REPORT OF GEOTECHNICAL ENGINEERING SERVICES

Proposed Abernethy Place Project
17th Street and Washington Street
Oregon City, Oregon

For
Hackett Hospitality Group, LLC
c/o Hill Architects
May 9, 2017

GeoDesign Project: HillArch-2-02

1.0 INTRODUCTION

This report presents the results of GeoDesign's geotechnical engineering evaluation for the proposed Abernethy Place project located north of the intersection of 17th Street and Washington Street in Oregon City, Oregon.

The site is shown relative to surrounding features on Figure 1. A site plan showing the location of our explorations and approximate site boundaries is presented on Figure 2. Acronyms and abbreviations used herein are defined at the end of this document.

2.0 PROJECT UNDERSTANDING

The site is approximately 4 acres in size and currently occupied by various businesses and the historic Hackett House. With the exception of the Hackett House, all the structures will be demolished. Based on information available in the RFP dated February 14, 2017, the proposed development will likely consist of two structures (Buildings A and B) up to five stories in height; basements are not planned. Building A is located on the western portion of the site and will include a hotel structure, and Building B is located on the eastern portion of the site and will likely be a mixed-use structure. In addition, development will include paved parking and drive aisles along with stormwater treatment facilities. Maximum column loads are estimated to be 400 kips and wall loads (dead and live loads) will be 14 kips per linear foot. We have assumed that floor slab loads will be less than 150 psf. Based on information provided by Sisul Engineering, site cuts will generally range from 0 to 2 feet above existing grades across the northwestern portion of the site and in a small area at the eastern edge. Site fills will range up 8 feet above existing grades across the southwestern portion of the site.

3.0 PURPOSE AND SCOPE

The purpose of our services was to provide geotechnical engineering recommendations for design and construction of the proposed development, including design parameters and foundation options. The specific scope of our services is summarized as follows:

- Reviewed readily available published geologic data and our in-house files for existing information on subsurface conditions in the site vicinity.
- Completed and submitted permit applications and appropriate fees to the City of Oregon City to conduct exploratory borings in the ROWs at the site.
- Coordinated and managed the field investigation, including locating utilities, coordination with existing tenants, and scheduling subcontractors.
- Completed a geotechnical exploration program that consists of the following:
  - Two borings in each of the building footprints (Buildings A and B) to depths of up to 55.4 feet BGS.
  - Four shallow borings to depths of up to 12.0 feet BGS within the parking and drive aisle areas. The borings were drilled using mud rotary and hollow-stem auger drilling methods.
- Conducted infiltration testing in two of the borings at a depth of 5.0 feet BGS. Test locations were discussed with Sisul Engineering.

- Collected soil samples for laboratory testing, and maintained a log of encountered soil and groundwater conditions in each exploration.
- Completed the following laboratory testing on selected soil samples:
    - Thirty-one moisture content determinations in general accordance with ASTM D 2216
    - One consolidation test in general accordance with ASTM D 2435
    - Four Atterberg limits tests in general accordance with ASTM D 4318
    - Four particle-size analyses in general accordance with ASTM D 1140
- Provide recommendations for site preparation and grading, including over-excavation, general excavation, temporary and permanent slopes, fill placement and compaction criteria, suitability of on-site soil for fill, subgrade preparation for buildings and pavements, and recommendations for wet weather construction.
- Provided foundation support recommendations for the proposed development. Our recommendations will include preferred foundation type, allowable bearing capacity, and lateral resistance parameters.
- Evaluated the liquefaction potential at the site.
- Assessed geologic hazard issues as of the City of Oregon City Master Plan process. This assessment was provided as a separate letter to our report.
- Provided seismic design recommendations in accordance with the procedures outlined in the 2012 IBC and 2014 SOSSC.
- Discussed groundwater conditions at the site, including recommendations for dewatering during construction and subsurface drainage (if required).
- Provided floor slabs recommendations.
- Provided trench backfill recommendations.
- Provided pavement recommendations for automobile driving and parking areas and heavy truck traffic areas in proposed parking and driveway areas.
- Prepared this geotechnical engineering report that presents our findings, conclusions, and recommendations. In addition, provided ten paper copies of the geotechnical report.

4.0 SITE CONDITIONS

4.1 SURFACE CONDITIONS

The site consists of an approximately 4-acre, irregular-shaped parcel. The site is currently occupied by various single-story structures with associated sheds and AC parking and drive aisles across the northeastern portions. The historic Hackett House and AC-paved parking area and vacant parcel with remnant AC areas are located to the southwest. Several large trees are located near the central and southwestern portions of the site near the Hackett House. Site elevations range from 39 feet (NAVD 1988) at the northeastern edge to 52 feet (NAVD 1988) at the southwestern corner. The topography of the site generally slopes slightly downward form south-southwest to northeast across the site.

Union Pacific Railroad tracks border the site to the west and Abernethy Creek is located to the south of the site across 17th Street. Land use in the vicinity of the site is mixed industrial, commercial, and residential.

4.2 SUBSURFACE CONDITIONS

We explored subsurface conditions at the site by drilling four borings (B-1 through B-4) to depths of up to 55.4 feet BGS. The borings were completed within the proposed building footprints. Approximate boring locations are shown on Figure 2. The details of our field exploration and laboratory testing programs, exploration logs, and laboratory test results are presented in Appendix A. Also, we reviewed boring logs and laboratory data from the previous geotechnical study at the train station located immediately north of the site. Relevant site plans, exploration logs, and laboratory data from the adjacent geotechnical study are presented in Appendix B.

In general, subsurface conditions in the borings consist of fill underlain by alluvial silt and sand; gravel underlies the silt and sand.

4.2.1 Fill

Fill is present all borings to depths ranging between 1 and 11.5 feet BGS (elevation 29.5 to 45.0 feet). The fill is comprised of loose to medium dense gravel and sand and soft to stiff silt. Organic material (woody debris) was also encountered in the fill. Fill of this type and consistency generally exhibits moderate strength characteristics and compressibility characteristics that are highly variable and unpredictable.

4.2.2 Alluvial Silt and Sand

The fill is underlain by layers of silt and sand. Based on the SPTs, the silt is very soft to medium stiff silt and contains varying proportions of sand and clay. SPTs show that the sand is very loose to loose and contains varying proportions of silt and extends to depths of approximately 34.5 to 52.5 feet BGS (elevation -0.5 foot to 4.5 feet) at the boring locations. Soils of this type and consistency generally exhibit low to moderate to high strength and moderate compressibility characteristics. Our laboratory testing program shows that the moisture content of the alluvial silt and sand varied between approximately 32 to 51 percent at the time of our exploration.

4.2.3 Gravel

The alluvial silt and sand is underlain by very dense gravel with varying proportions of silt and sand to the total depths explored of 42 to 55.4 feet BGS (elevation -3 to -8.3 feet) in borings B-1 through B-4. Cobbles are typically present in the gravel formation. Based on SPTs, the gravel is very dense. Gravel of this type and density generally exhibits moderate to high strength and low compressibility characteristics. Based on our laboratory testing program, the moisture content of the gravel varied between 10 and 36 percent at the time of our exploration.

4.2.4 Groundwater

Mud rotary drilling methods prevented groundwater observation in the borings. Free water was generally encountered in the borings at depths of 25 to 30 feet BGS. Groundwater was reported in the boring at the adjacent train station at similar depths. The depth to groundwater may fluctuate in response to seasonal changes, prolonged rainfall, changes in surface topography, and other factors not observed in this study.

The site is map both within the FEMA 100-year flood plain and 1996 flood inundation zone.

4.2.5 Infiltration

Infiltration testing was conducted in borings B-3 and B-6 located on the north and west sides of the site, respectively. Infiltration testing was conducted in general accordance with the local jurisdiction requirements and City of Portland 2016 Stormwater Management Manual.

Table 1 summarizes the infiltration test results. The exploration logs are presented in Appendix A.

Table 1. Infiltration Test Results

Exploration	Depth (feet BGS)	Soil Description	Observed Infiltration Rate (inches/hour)
B-3	5	Silt with trace sand (native)	Negligible
B-6	5	Silty sand (native)	Negligible

FIGURES

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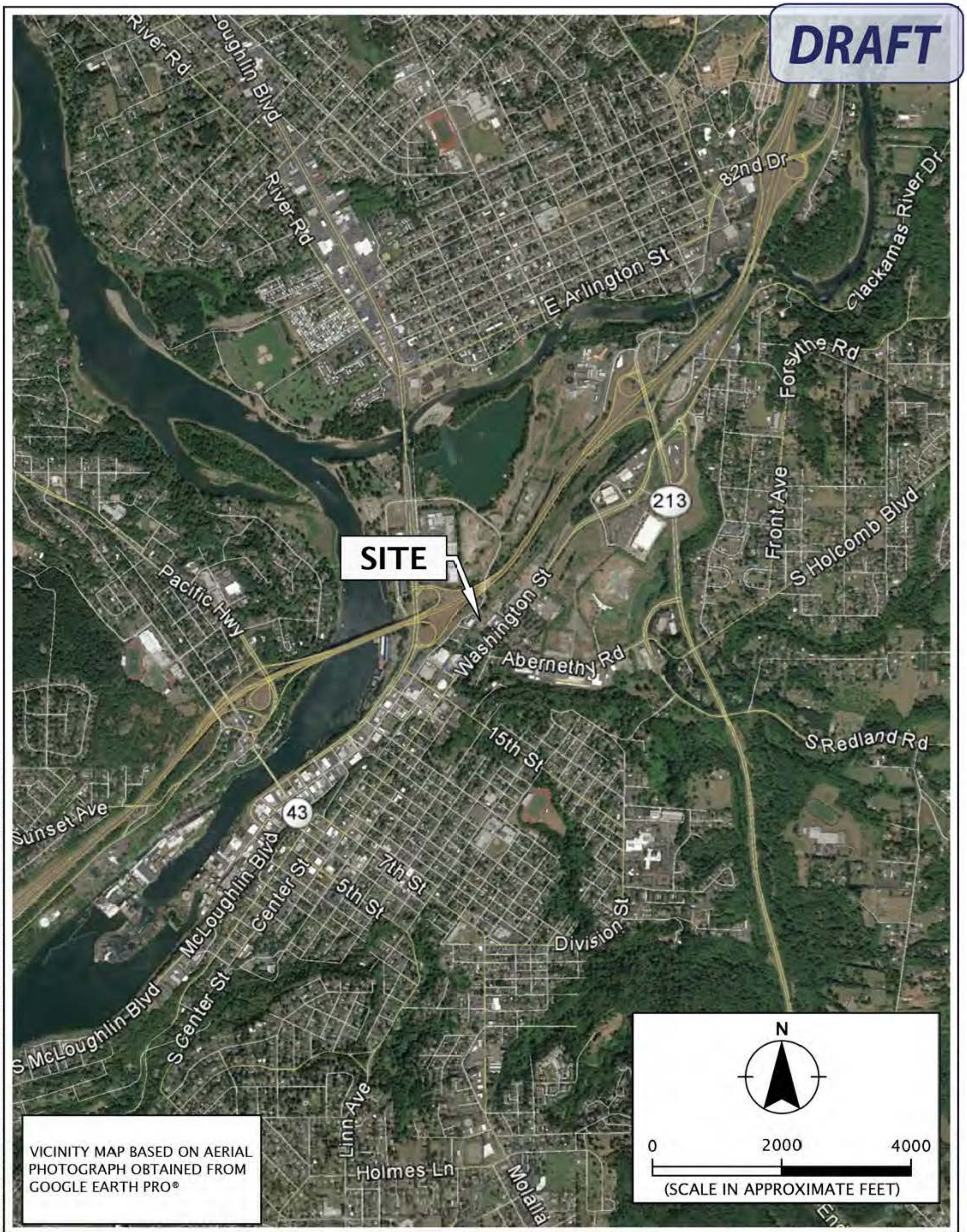
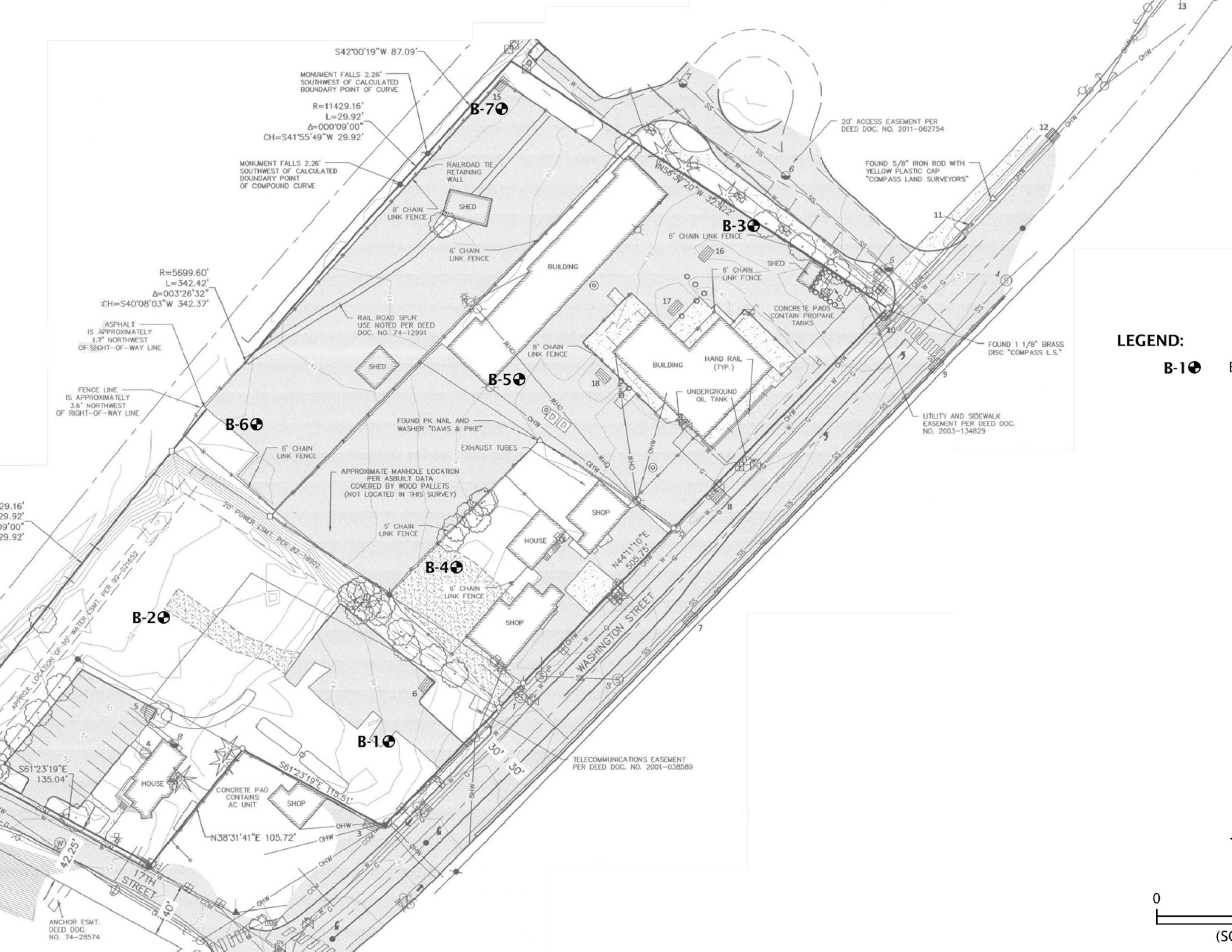


VICINITY MAP BASED ON AERIAL PHOTOGRAPH OBTAINED FROM GOOGLE EARTH PRO®

GEODESIGN INC 9450 SW Commerce Circle - Suite 300 Wilsonville OR 97070 503.968.8787 www.geodesigninc.com	HILLARCH-2-02	**VICINITY MAP**	
	MAY 2017	PROPOSED ABERNETHY PLACE PROJECT OREGON CITY, OR	**FIGURE 1**

Printed By: aday | Print Date: 5/3/2017 9:23:30 AM
File Name: J:\E-L\HillArch\HillArch-2-02\Figures\CAD\HillArch-2-02-VM01.dwg | Layout: FIGURE 1



LEGEND:
B-1
0
(SC
B-1
B-2
B-3
B-4
B-5
B-6
B-7
S42°00'19"W 87.09'
MONUMENT FALLS 2.26' SOUTHWEST OF CALCULATED BOUNDARY POINT OF CURVE
R=11429.16'
L=29.92'
Δ=000°09'00"
CH=S41°55'49"W 29.92'
MONUMENT FALLS 2.26' SOUTHWEST OF CALCULATED BOUNDARY POINT OF COMPOUND CURVE
RAILROAD TIE RETAINING WALL
8' CHAIN LINK FENCE
SHED
6' CHAIN LINK FENCE
BUILDING
N56°31'20"W 323.22'
20' ACCESS EASEMENT PER DEED DOC. NO. 2011-062754
FOUND 5/8" IRON ROD WITH YELLOW PLASTIC CAP "COMPASS LAND SURVEYORS"
6' CHAIN LINK FENCE
SHED
6' CHAIN LINK FENCE
CONCRETE PADS CONTAIN PROPANE TANKS
FOUND 1 1/8" BRASS DISC "COMPASS L.S."
R=5699.60'
L=342.42'
Δ=003°26'32"
CH=S40°08'03"W 342.37'
ASPHALT IS APPROXIMATELY 1.7' NORTHWEST OF RIGHT-OF-WAY LINE
RAIL ROAD SPUR USE NOTED PER DEED DOC. NO. 74-12991
SHED
8' CHAIN LINK FENCE
BUILDING
HAND RAIL (TYP.)
UNDERGROUND OIL TANK
FENCE LINE IS APPROXIMATELY 3.6' NORTHWEST OF RIGHT-OF-WAY LINE
FOUND PK NAIL AND WASHER "DAVIS & PIKE"
UTILITY AND SIDEWALK EASEMENT PER DEED DOC. NO. 2003-134829
6' CHAIN LINK FENCE
EXHAUST TUBES
APPROXIMATE MANHOLE LOCATION PER ASBUILT DATA COVERED BY WOOD PALLETS (NOT LOCATED IN THIS SURVEY)
SHOP
HOUSE
5' CHAIN LINK FENCE
N44°11'10"E 505.75'
20' POWER ESMT. PER 82-18922
APPROX. LOCATION OF 10' WATER ESMT. PER 99-021652
6' CHAIN LINK FENCE
SHOP
WASHINGTON STREET
29.16'
29.92'
09'00"
29.92'
S61°23'19"E 135.04'
HOUSE
CONCRETE PAD CONTAINS AC UNIT
SHOP
S61°23'19"E 115.51'
N38°31'41"E 105.72'
30'
30'
TELECOMMUNICATIONS EASEMENT PER DEED DOC. NO. 2001-038589
42.25'
17TH STREET
40'
ANCHOR ESMT. DEED DOC. NO. 74-28574
OHW
SS
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COM
1
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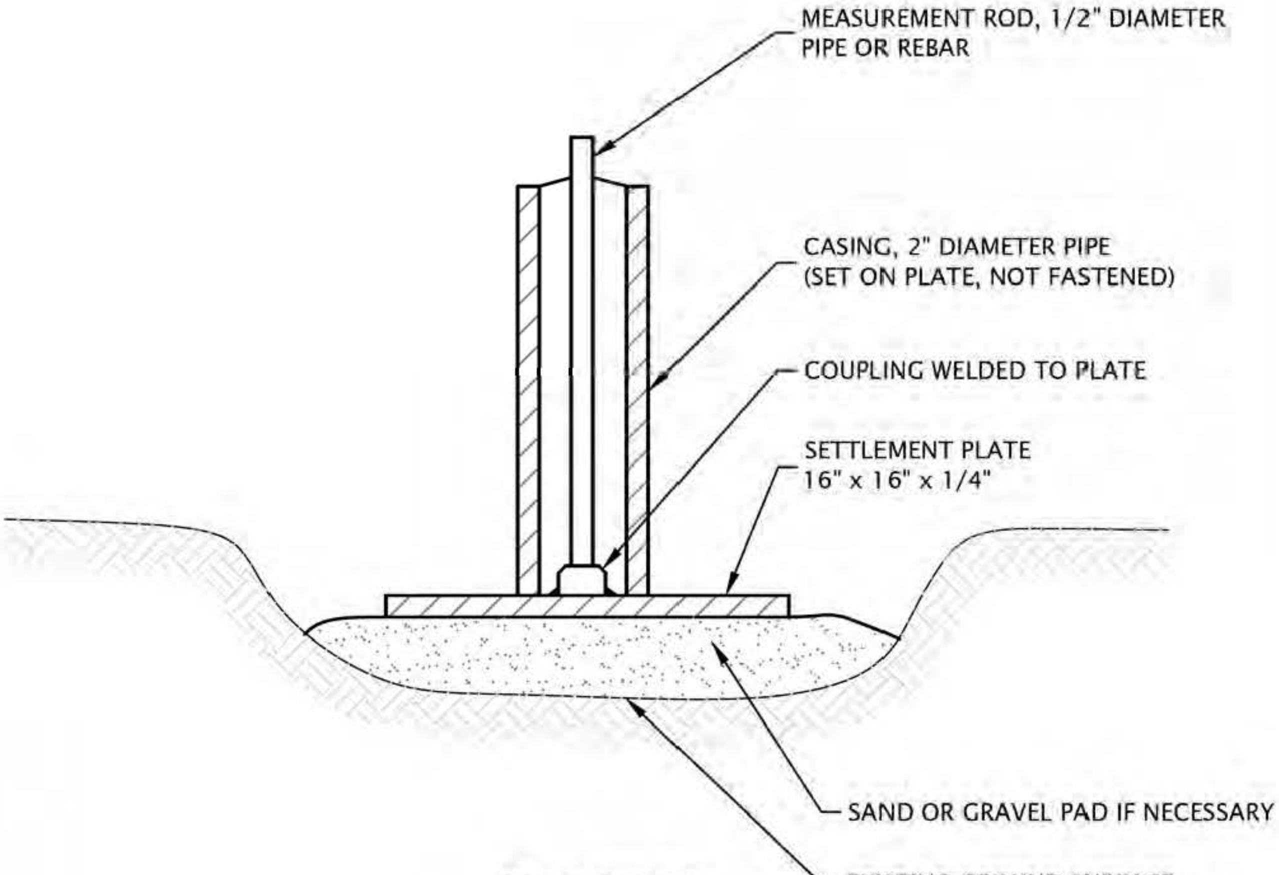

NOT TO SCALE

NOTES:

1. INSTALL MARKERS ON FIRM GROUND OR ON SAND OR GRAVEL PADS IF NEEDED FOR STABILITY. TAKE INITIAL READING ON TOP OF ROD AND AT ADJACENT GROUND LEVEL PRIOR TO PLACING ANY FILL.
2. FOR EASE IN HANDLING, ROD AND CASING ARE USUALLY INSTALLED IN 5-FOOT SECTIONS. AS FILL PROGRESSES, COUPLINGS ARE USED TO INSTALL ADDITIONAL LENGTHS. CONTINUITY IS MAINTAINED BY READING THE TOP OF THE MEASUREMENT ROD, THEN IMMEDIATELY ADDING THE NEW SECTION AND READING THE TOP OF THE ADDED ROD. BOTH READINGS ARE RECORDED.
3. RECORD THE ELEVATION OF THE TOP OF THE MEASUREMENT ROD IN EACH MARKER AT THE RECOMMENDED TIME INTERVALS. EACH TIME, NOTE THE ELEVATION OF THE ADJACENT FILL SURFACE.
4. READ THE MARKER TO THE NEAREST 0.01 FOOT, OR 0.005 FOOT IF POSSIBLE. NOTE THE FILL ELEVATION TO THE NEAREST 0.1 FOOT.

