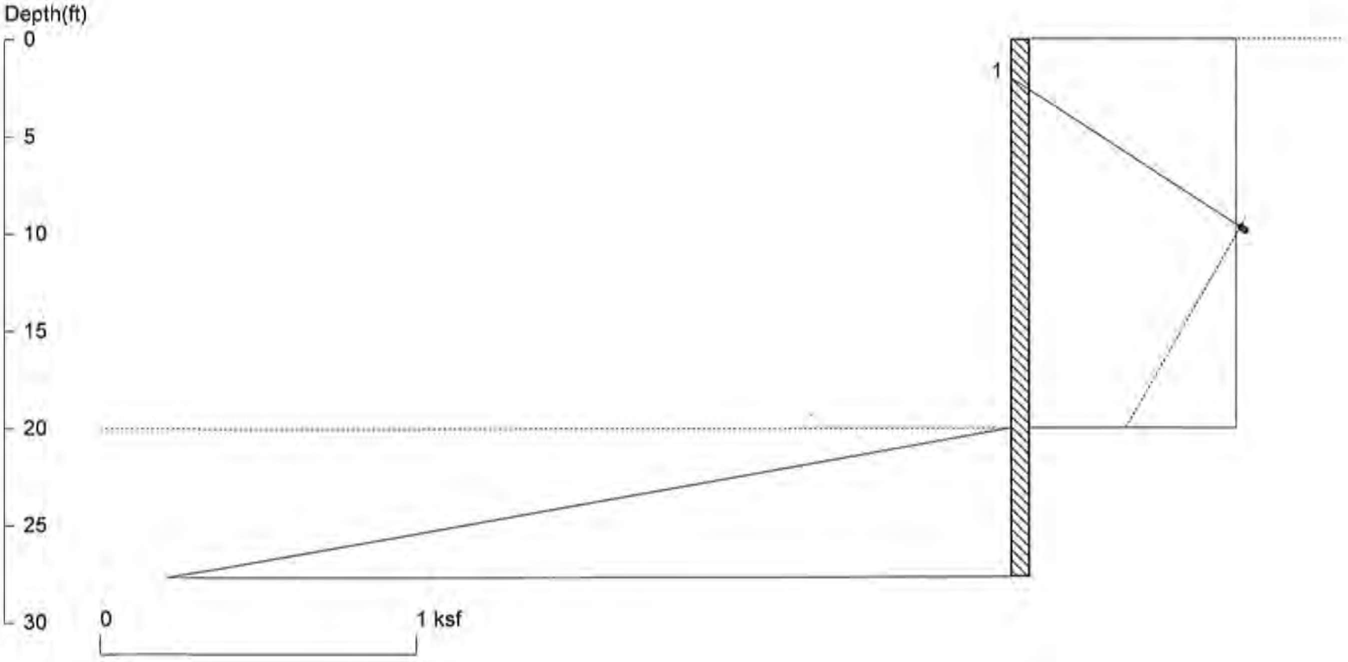
$$= \frac{\text{length of intact core pieces} > 4 \text{ in}}{\text{total length of core run (inches)}} \times 100\%$$

Earth Engineers, Inc.

ROCK HARDNESS (modified USBR, 2001)

Descriptor	Criteria
Extremely hard	Cannot be scratched with pocket knife or sharp pick; can only be chipped with repeated heavy hammer blows
Very hard	Cannot be scratched with pocket knife or sharp pick; breaks with repeated heavy hammer blows
Hard	Can be scratched with pocket knife or sharp pick with heavy pressure, heavy hammer blows required to break specimen
Moderately hard	Can be scratched with pocket knife or sharp pick with light or moderate pressure; breaks with moderate hammer blows
Moderately soft	Can be grooved 1/16 inch with pocket knife or sharp pick with moderate or heavy pressure; breaks with light hammer blow or heavy hand pressure
Soft	Can be grooved or gouged with pocket knife or sharp pick with light pressure; breaks with light to moderate hand pressure
Very soft	Can be readily indented, grooved, or gouged with fingernail, or carved with pocket knife; breaks with light hand pressure

APPENDIX F
SHORING SUITE CALCULATIONS



<ShoringSuite> CIVILTECH SOFTWARE USA www.civiltech.com

Licensed to Earth Engineers, Inc. Date: 4/11/2020
File: P:\Projects\2020 Projects\20-014 (Nenana Avenue Construction, Cannon Beach, OR) (AB-TH)\Shoring Suite Stuff\

Minimum
Wall Height=20.0 Pile Diameter=1.3 Pile Spacing=6.0 Wall Type: 2. Soldier Pile, Drilled
↖ assumed exposed pile length due to west slope slide.
~~PILE LENGTH: Min. Embedment=7.66 Min. Pile Length=27.66~~
~~MOMENT IN PILE: Max. Moment=235.01 per Pile Spacing=6.0 at Depth=13.08~~

BRACE FORCE: Strut, Tieback, Plate Anchor, Deadman, Sheet Pile as Anchor

No. & Type	Depth	Angle	Space	Total F.	Horiz. F.	Vert. F.	L_free	Fixed Length
1. Tieback	2.0	35.0	15.0	158.0	129.5	90.6	13.4	14.0

UNITS: Width,Diameter,Spacing,Length,Depth,and Height - ft; Force - kip; Bond Strength and Pressure - ksf

DRIVING PRESSURES (ACTIVE, WATER, & SURCHARGE):

Z1	P1	Z2	P2	Slope
0	0.660	20	0.660	0

PASSIVE PRESSURES: Pressures below will be divided by a Factor of Safety =1.5

Z1	P1	Z2	P2	Slope
20	0	40	7.000	0.350

ACTIVE SPACING:

No.	Z depth	Spacing
1	0.00	6.00
2	20.00	1.33

PASSIVE SPACING:

No.	Z depth	Spacing
1	20.00	4.00

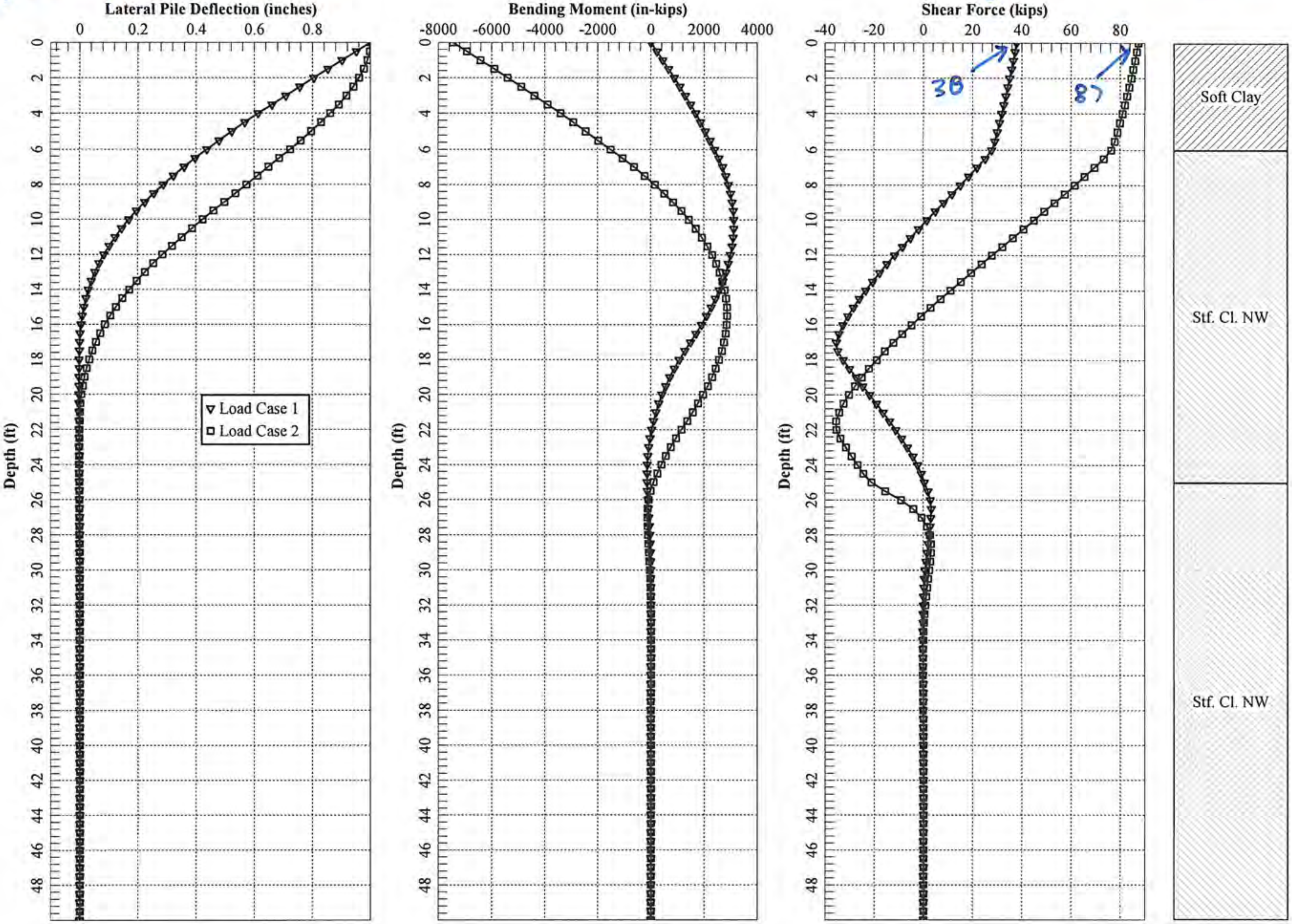
UNITS: Width,Spacing,Diameter,Length,and Depth - ft; Force - kip; Moment - kip-ft
Friction,Bearing,and Pressure - ksf; Pres. Slope - kip/ft3; Deflection - in

APPENDIX G
LPILE CALCULATIONS

W14 x 68 - STRONG AXIS
30" hole
4,000 psi concrete

ULTIMATE VALUES SHOWN

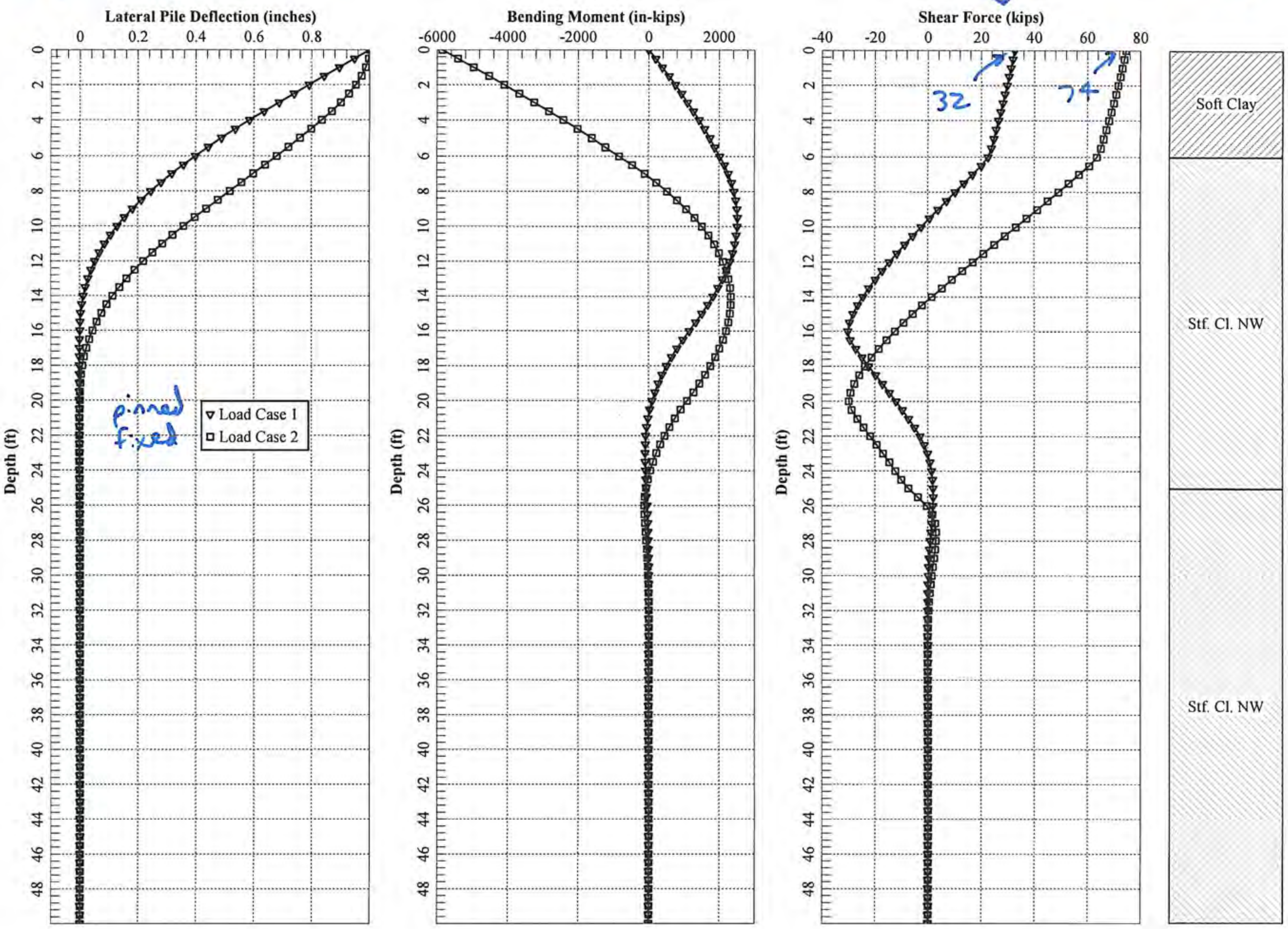
$$\text{ALLOWABLE} = \frac{\text{ULTIMATE}}{2}$$



W14 x 68 – WEAK AXIS
30" hole
4,000 psi concrete

ULTIMATE VALUES SHOWN

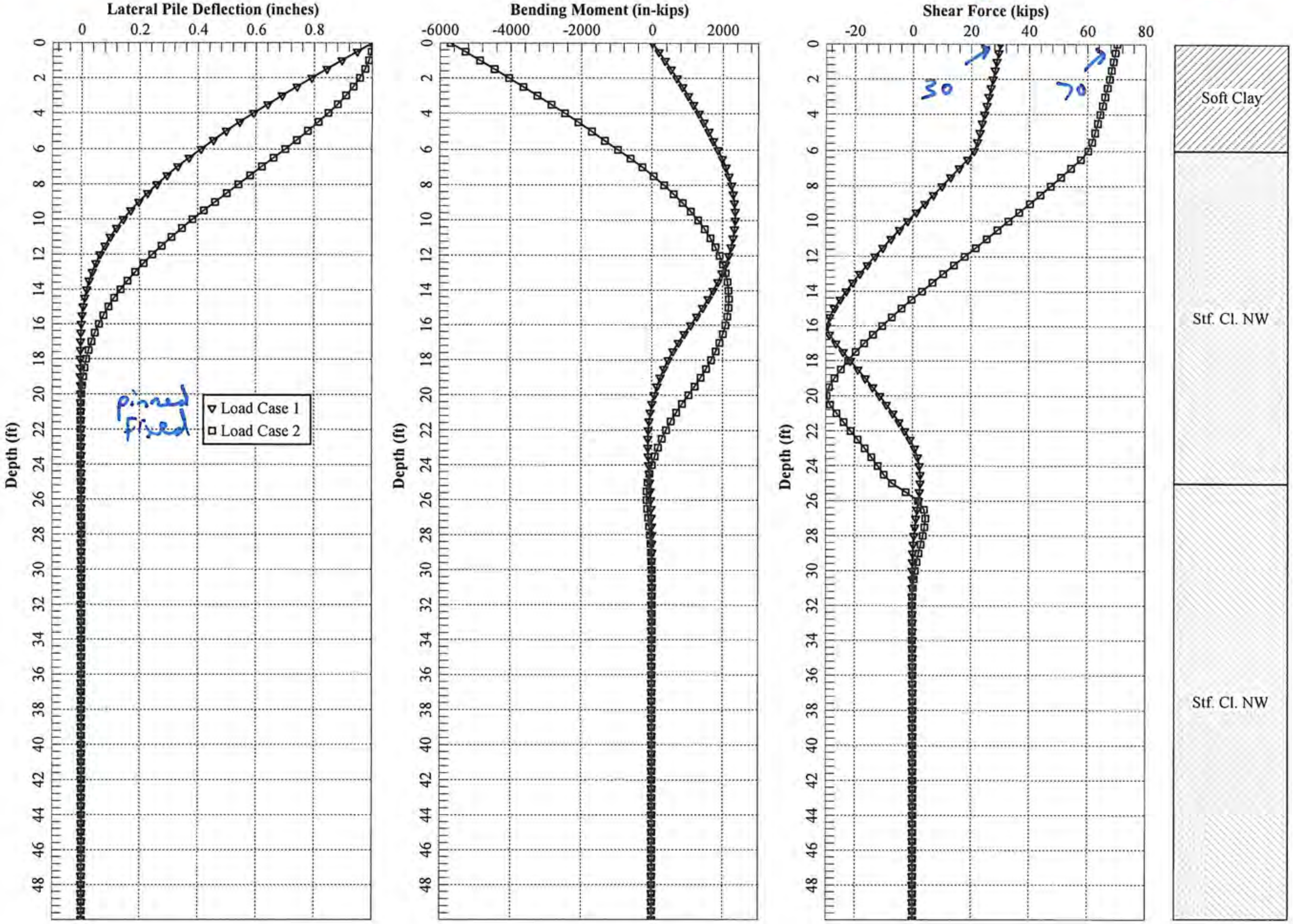
$\text{ALLOWABLE} = \frac{\text{ULTIMATE}}{2}$