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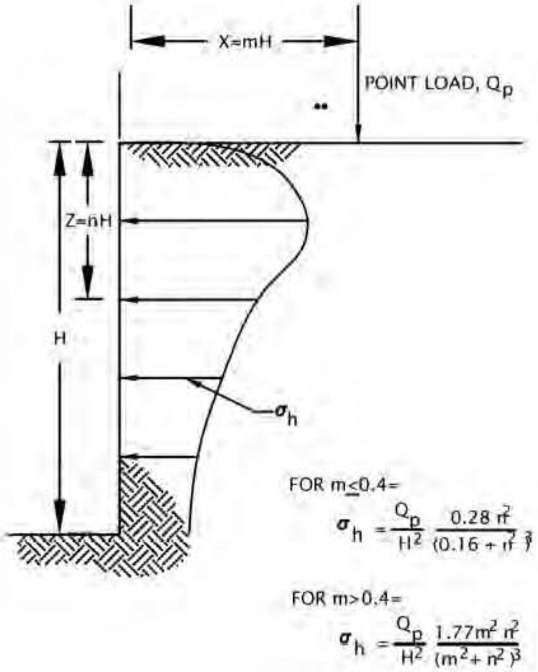
	MAY 2017	PROPOSED ABERNETHY PLACE PROJECT OREGON CITY, OR	**FIGURE 3**

Printed By: aday | Print Date: 5/3/2017 9:23:32 AM
File Name: J:\E-L\HillArch\HillArch-2\HillArch-2-02\Figures\CAD\HillArch-2-02-det-stlmnt-plt.dwg | Layout: FIGURE 3

Printed By: aday | Print Date: 5/3/2017 9:23:32 AM
File Name: J:\E-L\HillArch\HillArch-2\HillArch-2-02\Figures\CAD\HillArch-2-02-DET01.dwg | Layout: FIGURE 4

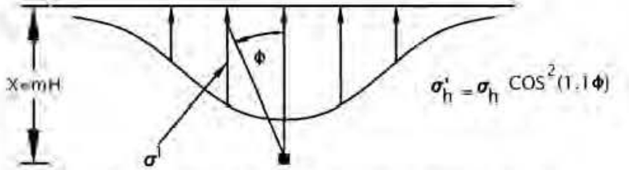


FOR m≤0.4=

$$\sigma_h = \frac{Q_p}{H^2}\,\frac{0.28\,n^2}{(0.16+n^2)^3}$$

FOR m>0.4=

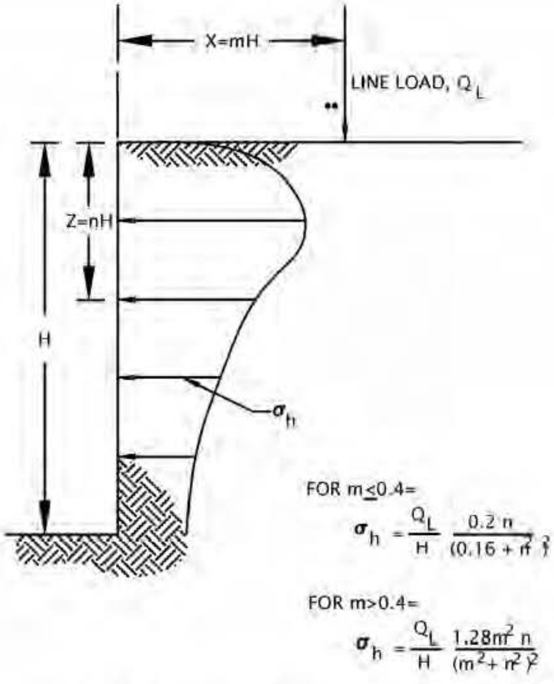
$$\sigma_h = \frac{Q_p}{H^2}\,\frac{1.77m^2\,n^2}{(m^2+n^2)^3}$$



$$\sigma'_h = \sigma_h\ \cos^2(1.1\Phi)$$

DISTRIBUTION OF HORIZONTAL PRESSURES

VERTICAL POINT LOAD



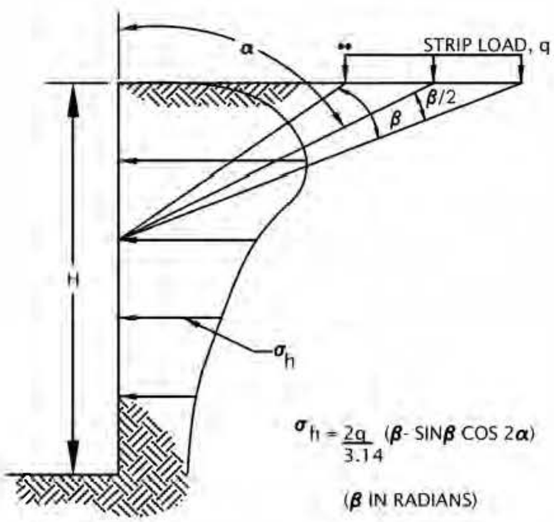
FOR m≤0.4=

$$\sigma_h = \frac{Q_L}{H}\,\frac{0.2\,n}{(0.16+n^2)^2}$$

FOR m>0.4=

$$\sigma_h = \frac{Q_L}{H}\,\frac{1.28m^2\,n}{(m^2+n^2)^2}$$

LINE LOAD PARALLEL TO WALL



$$\sigma_h = \frac{2q}{3.14}\,(\beta - \text{SIN}\beta\ \text{COS}\ 2\alpha)$$

(β IN RADIANS)

STRIP LOAD PARALLEL TO WALL

NOTES:

1. THESE GUIDELINES APPLY TO RIGID WALLS WITH POISSON'S RATIO ASSUMED TO BE 0.5 FOR BACKFILL MATERIALS.
2. LATERAL PRESSURES FROM ANY COMBINATION OF ABOVE LOADS MAY BE DETERMINED BY THE PRINCIPLE OF SUPERPOSITION.
3. VALUES IN THIS FIGURE ARE UNFACTORED.

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SURCHARGE-INDUCED LATERAL EARTH PRESSURES

PROPOSED ABERNETHY PLACE PROJECT
OREGON CITY, OR

FIGURE 4

APPENDIX A

APPENDIX A

FIELD EXPLORATIONS

GENERAL

We explored subsurface conditions at the site by drilling eight borings (B-1 through B-8) to depths of up to approximately 55.4 feet BGS. Figure 2 shows the approximate exploration locations. The borings were drilled on March 8 through 10, 2017 by Hard Core Drilling of Dundee, Oregon. The borings were drilled using mud rotary and hollow-stem auger drilling methods. The exploration logs are presented in this appendix.

The exploration locations were located in the field pacing from survey existing site features. This information should be considered accurate only to the degree implied by the methods used.

SOIL SAMPLING

Members of our geology staff observed the explorations. We collected representative samples of the various soil encountered in the explorations for geotechnical laboratory testing. Sampling methods and intervals are shown on the exploration logs.

Soil samples were collected from the borings using the following methods:

- SPTs were performed in general conformance with ASTM D 1586. The sampler was driven with a 140-pound automatic trip hammer free-falling 30 inches. The number of blows required to drive the sampler 1 foot, or as otherwise indicated, into the soil is shown adjacent to the sample symbols on the exploration logs. Disturbed samples were collected from the split barrel for subsequent classification and index testing.
- Relatively undisturbed samples were obtained at selected intervals by pushing a Shelby tube sampler 24 inches ahead of the boring front.

The calibration factor for the SPT hammer used by Hard Core Drilling was 87 percent. The results of the calibration testing are presented at the end of this appendix.

SOIL CLASSIFICATION

The soil samples were classified in accordance with the "Exploration Key" (Table A-1) and "Soil Classification System" (Table A-2), which are presented in this appendix. The exploration logs indicate the depths at which the soils or their characteristics change, although the change actually could be gradual. If the change occurred between sample locations, the depth was interpreted. Classifications are shown on the exploration logs.

LABORATORY TESTING

CLASSIFICATION

The soil samples were classified in the laboratory to confirm field classifications. The laboratory classifications are shown on the exploration logs if those classifications differed from the field classifications.

MOISTURE CONTENT
We tested the natural moisture content of selected soil samples in general accordance with ASTM D 2216. The natural moisture content is a ratio of the weight of the water to soil in a test sample and is expressed as a percentage. The test results are presented in this appendix.

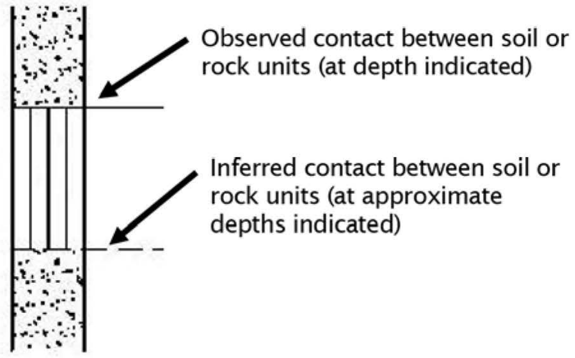
ATTERBERG LIMITS
The plastic limit and liquid limit (Atterberg limits) of selected soil samples were determined in accordance with ASTM D 4318. The Atterberg limits and the plasticity index were completed to aid in the classification of the soil. The test results are presented in this appendix.

PARTICLE-SIZE TESTING
Particle-size testing was completed on selected soil samples in general accordance with ASTM D 1140. The test results are presented in this appendix.

CONSOLIDATION TESTING
We performed one-dimensional consolidation testing in general accordance with ASTM D 2435 on a relatively undisturbed sample. The test measures the volume change of a soil sample under predetermined loads. The test results are presented in this appendix.

SYMBOL	SAMPLING DESCRIPTION
▯	Location of sample obtained in general accordance with ASTM D 1586 Standard Penetration Test with recovery
▮	Location of sample obtained using thin-wall Shelby tube or Geoprobe® sampler in general accordance with ASTM D 1587 with recovery
▥	Location of sample obtained using Dames & Moore sampler and 300-pound hammer or pushed with recovery
▥	Location of sample obtained using Dames & Moore and 140-pound hammer or pushed with recovery
⧗	Location of sample obtained using 3-inch-O.D. California split-spoon sampler and 140-pound hammer
☒	Location of grab sample
▮	Rock coring interval
▽	Water level during drilling
▼	Water level taken on date shown

Graphic Log of Soil and Rock Types



GEOTECHNICAL TESTING EXPLANATIONS

ATT	Atterberg Limits	PP	Pocket Penetrometer
CBR	California Bearing Ratio	P200	Percent Passing U.S. Standard No. 200 Sieve
CON	Consolidation		
DD	Dry Density	RES	Resilient Modulus
DS	Direct Shear	SIEV	Sieve Gradation
HYD	Hydrometer Gradation	TOR	Torvane
MC	Moisture Content	UC	Unconfined Compressive Strength
MD	Moisture-Density Relationship	VS	Vane Shear
OC	Organic Content	kPa	Kilopascal
P	Pushed Sample		

ENVIRONMENTAL TESTING EXPLANATIONS

CA	Sample Submitted for Chemical Analysis	ND	Not Detected
P	Pushed Sample	NS	No Visible Sheen
PID	Photoionization Detector Headspace Analysis	SS	Slight Sheen
		MS	Moderate Sheen
ppm	Parts per Million	HS	Heavy Sheen

EXPLORATION KEY

TABLE A-1

RELATIVE DENSITY - COARSE-GRAINED SOILS

Relative Density	Standard Penetration Resistance	Dames & Moore Sampler (140-pound hammer)	Dames & Moore Sampler (300-pound hammer)
Very Loose	0 – 4	0 - 11	0 - 4
Loose	4 – 10	11 - 26	4 - 10
Medium Dense	10 – 30	26 - 74	10 - 30
Dense	30 – 50	74 - 120	30 - 47
Very Dense	More than 50	More than 120	More than 47

CONSISTENCY - FINE-GRAINED SOILS

Consistency	Standard Penetration Resistance	Dames & Moore Sampler (140-pound hammer)	Dames & Moore Sampler (300-pound hammer)	Unconfined Compressive Strength (tsf)
Very Soft	Less than 2	Less than 3	Less than 2	Less than 0.25
Soft	2 - 4	3 – 6	2 - 5	0.25 - 0.50
Medium Stiff	4 - 8	6 – 12	5 - 9	0.50 - 1.0
Stiff	8 - 15	12 – 25	9 - 19	1.0 - 2.0
Very Stiff	15 - 30	25 – 65	19 – 31	2.0 - 4.0
Hard	More than 30	More than 65	More than 31	More than 4.0

PRIMARY SOIL DIVISIONS			GROUP SYMBOL	GROUP 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 GRAVELS (< 5% fines)	GW or GP	GRAVEL
		GRAVEL WITH FINES (≥ 5% and ≤ 12% fines)	GW-GM or GP-GM	GRAVEL with silt
			GW-GC or GP-GC	GRAVEL with clay
		GRAVELS WITH FINES (> 12% fines)	GM	silty GRAVEL
			GC	clayey GRAVEL
			GC-GM	silty, clayey GRAVEL
	SAND (50% or more of coarse fraction passing No. 4 sieve)	CLEAN SANDS (<5% fines)	SW or SP	SAND
		SANDS WITH FINES (≥ 5% and ≤ 12% fines)	SW-SM or SP-SM	SAND with silt
			SW-SC or SP-SC	SAND with clay
		SANDS WITH FINES (> 12% fines)	SM	silty SAND
			SC	clayey SAND
			SC-SM	silty, clayey SAND
FINE-GRAINED SOILS (50% or more passing No. 200 sieve)	SILT AND CLAY	Liquid limit less than 50	ML	SILT
			CL	CLAY
			CL-ML	silty CLAY
			OL	ORGANIC SILT or ORGANIC CLAY
		Liquid limit 50 or greater	MH	SILT
			CH	CLAY
			OH	ORGANIC SILT or ORGANIC CLAY
HIGHLY ORGANIC SOILS			PT	PEAT

MOISTURE CLASSIFICATION

Term	Field Test
dry	very low moisture, dry to touch
moist	damp, without visible moisture
wet	visible free water, usually saturated

ADDITIONAL CONSTITUENTS

Secondary granular components or other materials such as organics, man-made debris, etc.					
Percent	Silt and Clay In:		Percent	Sand and Gravel In:	
	Fine-Grained Soils	Coarse-Grained Soils		Fine-Grained Soils	Coarse-Grained Soils
< 5	trace	trace	< 5	trace	trace
5 – 12	minor	with	5 – 15	minor	minor
> 12	some	silty/clayey	15 – 30	with	with
			> 30	sandy/gravelly	Indicate %

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SOIL CLASSIFICATION SYSTEM

TABLE A-2

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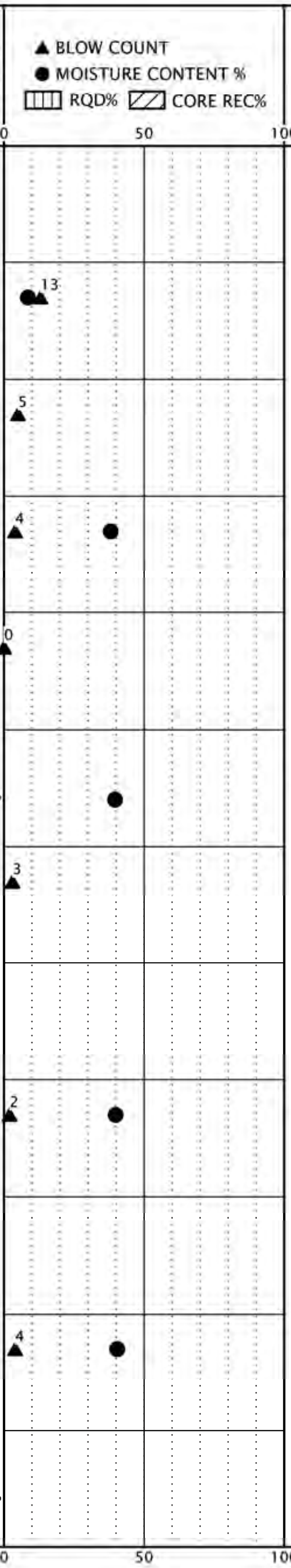
DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	BLOW COUNT	INSTALLATION AND COMMENTS
0.0	Medium dense, gray GRAVEL with silt and sand (GP-GM); moist - **FILL**.	46.0			
				13	
5.0	Medium stiff, gray-dark gray SILT with sand (ML), trace organics (woody debris) and gravel; moist to wet, sand is fine, organics are ~<1/2-inch diameter.	41.0 5.0		5	
7.5	soft to medium stiff, without gravel at 7.5 feet		P200	4	P200 = 27%
	Very loose to loose, light brown, silty SAND (SM); moist, fine (alluvium).	37.5 8.5			
10.0	Very soft, brown SILT with sand (ML), trace organics (carbonized wood); wet, sand is fine.	36.0 10.0		0	
			DD CON		DD = 79 pcf
15.0	soft, light brown, minor sand, trace clay, without organics; moist to wet, sand is fine, stratified beds of SILT and SILT with sand (~3 inches thick) at 15.0 feet		PP	3	PP = 0.25 tsf
20.0	trace sand; moist at 21.0 feet		PP	2	PP = 0.25 tsf
25.0	soft to medium stiff at 25.0 feet		ATT PP	4	PP = 0.5 tsf LL = 44% PL = 25%
30.0					



DRILLED BY: Hard Core Drilling

LOGGED BY: JGH

COMPLETED: 03/08/17

BORING METHOD: mud rotary (see document text)

BORING BIT DIAMETER: 4 7/8 inches

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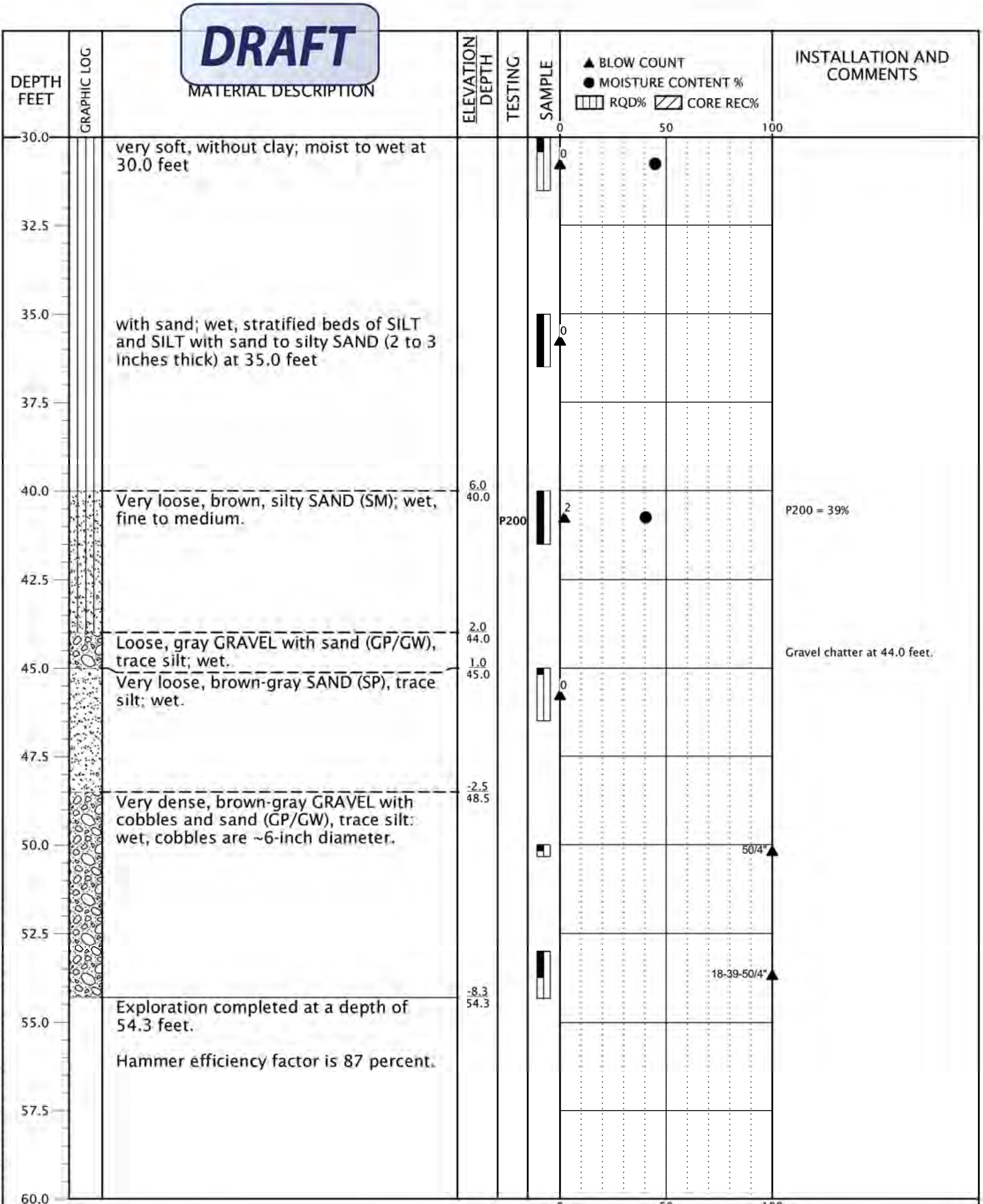
BORING B-1

PROPOSED ABERNETHY PLACE PROJECT
OREGON CITY, OR

FIGURE A-1



DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	BLOW COUNT	INSTALLATION AND COMMENTS
30.0	very soft, without clay; moist to wet at 30.0 feet			0	
35.0	with sand; wet, stratified beds of SILT and SILT with sand to silty SAND (2 to 3 inches thick) at 35.0 feet			0	
40.0	Very loose, brown, silty SAND (SM); wet, fine to medium.	6.0 40.0	P200	2	P200 = 39%
44.0	Loose, gray GRAVEL with sand (GP/GW), trace silt; wet.	2.0 44.0			Gravel chatter at 44.0 feet.
45.0	Very loose, brown-gray SAND (SP), trace silt; wet.	1.0 45.0		0	
48.5	Very dense, brown-gray GRAVEL with cobbles and sand (GP/GW), trace silt; wet, cobbles are ~6-inch diameter.	-2.5 48.5		50/4" 18-39-50/4"	
54.3	Exploration completed at a depth of 54.3 feet. Hammer efficiency factor is 87 percent.	-8.3 54.3			

DRILLED BY: Hard Core Drilling
LOGGED BY: JGH
COMPLETED: 03/08/17

BORING METHOD: mud rotary (see document text)
BORING BIT DIAMETER: 4 7/8 inches

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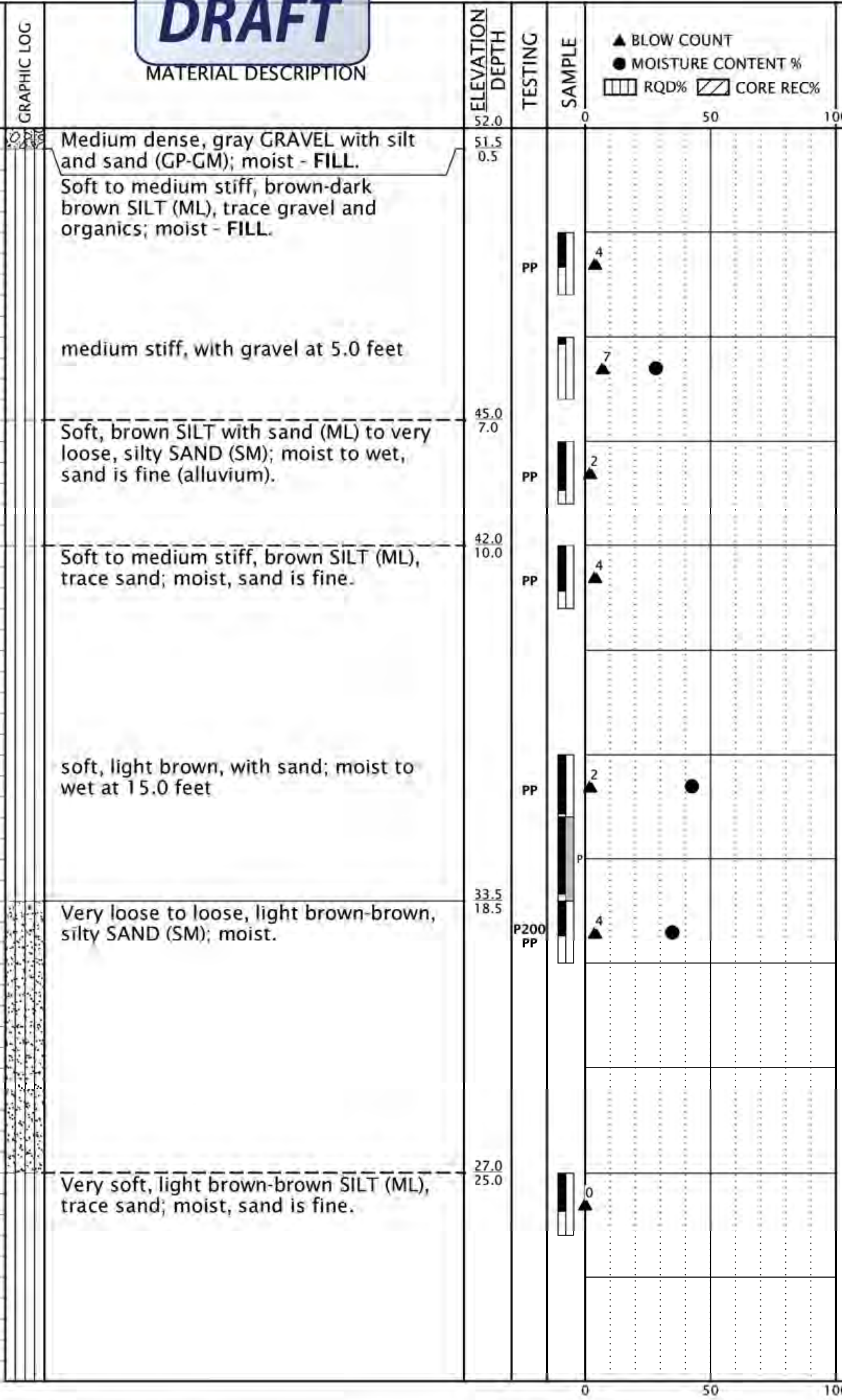
BORING B-1
(continued)

PROPOSED ABERNETHY PLACE PROJECT
OREGON CITY, OR

FIGURE A-1

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	INSTALLATION AND COMMENTS
0.0		52.0		
	Medium dense, gray GRAVEL with silt and sand (GP-GM); moist - **FILL**.	51.5 0.5		
	Soft to medium stiff, brown-dark brown SILT (ML), trace gravel and organics; moist - **FILL**.			
2.5			PP	PP = 0.5 tsf
5.0	medium stiff, with gravel at 5.0 feet			Gravel or chunk of concrete at 4.5 feet.
	Soft, brown SILT with sand (ML) to very loose, silty SAND (SM); moist to wet, sand is fine (alluvium).	45.0 7.0		
7.5			PP	PP = 0.0 tsf
10.0	Soft to medium stiff, brown SILT (ML), trace sand; moist, sand is fine.	42.0 10.0	PP	PP = 0.25 tsf
12.5				
15.0	soft, light brown, with sand; moist to wet at 15.0 feet		PP	PP = 0.25 tsf
17.5				
	Very loose to loose, light brown-brown, silty SAND (SM); moist.	33.5 18.5	P200 PP	P200 = 33% PP = 0.0 tsf
20.0				
22.5				
25.0	Very soft, light brown-brown SILT (ML), trace sand; moist, sand is fine.	27.0 25.0		
27.5				
30.0				



DRILLED BY: Hard Core Drilling

LOGGED BY: JGH/ZMR

COMPLETED: 03/08/17

BORING METHOD: mud rotary (see document text)

BORING BIT DIAMETER: 4 7/8 inches

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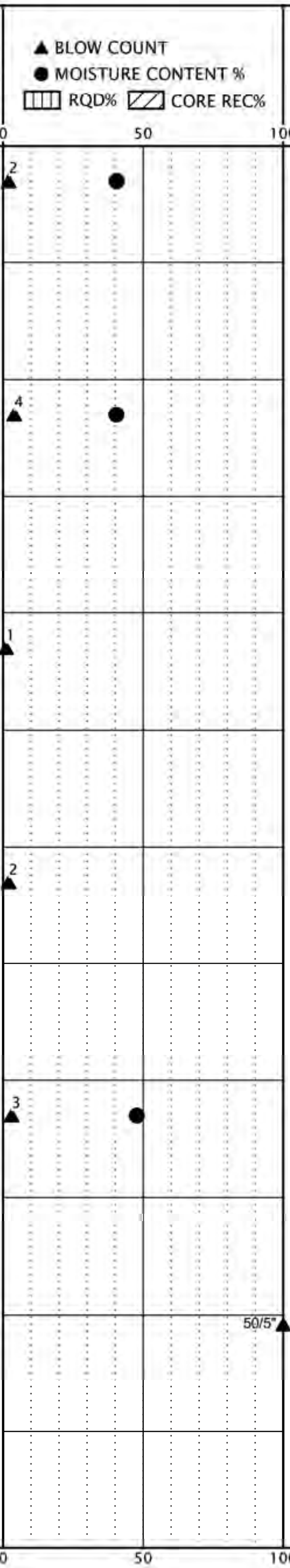
BORING B-2

PROPOSED ABERNETHY PLACE PROJECT
OREGON CITY, OR

FIGURE A-2

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	INSTALLATION AND COMMENTS
30.0	soft, minor sand; moist to wet at 30.0 feet		ATT PP	PP = 0.0 tsf LL = 37% PL = 28%
35.0	soft to medium stiff, with sand; wet, stratified beds of silty SAND (2 to 3 inches thick) at 35.0 feet		P200 PP	P200 = 55% PP = 0.25 tsf
40.0	very soft, trace sand; moist to wet at 40.0 feet		PP	PP = 0.0 tsf
45.0	Very loose, light brown-orange, silty SAND (SM); wet, fine, laminated to stratified beds of SILT (up to 1 inch thick).	7.0 45.0	PP	PP = 0.0 tsf
50.0	Soft, light brown-orange SILT with sand (ML) to silty SAND (SM); moist to wet, sand is fine, stratified beds of SILT; moist and silty SAND; wet.	2.0 50.0	PP	PP = 0.0 tsf
52.5	Very dense, brown-gray GRAVEL with sand (GP/GW), trace silt; wet.	-0.5 52.5		Driller Comment: gravelly at 52.5 feet.
55.4	Exploration completed at a depth of 55.4 feet. Hammer efficiency factor is 87 percent.	-3.4 55.4		



DRILLED BY: Hard Core Drilling
LOGGED BY: JGH/ZMR
COMPLETED: 03/08/17

BORING METHOD: mud rotary (see document text)
BORING BIT DIAMETER: 4 7/8 inches

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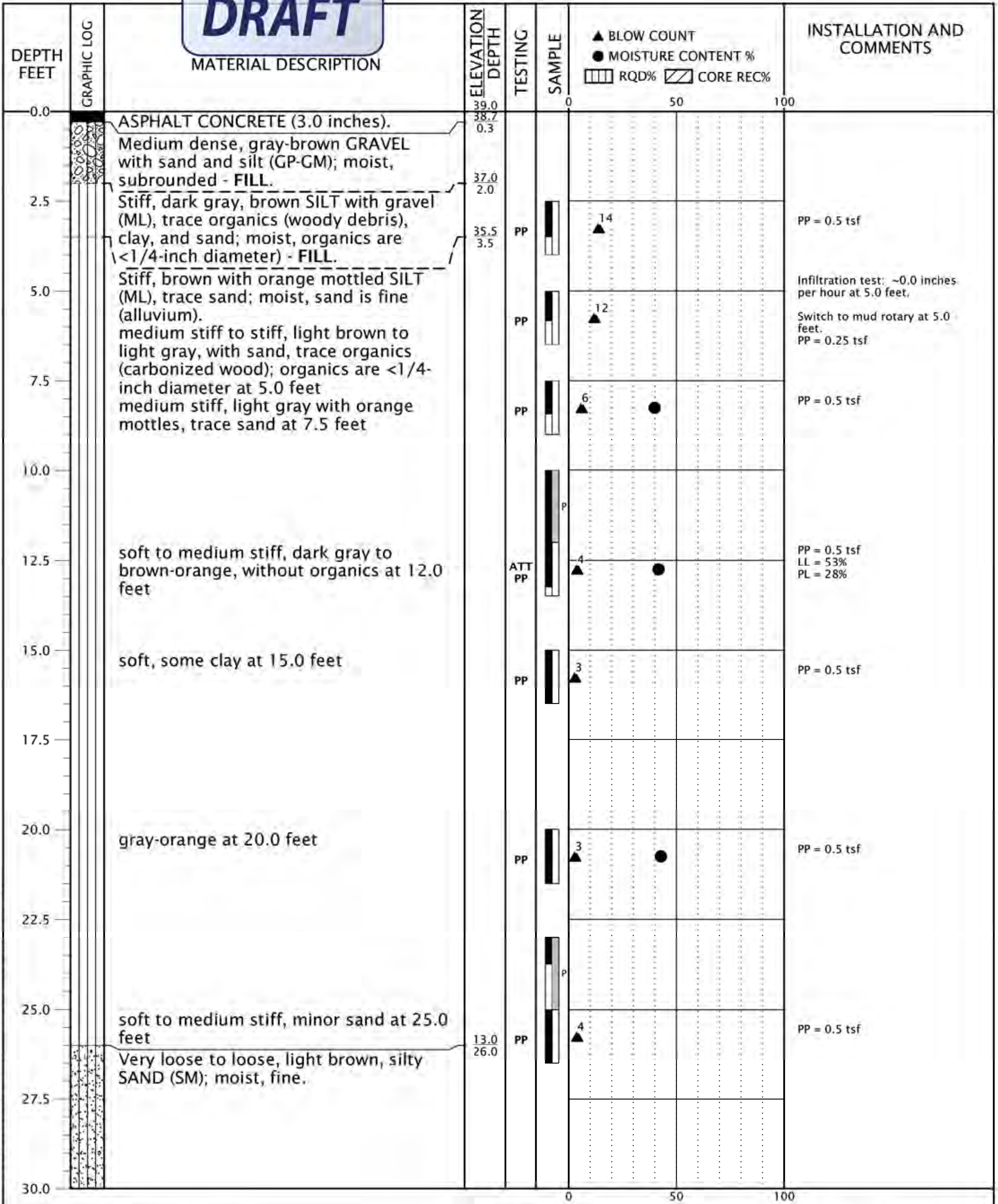
BORING B-2
(continued)

PROPOSED ABERNETHY PLACE PROJECT
OREGON CITY, OR

FIGURE A-2



ASPHALT CONCRETE (3.0 inches).

Medium dense, gray-brown GRAVEL with sand and silt (GP-GM); moist, subrounded - **FILL**.

Stiff, dark gray, brown SILT with gravel (ML), trace organics (woody debris), clay, and sand; moist, organics are <1/4-inch diameter) - **FILL**.

Stiff, brown with orange mottled SILT (ML), trace sand; moist, sand is fine (alluvium).
medium stiff to stiff, light brown to light gray, with sand, trace organics (carbonized wood); organics are <1/4-inch diameter at 5.0 feet
medium stiff, light gray with orange mottles, trace sand at 7.5 feet

soft to medium stiff, dark gray to brown-orange, without organics at 12.0 feet

soft, some clay at 15.0 feet

gray-orange at 20.0 feet

soft to medium stiff, minor sand at 25.0 feet

Very loose to loose, light brown, silty SAND (SM); moist, fine.

PP = 0.5 tsf

Infiltration test: ~0.0 inches per hour at 5.0 feet.

Switch to mud rotary at 5.0 feet.
PP = 0.25 tsf

PP = 0.5 tsf

PP = 0.5 tsf
LL = 53%
PL = 28%

PP = 0.5 tsf

PP = 0.5 tsf

PP = 0.5 tsf

DRILLED BY: Hard Core Drilling

LOGGED BY: JGH/ZMR

COMPLETED: 03/09/17

BORING METHOD: hollow-stem auger and mud rotary (see document text)

BORING BIT DIAMETER: 8 inches/4 7/8 inches

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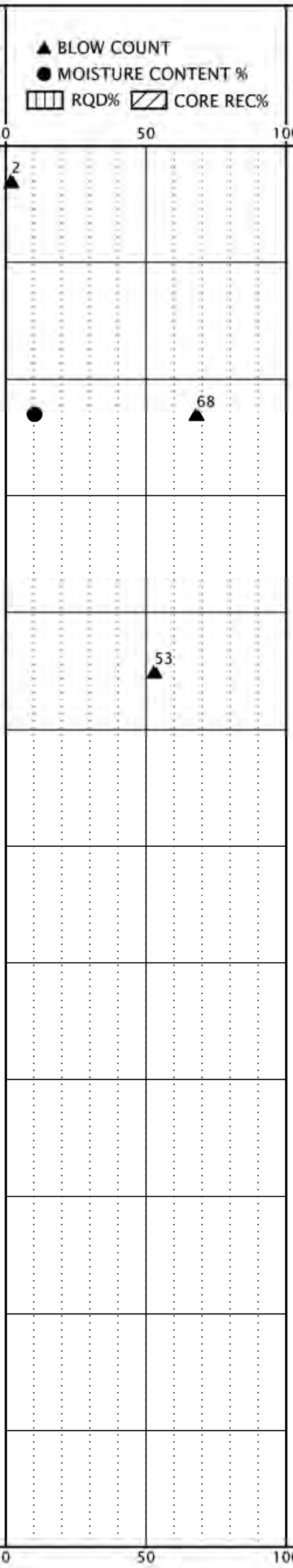
BORING B-3

PROPOSED ABERNETHY PLACE PROJECT
OREGON CITY, OR

FIGURE A-3

DRAFT

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	▲ BLOW COUNT ● MOISTURE CONTENT % ▥ RQD% ▨ CORE REC% 0 50 100	INSTALLATION AND COMMENTS
30.0		Very loose, light brown, silty SAND (SM) to SILT with sand (ML); wet, sand is fine, stratified beds of SILT with sand and silty SAND; sand is fine (up to 3 inches thick).	9.0 30.0	PP		▲ 2	PP = 0.0 tsf
32.5							
35.0		Very dense, gray-brown GRAVEL with sand (GP/GW), trace silt; wet.	4.5 34.5			● ▲ 68	Driller Comment: gravel at 34.5 feet.
37.5							
40.0		with cobbles at 40.0 feet				▲ 53	Cobble in sample at 40.0 feet.
42.5		Exploration completed at a depth of 42.0 feet. Hammer efficiency factor is 87 percent.	-3.0 42.0				
45.0							
47.5							
50.0							
52.5							
55.0							
57.5							
60.0							



DRILLED BY: Hard Core Drilling
LOGGED BY: JGH/ZMR
COMPLETED: 03/09/17

BORING METHOD: hollow-stem auger and mud rotary (see document text)
BORING BIT DIAMETER: 8 inches/4 7/8 inches