W14 x 68 - STRONG AXIS
24" hole
4,000 psi Concrete

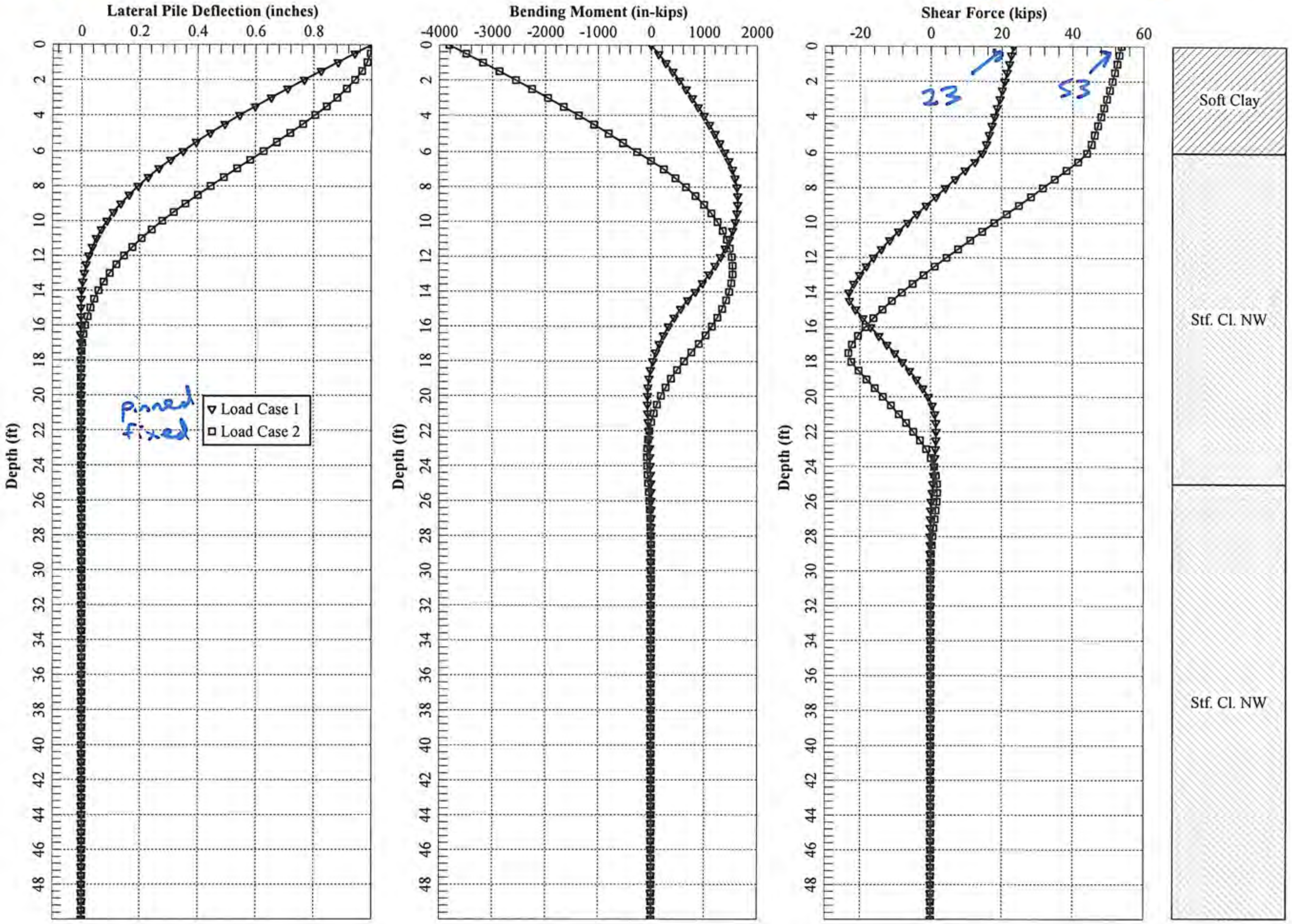
ULTIMATE VALUES SHOWN

ALLOWABLE = $\frac{\text{ULTIMATE}}{2}$



W14x68 – WEAK AXIS
24" hole
4,000 psi concrete

- ULTIMATE VALUES SHOWN
- ALLOWABLE = $\frac{\text{ULTIMATE}}{2}$



Pinned
fixed

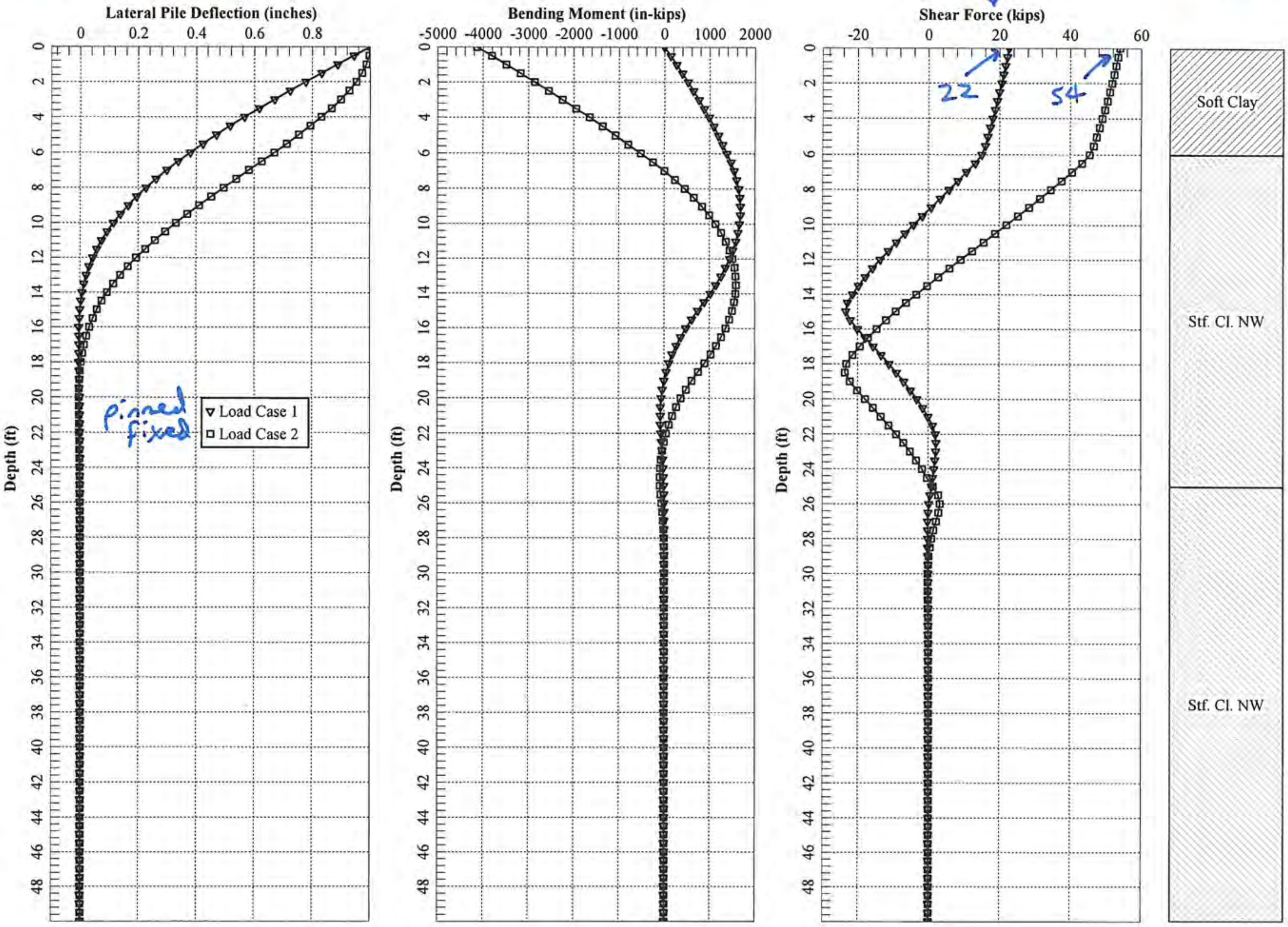
23

53

W12X72 - STRONG AXIS
18" hole
4,000 psi concrete

ULTIMATE VALUES SHOWN

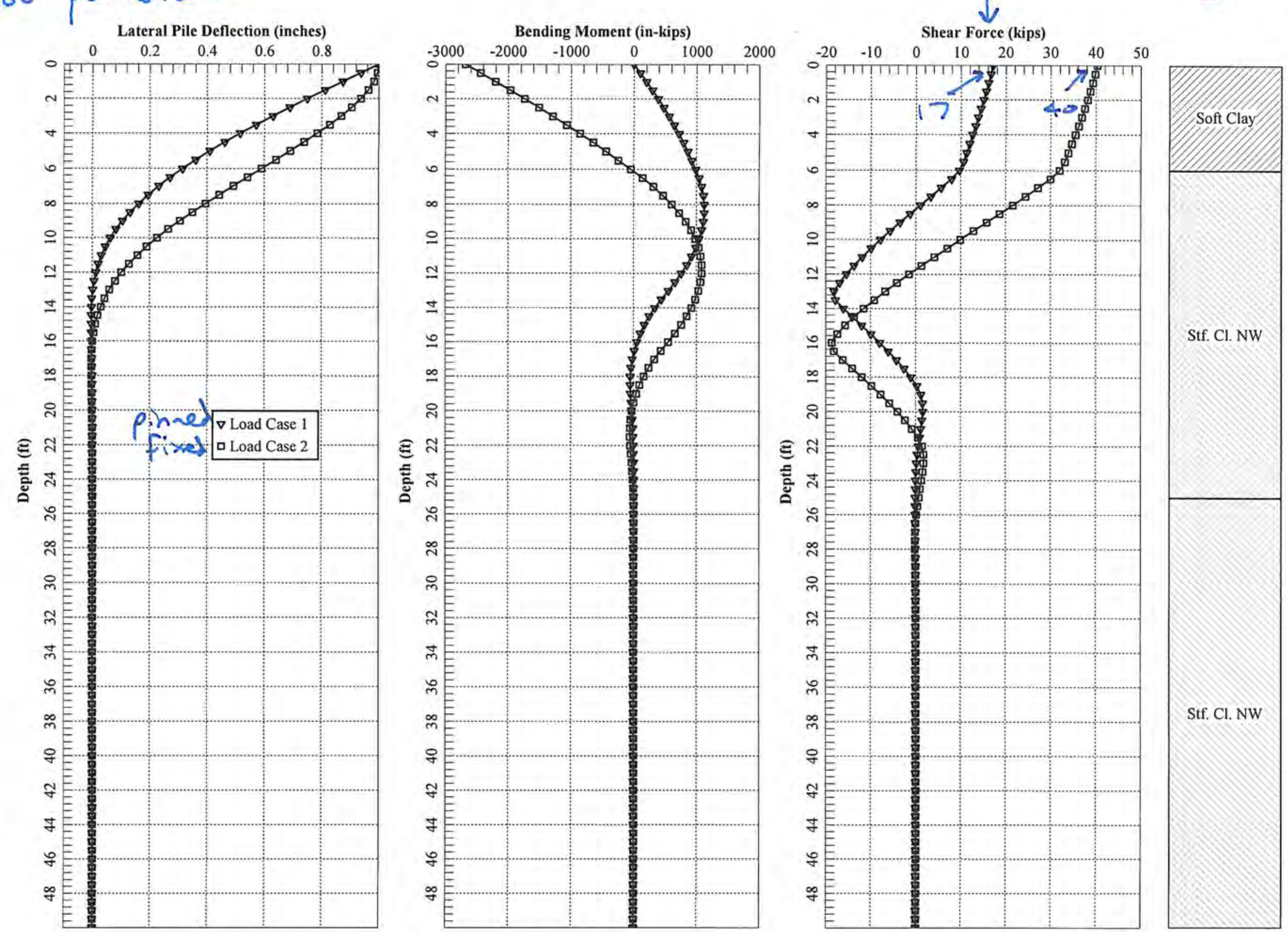
ALLOWABLE = $\frac{\text{ULTIMATE}}{2}$



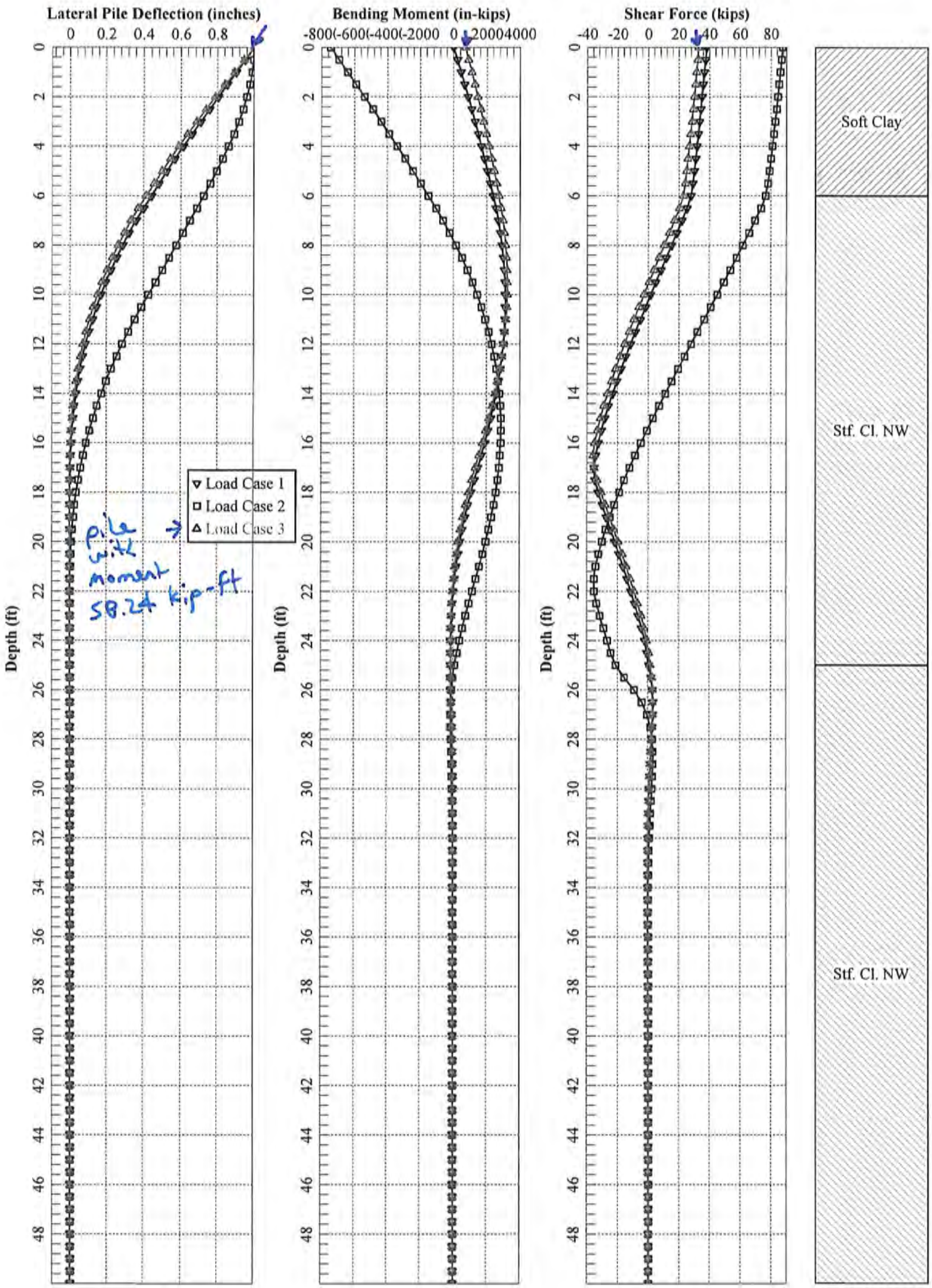
W12 x 72 - WEAK AXIS
18" hole
4,000 psi concrete

ULTIMATE VALUES SHOWN

$\text{ALLOWABLE} = \frac{\text{ULTIMATE}}{2}$



30" pile, strong axis. Moment = 58.24 k[illegible] inch-lbs.

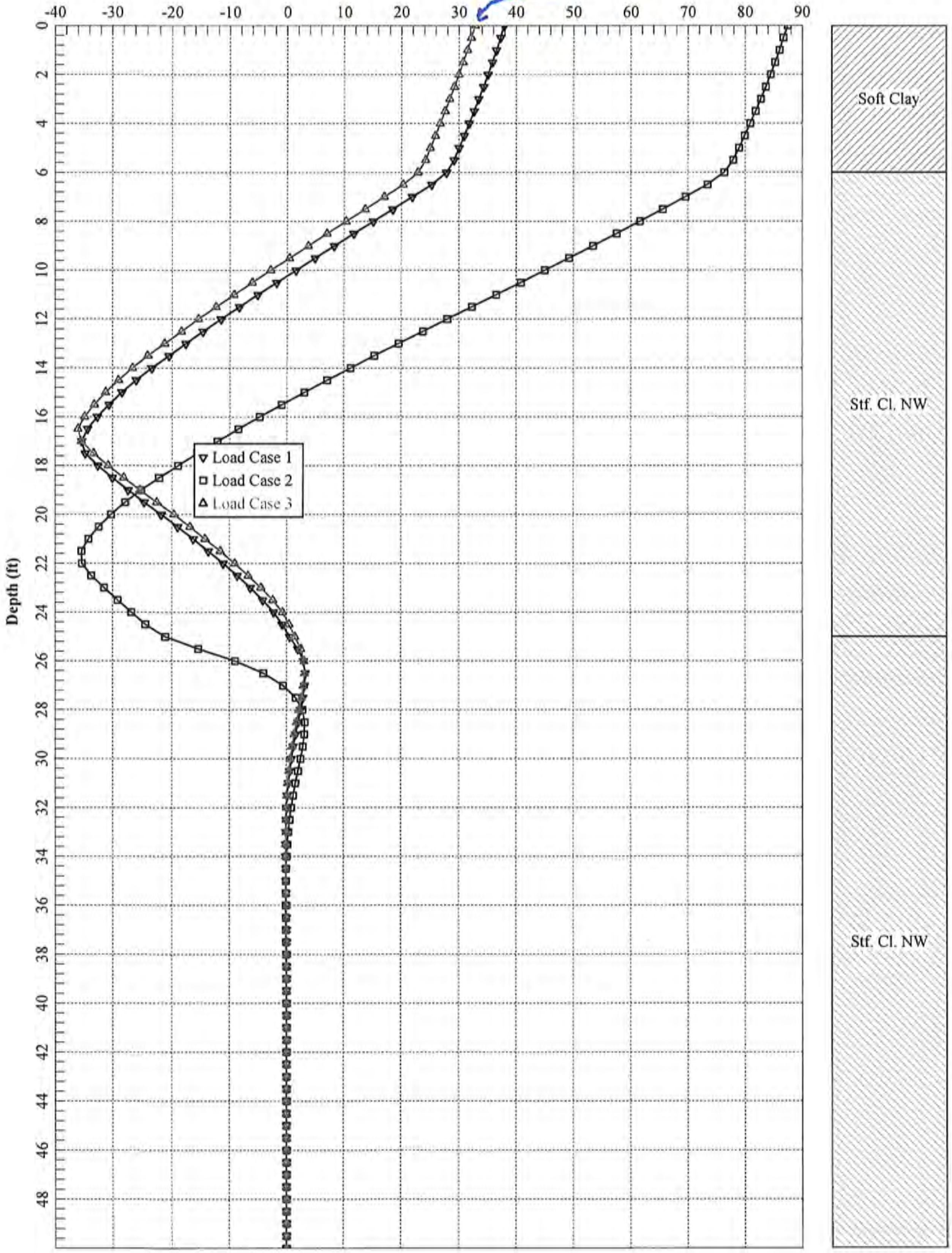


pile with moment 58.24 kip-ft

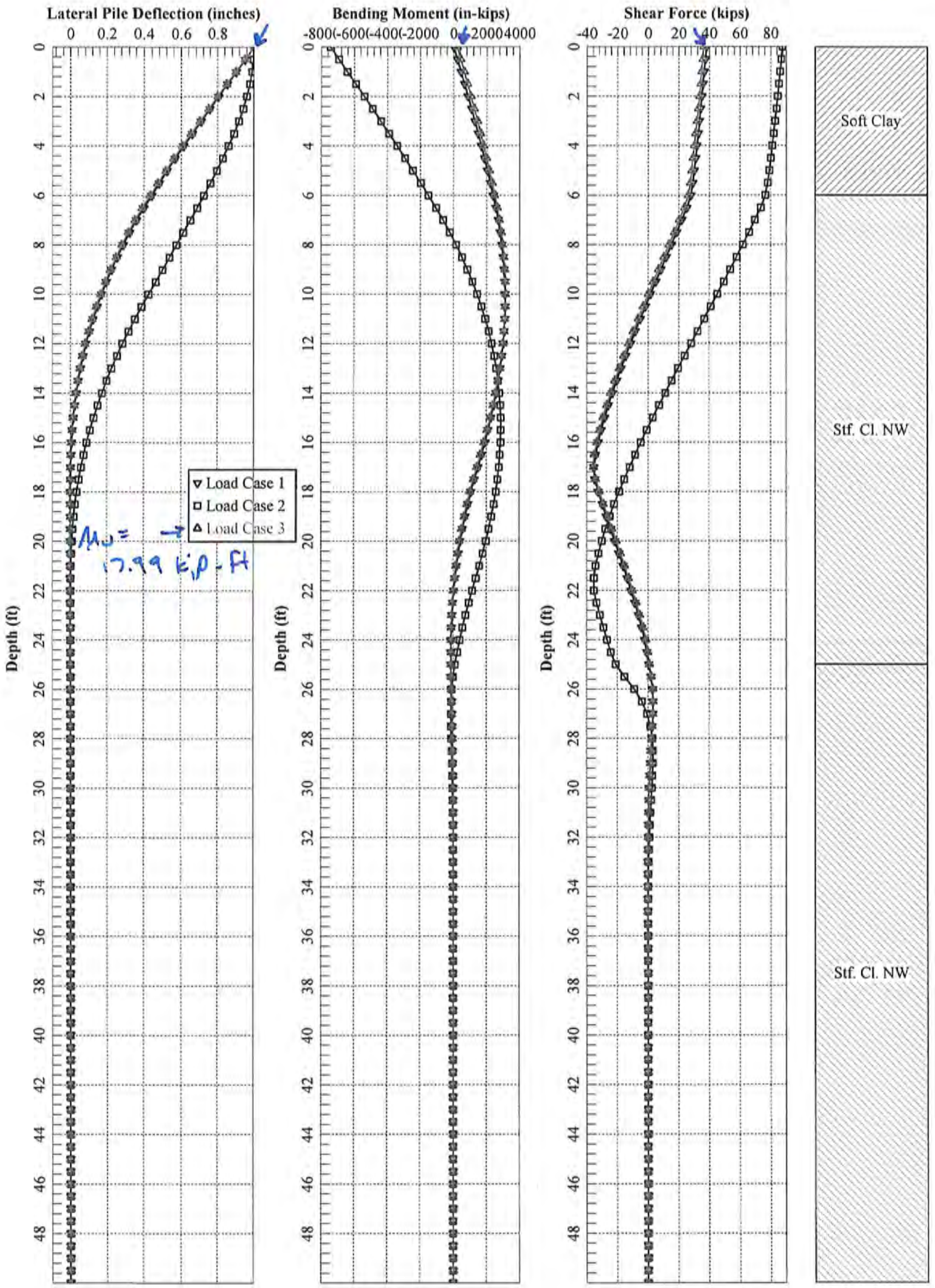
33 kips ultimate

30-INCH DIAMETER DRILLED PIER

Shear Force (kips)



30" pile, strong axis, Mu = 17.99 kip-ft (215,880 in-lb

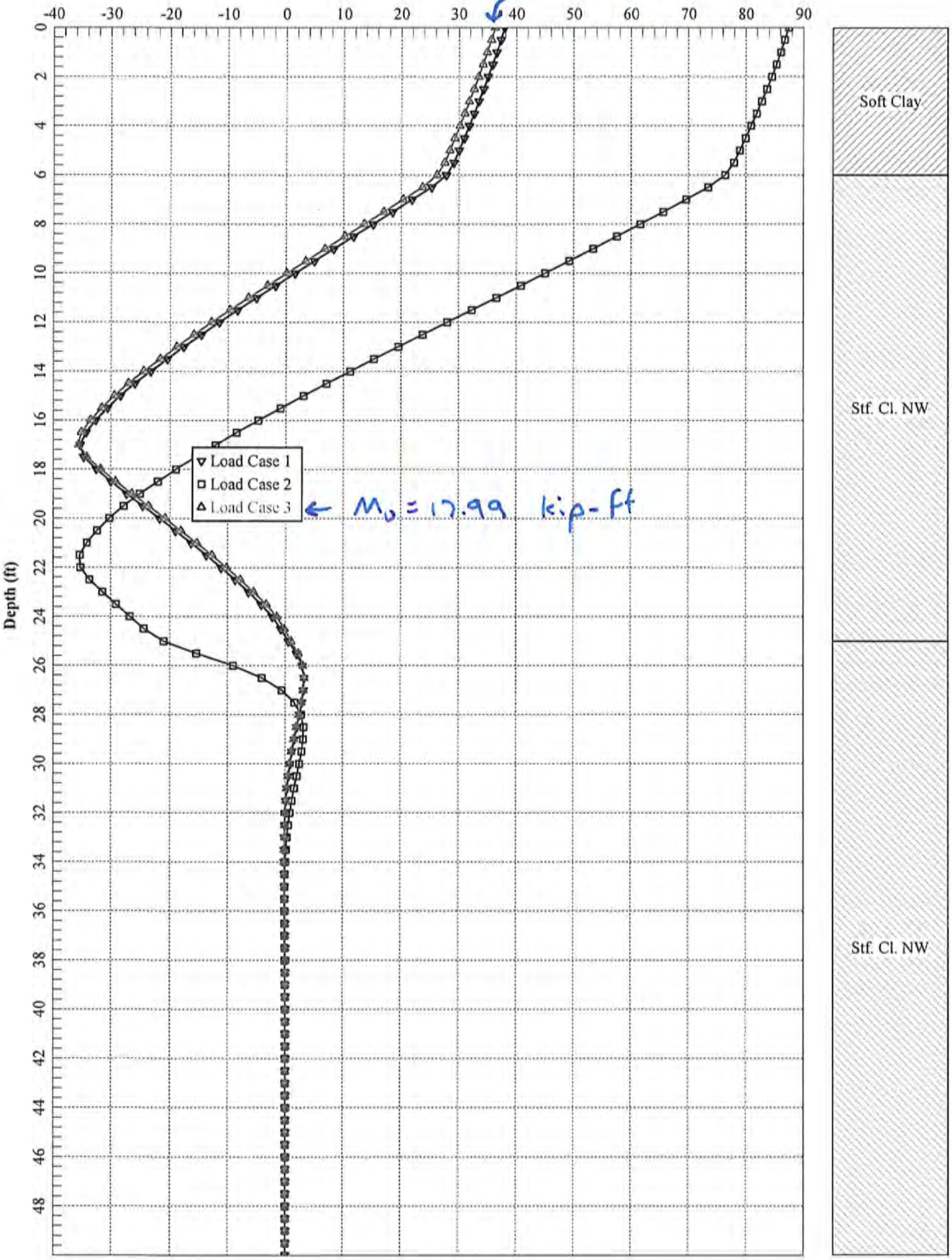


Mu = → 17.99 kip-ft

30" pile, strong axis, $M_u = 17.99$ kip-ft / 215[illegible]80 inch-lb

~~24~~ 30-INCH DIAMETER DRILLED PIER
Shear Force (kips)

36 kips ultimate



← $M_u = 17.99$ kip-ft

Geotech Solutions Inc.

June 20, 2020 *robertscannon-18-1-consultte*

Stanley and Rebecca Roberts
stan.milliman@gmail.com

Cc:
kevin@objectiveadvisorsllc.com
plandevelopment@msn.com
eric@miller-se.com
troy@earth-engineers.com

GEOTECHNICAL ENGINEERING CONSULTATION
Roadway Stability Impacts from Nenana Avenue Improvements
Cannon Beach, Oregon

This letter summarizes our review of geotechnical aspects of the structural engineering foundation plans and associated geotechnical report for the Nenana Avenue roadway west of Hemlock Street. The geotechnical report by Earth Engineers indicates drilled cast-in-place reinforced concrete piers embedded to siltstone used to support the roadway, with roadway runoff collected and routed to erosion protected discharge. Each of these approaches is suitable.