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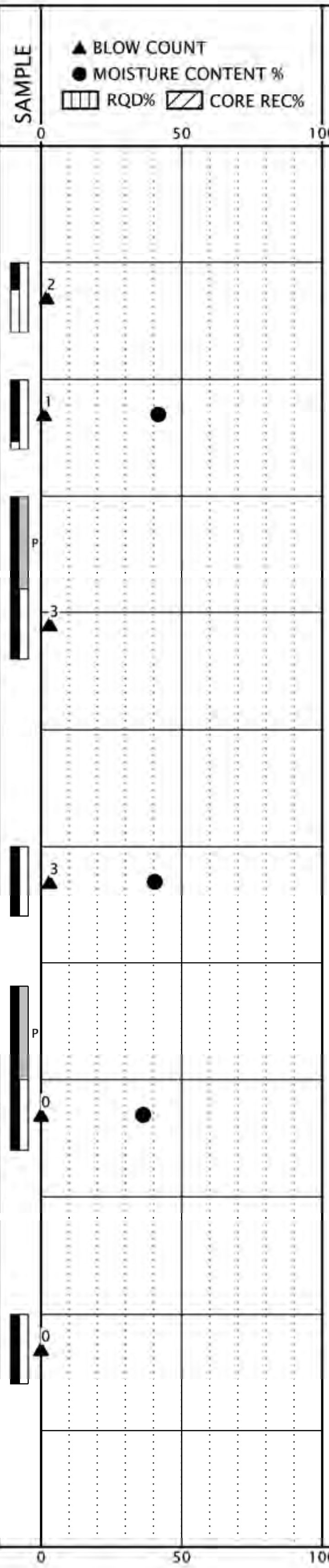
BORING B-3
(continued)

PROPOSED ABERNETHY PLACE PROJECT
OREGON CITY, OR

FIGURE A-3

DRAFT

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	INSTALLATION AND COMMENTS
0.0	Medium dense, gray GRAVEL (GP), trace sand and silt; moist - **FILL.**	41.0		
	Soft, brown SILT (ML), trace sand and organics (rootlets, carbonized wood); moist, sand is fine, organics are 1/4-inch diameter (alluvium).	40.0 1.0		
2.5			PP	PP = 0.0 tsf
5.0	very soft at 5.0 feet		PP	PP = 0.5 tsf
7.5				
10.0	soft, without organics at 9.5 feet		PP	PP = 0.5 tsf
12.5				
15.0			PP	PP = 0.0 tsf
17.5				
20.0	very soft, minor sand at 20.0 feet		ATT PP	PP = 0.0 tsf LL = 42% PL = 27%
22.5				
25.0	trace sand at 25.0 feet		PP	PP = 0.25 tsf
27.5				
30.0				



DRILLED BY: Hard Core Drilling

LOGGED BY: ZMR

COMPLETED: 03/09/17

BORING METHOD: mud rotary (see document text)

BORING BIT DIAMETER: 4 7/8 inches

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BORING B-4

PROPOSED ABERNETHY PLACE PROJECT
OREGON CITY, OR

FIGURE A-4

BORING LOG HILLARCH-2-02-B1_8.GPJ GEODESIGN.GDT PRINT DATE: 5/8/17:RC:KT

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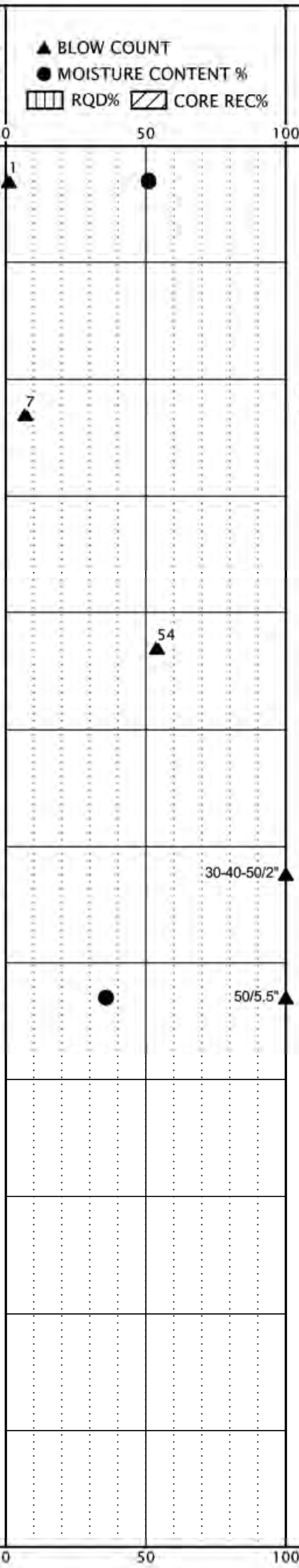
DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	INSTALLATION AND COMMENTS
30.0	Very soft, light brown-light gray SILT with sand (ML) to silty SAND (SM); wet, sand is fine.	11.0 30.0	PP	PP = 0.0 tsf
32.5				
35.0	Loose, brown SAND (SP), trace silt; wet.	6.0 35.0		Driller Comment: felt more stiff at 35.0 feet.
37.5				
40.0	medium dense at 40.0 feet			
	Very dense, gray-brown GRAVEL with sand (GP/GW), trace silt; wet.	0.5 40.5		
42.5				
45.0				
47.5				
	Exploration completed at a depth of 48.5 feet. Hammer efficiency factor is 87 percent.	-7.5 48.5		
50.0				
52.5				
55.0				
57.5				
60.0				



DRILLED BY: Hard Core Drilling

LOGGED BY: ZMR

COMPLETED: 03/09/17

BORING METHOD: mud rotary (see document text)

BORING BIT DIAMETER: 4 7/8 inches

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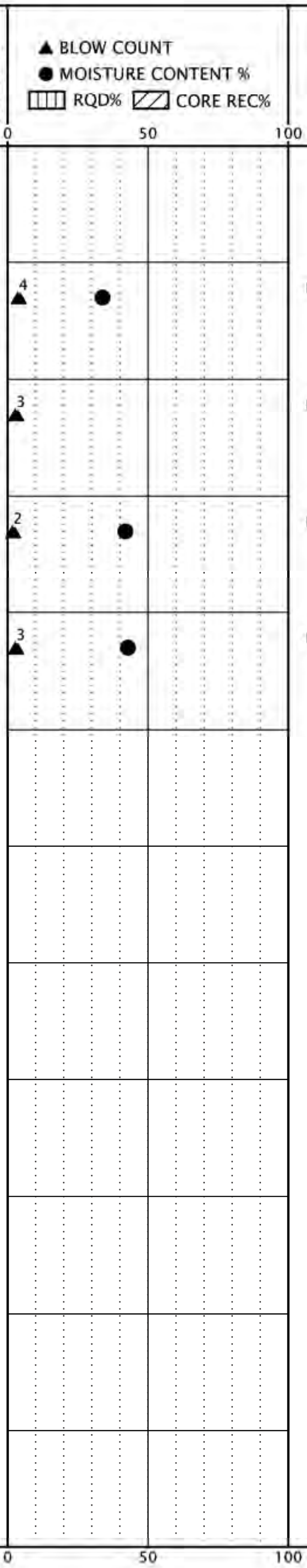
BORING B-4
(continued)

PROPOSED ABERNETHY PLACE PROJECT
OREGON CITY, OR

FIGURE A-4

DRAFT

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	INSTALLATION AND COMMENTS
0.0	ASPHALT CONCRETE (3.0 inches).	40.0		
	Medium dense, brown-gray GRAVEL (GP/GM), trace sand and silt; moist, subrounded - **FILL**.	39.7 0.3		
2.5	Soft to medium stiff, light brown SILT (ML), trace sand and organics (rootlets); moist (alluvium).	37.5 2.5	PP	PP = 0.0 tsf
5.0	Soft, light brown SILT (ML) to silty SAND (SM), trace organics (carbonized wood); wet, sand is fine, organics are 1/8-inch diameter.	35.0 5.0	PP	PP = 0.5 tsf
7.5	Soft, light brown, silty SAND (SM); wet, fine.	32.5 7.5	PP	PP = 0.25 tsf
10.0	Soft, light gray with brown-orange mottled SILT (ML), trace sand and organics (carbonized wood); wet, sand is fine, organics are 1/4-inch diameter.	30.0 10.0	PP	PP = 0.5 tsf
	Exploration completed at a depth of 11.5 feet. Hammer efficiency factor is 87 percent.	28.5 11.5		
12.5				
15.0				
17.5				
20.0				
22.5				
25.0				
27.5				
30.0				



DRILLED BY: Hard Core Drilling

LOGGED BY: ZMR

COMPLETED: 03/10/17

BORING METHOD: mud rotary (see document text)

BORING BIT DIAMETER: 4 7/8 inches

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BORING B-5

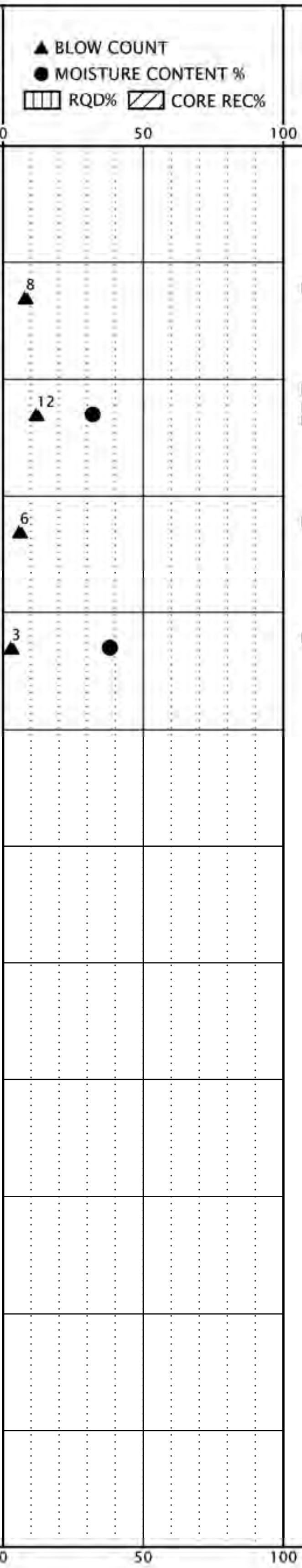
PROPOSED ABERNETHY PLACE PROJECT
OREGON CITY, OR

FIGURE A-5

BORING LOG HILLARCH-2-02-B1_8.GPJ GEODESIGN.GDT PRINT DATE: 5/8/17:RC:KT

DRAFT

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	INSTALLATION AND COMMENTS
0.0	ASPHALT CONCRETE (3.0 inches).	43.0 42.7 0.3		
	Medium dense, gray GRAVEL with sand and silt (GP-GW); moist, subrounded - **FILL**.			
2.5	Loose, light brown with orange mottled, silty SAND (SM); moist, fine (alluvium).	40.0 3.0	PP	PP = 0.5 tsf
5.0	medium dense, trace organics (carbonized wood); organics are 1/4-inch diameter at 5.0 feet		PP	Infiltration test: ~0.0 inches per hour at 5.0 feet. PP = 0.5 tsf
7.5	loose, light brown at 7.5 feet		PP	PP = 0.0 tsf
10.0	Very loose, light brown, silty SAND (SM) to SILT with sand (ML); moist, sand is fine.	33.0 10.0	PP	PP = 0.0 tsf
	Exploration completed at a depth of 11.5 feet.	31.5 11.5		
12.5	Hammer efficiency factor is 87 percent.			
15.0				
17.5				
20.0				
22.5				
25.0				
27.5				
30.0				



DRILLED BY: Hard Core Drilling

LOGGED BY: ZMR

COMPLETED: 03/10/17

BORING METHOD: hollow-stem auger and mud rotary (see document text)

BORING BIT DIAMETER: 8 inches/4 7/8 inches

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BORING B-6

PROPOSED ABERNETHY PLACE PROJECT
OREGON CITY, OR

FIGURE A-6

BORING LOG HILLARCH-2-02-B1_8.GPJ GEODESIGN.GDT PRINT DATE: 5/8/17:RC:KT

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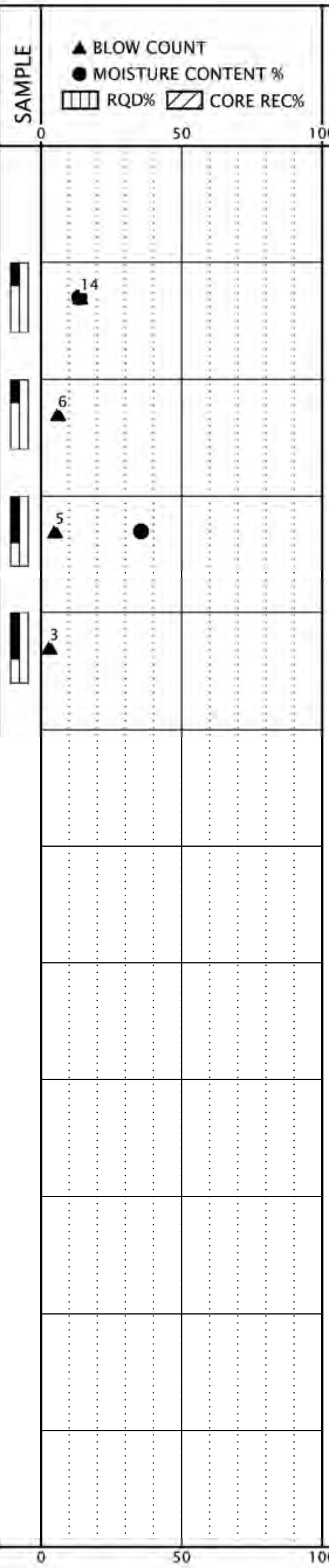
MATERIAL DESCRIPTION	ELEVATION DEPTH
	41.0
ASPHALT CONCRETE (3.0 inches).	40.7 0.3
Medium dense, gray-brown GRAVEL with sand and silt (GP-GW); moist, subrounded - **FILL**.	36.5 4.5
Loose, dark gray-brown, silty SAND with organics (wood chunks) (SM); wet, organics are ~2-inch diameter - **FILL**.	33.5 7.5
Medium stiff, dark gray, sandy SILT (ML), trace organics (wood chunks); moist, sand is fine, organics are 1/4-inch diameter - **FILL**.	31.0 10.0
Soft, dark gray, sandy SILT to SILT with sand (ML); moist, sand is fine - **FILL**.	29.5 11.5
Exploration completed at a depth of 11.5 feet.	
Hammer efficiency factor is 87 percent.	

DEPTH FEET: 0.0, 2.5, 5.0, 7.5, 10.0, 12.5, 15.0, 17.5, 20.0, 22.5, 25.0, 27.5, 30.0

GRAPHIC LOG | TESTING | SAMPLE | INSTALLATION AND COMMENTS



DRILLED BY: Hard Core Drilling

LOGGED BY: ZMR

COMPLETED: 03/10/17

BORING METHOD: mud rotary (see document text)

BORING BIT DIAMETER: 4 7/8 inches

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BORING B-7

PROPOSED ABERNETHY PLACE PROJECT
OREGON CITY, OR

FIGURE A-7

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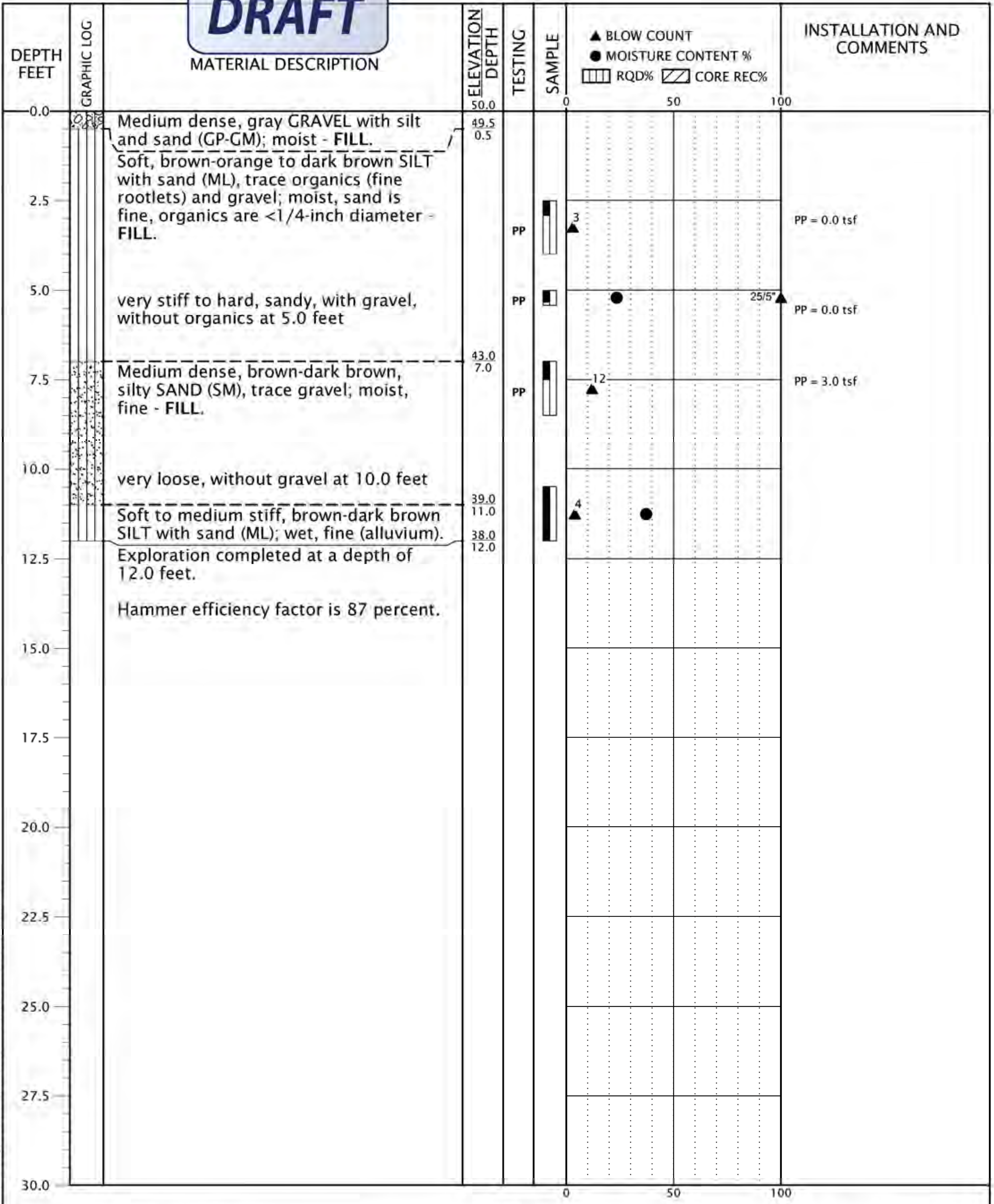

Medium dense, gray GRAVEL with silt and sand (GP-GM); moist - **FILL**.

Soft, brown-orange to dark brown SILT with sand (ML), trace organics (fine rootlets) and gravel; moist, sand is fine, organics are <1/4-inch diameter - **FILL**.

very stiff to hard, sandy, with gravel, without organics at 5.0 feet

Medium dense, brown-dark brown, silty SAND (SM), trace gravel; moist, fine - **FILL**.

very loose, without gravel at 10.0 feet

Soft to medium stiff, brown-dark brown SILT with sand (ML); wet, fine (alluvium).

Exploration completed at a depth of 12.0 feet.

Hammer efficiency factor is 87 percent.

BORING LOG HILLARCH-2-02-B1_8.GPJ GEODESIGN.GDT PRINT DATE: 5/8/17:RC:KT

DRILLED BY: Hard Core Drilling

LOGGED BY: ZMR

COMPLETED: 03/10/17

BORING METHOD: mud rotary (see document text)

BORING BIT DIAMETER: 4 7/8 inches

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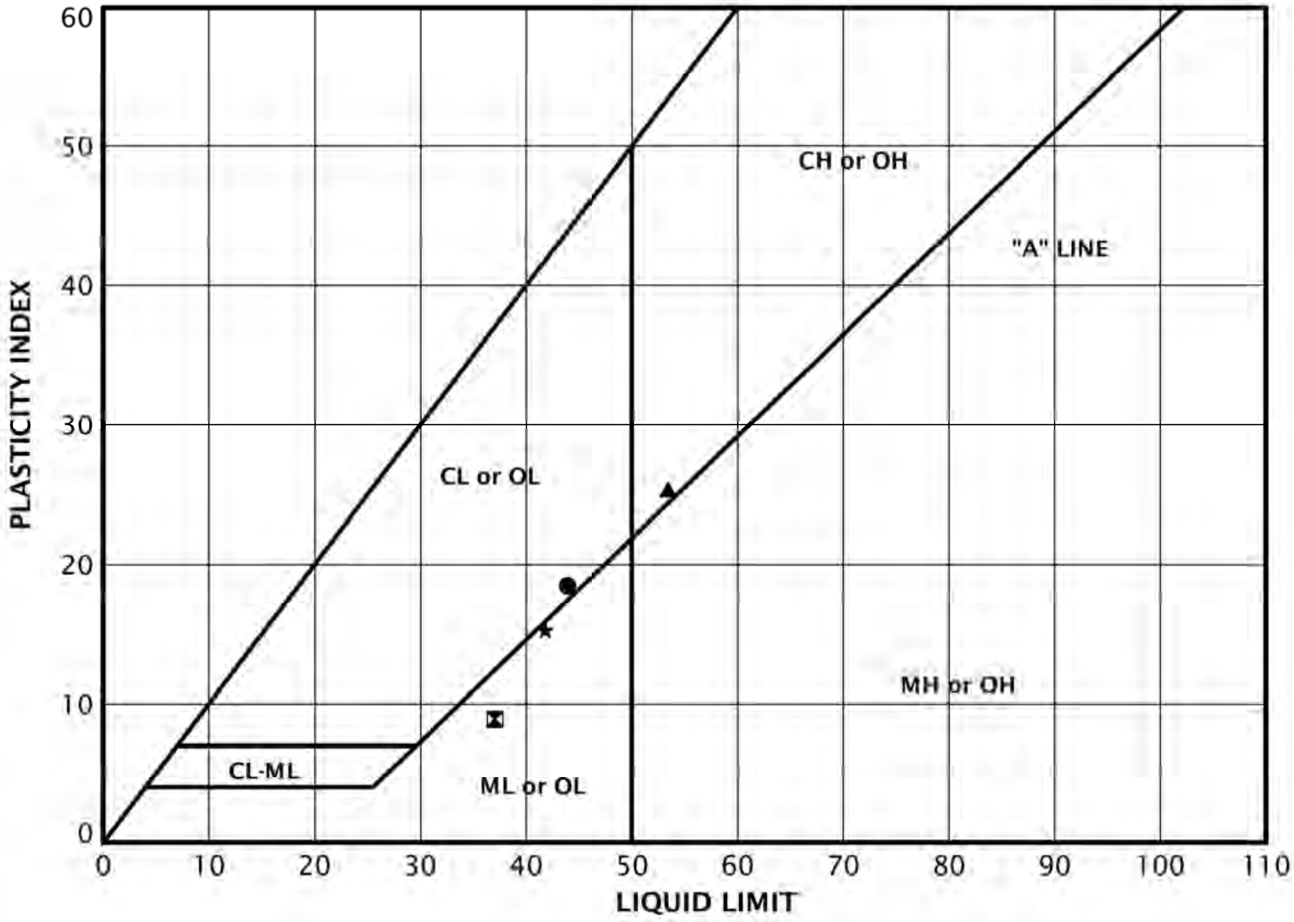
BORING B-8

PROPOSED ABERNETHY PLACE PROJECT
OREGON CITY, OR

FIGURE A-8



KEY	EXPLORATION NUMBER	SAMPLE DEPTH (FEET)	MOISTURE CONTENT (PERCENT)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
●	B-1	25.0	40	44	25	19
⧗	B-2	30.0	41	37	28	9
▲	B-3	12.0	42	53	28	25
★	B-4	20.0	36	42	27	15

ATTERBERG_LIMITS 7 HILLARCH-2-02-B1_8.GPJ GEODESIGN.GDT PRINT DATE 5/8/17:KT

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ATTERBERG LIMITS TEST RESULTS

PROPOSED ABERNETHY PLACE PROJECT
OREGON CITY, OR

FIGURE A-9

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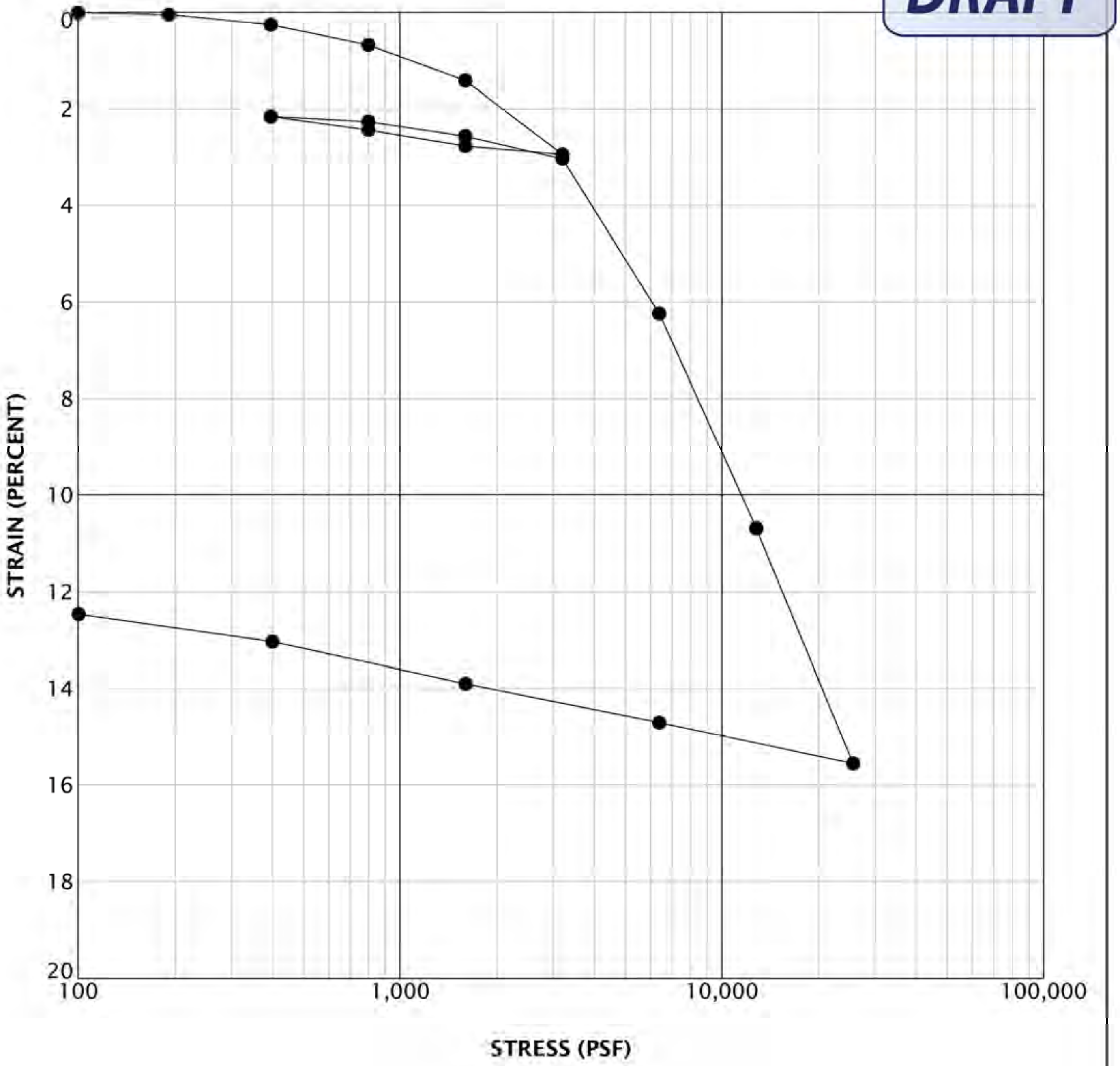

KEY	EXPLORATION NUMBER	SAMPLE DEPTH (FEET)	MOISTURE CONTENT (PERCENT)	DRY DENSITY (PCF)
●	B-1	13.0	40	79

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CONSOLIDATION TEST RESULTS

PROPOSED ABERNETHY PLACE PROJECT
OREGON CITY, OR

FIGURE A-10

CONSOL_STRAIN_100K HILLARCH-2-02-B1_8.GPJ GEODESIGN.GDT PRINT DATE: 5/8/17:KT

DRAFT

SAMPLE INFORMATION			MOISTURE CONTENT (PERCENT)	DRY DENSITY (PCF)	SIEVE			ATTERBERG LIMITS		
EXPLORATION NUMBER	SAMPLE DEPTH (FEET)	ELEVATION (FEET)			GRAVEL (PERCENT)	SAND (PERCENT)	P200 (PERCENT)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
B-1	2.5	43.5	9							
B-1	7.5	38.5	38				27			
B-1	13.0	33.0	40	79						
B-1	20.0	26.0	40							
B-1	25.0	21.0	40					44	25	19
B-1	30.0	16.0	45							
B-1	40.0	6.0	40				39			
B-2	5.0	47.0	28							
B-2	15.0	37.0	43							
B-2	18.5	33.5	35				33			
B-2	30.0	22.0	41					37	28	9
B-2	35.0	17.0	40				55			
B-2	50.0	2.0	48							
B-3	7.5	31.5	40							
B-3	12.0	27.0	42					53	28	25
B-3	20.0	19.0	43							
B-3	35.0	4.0	10							
B-4	5.0	36.0	42							
B-4	15.0	26.0	40							
B-4	20.0	21.0	36					42	27	15
B-4	30.0	11.0	51							
B-4	48.0	-7.0	36							
B-5	2.5	37.5	34							
B-5	7.5	32.5	42							
B-5	10.0	30.0	43							
B-6	5.0	38.0	32							
B-6	10.0	33.0	38							

LAB SUMMARY HILLARCH-2-02-B1_8.GPJ GEODESIGN.GDT PRINT DATE: 5/8/17:KT

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SUMMARY OF LABORATORY DATA

PROPOSED ABERNETHY PLACE PROJECT
OREGON CITY, OR

FIGURE A-11

SAMPLE INFORMATION			MOISTURE CONTENT (PERCENT)	DRY DENSITY (PCF)	SIEVE			ATTERBERG LIMITS		
EXPLORATION NUMBER	SAMPLE DEPTH (FEET)	ELEVATION (FEET)			GRAVEL (PERCENT)	SAND (PERCENT)	P200 (PERCENT)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
B-7	2.5	38.5	14							
B-7	7.5	33.5	36							
B-8	5.0	45.0	23							
B-8	10.5	39.5	37							

SUMMARY OF LABORATORY DATA
(continued)

PROPOSED ABERNETHY PLACE PROJECT
OREGON CITY, OR

Robert Miner Dynamic Testing, Inc.
Dynamic Measurements and Analyses for Deep Foundations

January 26, 2017

Mr. Matt Van Bergen
Hard Core Drilling, Inc.
18755 SW Niederberger Rd
Dundee, Oregon

Re: Penetration Test Energy Measurements
CME-75, Rig No. 103, CME Auto Hammer
Bore Hole: BH2, December 29, 2016
Newberg Steel Yard, Newberg, Oregon

RMDT Job No. 17F04b

Dear Mr. Van Bergen,

This letter presents energy transfer measurements made during Standard Penetration Tests for the drill hole and drill rig referenced above. Robert Miner Dynamic Testing, Inc. (RMDT) made dynamic measurements with a Pile Driving Analyzer® as a hammer advanced the NW rod during sampling with a split spoon sampler.

The purpose of RMDT's testing was the measurement of energy transferred to the drill rods. Measurements were made on a section of NW gauge rod at the top of the drill rod. Strain gages and accelerometers on the rod were connected to a Pile Driving Analyzer® (PDA) which generally processed acceleration and strain measurements from each hammer blow and stored both the measurements and computed results. Measurements and data processing generally followed the ASTM D 4633-16 standard. Energy transfer past the gage location, EFV, was computed by the PDA using force and velocity records as follows:

$$EFV = \int_a^b F(t)\ v(t)\ dt$$

The value "a" corresponds to the start of the record which is when the energy transfer begins and *"b"* is the time at which energy transferred to the rod reaches a maximum value. Appendix A contains more information on our measurement equipment and methods of analysis. The EFV energy calculation is identical to the EMX energy result discussed in Appendix A. The EFV and EMX values apply to the sensor location near the top of the rod.

TEST DETAILS

On December 29, 2016, a boring was advanced at the yard of Newberg Steel in Newberg, Oregon. The drill rig used during sampling was a truck mounted CME-75 auger unit manufactured by Central Mine Equipment (CME) and referred to as Rig 103 (OR Licence No.

YEAA724) by the operator. The CME-75 unit drilled to five predetermined depth intervals ranging from 20 to 60 ft below ground surface. The 40 and 50 ft Samples were over driven by 0.5 ft for a total of 2 ft of sample advancement per measurement. Over driving of a sample was performed to increase the data population available for analysis. The rod used to advance the spoon at each sample depth had a diameter matching that of NW rod. The automatic hammer in use during our testing was manufactured by CME and appeared to use a chain drive powered by a hydraulic motor, with the ram and chain drive enclosed within an outer casing.

RESULTS

A summary of testing and monitoring results is given in Table 1. The tabulated results include the starting sample depth, the penetration resistance, the number of hammers blows in our data set, measured energy transfer, EFV, the computed transfer efficiency, ETR, and the hammer blow rate, BPM. Appendix B contains detailed numeric results for each individual test.

Energy measurements must be divided by the theoretical free fall energy of the hammer to obtain an efficiency. A 140 lb ram raised 30 inches above an impact surface has 350 lb-ft of potential energy. Thus, the transfer energy results for sampling with the 140 lb ram may be divided by 350 lb-ft to yield the ratio of the delivered energy to the nominal potential energy. This efficiency ratio, ETR, is given for each sample interval as a percent efficiency.

Table 1. Summary of Test Details and Results for the 140-lb ram and Split Spoon Sampler

Sample Name and Sample Depth	Penetration Resistance (Blow/Set)	Number of Blows in Data Set	Average Transfer Energy EFV (lb-ft)	Average Transfer Efficiency ETR (percent)	Average Hammer Blow Rate BPM (blow/min)
20 ft Sample	5/1.0 ft	5	306	87	56
30 ft Sample	3/1.0 ft	2	306	88	42
40 ft Sample*	14/1.5 ft	13	302	86	57
50 ft Sample*	14/1.5 ft	12	298	85	56
60 ft Sample	23/1.0 ft	23	305	87	57
Average for Split Spoon samples:			**303**	**87**	**54**

**Note: Over Drove Sample 0.5 ft. Analyzed 1.5 ft of N-Values.*

Five sample returns were monitored while the 140 lb ram and standard split spoon sampler were in use. The overall average ETR and hammer blow rate was 87 percent and 54 blows per minute, respectively.

It was a pleasure to assist you and to participate on this project with the staff of Hard Core Drilling. Please do not hesitate to contact us if you or other project participants have any questions about this report.

Sincerely,

REGISTERED PROFESSIONAL
ENGINEER
81975PE
Andrew Banas
OREGON
MAR 10, 2015
ANDREW J BANAS
EXPIRES: 12/31/ 17

Andrew Banas, P.E.

Robert Miner Dynamic Testing, Inc.

APPENDIX A
AN INTRODUCTION INTO DYNAMIC PILE TESTING METHODS

The following has been written by Goble Rausche Likins and Associates, Inc. and may only be copied with its written permission.

BACKGROUND

Modern procedures of design and construction control require verification of bearing capacity and integrity of deep foundations during preconstruction test programs and also production installation. Dynamic pile testing methods meet this need economically and reliably, and therefore, form an important part of a quality assurance program when deep foundations are executed. Several dynamic pile testing methods exist; they have different benefits and limitations and different requirements for proper execution.

The Case Method of dynamic pile testing, named after the Case Institute of Technology where it was developed between 1964 and 1975, requires that a substantial ram mass (such as that of a pile driving hammer) impacts the pile top such that the pile undergoes at least a small permanent set. The method is therefore also referred to as a "High Strain Method". The Case Method requires dynamic measurements on the pile or shaft under the ram impact and then an evaluation of various quantities based on closed form solutions of the wave equation, a partial differential equation describing the motion of a rod under the effect of an impact. Conveniently, measurements and analyses are done by a single piece of equipment: the Pile Driving Analyzer® (PDA). However, for bearing capacity evaluations an important additional method is CAPWAP® which performs a much more rigorous analysis of the dynamic records than the simpler Case Method.

A related analysis method is the "Wave Equation Analysis" which calculates a relationship between bearing capacity and pile stress and field blow count. The GRLWEAP™ program performs this analysis and provides a complete set of helpful information and input data.

The following description deals primarily with the Case Method or "High Strain Test" Method of pile testing, however, for the sake of completeness, the "Low Strain Test" performed with the Pile Integrity Test™ (PIT), mainly for pile integrity evaluation, will also be described.

RESULTS FROM DYNAMIC TESTING

There are two main objectives of high strain dynamic pile testing:

- *Dynamic Pile Monitoring* and
- *Dynamic Load Testing*.

Dynamic pile monitoring is conducted during the installation of impact driven piles to achieve a safe and economical pile installation. Dynamic load testing, on the other hand, has as its primary goal the assessment of pile bearing capacity. It is applicable to both cast *insitu* piles or drilled shafts and impact driven piles during restrike.

Dynamic Pile Monitoring

During pile installation, the sensors attached to the pile measure pile top force and velocity. A PDA conditions and processes these signals and calculates or evaluates:

- Bearing capacity at the time of testing, including an assessment of shaft resistance development and driving resistance. This information supports formulation of a driving criterion.

- Dynamic pile stresses, axial and averaged over the pile cross section, both tensile and compressive, during pile driving to limit the potential of damage either near the pile top or along its length. Bending stresses can be evaluated at the point of sensor attachment.

- Pile integrity assessment by the PDA is based on the recognition of certain wave reflections from along the pile. If detected early enough, a pile may be saved from complete destruction. On the other hand, once damage is recognized measures can be taken to prevent reoccurrence.

- Hammer performance parameters including the energy transferred to the pile, the hammer speed in blows per minute and the stroke of open ended diesel hammers.

Dynamic Pile Load Testing

Bearing capacity testing of either driven piles or drilled shafts applies the same basic measurement approach of dynamic pile monitoring. However, the test is done independent of the pile installation process and therefore a pile driving hammer or other dynamic loading device may not be available. If a special ram has to be mobilized then its weight should be between 0.8 and 2% of the test load (e.g. between 4 and 10 tons for a 500 ton test load) to assure sufficient soil resistance activation.

For a successful test, it most important that the test is conducted after a sufficient waiting time following pile installation for soil properties approaching their long term condition or concrete to properly set. During testing, PDA results of pile/shaft stresses and transferred energy are used to maintain safe stresses and assure sufficient resistance activation. For safe and sufficient testing of drilled shafts, ram energies are often increased from blow to blow until the test capacity has been activated. On the other hand, restrike tests on driven piles may require a warm hammer so that the very first blow produces a complete resistance activation. Data must be evaluated by CAPWAP for bearing capacity.

After the dynamic load test has been conducted with sufficient energy and safe stresses, the CAPWAP analysis provides the following results:

- Bearing capacity i.e. the mobilized capacity present at the time of testing

- Resistance distribution including shaft resistance and end bearing components

- Stresses in pile or shaft calculated for both the static load application and the dynamic test. These stresses are averages over the cross section and do not include bending effects or nonuniform contact stresses, e.g. when the pile toe is on uneven rock.

- Shaft impedance vs depth; this is an estimate of the shaft shape if it differs substantially from the planned profile

- Dynamic soil parameters for shaft and toe, i.e. damping factors and quakes (related to the dynamic stiffness of the resistance at the pile/soil interface.)

MEASUREMENTS

PDA

The basis for the results calculated by the PDA are pile top strain and acceleration measurements which are converted to force and velocity records, respectively. The PDA conditions, calibrates and displays these signals and immediately computes average pile force and velocity thereby eliminating bending effects. Using closed form Case Method solutions, based on the one-dimensional linear wave equation, the PDA calculates the results described in the analytical solutions section below.

HPA

The ram velocity may be directly obtained using radar technology in the Hammer Performance Analyzer™. For this unit to be applicable, the ram must be visible. The impact velocity results can be automatically processed with a PC or recorded on a strip chart.

Saximeter™

For open end diesel hammers, the time between two impacts indicates the magnitude of the ram fall height or stroke. This information is not only measured and calculated by the PDA but also by the convenient, hand-held Saximeter.

PIT

The Pile Integrity Tester™ (PIT) can be used to evaluate defects in concrete piles or shafts which may have occurred during driving or casting. Also timber piles of limited length can be tested in that manner. This so-called "Low Strain Method" or “Pulse-Echo Method” of integrity testing requires only the measurement of acceleration at the pile top. The stress wave producing impact is then generated by a small hand-held hammer and the records interpreted in the time domain. PIT also supports the so-called “Transient Response Method” which requires the additional measurement of the hammer force and an analysis in the frequency domain. This method may also be used to evaluate the unknown length of deep foundations under existing structures.

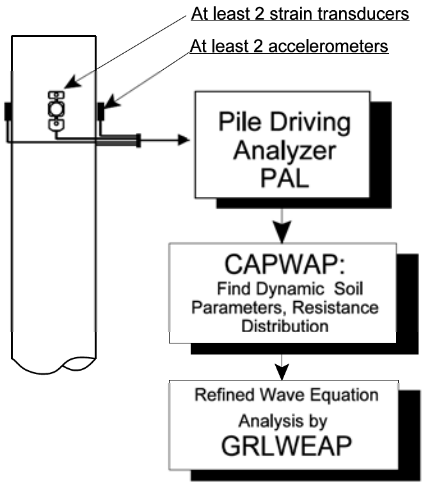
ANALYTICAL SOLUTIONS

BEARING CAPACITY

Wave Equation

GRL has written the GRLWEAP™ program which calculates a relationship between bearing capacity, pile stress and blow count. This relationship is often called the “bearing graph.” Once the blow count is known from pile installation logs, the bearing graph yields the bearing capacity. This approach requires no measurements and therefore can be performed during the design stage of a project, for example for the selection of hammer, cushion and pile size.

After dynamic pile monitoring and/or dynamic load testing has been performed, the “Refined Wave Equation Analysis” or RWEA (see schematic below) is often performed by inputting the PDA and CAPWAP calculated parameters. Then the bearing graph from the RWEA is the basis for a safe and sufficient driving criteria.