The roadway piers transfer load to the ground, which is part of the S-curves landslide. Mr. Rondema in our office has studied the S-curves slide for the City and other private parties for over 20 years. An extensive description of the slide and stability improvement is included in our report for Tax Lot 600. Stabilization solutions recommended by Geotech Solutions, Inc. for the S-curves slide were implemented in 2007-8 and included 19 horizontal drains below the S-curves including the Nenana ROW. These drains are cleaned annually by the City. Extensive long-term monitoring of the movement since drain installation indicates up to 0.2 inches of movement in the last 12 years on the shear surfaces at depth. These were documented from very heavy rainfall events in some years. No movement was detected in other years. Instruments and analyses were recently evaluated again prior to the City's installation of a new water line in Hemlock Street. Hemlock Street and its associated utilities have not shown slide damage since drain installation.

For the analyses herein we updated our established stability cross sections that extend from below the oceanfront toe onto the beach to up above Hemlock Street. Critical movement directions are as previously documented from slide movement, and combined with site topography the most conservative section was consistent with Profile 1 on the attached figure. Analyses parameters were based on extensive monitoring, multiple borings and inclinometer readings, and back calculation of soil strength on discrete observed shear surfaces, calibrated by years of data from movement, rainfall events, and digital water level data. A new detailed topographic survey was also requested and used in this work. From the preceding experience we established a stability model using the program SLIDE and limit equilibrium methods and derived a threshold factor of safety for the roadway, summarized in the attached figures. It should be noted that each of the neighboring lots and Hemlock Street itself north through Haystack Lane is subject to instability in earthquake loading, and the Nenana ROW is part of this, regardless of roadway improvements.

We applied the dead load capacity of piers as a distributed load over the roadway to capture varied bent spacing and pier placement and applied this to the portion of the roadway intersecting the critical cross section. This includes the upper portion of the roadway where loading is most critical. Based on our analyses, using the loading provided, there is no appreciable reduction (less than 1%) in the critical slope stability factors of safety for the S-curves slide from roadway loads, including neighboring lots and Hemlock Street. This is due to the very large size and depth, and therefore very high weight, of the slide itself. Compared to the slide, the roadway weight is very small. In addition, capture of runoff from this new roadway area will reduce infiltration into the upper part of the slide. A plot of our stability model and associated outputs for the road loading condition is also attached for reference.

The Limitations of our reports apply. If you have any questions, please contact us.

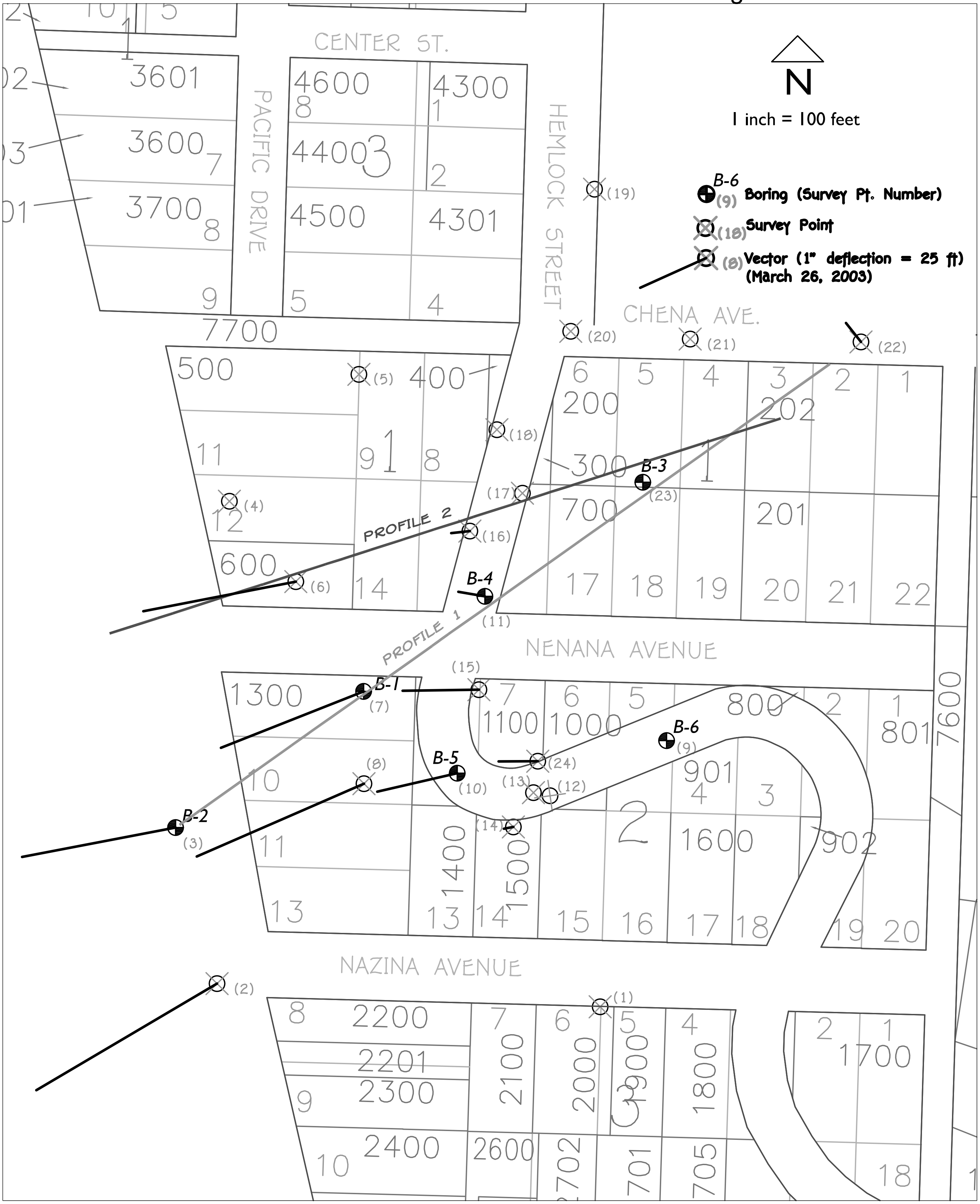
Sincerely,

Don Rondema, MS, PE, GE
Principal

DON RONDEMA
Expires 12/31/20

Attached:

Stability cross section Profile 1, Structural Plan S1.01 excerpt, stability models (4)



SITE PLAN

Cannon-02-01-gi

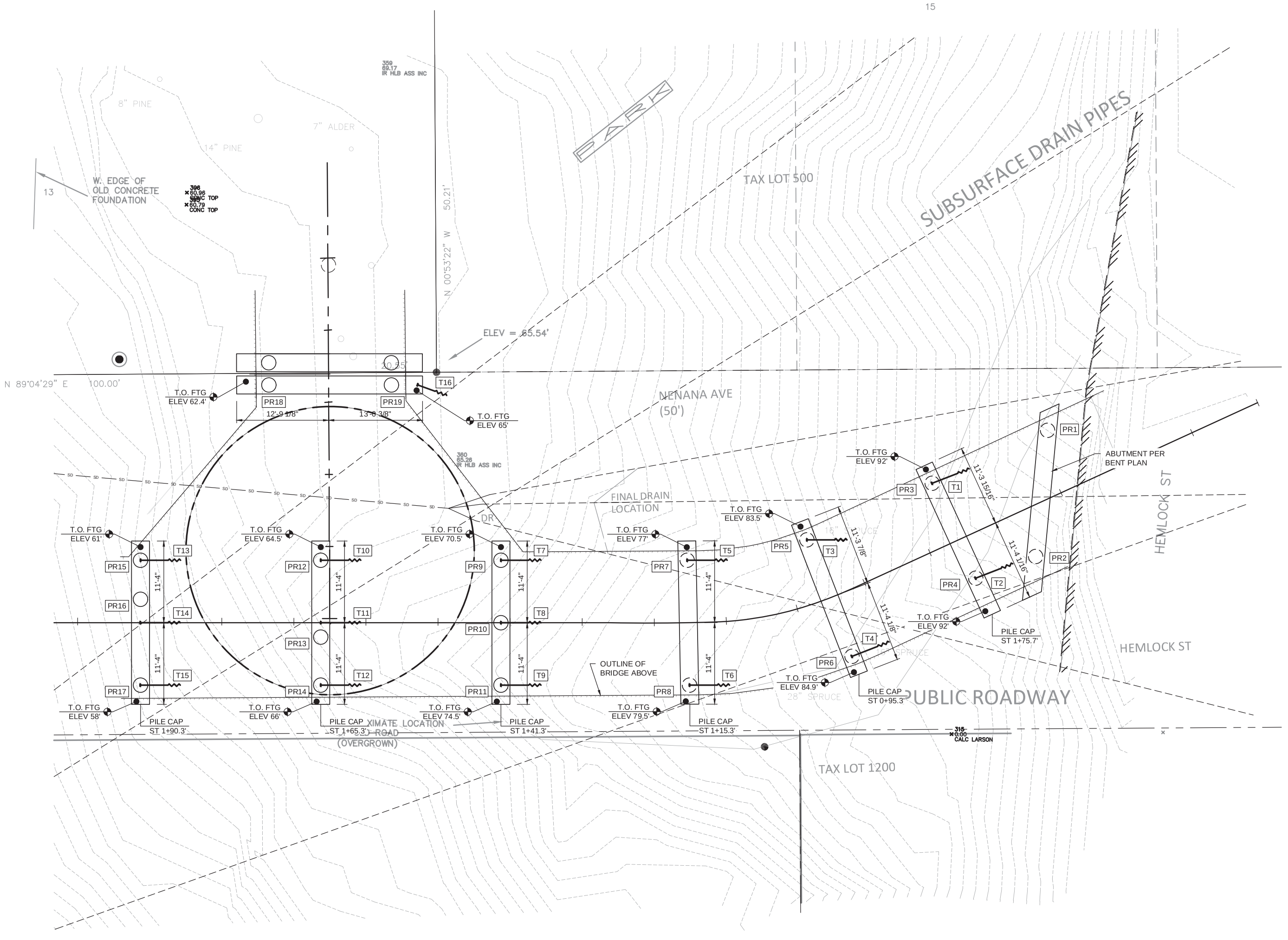
Geotech Solutions Inc.

GENERAL NOTES

1. FIELD VERIFY EXISTING CONDITIONS PRIOR TO PURCHASE OF MATERIALS AND CONSTRUCTION
2. THESE DRAWINGS ARE TO ACCOMPANY DRAWINGS BY MORGAN CIVIL ENGINEERING, INC. REFERENCE CIVIL DRAWING FOR ROAD FINISH ELEVATIONS AND STATIONING
3. REFERENCE SHEET S0.02 AND S0.03 FOR STRUCTURAL NOTES
4. REFERENCE SHEET S8.01 FOR TYPICAL CONCRETE DETAILS

DRILLED PIER SCHEDULE FOR S1.01

	PLAN ID	DEAD LOAD (KIPS)	GLOBAL LIVE LOAD (KIPS)
BENT 1	PR1	58.8	7.7
BENT 1	PR2	35.5	4.2
BENT 2	PR3	101.3	8.8
BENT 2	PR4	96.9	11.1
BENT 3	PR5	112.6	4.5
BENT 3	PR6	131.3	3.6
BENT 4	PR7	104.2	9.8
BENT 4	PR8	118.2	10.5
BENT 5	PR9	105.0	8.8
BENT 5	PR10	96.0	14.3
BENT 5	PR11	63.1	12.2
BENT 6	PR12	139.0	19.6
BENT 6	PR13	95.7	10.0
BENT 6	PR14	77.5	10.1
BENT 7	PR15	88.0	9.0
BENT 7	PR16	68.3	5.2
BENT 7	PR17	55.2	6.0
BENT 8	PR18	90.7	4.0
BENT 8	PR19	82.9	4.0



1 S1.01 PILE CAP AND DRILLED PIER PLAN

1/8" = 1'-0"

PRELIMINARY
NOT FOR CONSTRUCTION

MILLER
CONSULTING
ENGINEERS
9570 SW Barbur Blvd
Suite One Hundred
Portland, OR 97219
Phone 503.246.1250
www.miller-se.com

STREET EXTENSION

ROBERTS RESIDENCE

2150 S HEMLOCK STREET
CANNON BEACH, OREGON

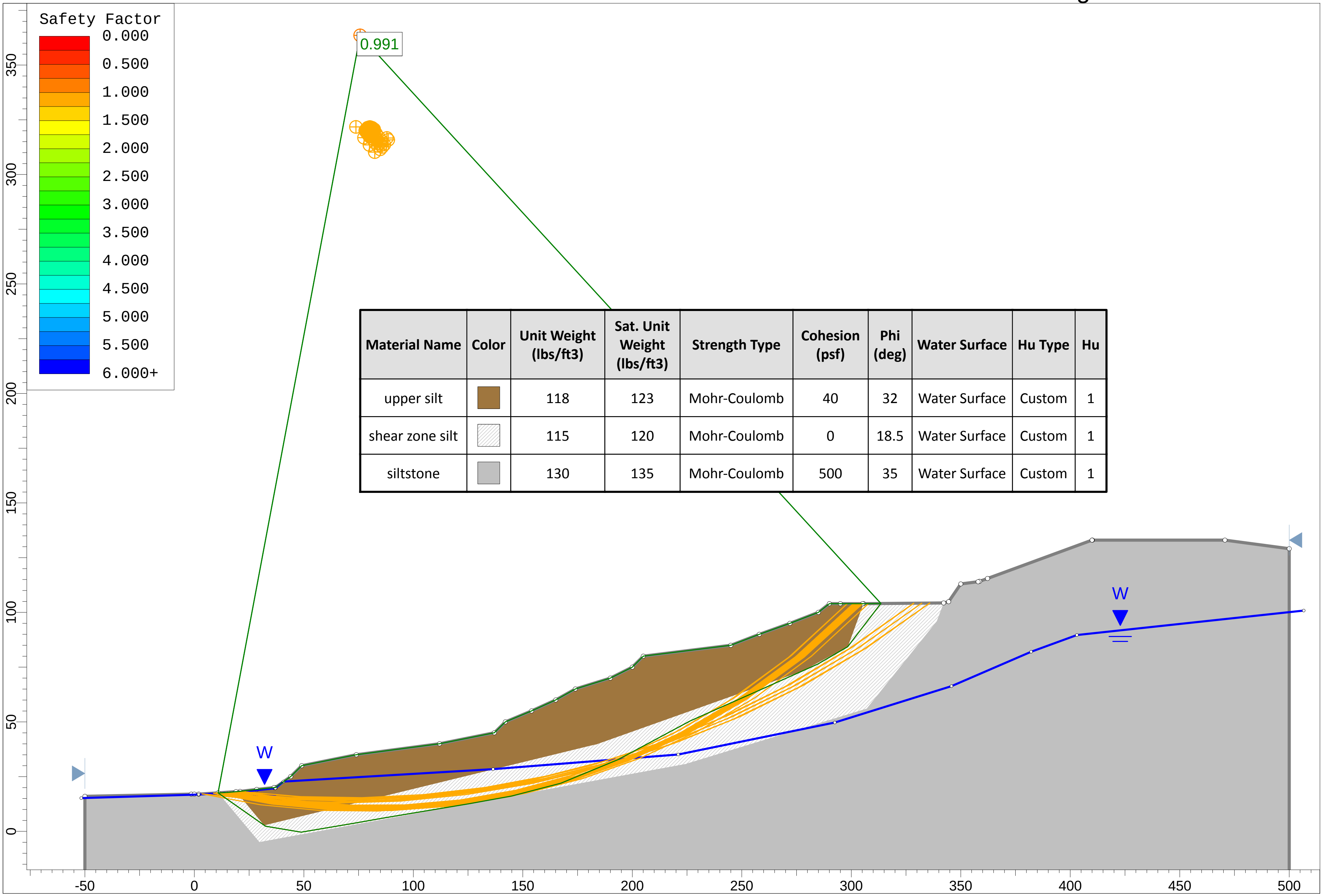
LINE IS 2 INCHES
AT FULL SCALE
(IF NOT 2" - SCALE ACCORDINGLY)

DRAWN BY: BCH
CHECKED BY:
MCE PROJECT NO: 200209
ISSUE DATE: 03.25.2020

REV.	DATE	DESCRIPTION

SHEET CONTENT
FOUNDATION PLAN

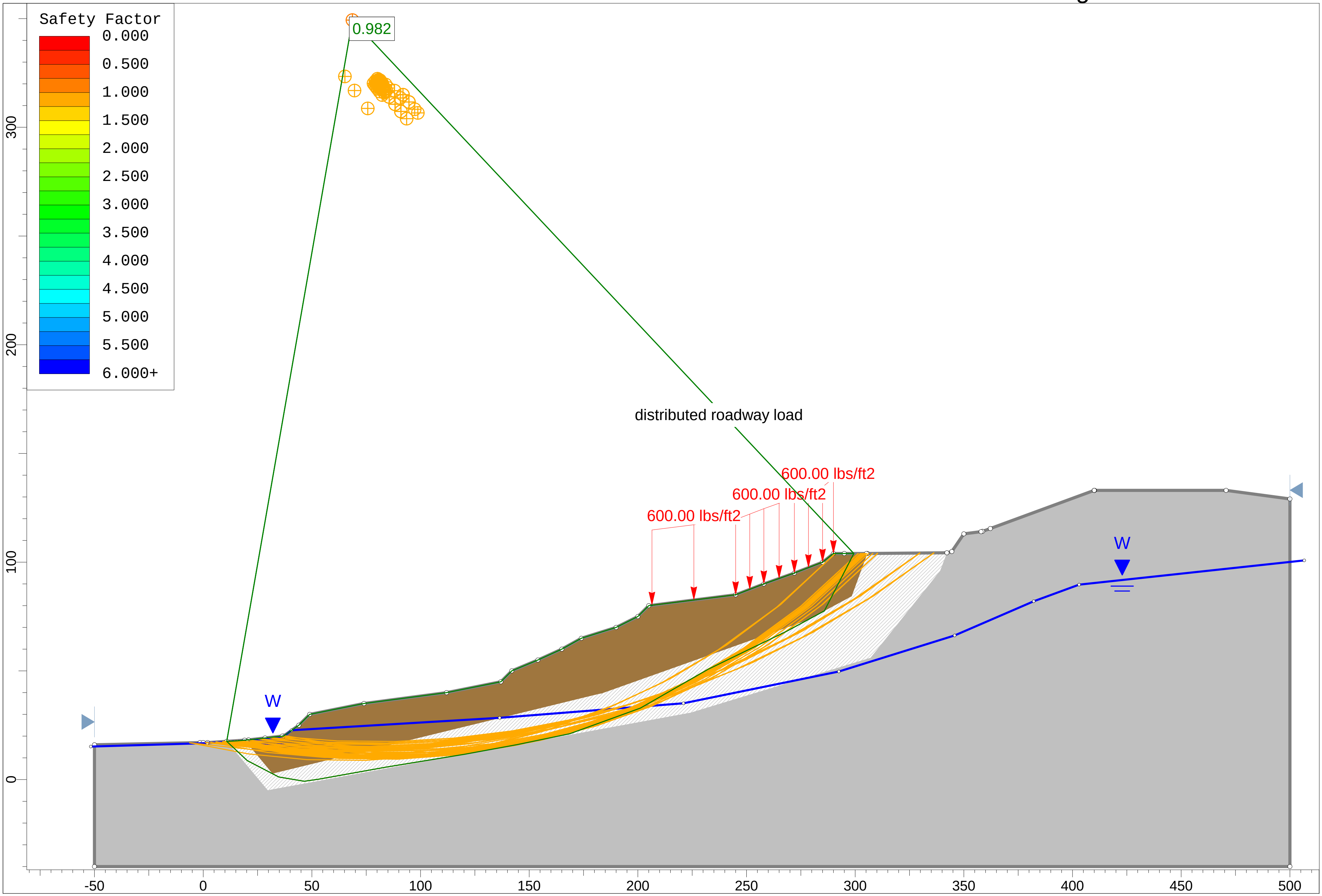
SHEET
S1.01



Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type	Hu
upper silt	🟫	118	123	Mohr-Coulomb	40	32	Water Surface	Custom	1
shear zone silt	▨	115	120	Mohr-Coulomb	0	18.5	Water Surface	Custom	1
siltstone	■	130	135	Mohr-Coulomb	500	35	Water Surface	Custom	1

nenana profile 1-winter drained defined shear zone.slim

Geotech Solutions, Inc.



nenana profile 1-winter drained defined shear zone.slim

Geotech Solutions, Inc.

Geotech Solutions Inc.

August 14, 2020 *robertscannon-18-1-consultte*

Stanley and Rebecca Roberts
stan.milliman@gmail.com

Cc:
kevin@objectiveadvisorsllc.com
plandevelopment@msn.com
eric@miller-se.com
troy@earth-engineers.com

GEOTECHNICAL ENGINEERING CONSULTATION
Purpose of Western Stability Pile System
Cannon Beach, Oregon

The purpose of our extensive detailed design of the western batter piles and associated grade beam is to improve stability of Tax Lot 600 (as summarized in our attached report and submitted reviews of geotechnical related structural plans). That grade beam is not relied on by any building foundation system for structural support. It is strictly dedicated to improving the lot stability. In that regard, we recommend its installation as soon as possible. Once this system is in, the lot stability is significantly improved, and any foundation elements could then be installed regardless of the season. Those foundation elements, whether they match a future approved building footprint or not, would also improve the stability of the lot, and any potential future house.

The Limitations of our reports apply. If you have any questions, please contact us.

Sincerely,

Don Rondema, MS, PE, GE
Principal

DON RONDEMA
Expires 12/31/20

REPORT OF GEOTECHNICAL ENGINEERING SERVICES

Proposed Residence at Tax Lot 600 - North of Nenana ROW
Cannon Beach, Oregon

Geotech Solutions Inc.

June 6, 2020

GSI Project: robertscannon-18-1-consult3

Geotech Solutions Inc.

June 6, 2020 *robertscannon-18-1-consult3*

Stanley and Rebecca Roberts
stan.milliman@gmail.com

Cc: Kevin Patrick; kevin@objectiveadvisorsllc.com

GEOTECHNICAL ENGINEERING CONSULTATION
House Foundation Support and Stability Analyses
Tax Lot 600, Nenana Avenue Oceanfront Lot - Cannon Beach, Oregon

As authorized, this report summarizes our geotechnical engineering consultation for the subject site's house foundation support and stability analyses. Mr. Rondema has studied the lot since 1995 for several owners and has extensive involvement in the S-curves slide evaluation and stability improvements for the City of Cannon Beach. The purpose of our work was to provide geotechnical engineering analyses and consultation to the design and planning team as requested for the house design. Our scope of services included the following:

- Provide principal level geotechnical project management, including review of analyses, report writing, and invoicing, as well as client communications.
- Construct a computerized stability model of the site and proposed house pad area from above the eastern fog line of Hemlock to past the western slope toe on the beach using 2-ft on ground surveyed topography as provided by others.
- Back calculate the existing tenuous stability condition using the model by reviewing previous explorations and slide data from our files, and complete analyses of up to 2 cross sections that incorporate various cut and fill scenarios and foundation elements compatible with provided plans.
- Complete sensitivity analyses of the preceding scenarios using a scoured slope toe condition. (A seismic condition will be unstable so will not be analyzed).
- Summarize our work in a letter report stamped by a PE/GE.
- Provide up to one site meeting and 4 hours of follow-up office consultation to the team after this report is issued.