Case Method

The Case Method is a closed form solution based on a few simplifying assumptions such as ideal plastic soil behavior and an ideally elastic and uniform pile. Given the measured pile top force F(t) and pile top velocity v(t), the total soil resistance is

$$R(t) = ½\{[F(t) + F(t_2)] + Z[v(t) - v(t_2)]\} \qquad (1)$$

where

t = a point in time after impact
t_2 = time $t + 2L/c$
L = pile length below gages
c = $(E/\rho)^{½}$ is the speed of the stress wave
ρ = pile mass density
Z = EA/c is the pile impedance
E = elastic modulus of the pile ($\rho\ c^2$)
A = pile cross sectional area

The total soil resistance consists of a dynamic (R_d) and a static (R_s) component. The static component is therefore

$$R_s(t) = R(t) - R_d(t) \qquad (2)$$

The dynamic component may be computed from a soil damping factor, J, and a pile toe velocity, $v_t(t)$ which is conveniently calculated for the pile toe. Using wave considerations, this approach leads immediately to the dynamic resistance

$$R_d(t) = J[F(t) + Zv(t) - R(t)] \qquad (3)$$

and finally to the static resistance by means of Equation 2.

There are a number of ways in which Eq. 1 through 3 can be evaluated. Most commonly, t_2 is set to that time at which the static resistance becomes maximum. The result is the so-called **RMX** capacity. Damping factors for RMX typically range between 0.5 for coarse grained materials to 1.0 for clays. The **RSP** capacity (this method is most commonly referred to in the literature, yet it is not very frequently used) requires damping factors between 0.1 for sand and 1.0 for clay. Another capacity, **RA2**, determines the capacity at a time when the pile is essentially at rest and thus damping is small; RA2 therefore requires no damping parameter. In any event, the proper Case Method and its associated damping parameter is most conveniently found after a CAPWAP analysis has been performed.

The static resistance calculated by Case Method or CAPWAP is the mobilized resistance at the time of testing. Consideration therefore has to be given to soil setup or relaxation effects and whether or not a sufficient set has been achieved under the test loading that would correspond to a full activation of the ultimate soil resistance.

The PDA also calculates an estimate of shaft resistance as the difference between force and velocity times impedance at the time immediately prior to the return of the stress wave from the pile toe. This shaft resistance is not reduced by damping effects and is therefore called the total shaft resistance **SFT**. A correction for damping effects produces the static shaft resistance estimate, **SFR**.

The Case Method solution is simple enough to be evaluated "in real time," i.e. between hammer blows, using the PDA. It is therefore possible to calculate all relevant results for all hammer blows and plot these results as a function of depth or blow number. This is done in the PDAPLOT program.

CAPWAP

The CAse Pile Wave Analysis Program combines the wave equation pile and soil model with the Case Method measurements. Thus, the solution includes not only the total and static bearing capacity values but also the shaft resistance, end bearing, damping factors and soil stiffnesses. The method iteratively calculates a number of unknowns by signal matching. While it is necessary to make hammer performance assumptions for a GRLWEAP analysis, the CAPWAP program works with the pile top measurements. Furthermore, while GRLWEAP and Case Method require certain assumptions regarding the soil behavior, CAPWAP calculates these soil parameters.

STRESSES

During pile monitoring, it is important that compressive stress maxima at pile top and toe and tensile stress maxima somewhere along the pile be calculated for each hammer blow.

At the pile top (location of sensors) both the maximum compression stress, **CSX**, and the maximum stress from individual strain transducers, **CSI**, are directly obtained from the measurements. Note that CSI is greater than or equal to CSX and the difference between CSI and CSX is a measure of bending in the plane of the strain transducers. Note also that all stresses calculated for locations below the sensors are averaged over the pile cross section and therefore do not include components from either bending or eccentric soil resistance effects.

The PDA calculates the compressive stress at the pile bottom, **CSB**, assuming (a) a uniform pile and (b) that the pile toe force is the maximum value of the total resistance R(t) minus the total shaft resistance, SFT. Again, for this stress estimation uniform resistance force are assumed (e.g. not a sloping rock.)

For concrete piles, the maximum tension stress, **TSX**, is also of great importance. It occurs at some point below the pile top. The maximum tension stress can be computed from the pile top measurements by finding the maximum tension wave (either traveling upward, W_U, or downward, W_d) and reducing it by the minimum compressive wave traveling in opposite direction.

$$W_u = ½[F(t) - Zv(t)] \qquad (4)$$

$$W_d = ½[F(t) + Zv(t)] \qquad (5)$$

CAPWAP also calculates tensile and compressive stresses along the pile and, in general, more accurately than the PDA. In fact, for non-uniform piles or piles with joints, cracks or other discontinuities, the closed form solutions from the PDA may be in error.

PILE INTEGRITY

High Strain Tests (PDA)

Stress waves in a pile are reflected wherever the pile impedance, $Z = EA/c = \rho cA = A\sqrt{(E\ \rho)}$, changes. Therefore, the pile impedance is a measure of the quality of the pile material (E, ρ, c) and the size of its cross section (A). The reflected waves arrive at the pile top at a time which is greater the farther away from the pile top the reflection occurs. The magnitude of the change of the upward traveling wave (calculated from the measured force and velocity, Eq. 4) indicates the extent of the cross sectional change. Thus, with β_i (**BTA**) being a relative integrity factor which is unity for no impedance change and zero for the pile end, the following is calculated by the PDA.

$$\beta_i = (1 - \alpha_i)/(1 + \alpha_i) \quad (6)$$

with

$$\alpha_i = ½(W_{UR} - W_{UD})/(W_{Di} - W_{UR}) \quad (7)$$

where

W_{UR} is the upward traveling wave at the onset of the reflected wave. It is caused by resistance.

W_{UD} is the upwards traveling wave due to the damage reflection.

W_{Di} is the maximum downward traveling wave due to impact.

It can be shown that this formulation is quite accurate as long as individual reflections from different pile impedance changes have no overlapping effects on the stress wave reflections.

Without rigorous derivation, it has been proposed to consider as slight damage when β is above 0.8 and a serious damage when β is less than 0.6.

Low Strain Tests (PIT)

The pile top is struck with a held hand hammer and the resulting pile top velocity is measured, displayed and interpreted for signs of wave reflections. In general, a comparison of the reflected acceleration leads to a relative measure of extent of damage, again the location of the problem is indicated by the arrival time of the reflection. PIT records can also be interpreted by the β-Method. However, low strain tests do not activate much resistance which simplifies Eq. 7 since W_{UR} is then equal to zero.

For drilled shafts and PIT records that clearly show a toe reflection, an approximate shaft profile can be calculated from low strain records using the PITSTOP program's PROFILE routine.

HAMMER PERFORMANCE

The PDA calculates the energy transferred to the pile top from:

$$E(t) = {}_o\int^t F(t)v(t)\, dt \quad (8a)$$

The maximum of the E(t) curve is the most important information for an overall evaluation of the performance of a hammer and driving system. This **EMX** value allows for a classification of the hammer's performance when presented as the rated transfer efficiency, also called energy transfer ratio (**ETR**) or global efficiency

$$e_T = EMX/E_R \quad (8b)$$

where

E_R is the manufacturer's rated energy value.

Both Saximeter and PDA calculate the stroke (**STK**) of an open end diesel hammer using

$$STK = (g/8)\, T_B^{\,2} - h_L \quad (9)$$

where

g is the earth's gravitational acceleration,
T_B is the time between two hammer blows,
h_L is a stroke loss value due to gas compression and time losses during impact (usually 0.3 ft or 0.1 m).

DETERMINATION OF WAVE SPEED

An important facet of dynamic pile testing is an assessment of pile material properties. Since in general force is determined from strain by multiplication with elastic modulus, E, and cross sectional area, A, the dynamic elastic modulus has to be determined for pile materials other than steel. In general, the records measured by the PDA clearly indicate a pile toe reflection as long as pile penetration per blow is greater than 1 mm or .04 inches. The time between the onset of the force and velocity records at impact and the onset of the reflection from the toe (usually apparent by a local maximum of the wave up curve) is the so-called wave travel time, T. Dividing 2L (L is here the length of the pile below sensors) by T leads to the stress wave speed in the pile:

$$c = 2L/T \quad (10)$$

The elastic modulus of the pile material is related to the wave speed according to the linear elastic wave equation theory by

$$E = c^2\rho \qquad (11)$$

Since the mass density of the pile material, ρ, is usually well known (an exception is timber for which samples should be weighed), the elastic modulus is easily found from the wave speed. Note, however, that this is a dynamic modulus which is generally higher than the static one and that the wave speed depends to some degree on the strain level of the stress wave. For example, experience shows that the wave speed from PIT is roughly 5% higher than the wave speed observed during a high strain test.

Other Notes:

- If the pile material is nonuniform then the wave speed c, according to Eq. 10, is an average wave speed and does not necessarily reflect the pile material properties of the location where the strain sensors are attached to the pile top. For example, pile driving often causes fine tension cracks some distance below the top of concrete piles. Then the average c is slower than that at the pile top. It is therefore recommended to determine E in the beginning of pile driving and not adjust it when the average c changes.

- If the pile has such a high resistance that there is no clear indication of a toe reflection then the wave speed of the pile material must be determined either by assumption or by taking a sample of the concrete and measuring its wave speed in a simple free column test. Another possibility is to use the proportionality relationship, discussed under “DATA QUALITY CHECKS” to find c as the ratio between the measured velocity and measured strain.

DATA QUALITY CHECKS

Quality data is the first and foremost requirement for accurate dynamic testing results. It is therefore important that the measurement engineer performing PDA or PIT tests has the experience necessary to recognize measurement problems and take appropriate corrective action should problems develop. Fortunately, dynamic pile testing allows for certain data quality checks because two independent measurements are taken that have to conform to certain relationships.

Proportionality

As long as there is only a wave traveling in one direction, as is the case during impact when only a downward traveling wave exists in the pile, force and velocity measured at the pile top are proportional

$$F = v\, Z = v\, (EA/c) \qquad (12a)$$

This relationship can also be expressed in terms of stress

$$\sigma = v\, (E/c) \qquad (12b)$$

or strain

$$\epsilon = v / c \qquad (12c)$$

This means that the early portion of strain times wave speed must be equal to the velocity unless the proportionality is affected by high friction near the pile top or by a pile cross sectional change not far below the sensors. Checking the proportionality is an excellent means of assuring meaningful measurements.

Measurements are always taken at opposite sides of the pile as a means of calculating the average force and velocity in the pile. The velocity on the two sides of the pile is very similar even when high bending exists. Thus, an independent check of the velocity measurements is easy and simple.

Strain measurements may differ greatly between the two sides of the pile when bending exists. It is even possible that tension is measured on one side while very high compression exists on the other side of the pile. In extreme cases, bending might be so high that it leads to a nonlinear stress distribution. The averaging of the two strain signals does then not lead to the average pile force and proportionality will not be achieved.

When testing drilled shafts, measurements of strain may also be affected by local concrete quality variations. It is then often necessary to use four strain transducers spaced at 90 degrees around the pile for an improved strain data quality. The use of four transducers is also recommended for large pile diameters, particularly when it is difficult to mount the sensors at least two pile widths or diameters below the pile top.

LIMITATIONS, ADDITIONAL CONSIDERATIONS

Mobilization of capacity

Estimates of pile capacity from dynamic testing indicate the **mobilized pile capacity at the time of testing**. At very high blow counts (low set per blow), dynamic test methods tend to produce lower bound capacity estimates as not all resistance (particularly at and near the toe) is fully activated.

Time dependent soil resistance effects

Static pile capacity from dynamic method calculations provide an estimate of the axial pile capacity. Increases and decreases in the pile capacity with time typically occur (soil setup/relaxation). Therefore, **restrike testing usually yields a better indication of long term pile capacity than a test at the end of pile driving**. Often a wait period of one or two days between end of driving and restrike is satisfactory for a realistic prediction of pile capacity but this waiting time depends, among other factors, on the permeability of the soil.

(A) Soil setup

Because excess positive pore pressures often develop during pile driving in fine grained soil (clays, silts or even fine sands), the capacity of a pile at the time of driving may often be less than the long term pile capacity. These pore pressures reduce the effective stress acting on the pile thereby reducing the soil resistance to pile penetration, and thus the pile capacity at the time of driving. As these pore pressures dissipate, the soil resistance acting on the pile increases as does the axial pile capacity. This phenomena is routinely called soil setup or soil freeze.

(*B) Relaxation*

Relaxation (capacity reduction with time) has been observed for piles driven into weathered shale, and may take several days to fully develop. Pile capacity estimates based upon initial driving or short term restrike tests can significantly overpredict long term pile capacity. Therefore, piles driven into shale should be tested after a minimum one week wait either statically or dynamically (with particular emphasis than on the first few blows). Relaxation has also been observed for displacement piles driven into dense saturated silts or fine sands due to a negative pore pressure effect at the pile toe. Again, restrike tests should be used, with great emphasis on early blows.

Capacity results for open pile profiles

Larger diameter open ended pipe piles (or H-piles which do not bear on rock) may behave differently under dynamic and static loading conditions. Under dynamic loads the soil inside the pile or between its flanges may slip and produce internal friction while under static loads the plug may move with the pile, thereby creating end bearing over the full pile cross section. As a result both friction and end bearing components may be different under static and dynamic conditions.

CAPWAP Analysis Results

A portion of the soil resistance calculated on an individual soil segment in a CAPWAP analysis can usually be shifted up or down the shaft one soil segment without significantly altering the match quality. Therefore, use of the CAPWAP resistance distribution for uplift, downdrag, scour, or other geotechnical considerations should be made with an understanding of these analysis limitations.

Stresses

PDA and CAPWAP calculated stresses are average values over the cross section. Additional allowance has to be made for bending or non-uniform contact stresses. To prevent damage it is therefore important to maintain good hammer-pile alignment and to protect the pile toes using appropriate devices or an increased cross sectional area.

In the United States is has become generally acceptable to limit the dynamic installation stresses of driven piles to the following levels:

90%	of yield strength for steel piles
85%	of the concrete compressive strength - after subtraction of the effective prestress - for concrete piles in compression

100% of effective prestress plus ½ of the concrete's tension strength for prestressed piles in tension

70% of the reinforcement strength for regularly reinforced concrete piles in tension

300% of the static design allowable stress for timber

Note that the dynamic stresses may either be directly measured at the pile top by the PDA or calculated by the PDA for other locations along the pile based on the pile top measurements.

Additional design considerations

Numerous factors have to be considered in pile foundation design. Some of these considerations include

- additional pile loading from downdrag or negative skin friction,
- lateral and uplift loading requirements
- effective stress changes (due to changes in water table, excavations, fills or other changes in overburden),
- long term settlements in general and settlement from underlying weaker layers and/or pile group effects,

These factors have not been evaluated by GRL and have not been considered in the interpretation of the dynamic testing results. The foundation designer should determine if these or any other considerations are applicable to this project and the foundation design.

Wave equation analysis results

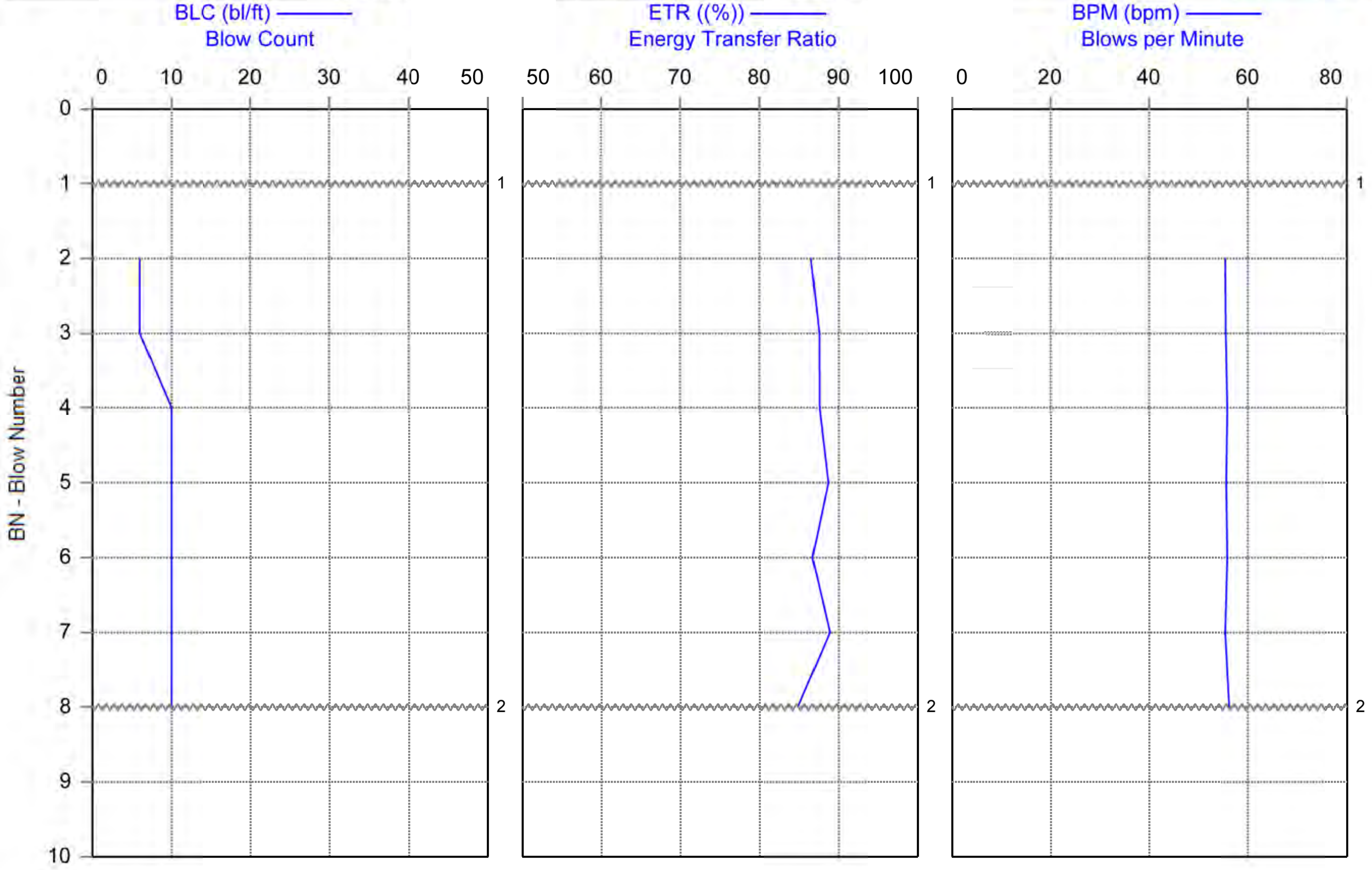
The results calculated by the wave equation analysis program depend on a variety of hammer, pile and soil input parameters. Although attempts have been made to base the analysis on the best available information, actual field conditions may vary and therefore stresses and blow counts may differ from the predictions reported. Capacity predictions derived from wave equation analyses should use restrike information. However, because of the uncertainties associated with restrike blow counts and restrike hammer energies, correlations of such results with static test capacities with have often displayed considerable scatter.

As for PDA and CAPWAP, the theory on which GRLWEAP is based is the one-dimensional wave equation. For that reason, stress predictions by the wave equation analysis can only be averages over the pile cross section. Thus, bending stresses or stress concentrations due to non-uniform impact or uneven soil or rock resistance are not considered in these results. Stress maxima calculated by the wave equation are usually subjected to the same limits as those measured directly or calculated from measurements by the PDA.

Appendix B

Summary of Case Method Results

Robert Miner Dynamic Testing, Inc. - PDIPLOT2 Ver 2016.1.56.1 - Case Method & iCAP® Results
Printed: 03-January-2017
Test started: 29-December-2016
HARDCORE, NWBRG STEEL, NO 103 - 20 FT SAMPLE



1 - Start of test on 12/29/2016 at 11:42 AM

2 - End of test on 12/29/2016 at 11:42 AM

Robert Miner Dynamic Testing, Inc.

Case Method & iCAP® Results

PDIPLOT2 2016.1.56.1 - Printed 03-January-2017

HARDCORE, NWBRG STEEL, NO 103 - 20 FT SAMPLE
OP: RMDT

RIG 103, CME-75, 140LB, NWJ
Date: 29-December-2016

AR:	1.44 in²	SP:	0.492 k/ft³
LE:	26.00 ft	EM:	30,000 ksi
WS:	16,807.9 f/s	JC:	0.00 []

CSX: Max Measured Compr. Stress
BPM: Blows per Minute
EFV: Energy of FV
ETR: Energy Transfer Ratio
FMX: Maximum Force
VMX: Maximum Velocity
RAT: SPT Length Ratio
EF2: Energy of F^2

BL#	Depth ft	BLC bl/ft	CSX ksi	BPM bpm	EFV k-ft	ETR (%)	FMX kips	VMX f/s	RAT []	EF2 k-ft
4	21.10	10	29.6	55.8	306.7	87.6	43	14.3	1.1	404.05
5	21.20	10	30.5	55.6	310.5	88.7	44	14.7	1.1	408.50
6	21.30	10	29.9	55.8	303.6	86.7	43	14.2	1.1	401.89
7	21.40	10	31.0	55.4	311.3	88.9	45	14.6	1.1	416.67
8	21.50	10	29.8	56.2	297.1	84.9	43	14.4	1.1	396.70
		Average	30.2	55.8	305.8	87.4	43	14.5	1.1	405.56
		Std. Dev.	0.5	0.3	5.2	1.5	1	0.2	0.0	6.73
		Maximum	31.0	56.2	311.3	88.9	45	14.7	1.1	416.67
		Minimum	29.6	55.4	297.1	84.9	43	14.2	1.1	396.70

Total number of blows analyzed: 5

BL#	Sensors
2-8	F1: [62NWJ-1] 216.9 (1.00); F2: [62NWJ-2] 217.3 (1.00); A1: [K2445] 307.0 (1.00); A2: [K847] 310.0 (1.00)

BL#	Comments
1	Start of test on 12/29/2016 at 11:42 AM
8	End of test on 12/29/2016 at 11:42 AM

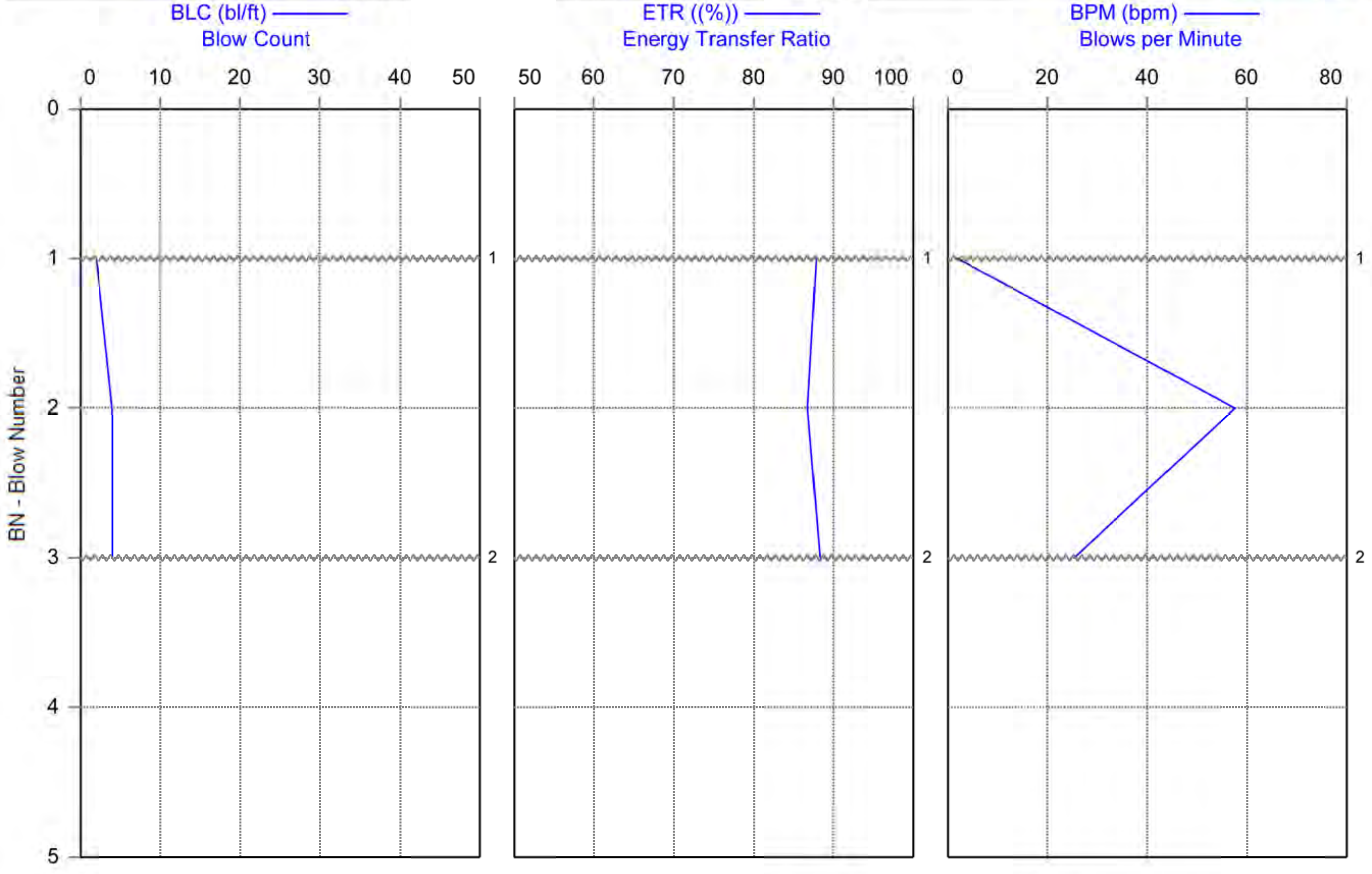
Time Summary

Drive 7 seconds 11:42 AM - 11:42 AM BN 1 - 8

Robert Miner Dynamic Testing, Inc. - PDIPLOT2 Ver 2016.1.56.1 - Case Method & iCAP® Results

Printed: 03-January-2017 Test started: 29-December-2016

HARDCORE, NWBRG STEEL, NO 103 - 30 FT SAMPLE



1 - Start of test on 12/29/2016 at 11:58 AM

2 - End of test on 12/29/2016 at 11:58 AM

HARDCORE, NWBRG STEEL, NO 103 - 30 FT SAMPLE RIG 103, CME-75, 140LB, NWJ
OP: RMDT Date: 29-December-2016

AR: 1.44 in^2
LE: 36.00 ft
WS: 16,807.9 f/s

SP: 0.492 k/ft^3
EM: 30,000 ksi
JC: 0.00 []

CSX: Max Measured Compr. Stress
BPM: Blows per Minute
EFV: Energy of FV
ETR: Energy Transfer Ratio

FMX: Maximum Force
VMX: Maximum Velocity
RAT: SPT Length Ratio
EF2: Energy of F^2

BL#	Depth	BLC	CSX	BPM	EFV	ETR	FMX	VMX	RAT	EF2
	ft	bl/ft	ksi	bpm	k-ft	(%)	kips	f/s	[]	k-ft
2	31.25	4	28.8	57.6	303.7	86.8	41	14.5	1.1	393.31
3	31.50	4	29.6	25.3	309.1	88.3	43	14.4	1.0	398.70
		Average	29.2	41.5	306.4	87.5	42	14.4	1.1	396.01
		Std. Dev.	0.4	16.2	2.7	0.8	1	0.1	0.0	2.69
		Maximum	29.6	57.6	309.1	88.3	43	14.5	1.1	398.70
		Minimum	28.8	25.3	303.7	86.8	41	14.4	1.0	393.31

Total number of blows analyzed: 2

BL# Sensors

1-3 F1: [62NWJ-1] 216.9 (1.00); F2: [62NWJ-2] 217.3 (1.00); A1: [K2445] 307.0 (1.00);
A2: [K847] 310.0 (1.00)

BL# Comments

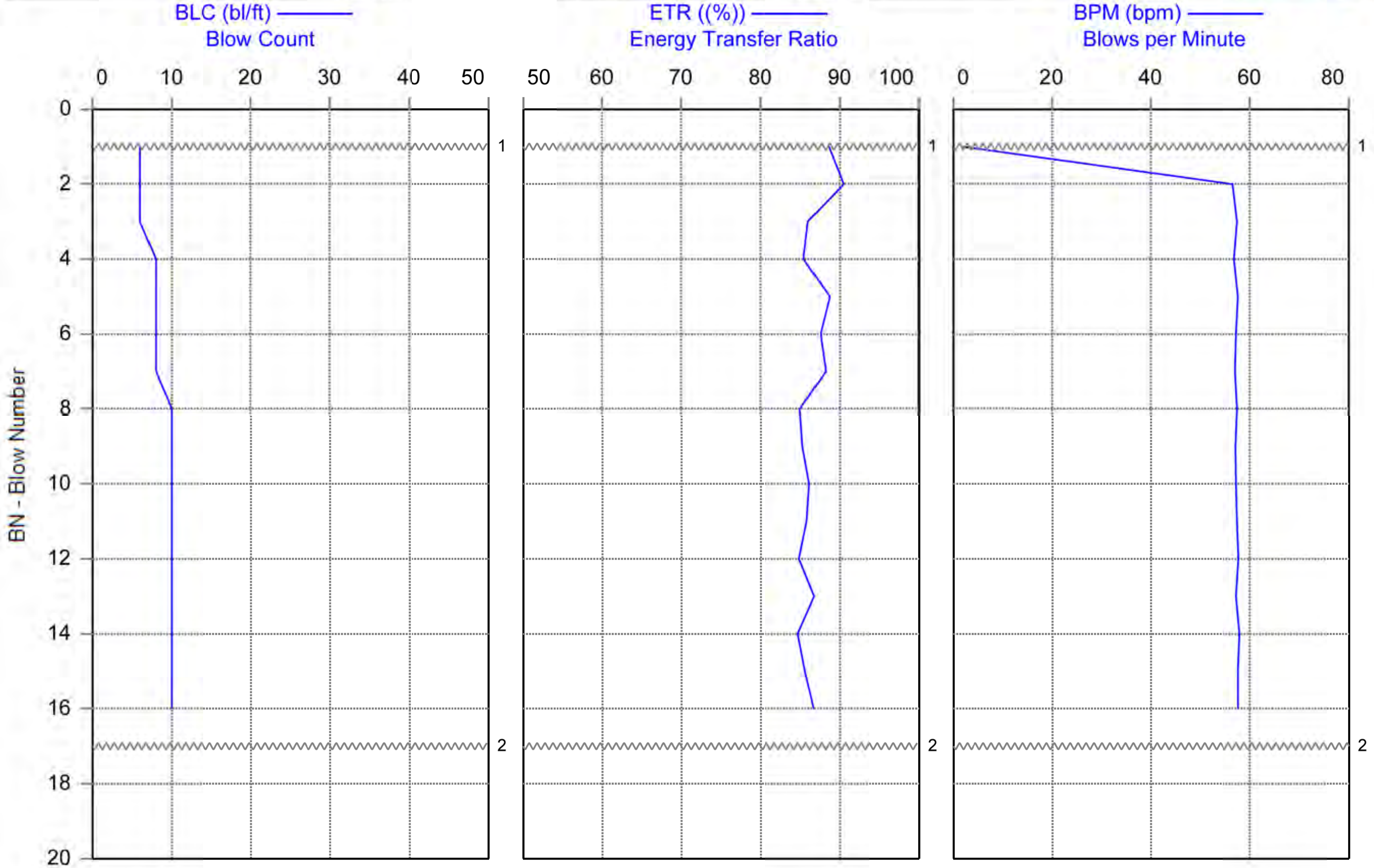
1 Start of test on 12/29/2016 at 11:58 AM
3 End of test on 12/29/2016 at 11:58 AM

Time Summary

Drive 3 seconds 11:58 AM - 11:58 AM BN 1 - 3

Robert Miner Dynamic Testing, Inc. - PDIPLOT2 Ver 2016.1.56.1 - Case Method & iCAP® Results
Printed: 23-January-2017
Test started: 29-December-2016

HARDCORE, NWBRG STEEL, NO 103 - 40 FT SAMPLE



1 - Start of test on 12/29/2016 at 12:14 PM

2 - End of test on 12/29/2016 at 12:15 PM

HARDCORE, NWBRG STEEL, NO 103 - 40 FT SAMPLE RIG 103, CME-75, 140LB, NWJ
OP: RMDT Date: 29-December-2016

AR: 1.44 in² SP: 0.492 k/ft³
LE: 46.00 ft EM: 30,000 ksi
WS: 16,807.9 f/s JC: 0.00 []

CSX: Max Measured Compr. Stress FMX: Maximum Force
BPM: Blows per Minute VMX: Maximum Velocity
EFV: Energy of FV RAT: SPT Length Ratio
ETR: Energy Transfer Ratio EF2: Energy of F^2

BL#	Depth ft	BLC bl/ft	CSX ksi	BPM bpm	EFV k-ft	ETR (%)	FMX kips	VMX f/s	RAT []	EF2 k-ft
4	40.63	8	29.4	56.8	299.1	85.5	42	13.7	1.1	413.00
5	40.75	8	30.6	57.6	310.7	88.8	44	15.0	0.7	420.53
6	40.88	8	30.7	57.2	306.6	87.6	44	14.9	0.7	421.48
7	41.00	8	31.0	57.0	309.0	88.3	45	14.4	0.7	426.29
8	41.10	10	29.8	57.4	297.5	85.0	43	13.7	1.1	428.43
9	41.20	10	30.0	57.1	298.6	85.3	43	13.6	1.1	431.58
10	41.30	10	30.1	57.2	301.7	86.2	43	13.9	0.7	422.40
11	41.40	10	29.7	57.4	300.5	85.9	43	13.3	1.1	430.27
12	41.50	10	29.7	57.7	297.1	84.9	43	13.5	1.1	419.07
13	41.60	10	29.5	57.2	303.9	86.8	42	13.4	1.1	425.69
14	41.70	10	30.4	57.9	296.6	84.7	44	14.0	0.7	408.54
15	41.80	10	30.0	57.6	299.8	85.7	43	14.0	0.7	405.11
16	41.90	10	30.4	57.6	303.8	86.8	44	14.1	0.7	410.99
		Average	30.1	57.4	301.9	86.3	43	14.0	0.9	420.26
		Std. Dev.	0.5	0.3	4.4	1.3	1	0.5	0.2	8.20
		Maximum	31.0	57.9	310.7	88.8	45	15.0	1.1	431.58
		Minimum	29.4	56.8	296.6	84.7	42	13.3	0.7	405.11

Total number of blows analyzed: 13

BL# Sensors

1-16 F1: [62NWJ-1] 216.9 (1.00); F2: [62NWJ-2] 217.3 (1.00); A1: [K2445] 307.0 (1.00);
A2: [K847] 310.0 (1.00)

BL# Comments

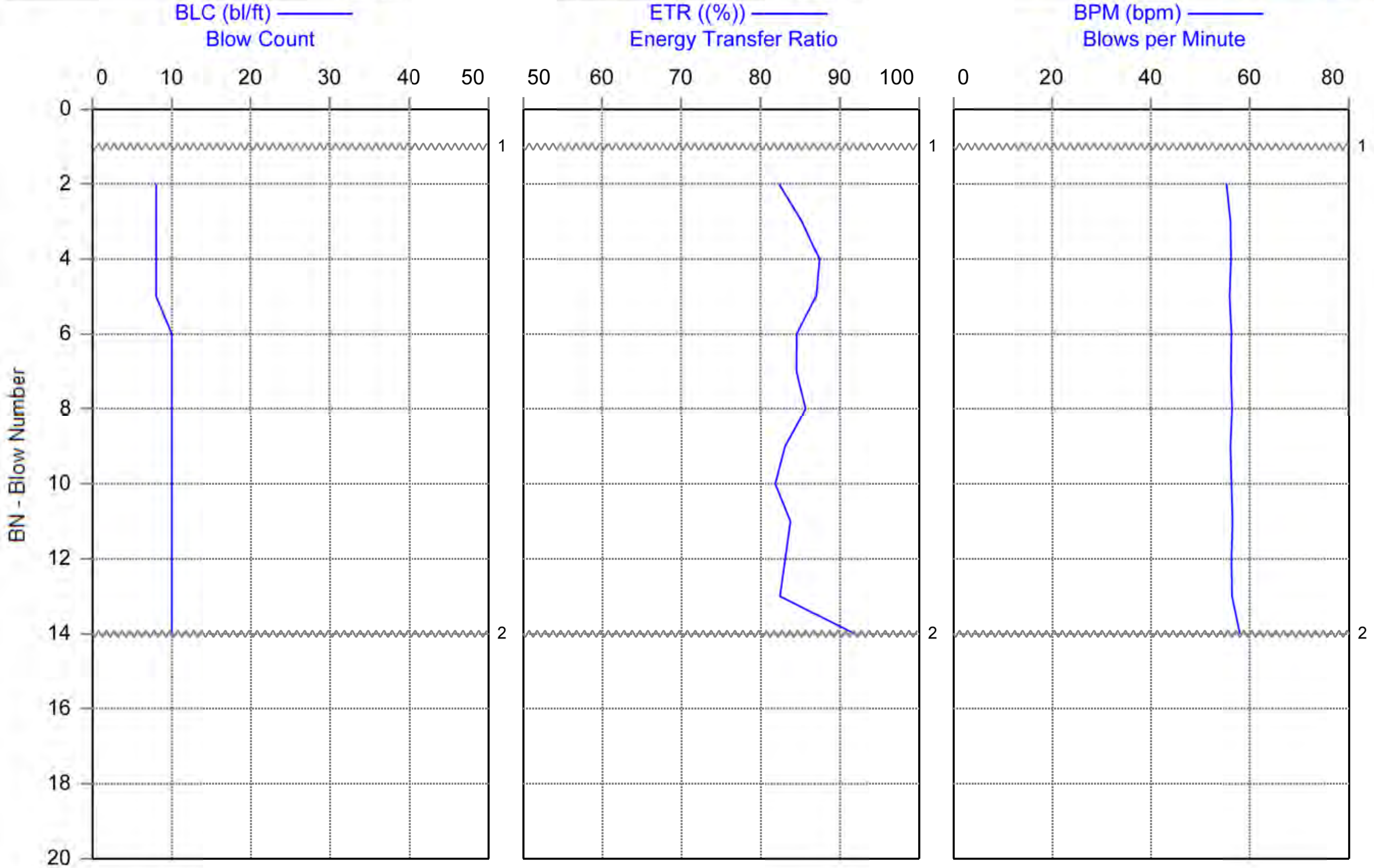
1 Start of test on 12/29/2016 at 12:14 PM
17 End of test on 12/29/2016 at 12:15 PM

Time Summary

Drive 16 seconds 12:14 PM - 12:15 PM BN 1 - 17

Robert Miner Dynamic Testing, Inc. - PDIPLOT2 Ver 2016.1.56.1 - Case Method & iCAP® Results
Printed: 03-January-2017
Test started: 29-December-2016

HARDCORE, NWBRG STEEL, NO 103 - 50 FT SAMPLE



1 - Start of test on 12/29/2016 at 12:37 PM

2 - End of test on 12/29/2016 at 12:38 PM

HARDCORE, NWBRG STEEL, NO 103 - 50 FT SAMPLE RIG 103, CME-75, 140LB, NWJ
OP: RMDT Date: 29-December-2016

AR: 1.44 in²
LE: 56.00 ft
WS: 16,807.9 f/s
SP: 0.492 k/ft³
EM: 30,000 ksi
JC: 0.00 []

CSX: Max Measured Compr. Stress
BPM: Blows per Minute
EFV: Energy of FV
ETR: Energy Transfer Ratio
FMX: Maximum Force
VMX: Maximum Velocity
RAT: SPT Length Ratio
EF2: Energy of F^2

BL#	Depth	BLC	CSX	BPM	EFV	ETR	FMX	VMX	RAT	EF2
	ft	bl/ft	ksi	bpm	k-ft	(%)	kips	f/s	[]	k-ft
3	50.75	8	29.5	56.1	298.5	85.3	42	15.7	0.5	401.39
4	50.88	8	30.0	56.2	306.1	87.5	43	16.6	0.5	402.44
5	51.00	8	29.9	55.9	304.6	87.0	43	16.9	0.5	401.97
6	51.10	10	30.9	56.3	296.2	84.6	44	16.0	0.5	410.18
7	51.20	10	30.6	56.2	296.0	84.6	44	16.4	0.5	409.39
8	51.30	10	30.5	56.4	300.1	85.8	44	15.9	0.5	409.99
9	51.40	10	30.2	56.1	291.0	83.1	43	16.0	0.5	404.81
10	51.50	10	30.7	56.3	286.7	81.9	44	15.9	0.5	401.46
11	51.60	10	30.3	56.5	293.4	83.8	44	15.6	0.5	396.79
12	51.70	10	29.8	56.3	291.1	83.2	43	16.5	0.5	394.38
13	51.80	10	29.7	56.4	288.8	82.5	43	15.5	0.5	390.58
14	51.90	10	28.8	58.0	321.6	91.9	41	17.2	0.5	378.35
		Average	30.1	56.4	297.9	85.1	43	16.2	0.5	400.15
		Std. Dev.	0.6	0.5	9.2	2.6	1	0.5	0.0	8.79
		Maximum	30.9	58.0	321.6	91.9	44	17.2	0.5	410.18
		Minimum	28.8	55.9	286.7	81.9	41	15.5	0.5	378.35

Total number of blows analyzed: 12

BL#	Sensors
2-14	F1: [62NWJ-1] 216.9 (1.00); F2: [62NWJ-2] 217.3 (1.00); A1: [K2445] 307.0 (1.00); A2: [K847] 310.0 (1.00)

BL#	Comments
1	Start of test on 12/29/2016 at 12:37 PM
14	End of test on 12/29/2016 at 12:38 PM