LITERATURE REVIEW

We reviewed the following geologic information and geotechnical reports available in our files and as read from others as part of our study.

- DOGAMI Bulletin 74, 1972.
- 'Field Investigation of Geologic Hazards in Cannon Beach, Oregon', Martin E. Ross, June 3, 1977.
- DOGAMI O-09-06
- 'Geotechnical Engineering Report, Tax Lot 600 – Nenana Avenue Oceanfront Lot, Cannon Beach, Oregon', GeoEngineers, December 7, 1995.
- 'Phase 1 Report of Geotechnical Engineering Services, Residential Site Development, Lots 9-12, Block 2, Tolovana Park, Cannon Beach, Oregon', GeoEngineers, May 13, 1998.

- 'Geotechnical Feasibility Study – Phase 1, Nenana Avenue Lot, Cannon Beach, Oregon', GeoDesign, Inc., November 18, 1999.
- 'Geotechnical Engineering Report, Geotechnical Investigation and Monitoring – Phase II, Tax Lot 600 – Nenana Avenue, Cannon Beach, Oregon', GeoDesign, Inc., May 25, 2001.
- S-Curves Landslide Investigation, Stabilization, and Monitoring, Geotech Solutions, 2002 to present. Original report dated May 12, 2003.
- S-Curves water line, Geotech Solutions, Inc., August 15, 2018.
- Geotechnical feasibility for planning phase, Geotech Solutions, Inc., July 2, 2019.
- Borings from Earth Engineers for Nenana ROW, 2020.
- Updated Stability Analyses of S-curves slide for Nenana Roadway improvements – in process, April 2020.

These references contain geologic and geotechnical information in the immediate vicinity and on the site itself. On-site studies included three borings and installation of inclinometer casings for measurement of ground movements and ground water levels. Off-site work is also extensive and includes borings and inclinometer casings, ground water instrumentation from the beach level to above Hemlock Street, slope stability modeling, and installation of horizontal drains for slide stabilization improvements coupled with over 12 years of monitoring. Locations of explorations are shown on the attached plans and feasibility report.

HISTORY AND BACKGROUND

The site is located near the southern end of an ancient landslide mass as mapped by Ross, 1977. A copy of this map is provided in the report attachment by others. The slide extends past the northern ends of Pacific Street and Haystack Lane north of the site. The central and northern portions of the ancient landslide are developed with several roadways and numerous residences. Small incremental movement of these areas of the slide mass is likely ongoing, especially during wet season heavy rainfall events, but we are not aware of recent large displacements in those areas that damaged structures or roadways.

The southern end of the ancient slide, including the site and areas to the east and south, has been more active for many decades. This portion of the slide is commonly referred to as the 'S-Curves slide'. Interviews completed by others (1998) describe downward movement of approximately 6 to 8 feet in Hemlock Street during the winter of 1972. This report also indicates that a previous residence was removed from the subject site in 1972. The report does not specifically indicate whether the residence was damaged by the ground movements, although cracks and displacement of the remnant slab of up to 5 inches were noted. Remnants of the residence are still present at the site including at least portions of the concrete slab, rubble fill, and evidence of previous grading.

A geotechnical investigation was done by others in 1998 for a group of tax lots located immediately south of the Nenana Avenue easement, property since purchased by the City as "Inspiration Point". This report describes evidence of shallow landslides and the investigation included two borings with slope inclinometers that measured active movements associated with a deeper landslide surface.

Ground movement in the S-Curves slide area has occurred many times, and in 1999 resulted in ground and pavement ruptures in Hemlock Street and abutting sites of 4- 6 inches vertically and several inches horizontally, and rupture of the south end City sewer force main. These movements and related events also historically deformed utilities and the roadway in Chena Street (which has since been repaired/improved) and damaged several houses. Geotech Solutions began a landslide investigation of the S-Curves slide in 2002 and groups of horizontal drains were installed under our consultation in 2007 and 2008 by the City of Cannon Beach. The goal was to decrease storm rainfall related spikes in groundwater levels and therefore reduce movement. Geotech Solutions' investigation included numerous borings and piezometers and related measurements of ground movement and ground water levels correlated to storm event rainfall. Monitoring of rainfall, drain discharge, water levels and movement has been ongoing since installation, with the latest readings in June 2018 prior to roadway water line improvements. A memo summarizing the most recent readings is attached.

Geotechnical investigations regarding private development on the subject site were done by GeoEngineers (1995) and GeoDesign (1999 and 2001) with Mr. Rondema's involvement. These investigations included three borings on site and installation of slope inclinometer casings for measurement of ground movements. In summary the inclinometers on site indicated movement at depths of 35 to 45 feet below the ground surface on the eastern portion of the lot from 2000 to 2001, with massive siltstone below that to depths of over 70 feet (roughly elevation -5 ft, 25 feet below current beach levels at the slope toe). In 2001 these casings were deformed to the point they could not be read. Logs of these borings and an inclinometer plot are attached.

S-CURVES STABILITY IMPROVEMENTS

Monitoring completed by Geotech Solutions in the S-Curves slide area has indicated that groundwater levels and slide movements have decreased, but not stopped, since installation of the network of 19 horizontal drains in areas immediately south and east of the subject site (as shown in the attached figure). No cracking or deformations of the Hemlock Street pavement have been observed since drain installation. However, our instruments indicate that small movements less than 0.2 inches have occurred at depth following at least three significant rainfall events in the last 12 years. The City of Cannon Beach annually has cleaned these drains, and one drain can no longer allow passage of the drain cleaning head for the last several years. A few of the 19 drains possibly being partially blocked is not expected to impact the S-curves or site (as that drain may still fully function anyway) due to redundancy in the drain system. Annual maintenance of the drains is required to maintain the current S-curves condition.

SITE OBSERVATIONS

We visited the site for Dave Roberts on February 18, 2010 to observe existing site conditions. At that time the existing concrete slab at the site was moderately to severely cracked with horizontal separations up to approximately 5 inches. Crack orientation was variable but larger cracks were roughly oriented north-south, parallel to the crest of the oceanfront slope.

Evidence of shallow landslide scarps and sloughing along the crest of the oceanfront slope is present (also as mapped by Ross '77). The crest of the slope is currently located approximately as close as 45 feet west of the east property line. These features can also be interpreted from the recent topographical survey. Surface water is present at the ground surface in wet conditions near the southeast property corner and is likely associated with the group of three horizontal drains located off-site to the east.

CONCLUSIONS AND RECOMMENDATIONS

General

Previous reports concluded that improved stability of the overall S-curves slide could allow for development, albeit still with some risk of damage from slide movement. As discussed in the preceding, this stability improvement in the overall S-curves slide has occurred with the installation of horizontal drains that have now been in place and monitored for over 12 years. Groundwater in the S-curves responds quickly to rainfall infiltration, with peaks occurring within hours during the wet season. Monitoring data shows that the network of horizontal drains has decreased these peaks in ground water levels, as well as baseline levels, and increased stability of the S-Curves slide. However, it should be noted that the slide is still moving fractions of an inch on deep shear surfaces in high intensity wet winter rainfall events. The measures herein are not intended to arrest overall S-curves slide movements, as such measures are not feasible on this small lot. Rather, the measures are to improve localized lot stability relative to the oceanfront slope.

The analyses done for this lot does indicate that overall S-curves slide stability conditions will not be reduced, and that sections through the lot will be slightly improved, if the recommendations herein are followed.

"Setback"/Active Instability Margin

The critical slide issue for house foundation support design is failure of the oceanfront slope in an eastward progression into the building pad. In general, the existing ocean front slopes are unstable west of the "setback" /active instability margin on the attached figure. This is not a "setback" for conventional foundations. It is rather a margin of active instability. Building west of this margin is not feasible and may further destabilize the lower slopes. The instability margin is generally above the 61 ft elevation to the south, and the 64 ft elevation to the north. Development east of the proposed margin is only suitable if the stability improvements and deep foundation recommendations of this report are followed. On-grade settlement sensitive hardscaping features (such as concrete patios and sidewalks) west of the proposed margin are not recommended. Although foundation support must be derived east of this margin, cantilevered features may be feasible west of the margin per a structural engineers' design.

It should be understood the recommendations herein are to improve stability conditions for localized stability in static conditions (no earthquake). A CSZ interface earthquake will result in failures of the oceanfront slopes, the S-curves, and likely the overall slide that extends far to the north. The measures herein for localized stability and house support improvements are intended to allow the structural engineer to design for egress during such an earthquake. House damage will still occur and will likely be irreparable following tsunami impacts to the slope and stability. Re-occupancy or even the feasibility of rebuilding is unlikely. This seismic instability condition is similar to adjacent developed properties. Our specific analyses and recommendations are detailed in the following sections.

Slope Stability Analyses

Over the last several decades we have evaluated the stability of the S-curves. This included the explorations, data acquisition, surveying, and observations described earlier in this report. From this information and the provided site topographic survey, we developed stability models for the site using the program SLIDE and limit equilibrium methods. The critical section through the site is shown on the attached stability figure. Factors of safety within the western slope were as low as 0.80 (failure is less than 1.0) and increased to just over 1.0 east of the instability margin, and near 1.1 at the east side of the lot. These factors of safety are consistent with site observations in the current S-curves "dewatered" slide context.

The preceding existing factor of safety of 1.0 to 1.1 on the east portion of the lot is unsuitable for building without improvement. Typically for active slide areas and owner accepted damage risk, a factor of safety of 1.3 is used. Therefore, measures to improve stability to this level were evaluated. This did not include buttressing or armoring the oceanfront slope as it was assumed to not be permittable and would still only be part of a solution. This also did not include new horizontal drains, as installation of such drains could exacerbate the localized oceanfront slope instability (phase 1 drain installation for the overall slide caused slight temporary mobilization in the Nenana ROW B-1 inclinometer casing).

Sensitivity analyses were performed on many variables. For example, embedding a basement would decrease stability upslope, and adding significant fill to the site would increase instability of the oceanfront slope. Extreme beach front toe slope scour, such as observed in the 1999 El Nino and winter storm surge events, could also decrease stability. An eroded toe condition is addressed with the lot stabilization measures herein but would reduce the overall S-curves stability by roughly 5%.

Stabilization systems in the form of deep foundations and "shear piles" were evaluated with various configurations, sizes, and frequency to achieve a relative factor of safety for localized stability of 1.3 (up to a 30% increase over the existing condition). Detailed descriptions of these systems are included in following sections of this report.

Erosion Protection

Erosion protection of the slopes is vital to maintain some resistance to ongoing sloughing which may impact surface features and stability upslope. The existing slope vegetation is well developed and thick and should not be disturbed. Root intensive salt tolerant plantings such as hooker willows would aid in toe stabilization if any exposed soils are present. If needed, we recommend a local expert on oceanfront erosion control plantings be consulted to provide recommended planting details and address possible permitting issues.

Earthwork

Site Preparation - Site preparation for earthwork will require removal of vegetation, existing debris and slabs, and other unsuitable materials within proposed foundation support and building footprint areas. Existing bollards and casings should be removed, and the casings filled with grout. Root balls from trees or shrubs may extend several feet and grubbing operations can cause considerable subgrade disturbance. All disturbed material should be removed to undisturbed subgrade and backfilled with structural fill. In general, roots greater than one inch in diameter should be removed.

Temporary Cut Slopes - Temporary and permanent cut slopes should be no more than 2 feet high.

Fill Height Limitation – Site stability modeling indicates an average applied load of 200 to 250 psf to the lot does not significantly impact instability. Thererfore, fills must be limited to an average of less than 2 feet above existing grades, including that needed around grade beams and pile caps. Likewise, landscape fills must not increase site elevations on average more than two feet.

Stabilization and Soft Areas - After stripping, we must be contacted to evaluate the exposed subgrade in any on-grade structure areas such as flatwork, etc. Soft areas will require over-excavation and backfilling with well graded, clean angular gravel compacted as structural fill. A separation geosynthetic will also be required, such as a Propex Geotex 801 or equivalent.

Working Blankets and Haul Roads - Construction equipment should not operate directly on the subgrade when wet, as it is susceptible to disturbance and softening. Rock working blankets and haul roads placed over the preceding geosynthetic can be used to protect subgrades. We recommend that sound, angular, pit run or crushed basalt with no more than 6 percent passing a #200 sieve be used to construct haul roads and working blankets. Working blankets should be at least 12 inches thick, and haul roads at least 20 inches thick. The preceding rock thicknesses are the minimum recommended. Subgrade protection is the responsibility of the contractor and thicker sections may be required based on subgrade conditions and type and frequency of construction equipment.

Imported Granular Fill - Imported granular fill, such as clean sand or rock, should have a maximum particle size of 6-inches, be well graded, and have less than 5 percent passing the #200 sieve. This material should be compacted to 95 percent relative to ASTM D 1557.

Trenches - Utility trenches may encounter groundwater seepage and caving should be expected where seepage is present and in soft and/or loose soils. Shoring of utility trenches will be required for depths greater than 4 feet. We recommend that the type and design of the shoring system be the responsibility of the contractor, who is in the best position to choose a system that fits the overall plan of operation. At building connections, tolerance of deflection should be part of the design, as the building is expected to move less than areas off site. No infiltration of collected storm water is allowed.

Pipe bedding should be installed in accordance with the pipe manufacturers' recommendations. If groundwater seepage is present in the base of the utility trench excavation, we recommend over-excavating the trench by 12 inches and placing trench stabilization material in the base. Trench stabilization material should consist of well-graded, crushed rock or crushed gravel with a maximum particle size of 4 inches and be free of deleterious materials. The percent passing the U.S. Standard #200 Sieve shall be less than 5 percent by weight when tested in accordance with ASTM C 117.

Trench backfill above the pipe zone should consist of well graded, angular crushed rock or sand fill with no more than 7 percent passing a #200 sieve. Trench backfill should be compacted to 92 percent relative to ASTM D 1557, and construction of hard surfaces, such as sidewalks or pavement, should not occur within two weeks of backfilling.

Stability and Foundations – Grouted Micropiles

Localized oceanfront slope stability is a high risk that can be decreased by improved resistance across the slide surface(s) as well as by providing a relatively rigid house foundation system. The risk cannot be made zero, but the intent is to improve conditions enough to prolong movement damage within current static (non-earthquake) conditions. An actual CSZ interface earthquake will induce S-curves slide movement regardless of what is done on this site, as the site is a very small part of the slide. In that scenario, the design goal is again to provide a rigid enough system that structural collapse will not occur and that egress prior to tsunami arrival is accommodated. Although technically above the inundation elevation, tsunamis may runup the slope and may cause immediate irreparable damage on its own, and certainly long-term slope damage.

Western Pile Stability Improvement System - As the overall slide is relatively deep and within hard siltstone, drilled grouted micropiles are the recommended approach to penetrate through this zone to massive siltstone. A westerly location of a stabilization micropile system at or just east of the instability margin is required to limit failures up into the building pad. To this end a westerly grade beam with paired battered piles is recommended. These have significant lateral shear and bending resistance. FHWA based micropile slide stabilization "up-down" coupled moment analyses procedures were used in conjunction with SLIDE slope stability analyses to evaluate stability improvements and pile types and sizes.

We recommend paired (one battered down to the west, one down to the east) 7-inch diameter, 0.45 wall thickness API N80 casing enclosed in a corrosion protection grout column (and with a grout filled interior). These piles will need to be inclined at 30 degrees from vertical to allow for mobilization of axial strength and reduction in bending. These pairings must be spaced no greater than 6 feet on center for the full N-S width of the property (as movement direction is not orthogonal E-W). The heads can be two feet apart, with the piles down to the east set west of the opposing piles (a staggered overlap). The encompassing western grade beam must be designed to be free-standing. It must be noted that overall stability is dependent on the lower water level conditions maintained by the system of horizontal drains employed and cleaned by the City.

Forces generated by pile strength mobilization resisting the slide are shown in the attached sketch, which includes a conceptual layout.

Based on previous observation of the on-site inclinometer casing (the SE bollard/casing on site) movement occurred as deep as 45 feet, roughly elevation 20 feet. Piles will need to penetrate at least 10 feet past this depth into hard siltstone (estimated near elevation 10 feet) to provide enough bond to resist lateral slide forces and their corresponding moments.

The preceding piles must not be included in the structural engineer's house support or lateral resistance calculations (but can be used for wind loading) as they are fully engaged in slide resistance. However, due to physical constraints, house support piles can be included in this grade beam.

Vertical House Support Piles – Grouted micro-piles are also recommended for house foundation support. As vertical house loads are modest, 6-inch diameter grouted Titan 40/16 micropiles are recommended. Embedment must again reach the required 10 feet past the shear zone and be at or below elevation 10 feet. For the preceding pile an allowable capacity of 53 kips may be used for design. This accounts for some reduction from the shear zone. The structural engineer should determine the appropriate layout and spacing to optimize design. These piles also slightly increase the factor of safety for stability if spaced no more than 10 feet apart.

No isolated pier caps are allowed, and all piles must be connected with grade beams in the east-west direction roughly perpendicular to the slope. For resistance to lateral loads, 5 kips can be used for these vertical piles. Other battered piles for the house loading may be required, and the horizontal vector of the preceding pile load can be used with batters up to 30 degrees. Grade beams are not to be used for lateral design due to ground settlement and must be designed as self-supporting.

Capacities for additional pile sizes and inclinations can be provided upon request. We must be retained to review pile support design and called to the site to observe installation of piles.

Seismic Design

In accordance with the International Building Code (IBC) as adopted by SOSSC, the subject project should be evaluated using the parameters associated with Site Class D. Tsunami hazard maps (TIM-Clat-09) indicate that the western portions of the site may be inundated by the largest expected CSZ interface earthquake event of Mw=9.1. We recommend the occupants have an evacuation plan. Instability and tsunami damage are expected to the oceanfront slope as described herein.

Ground Moisture and Drainage

General - The perimeter ground surface and hard-scaping must be sloped to drain away from all structures, and rain drains must be routed to suitable erosion protected discharge near the base of the oceanfront slope. This includes collection and routing of the horizontal drain outlets east of the site. Gutters must be tight-lined to a suitable discharge and maintained as free-flowing. All crawl spaces must be adequately ventilated.

Slope stability, settlement, and foundation support can be reduced by increased surface infiltration and erosion. Therefore, we recommend that all surface runoff from hard surfaces, including downspouts, be collected and routed by tight line to suitable erosion protected discharge at the base of the western oceanfront slope. Gutters must be maintained as free flowing. Ground surface slopes must be inclined away from the structure and be graded to prevent ponding. Periodic grading may be required to maintain proper slopes due to ground distortion or settlement.

Perimeter Drain - A perimeter foundation drain is required at the base of the exterior grade beams. The drain should consist of a one-foot wide zone of drain rock encompassing a 4-inch diameter perforated pipe, all enclosed with a nonwoven geosynthetic. The drain rock should have no more than 2 percent passing a #200 sieve and should extend to within one foot of the ground surface. The geosynthetic should have an AOS of a #70 sieve, a minimum permittivity of 1.0 sec^{-1}, and a minimum puncture resistance of 80 pounds (such as a Propex Geotex 601 or equivalent). As an alternative, a composite drain board (such as an Amerdrain 500/520 or equivalent) can be used above and encompassing the perimeter drain pipe. One foot of low permeability soil (such as the on-site silt) should be placed over the fabric at the top of the drain to isolate the drain from surface runoff.

Vapor Flow Retardant - Some flooring manufacturers require specific slab moisture levels and/or vapor barriers to validate the warranties on their products. A properly installed and protected vapor flow retardant can reduce slab moistures. If moisture sensitive floor coverings or operations are planned, we recommend a vapor barrier be used. Typically, a reinforced product or thick product (such as a 15 mil STEGO wrap or equivalent) can be used. Experienced contractors using appropriate concrete mix designs and placement commonly place concrete directly over the vapor barrier which overlies the base rock/underslab rock. This avoids the issue of water trapped in the rock between the slab and vapor barrier, which otherwise requires removal. In either case, slab moisture must be tested until it meets floor covering manufacturer's recommendations.

Limitations and Observation During Construction

We have prepared the preceding information for use by Stan and Rebecca Roberts and members of their design and construction team for this lot and project only. The information herein can be used for bidding or estimating purposes but must not be construed as a warranty of subsurface conditions. We have made observations only at the aforementioned locations, and only at the stated depths. These observations do not reflect soil types, strata thicknesses, water levels or seepage that may exist between observations or at other areas of the site. We must be consulted to review final design and specifications in order to see that our recommendations are suitably followed. If any changes are made to the anticipated locations, loads, configurations, or construction timing, our recommendations may not be applicable, and we should be consulted. The preceding recommendations must be considered preliminary, as actual soil conditions may vary. In order for our recommendations to be final, we must be retained to review final plans, to observe actual subsurface conditions encountered, and to observe underpinning installation. Our observations will allow us to adapt to actual conditions and to update our recommendations if needed.

< >

We appreciate the opportunity to work with you on this project and look forward to our continued involvement. Please contact us if you have any questions.

Sincerely,

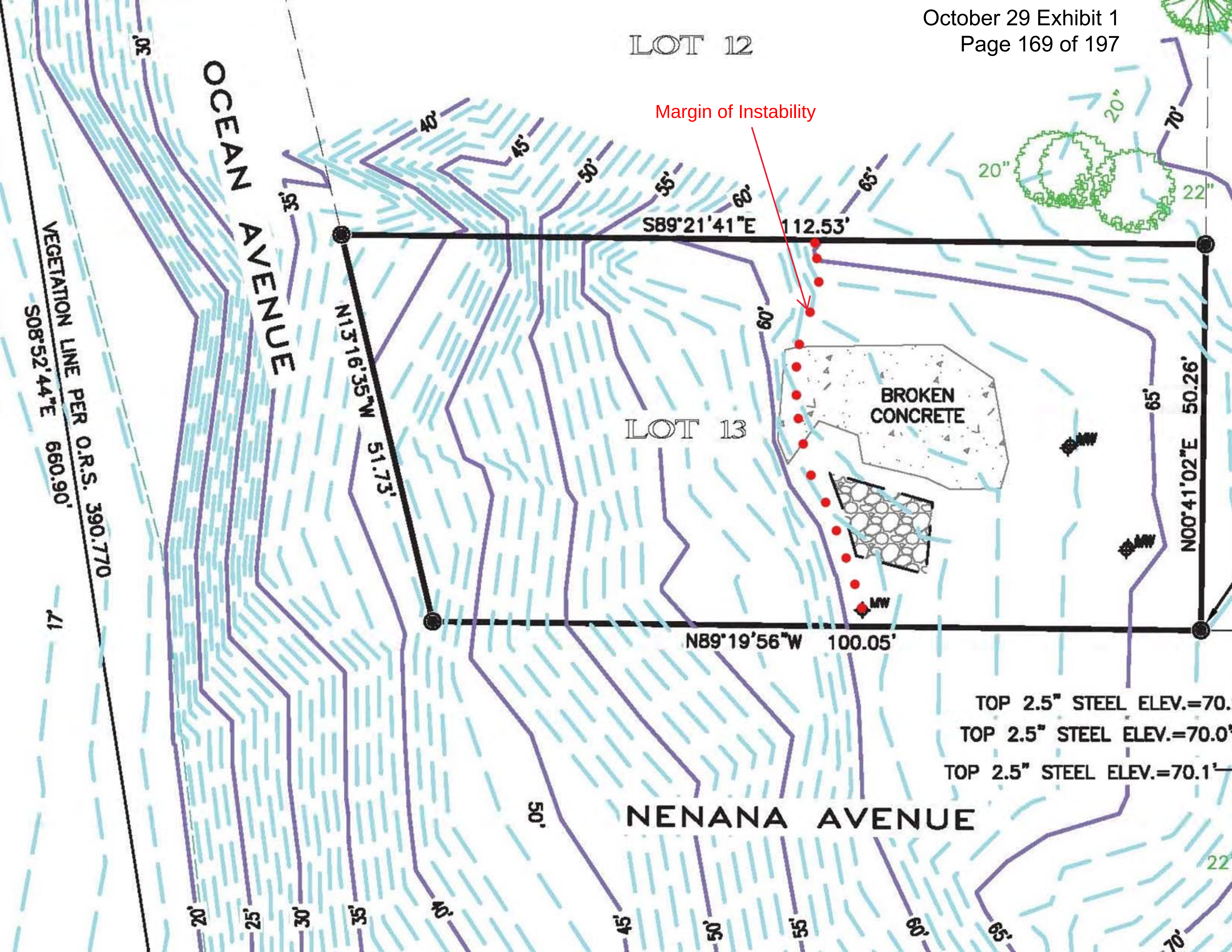
Don Rondema, MS, PE, GE
Principal

16,0[illegible]
JULY 21, 1992
DON RONDEMA
Expires 12/31/20

Attachments:

Site Aerial Photo with stability sections
Instability margin sketch on topo
Stability Analyses (4)
Pile Force and Concept Sketch
Geotech Solutions feasibility report
S-Curves Slide update memo
Horizontal drain layout
previous explorations by others


Pacific Dr
E Chena
SW cross section TL 600
robertscannon-9,18
critical slide section for TL600
cannon-2-S curves
N
200 ft
Google Earth
© 2020 Google


LOT 12
Margin of Instability
OCEAN AVENUE
VEGETATION LINE PER O.R.S. 390.770
S08°52'44"E 660.90'
S89°21'41"E 112.53'
N13°16'35"W 51.73'
N00°41'02"E 50.26'
N89°19'56"W 100.05'
LOT 13
BROKEN CONCRETE
MW
MW
MW
20"
20"
22"
22"
TOP 2.5" STEEL ELEV.=70.
TOP 2.5" STEEL ELEV.=70.0
TOP 2.5" STEEL ELEV.=70.1
NENANA AVENUE
17'
20'
25'
30'
30'
35'
35'
40'
40'
45'
45'
50'
50'
50'
55'
55'
60'
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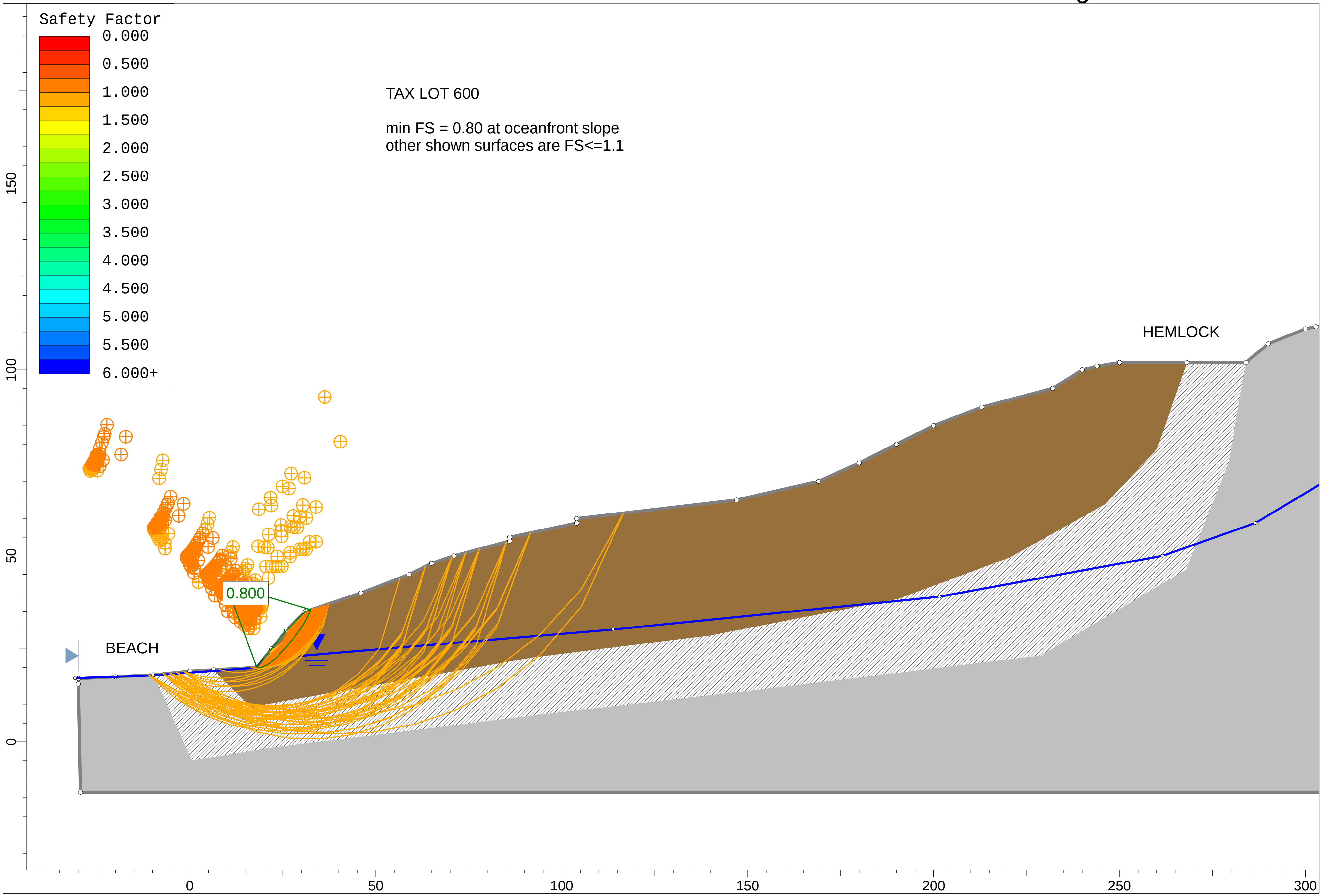
65'
65'
65'
70'
70'



tl600-winter drained crit slope dxn.slim

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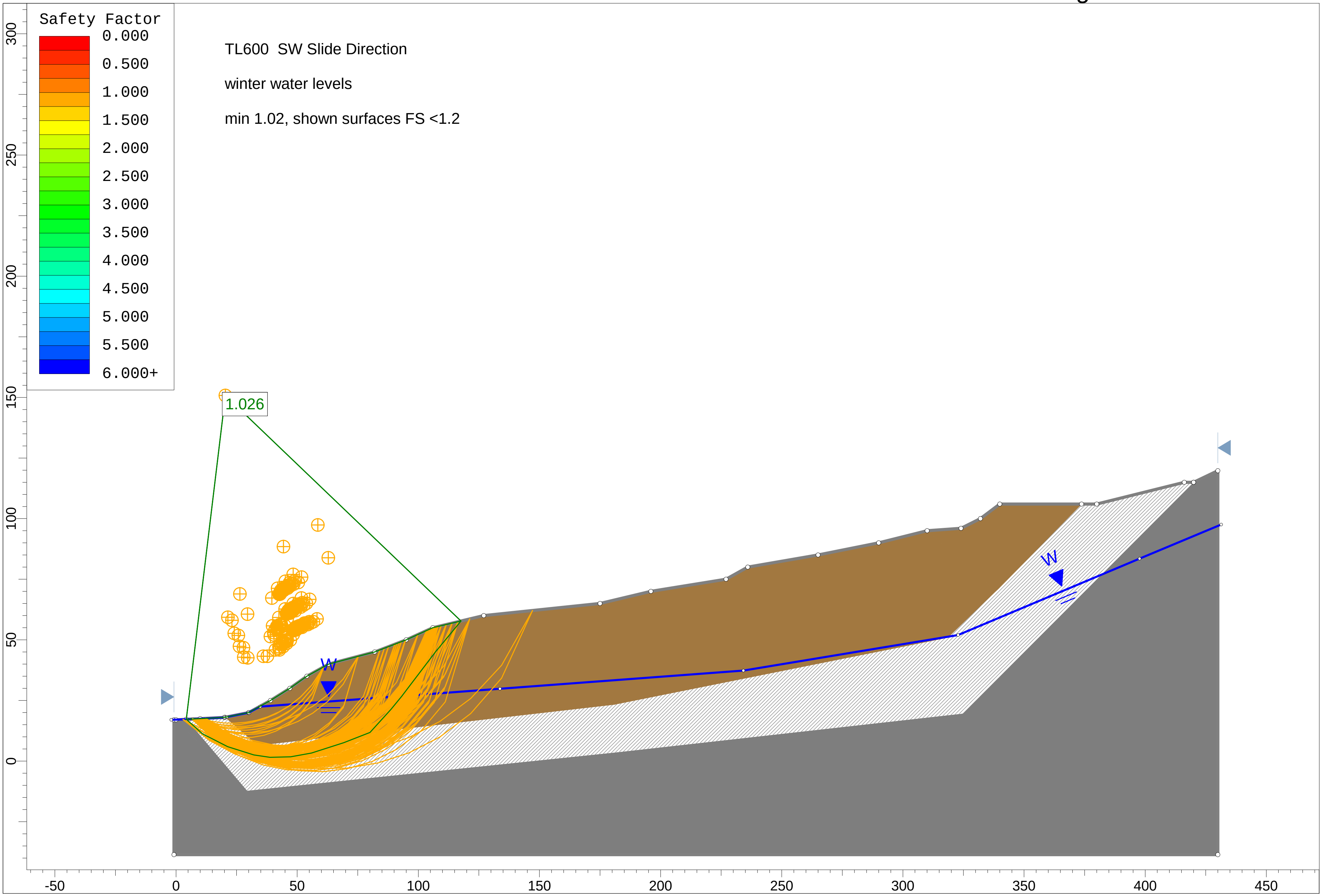
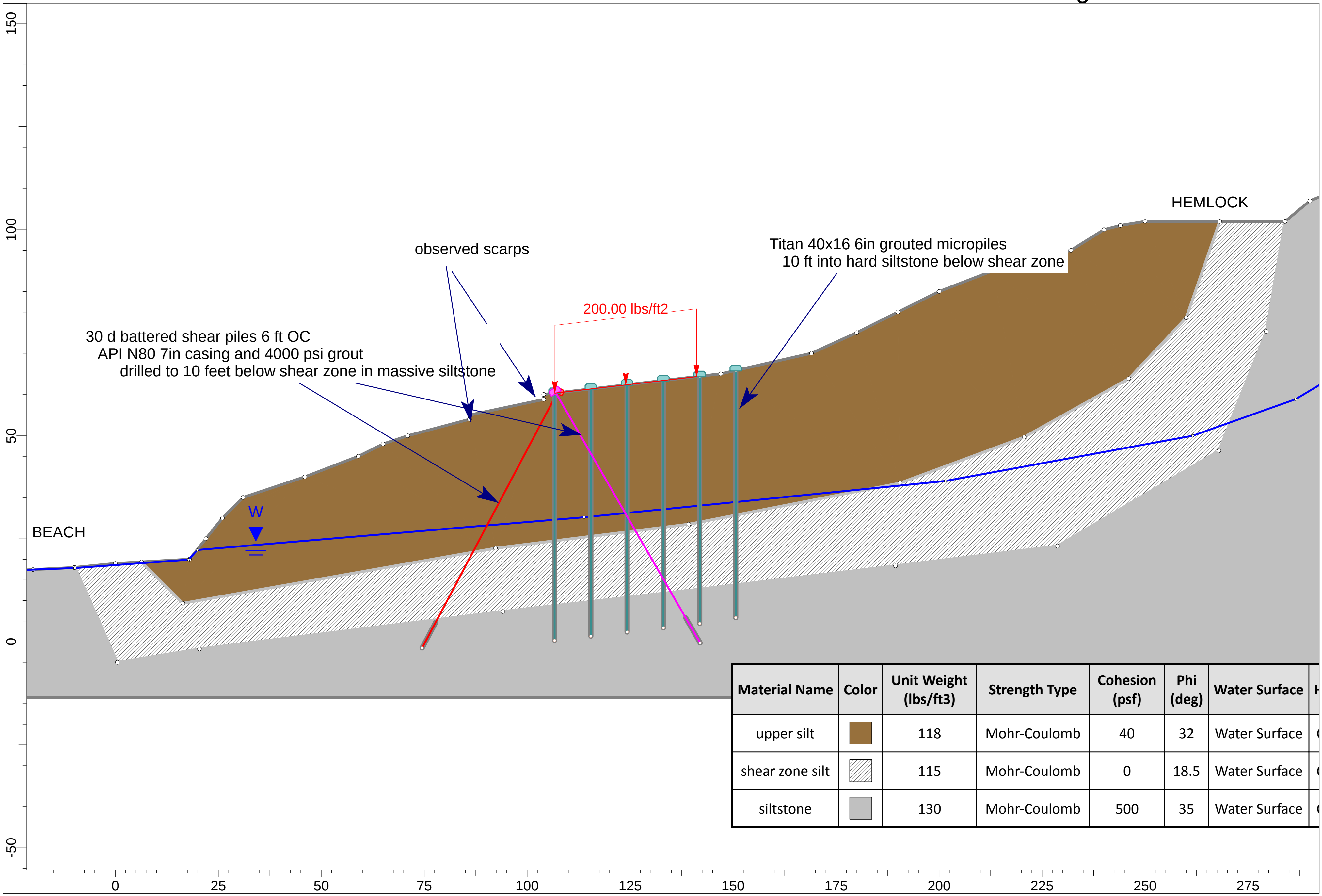


TL600 SW Slide Direction

winter water levels

min 1.02, shown surfaces FS <1.2

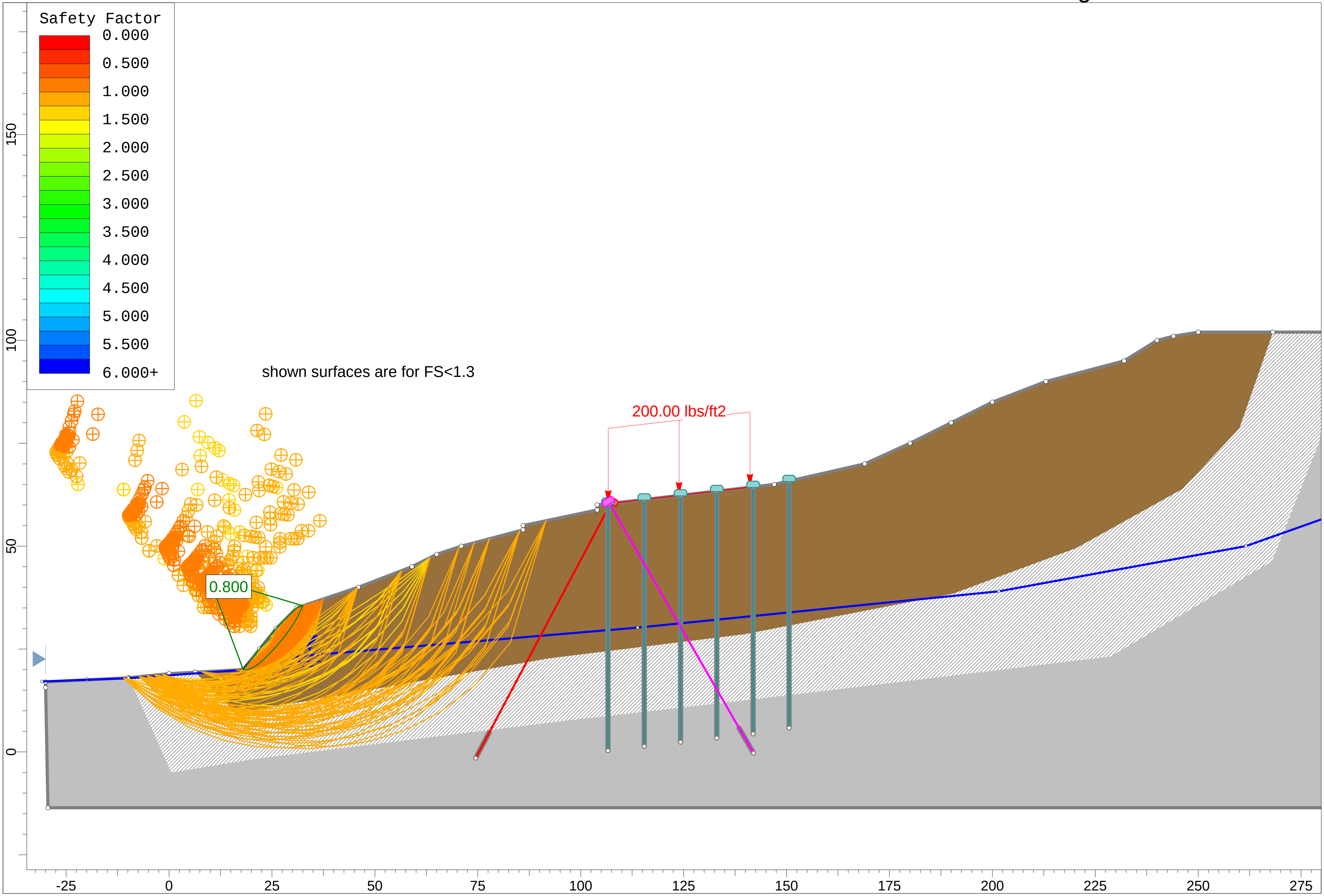


observed scarps

30 d battered shear piles 6 ft OC
API N80 7in casing and 4000 psi grout
drilled to 10 feet below shear zone in massive siltstone

Titan 40x16 6in grouted micropiles
10 ft into hard siltstone below shear zone

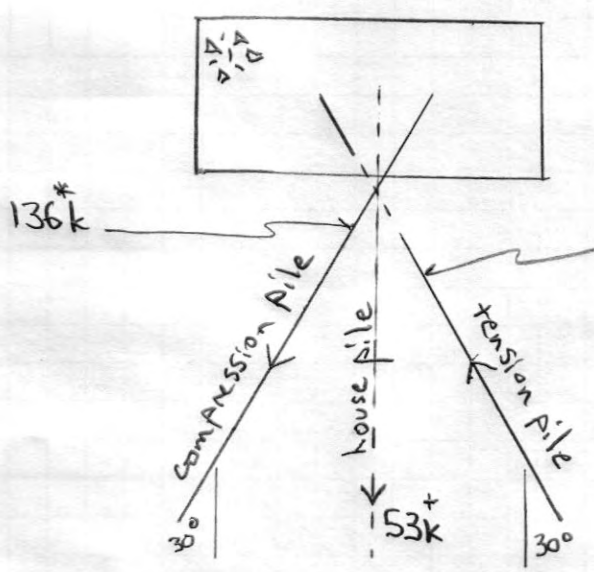
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface
upper silt	🟫	118	Mohr-Coulomb	40	32	Water Surface
shear zone silt	▨	115	Mohr-Coulomb	0	18.5	Water Surface
siltstone	⬜	130	Mohr-Coulomb	500	35	Water Surface



tl600-winter drained crit slope dxn - w 200 psf load.slim

West Grade Beam Pile Forces

NTS

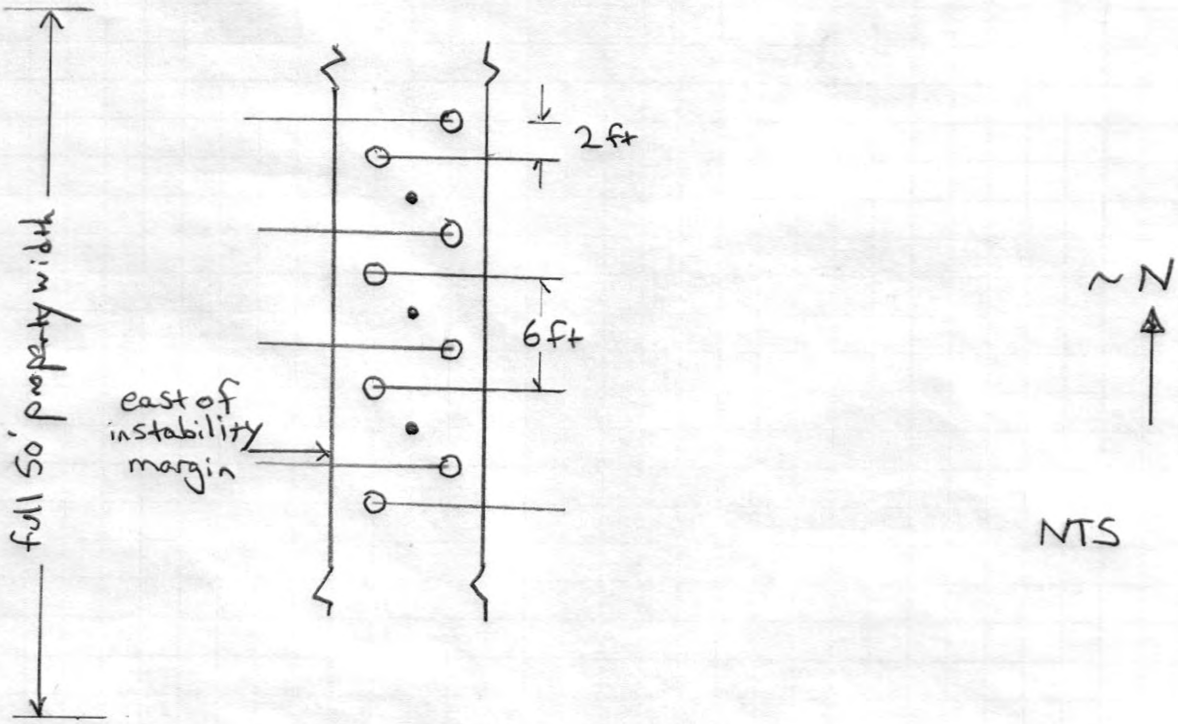


* 7" 0.45" wall grouted (4ksi) API N80 casing micropile

+ 6" titan 40/16 grouted (4ksi) micropile

embedment per report

Conceptual Layout



NTS

Geotech Solutions Inc.

July 2, 2019 *robertscannon-18-1-consult*

Stanley and Rebecca Roberts
Stan.milliman@gmail.com

Cc:
jay@jayraskinarchitect.com
rec@opusnet.com
kevin@objectiveadvisorsllc.com
plandevelopment@msn.com

GEOTECHNICAL ENGINEERING CONSULTATION
Planning Phase
Tax Lot 600, Nenana Avenue Oceanfront Lot - Cannon Beach, Oregon

Purpose and Scope

As authorized this report summarizes our geotechnical engineering consultation for the planning phase of the subject oceanfront lot located immediately north of the (unimproved) Nenana Avenue easement west of Hemlock Street in Cannon Beach, Oregon. We understand the feasibility of developing the site is to be evaluated, and our purpose was to assist in the geotechnical aspects of planning. This did not include actual foundation design recommendations and detailed stability analyses which are required for the design phase. Our specific scope of services included the following:

- Review vicinity geological and geotechnical information available in our files including recent summaries of landslide movement and our 2018 water line study.
- Review our work on the S-Curves slide to evaluate relative stability of the site and impact of stabilization efforts at the S-Curves, including movement rates and water level impacts.
- Attend up to 2 meetings as requested by the owner or architect.
- Provide a qualitative opinion on current stability condition and provide preliminary recommendations to reduce impacts to stability such as earthwork limitations and drainage requirements.
- Provide a qualitative discussion of preliminary foundation options and related considerations such as relative costs, risks and constructability.
- Provide a letter report summarizing our review, opinion of geotechnical feasibility, and preliminary options for foundation types.

Site Stability Background

The site is located within an active portion of an ancient landslide and is mapped in a geologic hazard area as mapped by the City of Cannon Beach (mapping excerpt attached). The site is part of a "down-dropped" area of the slide that is subject to storm surge wave attack. We have completed previous work on this property and adjacent properties, and have extensive work for the City of Cannon Beach in efforts to slow movement of the active portion of the slide at and above the site. That active portion has ruptured pavements on the S-curves and caused ground movement of several properties, including tax lot 600 and movement below the beach.

Mr. Rondema's involvement on this slide goes back to 1999, and Geotech Solutions previous work for the City on the S-curves slide began in 2002. That has included 6 borings up to 90- feet deep with

subsurface instruments and analyses, as well as survey monitoring for movement and acquisition of water level fluctuation data. Single event deformations were up to one foot vertically and horizontally in a west-southwest direction in response to high winter rainfall events in eroded toe conditions. In 2007 and 2008 horizontal drains were installed to reduce peak ground water levels during high rainfall events. This has significantly slowed, but not stopped, slide movement. The drains have been cleaned by the City each fall since installation, and drains flow during and after rainfall events with seasonal increases. Current slide movement has been measured near the active center at 0.3 inches in the primary shear zone in the last 6 years. Movement has been in response to high groundwater events induced by heavy rainfall storms. Most recently in 2018 we issued the attached slide movement update to the City, and in 2019 we completed work for a new water line in Hemlock Street. That water line in Hemlock is of a type of pipe and layout that can withstand some small slide movements, but is assumed to be ruptured in a CSZ earthquake event as is the sewer force main.

Risk

As stated, the site is part of an active landslide. Although movement has been slowed by horizontal drains reducing groundwater peaks in high rainfall events, this slowing is tenuous. Events that could accelerate movement include beach erosion, slope and toe erosion, new threshold rainfall events, and changes in slope loading such as cuts and fills, and site drainage. In addition, large movement is likely in earthquake ground motions from a CSZ interface earthquake (which has roughly a 30% chance of occurring in the next 50 years). Any of these issues, or a combination, could cause movement of the site that is structurally damaging. Damage could range from cracking and settlement to extensive movement and damage that requires rebuilding. The seismic motions of a CSZ interface earthquake (not to mention the subsequent tsunami impacts) would certainly result in extensive site damage and likely a loss of occupancy condition, and may render the site unusable. Because of these circumstances, in our opinion designing a structure for safe egress is the highest reasonable long term goal.

Localized ocean front slope regression is another risk, as the high bank erodes eastward to impact the building envelope. In this area of the coast regression averages roughly one foot per year, but is episodic, and may regress 10 or more feet in one year. Regression is typically more prevalent during strong southwestern storm surges and high sea level El Nino events which can coincide with total sand removal to siltstone on the beach (we observed this condition below the site in 1999, when the passive shear wedge of the slide was also visible on the beach).

Foundation Support

If the preceding risks are understood by the owner and the design team, and can be tolerated, foundation support is achievable. The types of approaches are likely limited by site access with equipment as well as high costs. We believe two approaches should be considered. A rigid reinforced structural mat supported by fixed deep foundations would be the lower risk - higher cost approach. Another approach could be a rigid mat designed for re-levelling. This has more risk of overall movement but lower initial cost, and also more risk of slope regression and utility impacts.

In any case drilling and underground work must be done when ground water levels are low with better stability, typically May through September.

Deep Foundation Supported Structural Mat - Within the site slide mass there are several rupture and movement zones at varying depths. These zones have been observed in adjacent inclinometer readings (below and next to the site), and were plotted 3 dimensionally from "communication" during

pressurized drilling/installation of horizontal drains. For foundation support to reduce overall movement these zones must be fully penetrated and the deep foundation elements designed to resist the resulting forces. The deep foundations would likely be large, heavily reinforced drilled shafts due to the high bending moments near the rupture zone interfaces. Shaft reinforcement may include W-shape beams (if they can be delivered to the site), or substantial rebar cages. Shaft size is likely limited to equipment access size and cost. Drilling will be difficult to adequately penetrate hard underlying siltstone. Special tooling as well as casing and dewatering will likely needed. The mat would need to structurally span between shafts, using grade support only for forming during construction.

Rigid mat designed for future Relevelling – A rigid mat designed to be stiff enough to accommodate relevelling is another possible option, but carries more risk. Increased risk is from distortion related damage to utilities and hardscaping, and exposure to undermining from shoreline regression. The structural engineer would need to design for significant free spans to accommodate slide grabens, as well as perimeter uplift and bending forces for relevelling. Relevelling could be done with push piers (hydraulically/reaction drive pipe piles) that are in place as part of the original construction. Reduction in regression risk could be accommodated by adding reinforced drilled shafts to the oceanfront side.

Access

The civil engineer must be consulted to design access at suitable inclinations and turning/egress. On-grade access will be difficult due to the very steep narrow roadway transition at Hemlock and the restraints to cutting and filling that may otherwise destabilize the slide. An initial estimate is that cuts must not be made in the slopes more than 2 feet deep and must be limited horizontally, and no cuts are allowed on the slope abutting Hemlock (just west of Hemlock, south of the existing "entry"). Likewise, fills would likely need to be limited to the equivalent weight of 2 feet of soil or rock. Detailed stability analyses of alternative grading sections would need to be done to better quantify these limits. For on-grade approaches a potential solution would be a near grade and pile restrained lightweight fill option on the downslope side of the entry drive. This could employ horizontally seated and connected EPS blocks shaped to desired grades. Shaped EPS for these approach inclinations may be difficult and costly, and may require a reinforced raked concrete wearing course depending on the final inclination. A viable alternative may be a pile supported structural approach and/or platform.

Drainage

Maintaining low ground water levels and limiting erosion are critical to stability. The mid-slope horizontal drain discharges for slide improvement abutting the east side of the lot complicate drainage as they will need to be accessible and maintained, with discharge collected to hard pipe. All runoff from structures and hard scaping must be collected and routed to suitable erosion protected discharge, preferably to the swale to the north if permissible.

Utility Connections

Utility connections that are designed to allow movement without damage are recommended. Such pipe connections are present in Hemlock for the sewer force main along the S-curves. Pipe with some flexibility in curved alignments can also help, such as the new water line in Hemlock. Again the civil engineer should be consulted on these options.

Limitations

We have prepared this report for use by Stanley Roberts and members of the planning team for this project only. The preceding recommendations should be considered preliminary, as actual soil conditions may vary. The information herein could be used for planning purposes but should not be construed as a warranty of surface or subsurface conditions. We have made observations only from the aforementioned information. These observations do not reflect soil types, strata thicknesses, water levels, seepage or stability conditions that may exist between observations, or after the present time. We must be consulted to complete stability and foundation support analyses design for any structures, as well as observe actual conditions encountered during construction in order for our recommendations to be final. Our observations will allow us to interpret actual conditions and adapt our recommendations if needed. Within the limitations of scope, schedule and budget, our services have been executed in accordance with the generally accepted practices in this area at the time this report was prepared. No warranty, expressed or implied, is given.

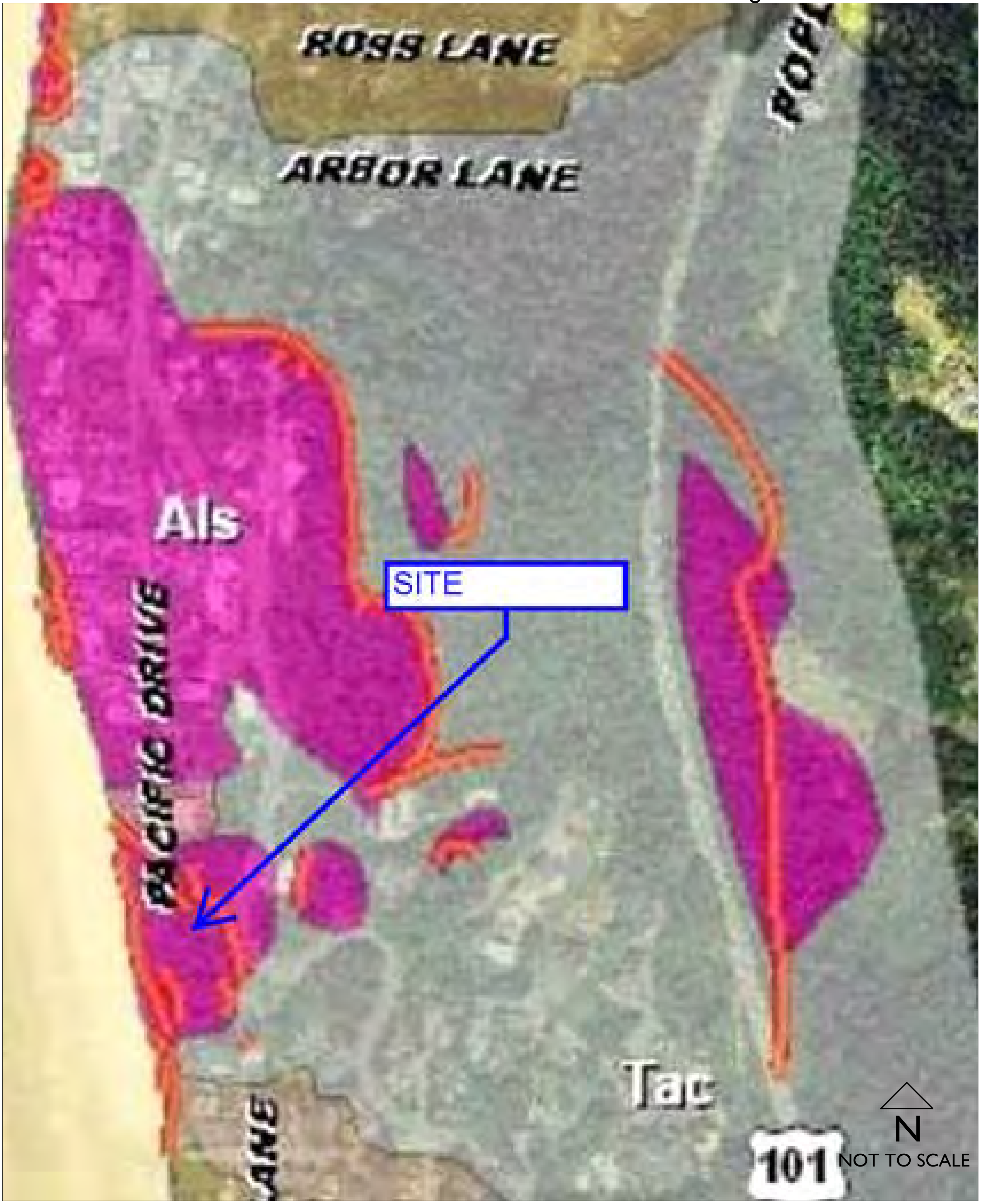
* * *

We appreciate the opportunity to work with you on this project and look forward to our continued involvement. Please call if you have questions.

Sincerely,

Don Rondema, MS, PE, GE
Principal

JULY 21, 1992
DON RONDEMA
Expires 12/31/20

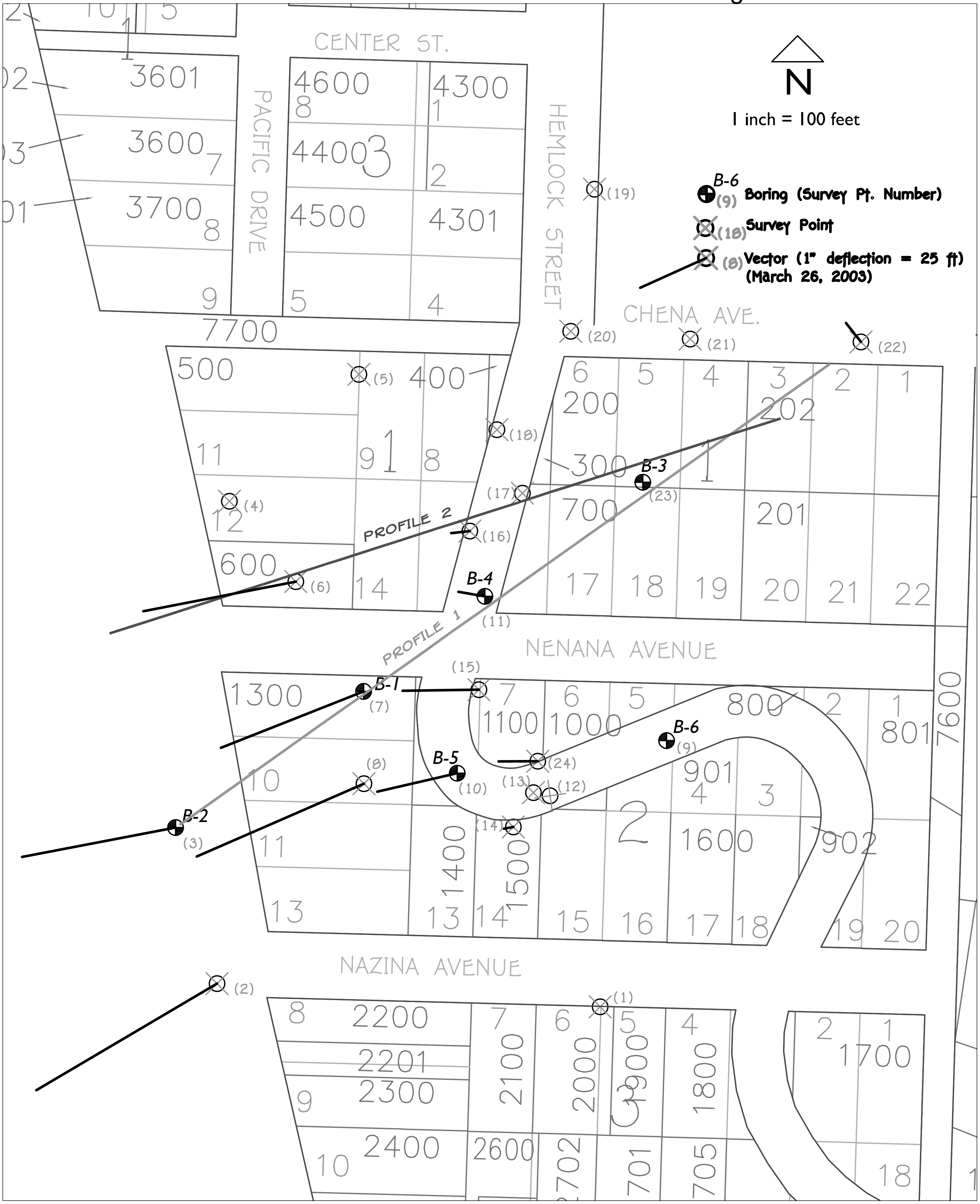


base from DOGAMI O-90-06

Geotech Solutions Inc.

SITE GEOLOGICAL MAPPING

robertscannon-18-1-consult



Geotech Solutions Inc.

SITE PLAN

Cannon-02-01-gi

Geotech
Solutions Inc.

MEMORANDUM *cannon-18-1-consult*

To: Karen LaBonte, Public Works Director, City of Cannon Beach; labonte@ci.cannon-beach.or.us

Date: June 26, 2018

Subject: Hemlock Street S-Curves Slide: Status Update

Introduction and Background

This memorandum provides an update to the status of the inclinometer data from the S-curves slide as read on June 23, 2018. The previous last reading was in 2015. The reason for this reading was a centerline crack appearing in the last month or so near the apex of the curve above and slightly south of the B-1r instrument. This crack is roughly 10-15 feet in length, and open up to roughly ¼" with perhaps a slight vertical offset down to the west. In addition, and perhaps relevant to tangential slide restraint and equilibrium, slope cuts and net mass removal has occurred on an adjacent project over roughly the past year. That project abuts previous lateral shear zones observed at the southern portion of the active slide.

The water levels in the slide are no longer being recorded as the instruments have expired, and new winter storm rainfall levels had not exceeded those previously recorded. The data attached are inclinometer readings for only one instrument near the center of the slide (B-1r) which has been shown over many years to correlate well with rainfall response and water levels and other previous movement in other, now irrevocably damaged, casings. It should be understood that this correlation is in the context of the general beach/slide toe elevations and erosion conditions experienced since 2008.

Conclusions and Recommendations

Roughly 0.2 inches of movement has occurred above/near the primary shear surface since the last reading roughly 3 years ago. The previous 3 years had roughly 0.1 inches of movement. Overall readings show a total of roughly 2.5 inches of movement on this replacement casing. A plot is attached. This movement is not out of recorded context movement rates for the slide.

Based on our site observations, in our opinion the surface cracking is not discernible from an aging panel joint or related thermal separation crack. It is possible that the crack was caused by accumulated underlying movement of the slide and is exhibiting at the previously placed grid overlap joint, but it does not coincide with previous slide induced crack locations which trended southwesterly with vertical offsets greater than horizontal, and at locations north and south of this crack location.

Although B-1r is approaching its deflection life, it is still functional and in our opinion does not need replacement at this time. Replacement/redrilling for a new casing (including initial baseline readings) is estimated at roughly $10,000 as access is difficult. If additional cracking occurs that is more indicative of slide movement, then a new water level logger is recommended for the paired B-1 standpipe (P-1).

Based on our current monitoring, we still expect movement of the S-Curves to be ongoing. However, the reduction in ground water levels and movement in large rainfall events has been greatly reduced by the functional horizontal drains compared to historical observations. No measures in addition to frequent roadway surface observation and annual drain cleaning are recommended at this time.

Provided the existing drains are maintained and cleaned annually and are functional, it is our opinion that they are sufficient to continue to slow the slide for the rainfall event intensities experienced since drain installation. Exceptions would be from earthquake ground motions or significant beach toe erosion. Any significant beach level erosion (such as exposure of siltstone below the sand similar to the El Nino cycle of 1999), or toe slumping, would be cause to take inclinometer readings, as would experiencing a new threshold rainfall event. These would be anything in excess of the storm events recorded since drain installation which are 4.37"-1 day, 6.26"-2day, 6.29"-3day, or 10.21"-5day. Please alert us if any of these thresholds are met.

The Limitations of our report apply, and that report and a few predrain install crack photos are attached here for background.

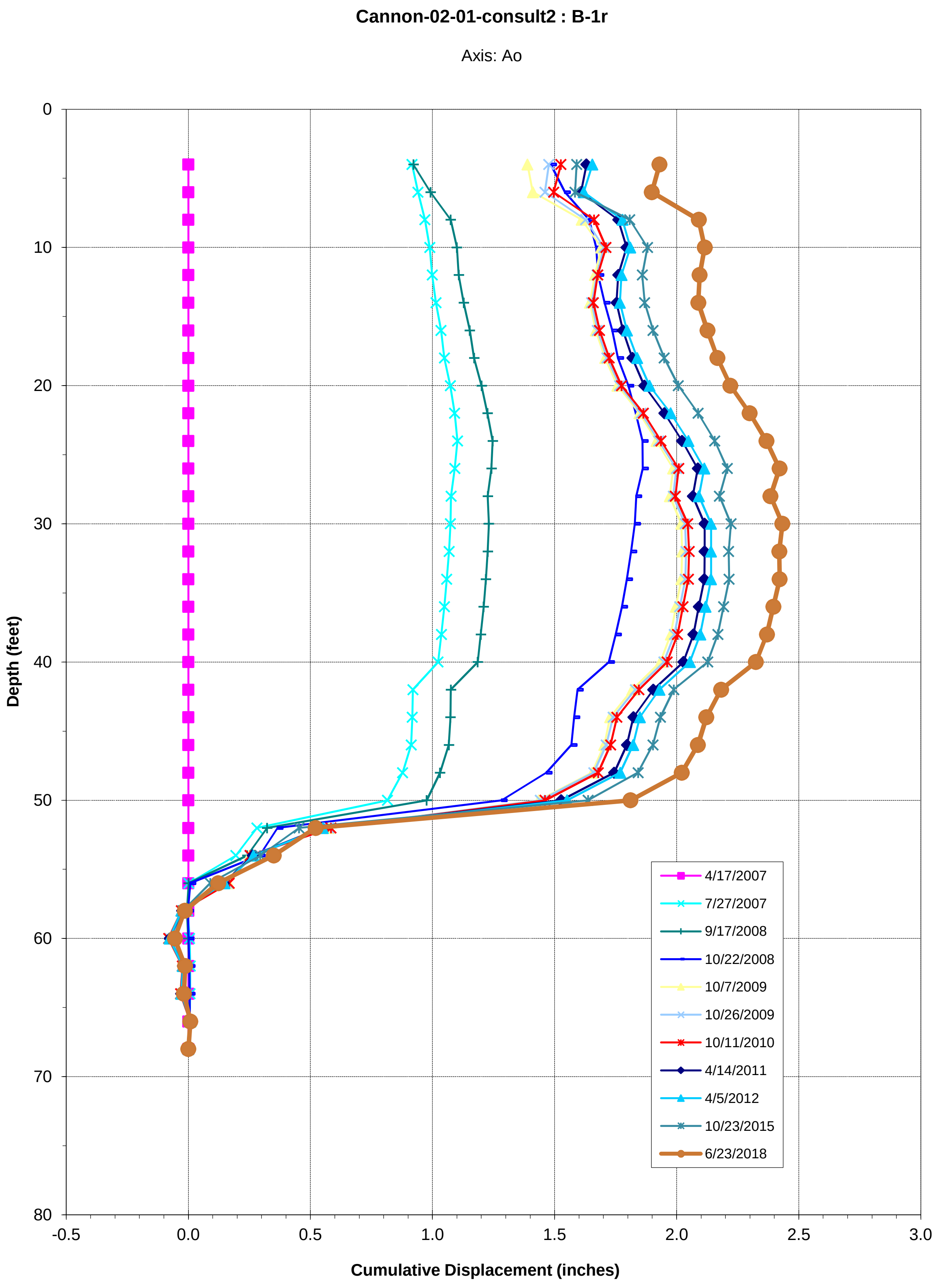
‹ ›

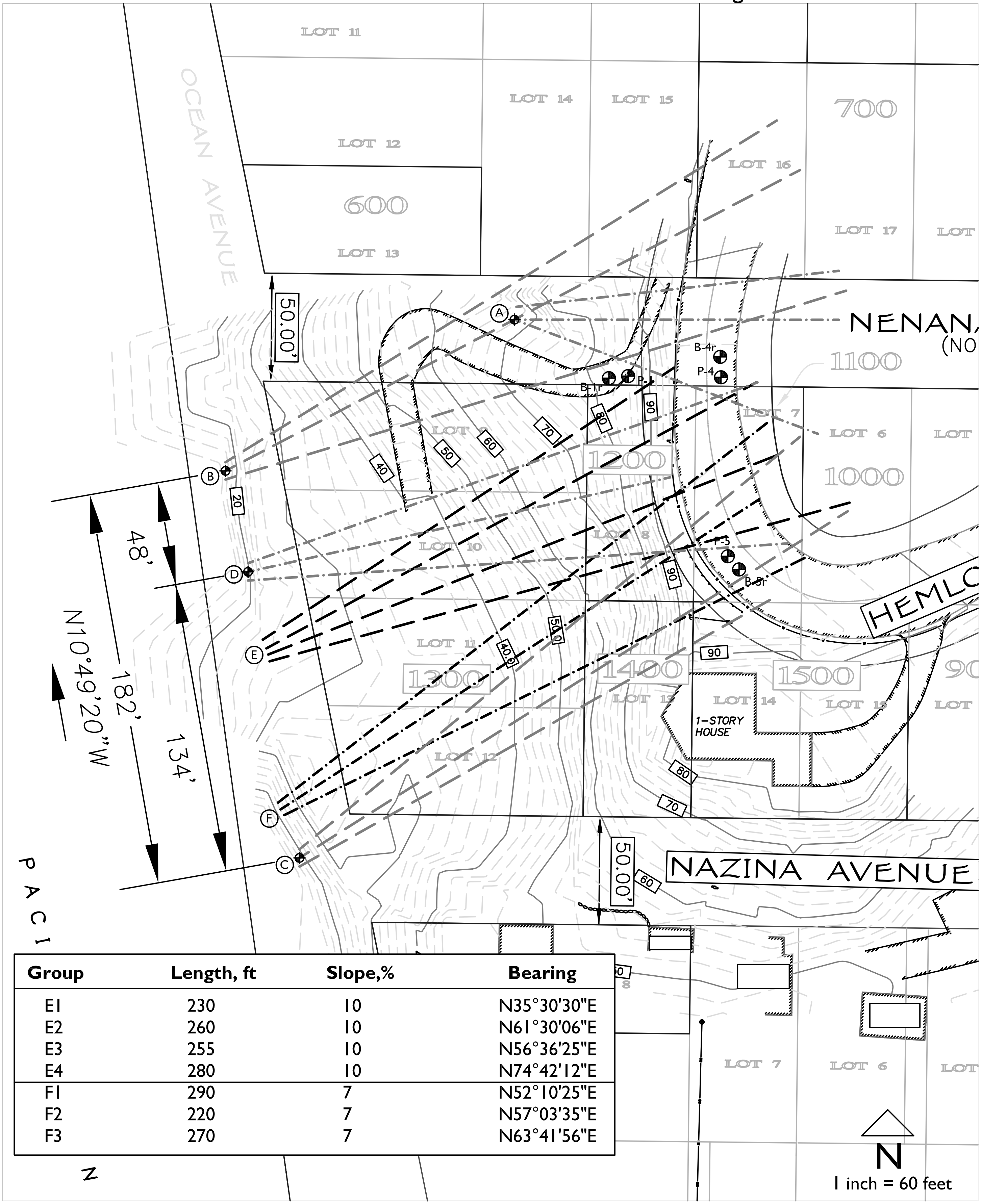
We appreciate the opportunity to work with you on this project and look forward to our continued involvement. Please call if you have questions.

Sincerely,

Don Rondema, MS, PE, GE
Principal

DON RONDEMA
Expires 12/31/18


Cannon-02-01-consult2 : B-1r
Axis: Ao
Depth (feet)
0
10
20
30
40
50
60
70
80
-0.5
0.0
0.5
1.0
1.5
2.0
2.5
3.0
Cumulative Displacement (inches)
4/17/2007
7/27/2007
9/17/2008
10/22/2008
10/7/2009
10/26/2009
10/11/2010
4/14/2011
4/5/2012
10/23/2015
6/23/2018



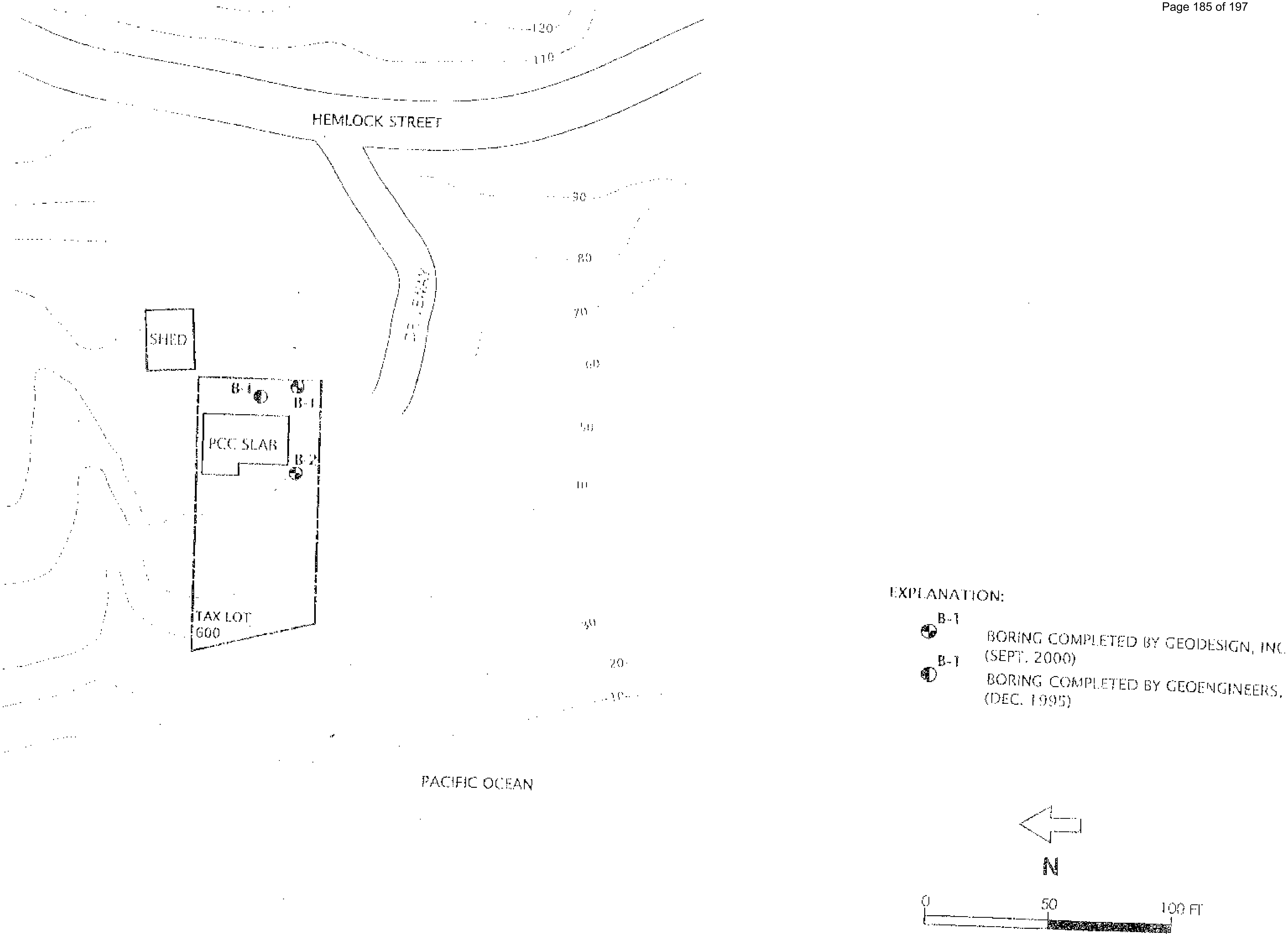
Group	Length, ft	Slope,%	Bearing
E1	230	10	N35°30'30"E
E2	260	10	N61°30'06"E
E3	255	10	N56°36'25"E
E4	280	10	N74°42'12"E
F1	290	7	N52°10'25"E
F2	220	7	N57°03'35"E
F3	270	7	N63°41'56"E

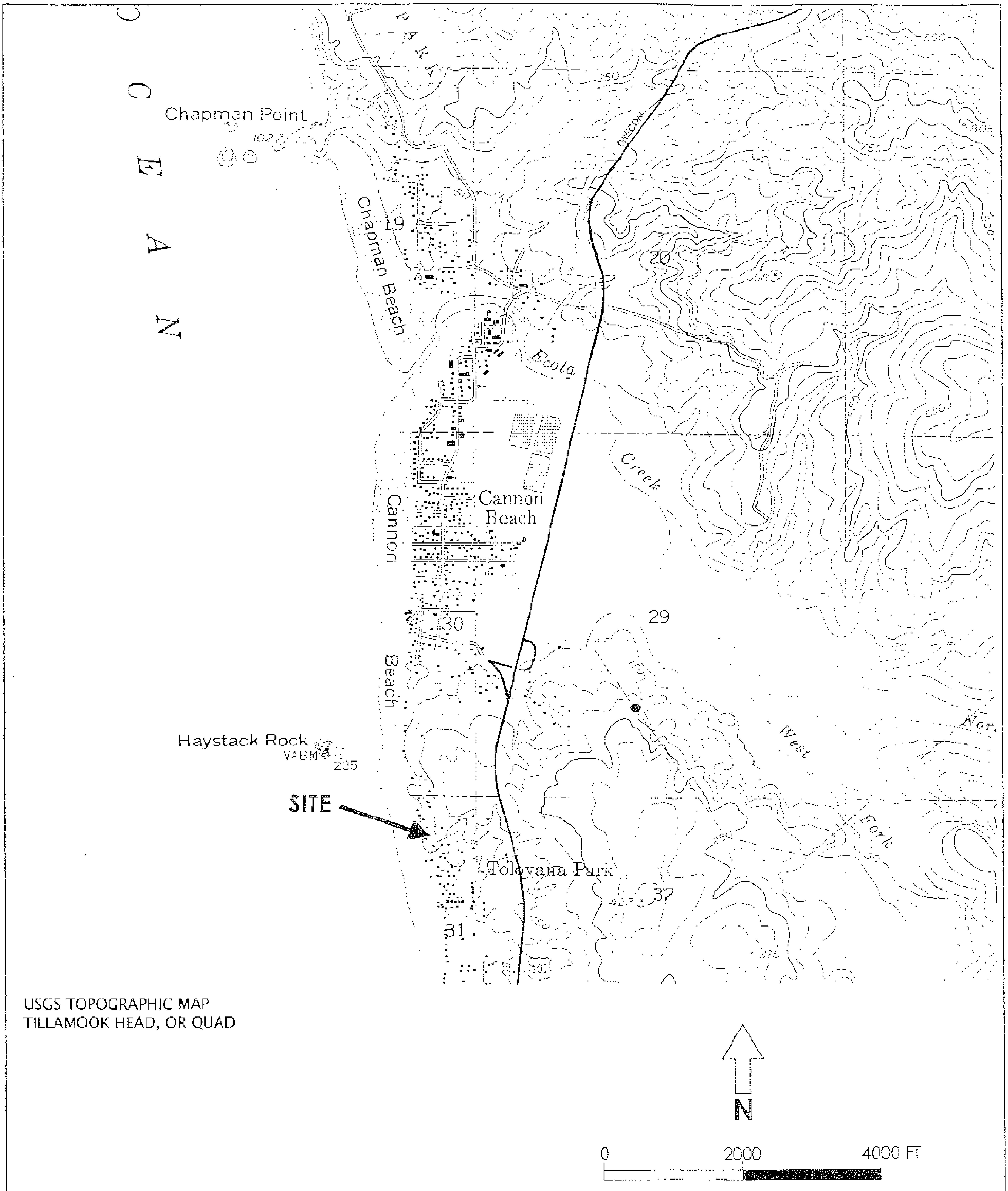
BASE DRAWING PROVIDED BY HLB-OTAK, IINC.

Geotech Solutions Inc.

SITE PLAN

cannon-02-01-consult2


120
110
HEMLOCK STREET
90
80
70
60
50
40
30
20
10
SHED
B-1
B-1
PCC SLAB
B-2
TAX LOT
600
PACIFIC OCEAN
EXPLANATION:
B-1
BORING COMPLETED BY GEODESIGN, INC
(SEPT. 2000)
B-1
BORING COMPLETED BY GEOENGINEERS,
(DEC. 1995)
N
0
50
100 FT



USGS TOPOGRAPHIC MAP
TILLAMOOK HEAD, OR QUAD

GEODESIGN INC

VICINITY MAP

BERTMAN-2	FEBRUARY 2001	FIGURE 1

APPENDIX A

FIELD EXPLORATIONS

We explored subsurface conditions at the site by advancing two borings (B-1 and B-2) at the approximate locations shown in Figure 2. Geo-Tech Explorations of Tualatin, Oregon, drilled the borings using a track-mounted drill rig equipped with mud rotary methods to depths of up 70.0 feet in on September 20 and 21, 2000.

We determined the exploration locations in the field from existing site features. The locations shown on Figure 2 should be considered approximate. A qualified member of GeoDesign's staff observed and documented all field activities.

We obtained representative samples of the various soils encountered for geotechnical laboratory testing. Classifications and sampling intervals are shown on the logs included in this appendix.

We classified the materials present in the samplers in the field in accordance with the "Key to Test Pit and Boring Log Symbols," "Soil Classification System and Guidelines," and "Rock Classification Guidelines," copies of which are included in this appendix. The boring logs indicate the depths at which the soils or their characteristics change, although the change actually may be gradual. If the change occurred between sample locations, the depth was interpreted.

LABORATORY TESTING

We classified soil samples in the laboratory to confirm field classifications. The laboratory classifications are included in the boring logs if those classifications differed from the field classifications.

We tested the natural moisture content of selected soil samples in general accordance with guidelines presented in ASTM D 2216. The moisture contents are included in the boring logs in this appendix.

We also completed unconfined compression testing of plaster capped siltstone cores of select samples. Results of this testing are attached.

KEY TO TEST PIT AND BORING LOG SYMBOLS

SYMBOL	SOIL DESCRIPTION
▮	Location of sample obtained in general accordance with ASTM D 1586 Standard Penetration Test
▯	Location of SPT sampling attempt with no sample recovery
◩	Location of sample obtained using thin wall, shelby tube, or Geoprobe® sampler in general accordance with ASTM D 1587
⧄	Location of thin wall, shelby tube, or Geoprobe® sampling attempt with no sample recovery
▤	Location of sample obtained using Dames and Moore sampler and 300 pound hammer or pushed
☰	Location of Dames and Moore sampling attempt (300 pound hammer or pushed) with no sample recovery
⊠	Location of grab sample
■	Rock Coring Interval
▽	Water level

GEOTECHNICAL TESTING EXPLANATIONS

PP	Pocket Penetrometer	LL	Liquid Limit
TOR	Torvane	PI	Plasticity Index
CONSOL	Consolidation	PCF	Pounds Per Cubic Foot
DS	Direct Shear	PSF	Pounds Per Square Foot
P200	Percent Passing U.S. No. 200 Sieve	TSF	Tons Per Square Foot
W	Moisture Content	P	Pushed Sample
DD	Dry Density	OC	Organic Content

ENVIRONMENTAL TESTING EXPLANATIONS

CA	Sample Submitted for Chemical Analysis	ND	Not Detected
PID	Photoionization Detector Headspace Analysis	NS	No Visible Sheen
PPM	Parts Per Million	SS	Slight Sheen
MG/KG	Milligrams Per Kilogram	MS	Moderate Sheen
P	Pushed Sample	HS	Heavy Sheen

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KEY TO TEST PIT AND BORING LOG SYMBOLS

TABLE A-1

SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			SYMBOL	NAME
Coarse Grained Soils More than 50% retained on No. 200 Sieve	Gravel More than 50% of coarse fraction retained on No. 4 Sieve	Clean Gravel	GW	Well graded, fine to coarse gravel
			GP	Poorly graded gravel
		Gravel with Fines	GM	Silty gravel
			GC	Clayey gravel
	Sand More than 50% of coarse fraction passes No. 4 Sieve	Clean Sand	SW	Well graded, fine to coarse sand
			SP	Poorly graded sand
		Sand with Fines	SM	Silty sand
			SC	Clayey sand
Fine Grained Soils More than 50% passes No. 200 Sieve	Silt and Clay Liquid Limit less than 50%	Inorganic	ML	Low plasticity silt
			CL	Low plasticity clay
		Organic	OL	Organic silt, organic clay
	Silt and Clay Liquid Limit greater than 50%	Inorganic	MH	High plasticity silt
			CH	High plasticity clay, fat clay
		Organic	OH	Organic clay, organic silt
Highly Organic Soils			PT	Peat

SOIL CLASSIFICATION GUIDELINES

GRANULAR SOILS

Relative Density	**Standard Penetration Resistance**
Very Loose	0 - 4
Loose	4 - 10
Medium Dense	10 - 30
Dense	30 - 50
Very Dense	More than 50

COHESIVE SOILS

Consistency	**Standard Penetration Resistance**	**Unconfined Compressive Strength (tsf)**
Very Soft	Less than 2	Less than 0.25
Soft	2 - 4	0.25 - 0.50
Medium Stiff	4 - 8	0.50 - 1.0
Stiff	8 - 15	1.0 - 2.0
Very Stiff	15 - 30	2.0 - 4.0
Hard	More than 30	More than 4.0

GRAIN SIZE CLASSIFICATION

Boulders	12 - 36 inches
Cobbles	3 - 12 inches
Gravel	¾ - 3 inches (coarse) ¼ - ¾ inches (fine)
Sand	No. 10 - No. 4 Sieve (coarse) No. 10 - No. 40 Sieve (medium) No. 40 - No. 200 Sieve (fine)

Subclassifications

	Percentage of other material in sample
Clean	0 - 2
Trace	2 - 10
Some	10 - 30
Sandy, Silty, Clayey, etc.	30 - 50

Dry = very low moisture, dry to the touch; Moist = damp, without visible moisture; Wet = saturated, with visible free water.

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SOIL CLASSIFICATION SYSTEM
AND GUIDELINES

TABLE A-2

ROCK CLASSIFICATION GUIDELINES

HARDNESS	DESCRIPTION
Very soft (RH-0)	For plastic material only
Soft (RH-1)	Carved or gouged with a knife
Moderate (RH-2)	Scratched with a knife
Hard (RH-3)	Difficult to scratch with a knife
Very hard (RH-4)	Rock scratches metal; rock cannot be scratched with a knife

STRENGTH	DESCRIPTION
Plastic	Easily deformable with finger pressure
Friable	Crumbles by rubbing with fingers
Weak	Crumbles only under light hammer blows
Moderately Strong	Few heavy hammer blows before breaking
Strong	Withstands few heavy hammer blows and yields large fragments
Very Strong	Withstands many heavy hammer blows, yields dust and small fragments

WEATHERING	DESCRIPTION
Severe	Rock decomposed; thorough discoloration; all fractures extensively coated with clay, oxides, or carbonates
Moderate	Intense localized discoloration of rock; fracture surfaces coated with weathering minerals
Little	Slight and intermittent discoloration of rock; few stains on fracture surfaces
Fresh	Rock unaffected by weathering

FRACTURING	FRACTURE SPACING
Crushed	Less than 5/8 inch to contains clay
Highly Fractured	5/8 inch to 2 inches
Closely Fractured	2 inches to 6 inches
Moderately fractured	6 inches to 1 foot
Little Fractured	1 foot to 4 feet
Massive	Greater than 4 feet

JOINT SPACING	DESCRIPTION
Papery	Less than 1/8 inch
Shaley or Platey	1/8 inch to 5/8 inch
Very Close	5/8 inch to 3 inches
Close	3 inches to 2 feet
Blocky	2 to 4 feet
Massive	Greater than 4 feet

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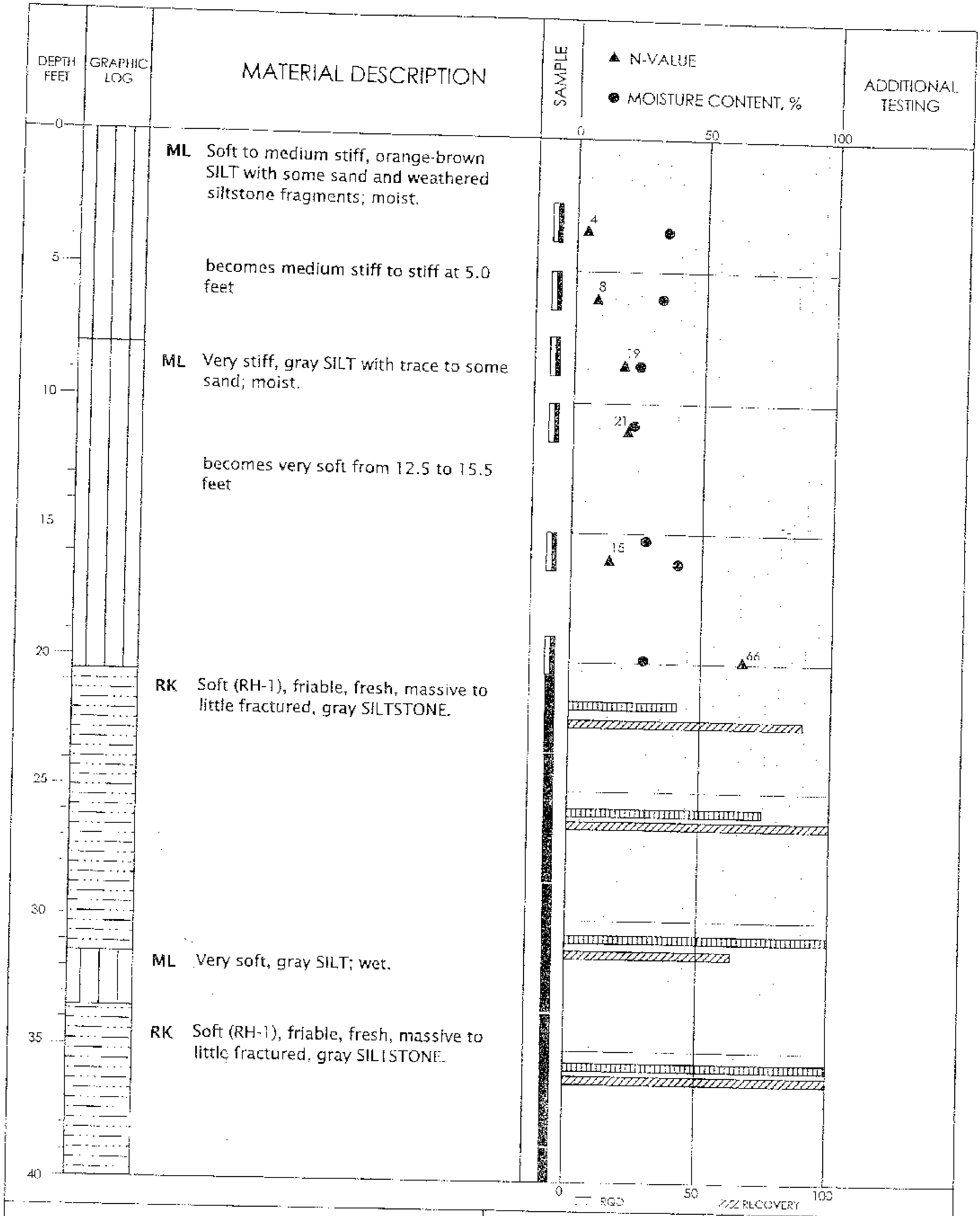
ROCK CLASSIFICATION GUIDELINES

TABLE A-3



ML Soft to medium stiff, orange-brown SILT with some sand and weathered siltstone fragments; moist.

becomes medium stiff to stiff at 5.0 feet

ML Very stiff, gray SILT with trace to some sand; moist.

becomes very soft from 12.5 to 15.5 feet

RK Soft (RH-1), friable, fresh, massive to little fractured, gray SILTSTONE.

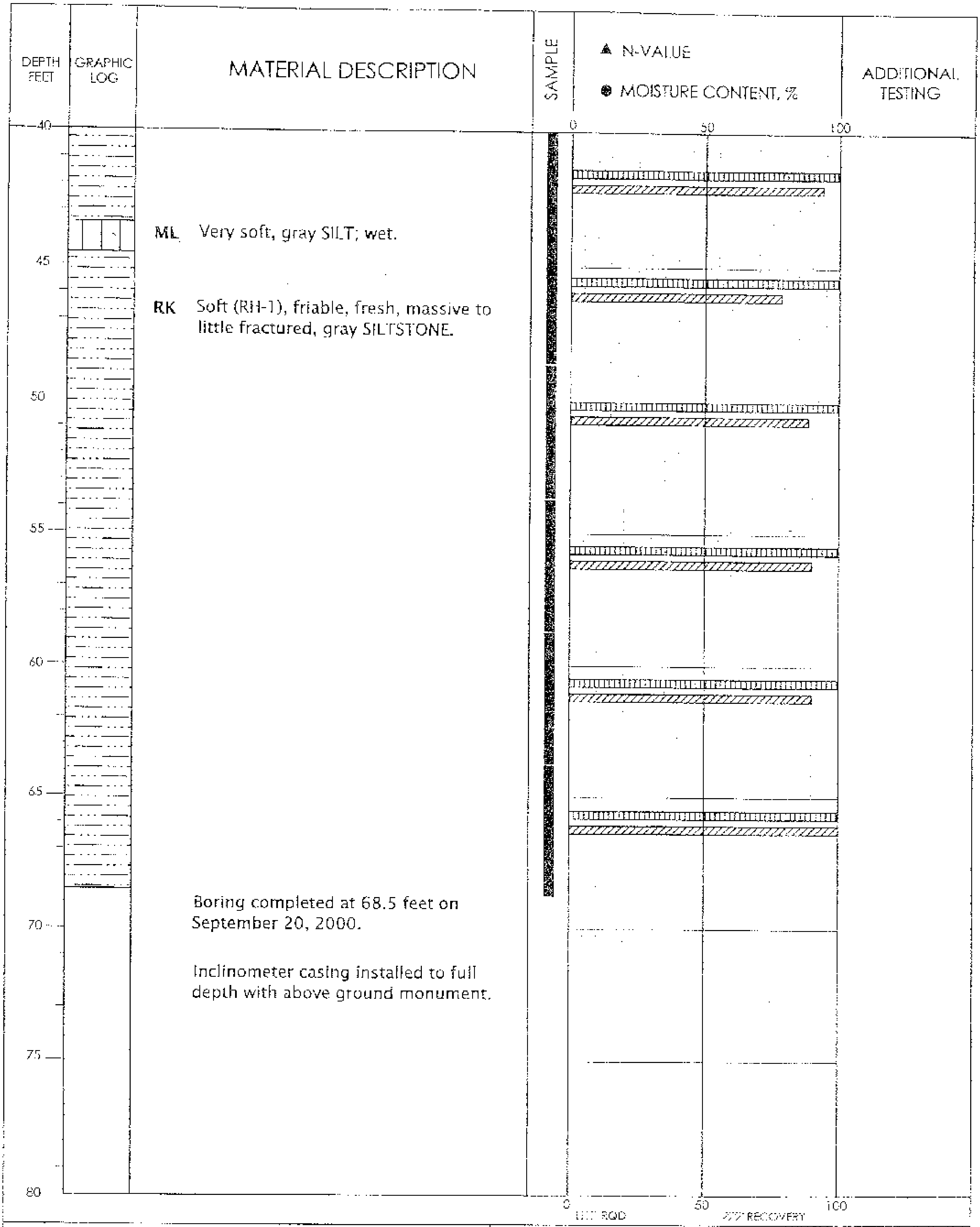
ML Very soft, gray SILT; wet.

RK Soft (RH-1), friable, fresh, massive to little fractured, gray SILTSTONE.

GEODESIGN INC

BORING B-1

BERTMAN-2 | FEBRUARY 2001 | FIGURE A-1



ML Very soft, gray SILT; wet.

RK Soft (RH-1), friable, fresh, massive to little fractured, gray SILTSTONE.

Boring completed at 68.5 feet on September 20, 2000.

Inclinometer casing installed to full depth with above ground monument.

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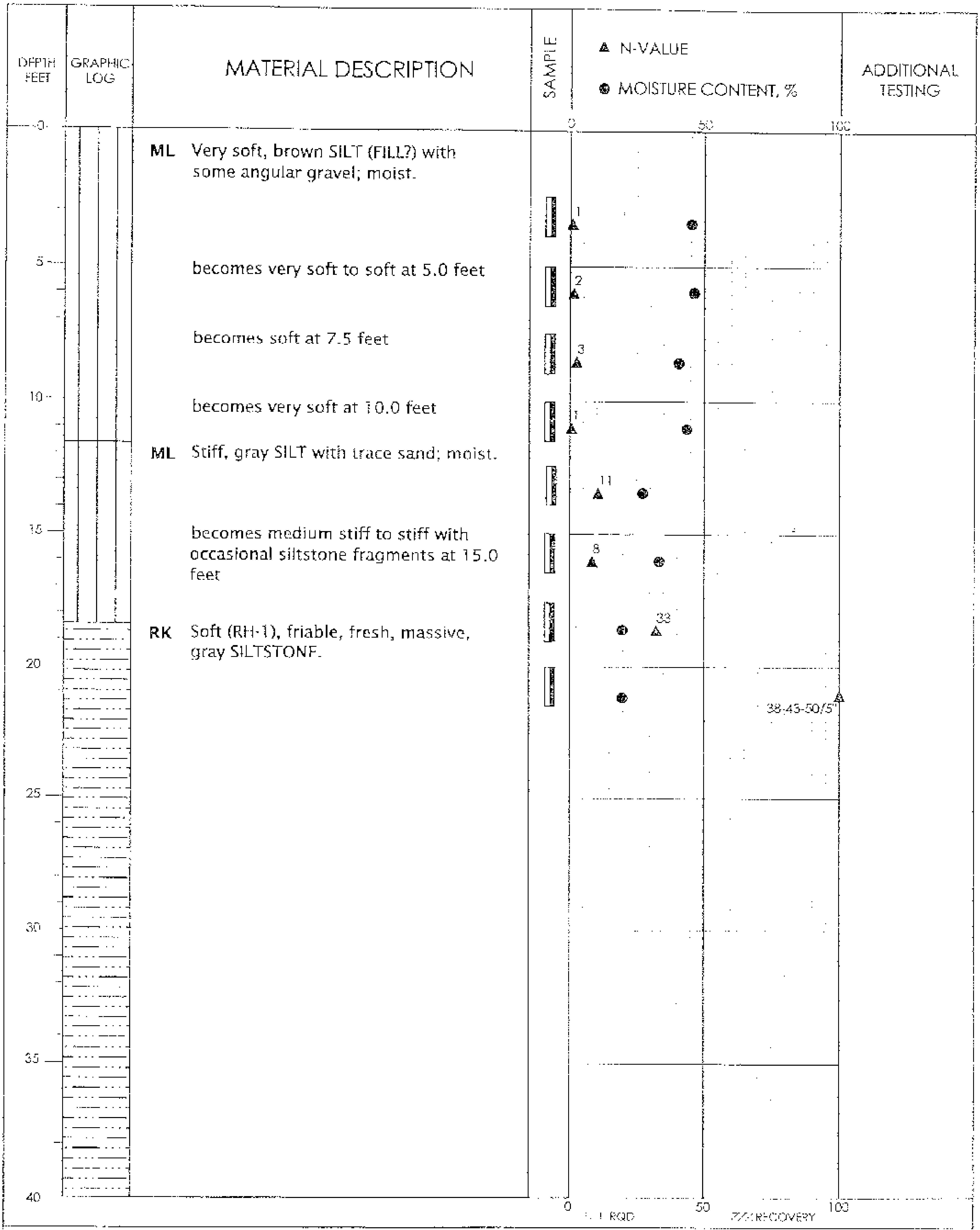
BORING B-1 (CONT.)

BERTMAN-2	FEBRUARY 2001	FIGURE A-1



ML Very soft, brown SILT (FILL?) with some angular gravel; moist.

becomes very soft to soft at 5.0 feet

becomes soft at 7.5 feet

becomes very soft at 10.0 feet

ML Stiff, gray SILT with trace sand; moist.

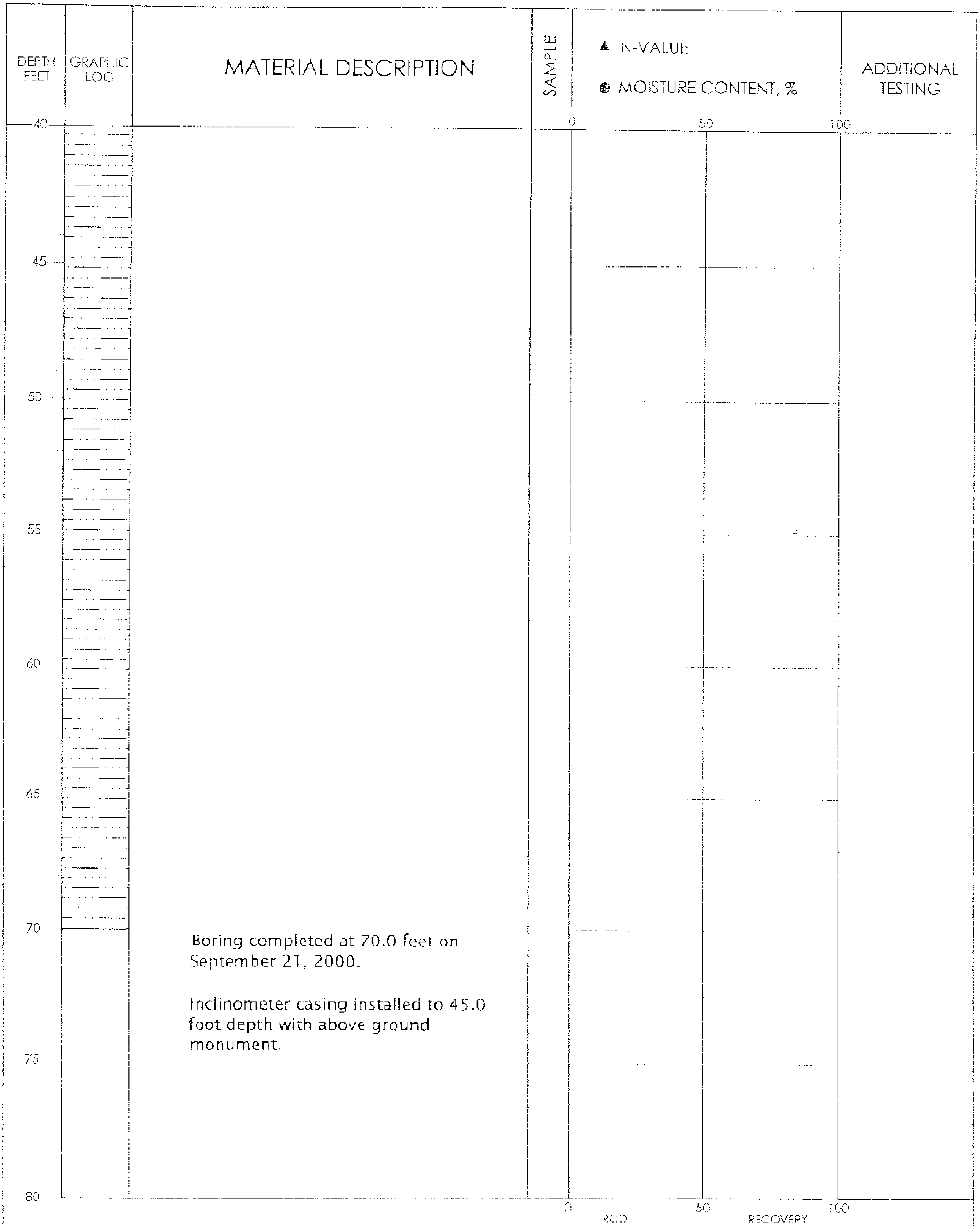
becomes medium stiff to stiff with occasional siltstone fragments at 15.0 feet

RK Soft (RH-1), friable, fresh, massive, gray SILTSTONE.

GEODESIGN INC

BORING B-2

BERTMAN-2 | FEBRUARY 2001 | FIGURE A 2



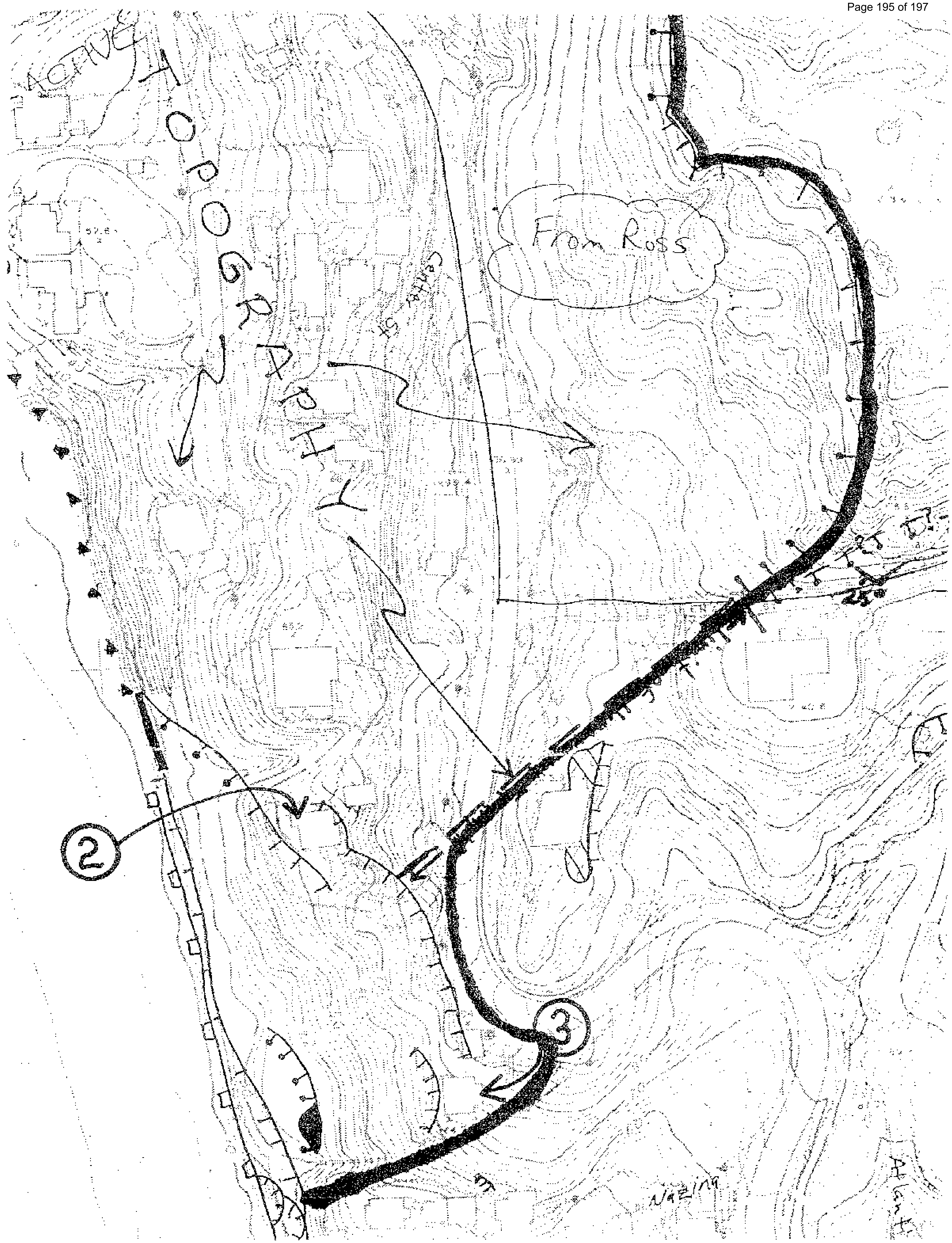
Boring completed at 70.0 feet on September 21, 2000.

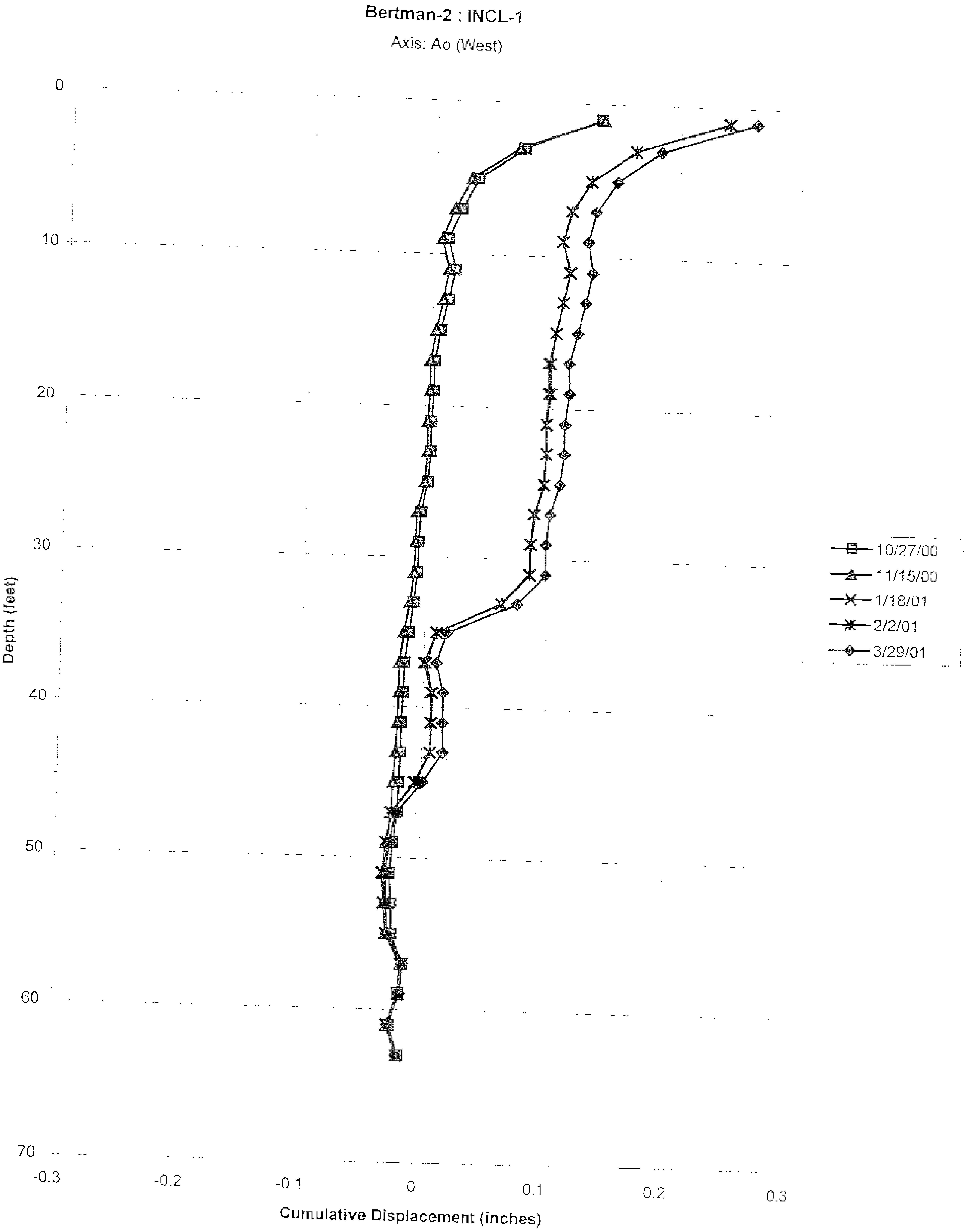
Inclinometer casing installed to 45.0 foot depth with above ground monument.

GEODESIGN INC

BORING B-2 (CONT.)

BERTMAN-2	FEBRUARY 2001	FIGURE A-2


ACTIVE
TOPOGRAPHY
From Ross
②
③
52.6
1/42/19
Alan F


Bertman-2 : INCL-1
Axis: Ao (West)
Depth (feet)
0
10
20
30
40
50
60
70
-0.3
-0.2
-0.1
0
0.1
0.2
0.3
Cumulative Displacement (inches)
10/27/00
11/15/00
1/18/01
2/2/01
3/29/01

Geotech Solutions Inc.

September 2, 2020 *robertscannon-18-1-consultte*

Stanley and Rebecca Roberts
stan.milliman@gmail.com

Cc:
kevin@objectiveadvisorsllc.com
plandevelopment@msn.com
eric@miller-se.com
troy@earth-engineers.com

GEOTECHNICAL ENGINEERING CONSULTATION
Plan Review of Western Stability Piles
Tax Lot 600
Cannon Beach, Oregon

This letter summarizes our review of the structural engineer's plans for the western stability piles on Tax lot 600. We have reviewed the plans and they conform to our geotechnical analyses and report recommendations for the stability pile system. The purpose of our analyses of the western batter piles and associated grade beam was to improve stability of Tax Lot 600 (as summarized in our attached report and submitted reviews of geotechnical related structural plans). These piles will also improve the stability of adjacent and upslope land, including the existing homes and infrastructure such as Hemlock Street and its associated utilities. This pile system is not relied on by any building foundation system for structural support. It is strictly dedicated to improving the lot stability. In that regard, we recommend its installation as soon as possible.

Once this system is in, the lot stability will be significantly improved, and construction during the wet season on the lot would be acceptable and still result in a higher stability condition than is currently present.

The Limitations of our reports apply. If you have any questions, please contact us.

Sincerely,

Don Rondema, MS, PE, GE
Principal

REGISTERED PROFESSIONAL ENGINEER
OREGON
JULY 21, 1992
DON RONDEMA
Expires 12/31/20