Time Summary

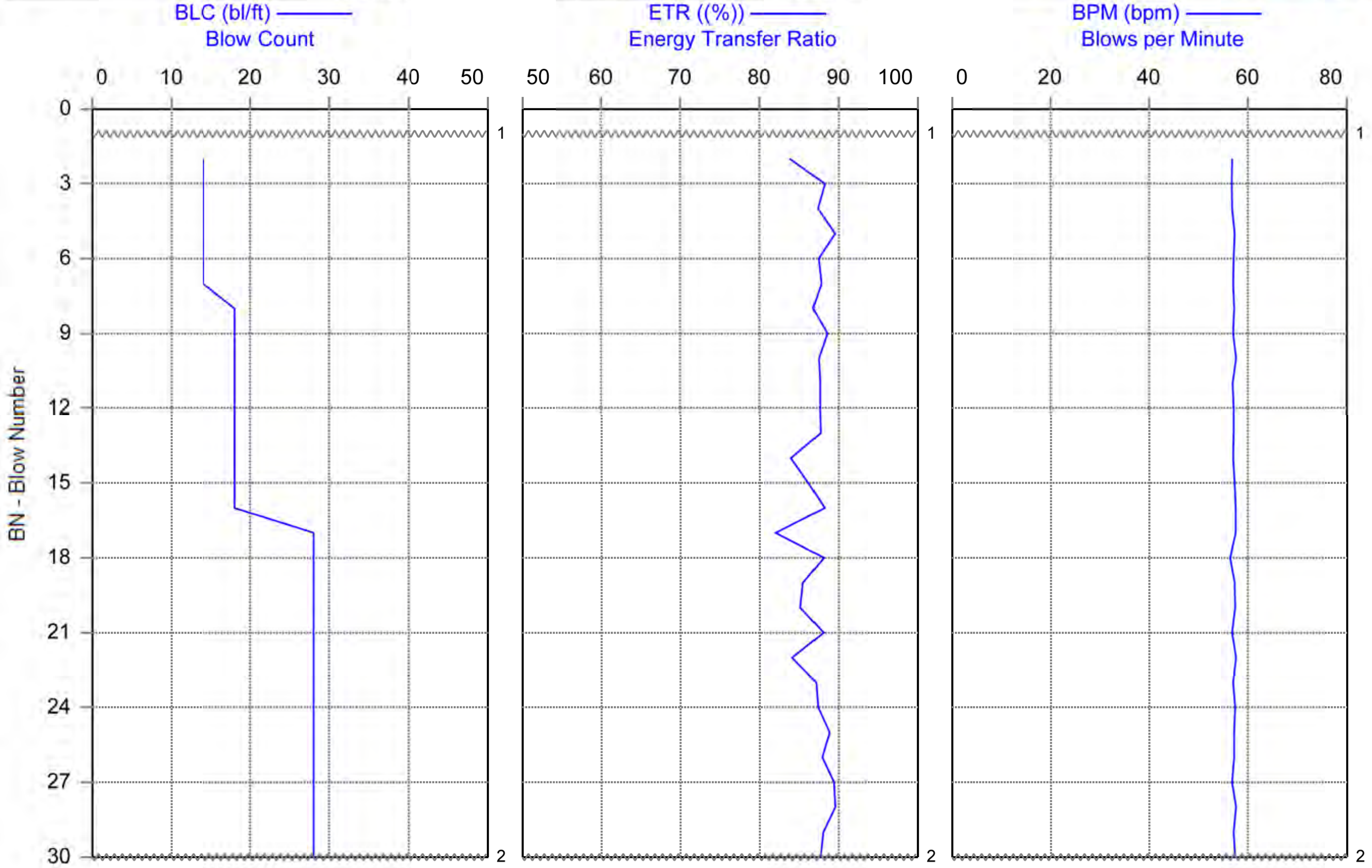
Drive 13 seconds 12:37 PM - 12:38 PM BN 1 - 14

Robert Miner Dynamic Testing, Inc. - PDIPLOT2 Ver 2016.1.56.1 - Case Method & iCAP® Results

Printed: 03-January-2017

Test started: 29-December-2016

HARDCORE, NWBRG STEEL, NO 103 - 60 FT SAMPLE



1 - Start of test on 12/29/2016 at 1:02 PM

2 - End of test on 12/29/2016 at 1:02 PM

HARDCORE, NWBRG STEEL, NO 103 - 60 FT SAMPLE
RIG 103, CME-75, 140LB, NWJ
OP: RMDT
Date: 29-December-2016

AR: 1.44 in²
LE: 66.00 ft
WS: 16,807.9 f/s
SP: 0.492 k/ft³
EM: 30,000 ksi
JC: 0.00 []

CSX: Max Measured Compr. Stress
BPM: Blows per Minute
EFV: Energy of FV
ETR: Energy Transfer Ratio
FMX: Maximum Force
VMX: Maximum Velocity
RAT: SPT Length Ratio
EF2: Energy of F^2

BL#	Depth	BLC	CSX	BPM	EFV	ETR	FMX	VMX	RAT	EF2
	ft	bl/ft	ksi	bpm	k-ft	(%)	kips	f/s	[]	k-ft
8	60.56	18	30.0	57.2	303.8	86.8	43	15.1	0.6	402.00
9	60.61	18	30.8	57.0	310.2	88.6	44	14.6	0.6	419.61
10	60.67	18	30.0	57.6	306.4	87.5	43	14.5	0.6	409.93
11	60.72	18	30.6	56.9	307.0	87.7	44	15.1	0.5	415.23
12	60.78	18	31.1	57.1	306.9	87.7	45	14.8	0.6	422.47
13	60.83	18	31.2	57.1	307.2	87.8	45	15.1	0.6	422.30
14	60.89	18	30.6	57.0	294.1	84.0	44	14.3	0.6	416.37
15	60.94	18	31.3	57.3	301.8	86.2	45	14.9	0.6	423.34
16	61.00	18	31.2	57.5	309.0	88.3	45	15.0	0.6	411.31
17	61.04	28	29.2	57.5	287.4	82.1	42	14.5	0.6	387.70
18	61.07	28	30.3	56.4	308.6	88.2	44	14.3	0.6	406.84
19	61.11	28	30.6	57.3	299.4	85.6	44	15.0	0.6	408.23
20	61.14	28	29.4	57.4	298.2	85.2	42	14.0	0.6	400.47
21	61.18	28	30.9	56.8	308.6	88.2	45	14.5	0.6	412.52
22	61.21	28	30.2	57.6	294.7	84.2	43	14.1	0.6	402.38
23	61.25	28	30.5	57.0	305.2	87.2	44	14.5	0.6	404.49
24	61.29	28	30.8	57.4	306.1	87.4	44	14.3	0.6	412.10
25	61.32	28	31.4	57.2	311.1	88.9	45	14.2	0.6	423.11
26	61.36	28	31.3	57.2	307.8	88.0	45	14.1	0.5	420.85
27	61.39	28	31.3	56.8	313.0	89.4	45	14.1	0.6	423.91
28	61.43	28	30.4	57.6	313.6	89.6	44	14.3	0.6	411.63
29	61.46	28	30.3	57.1	308.3	88.1	44	14.0	0.6	410.16
30	61.50	28	30.7	57.4	307.2	87.8	44	14.2	0.6	404.38
		Average	30.6	57.2	305.0	87.2	44	14.5	0.6	411.80
		Std. Dev.	0.6	0.3	6.3	1.8	1	0.4	0.0	8.98
		Maximum	31.4	57.6	313.6	89.6	45	15.1	0.6	423.91
		Minimum	29.2	56.4	287.4	82.1	42	14.0	0.5	387.70

Total number of blows analyzed: 23

BL#	Sensors
2-30	F1: [62NWJ-1] 216.9 (1.00); F2: [62NWJ-2] 217.3 (1.00); A1: [K2445] 307.0 (1.00); A2: [K847] 310.0 (1.00)

BL#	Comments
1	Start of test on 12/29/2016 at 1:02 PM
30	End of test on 12/29/2016 at 1:02 PM

Time Summary

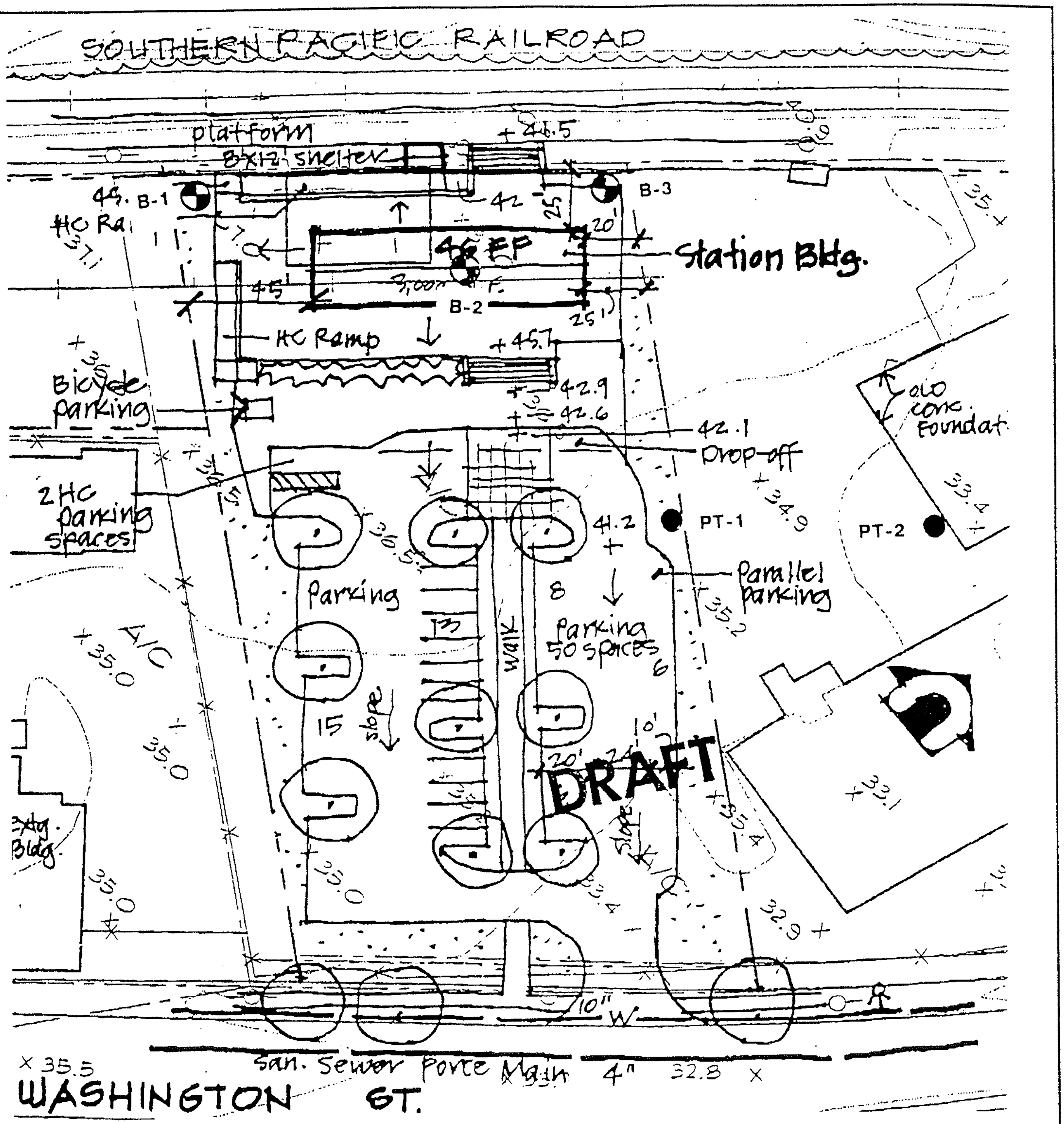
Drive 30 seconds 1:02 PM - 1:02 PM BN 1 - 30

APPENDIX B

APPENDIX B

PRIOR EXPLORATIONS AT ADJACENT SITE TO NORTH

We reviewed geotechnical information and laboratory data from the previous geotechnical study completed for the train station located to the north of the site. The site plan, relevant explorations logs, and applicable laboratory results from the report are presented in this appendix.



LEGEND

⊕	B-1	Location and designation of exploratory boring by CDM December 2000
●	PT-1	Location and designation of infiltration test by CDM December 2000

Reference: DEA Concetptual Site Plan Oct. 11, 2000

CDM

Site Exploration Plan

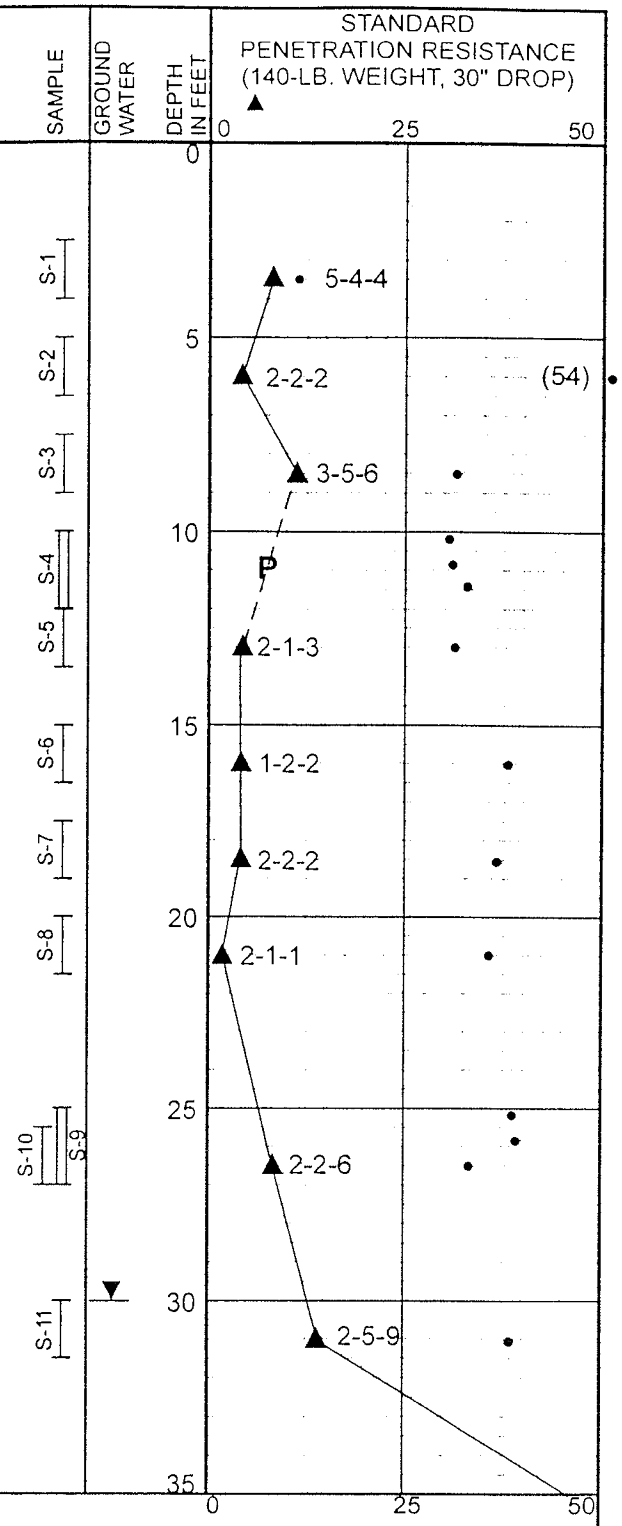
DEA/ OR City/ Amtrak Station
Oregon City, Oregon

FIGURE
2

DRAWING NAME	CLIENT NO	PROJECT NO	TASK	DRAWN	DATE	APPROVED
Site cdr	20136	31297		LJR	23 Jan 01	

GRAPHIC LOG	SOIL DESCRIPTION SURFACE ELEVATION = Approximately +35 ft MSL	
	Surface - Asphalt and GRAVEL Loose to medium-dense, brown, silty GRAVEL, GM, damp (FILL). Gravel >2" diameter.	4.5
	Soft to medium-stiff, gray, SILT with organics, MH, damp, medium to high plasticity (Original Topsoil?)	7.0
	Medium-stiff, gray, SILT, ML, damp, low plasticity. - grades sandy - trace clay-grades sandy - sand lens	30.0
	Medium-dense, brown to gray, silty SAND, SM, wet. 2" thick lens of rust colored SILT.	

DRAFT



2.0" O.D. SPLIT SPOON SAMPLE

3.25" O.D. SPLIT SPOON SAMPLE

3.0" O.D. THIN WALL SAMPLE

CORE RUN SAMPLE

P SAMPLE PUSHED

* SAMPLE NOT RECOVERED

ATTERBERG LIMITS

PLASTIC LIMIT — NATURAL WATER CONTENT — LIQUID LIMIT

IMPERVIOUS SEAL

▽ WATER LEVEL

OBSERVATION WELL TIP

• WATER CONTENT IN PERCENT

1. SOIL DESCRIPTIONS AND INTERFACES ARE INTERPRETIVE, ACTUAL CHANGES MAY BE GRADUAL.

2. WATER LEVEL IS FOR DATE SHOWN AND MAY VARY WITH TIME OF YEAR

CDM

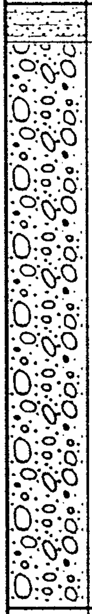
Log of Boring B-1

DEA/OR City/Amtrak Station
Oregon City, Oregon

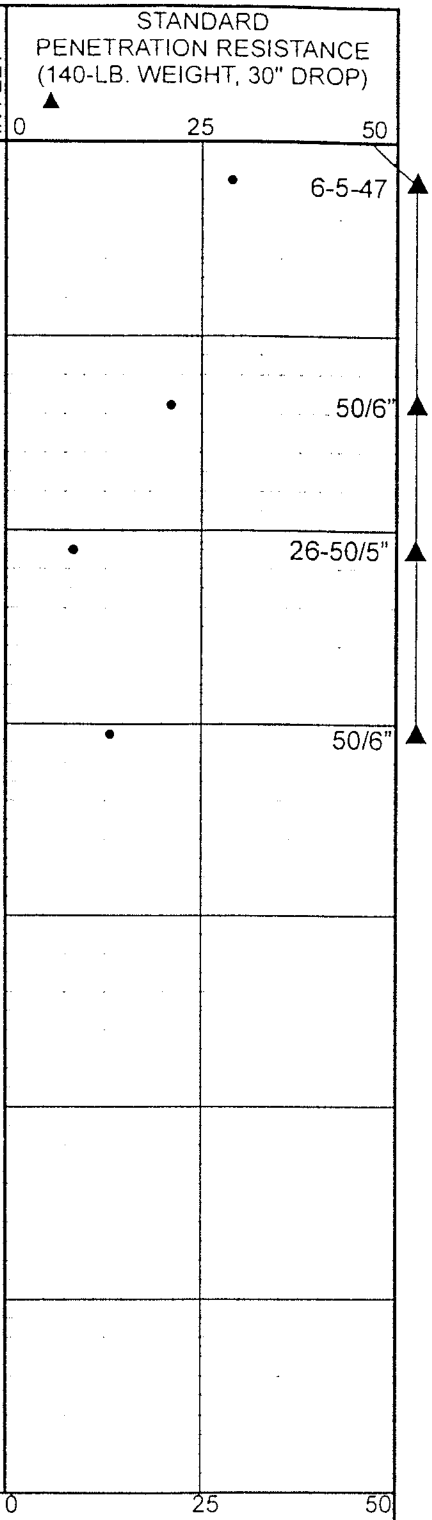
FIGURE
A-2a

DRAWING NAME	CLIENT NO	PROJECT NO	TASK	DRAWN	DATE	APPROVED
Boring.cdr	20136	31297	INV FLD	BBP	05 Jan 00	

SURFACE ELEVATION = Approximately +35 feet MSL



SOIL DESCRIPTION		SAMPLE	GROUND WATER	DEPTH IN FEET
Medium-dense, brown to gray, silty SAND, SM, wet.	36.0	S-12		35
Very dense, dark gray to brown, fine to coarse grained GRAVEL, GW, wet. Gravel ranges from 0.5" to >2" diameter.		S-13		40
		S-14		45
- Driller reports cobbles 6" to 8" in diameter.	50.5	S-15		50
Bottom of boring at 50.5 ft, completed 12/11/00 Groundwater at 30 feet bgs.				55



DRAFT

LAB DATA

Sample Number	Depth (ft)	γw (pcf)	γd (pcf)	wc (%)
S-4B	10.2 - 10.4	107.0	81.3	31.5
S-4A	10.5 - 11.1	104.9	79.2	32.4
S-9A	25.1 - 25.4	110.6	79.2	39.7

I 2.0" O.D. SPLIT SPOON SAMPLE
] 3.25" O.D. SPLIT SPOON SAMPLE
▯ 3.0" O.D. THIN WALL SAMPLE
▯ CORE RUN SAMPLE

P SAMPLE PUSHED
* SAMPLE NOT RECOVERED
ATTERBERG LIMITS
├─●─┤ LIQUID LIMIT, NATURAL WATER CONTENT, PLASTIC LIMIT

IMPERVIOUS SEAL
∇ WATER LEVEL
▌ OBSERVATION WELL TIP

• WATER CONTENT IN PERCENT

1. SOIL DESCRIPTIONS AND INTERFACES ARE INTERPRETIVE, ACTUAL CHANGES MAY BE GRADUAL.
2. WATER LEVEL IS FOR DATE SHOWN AND MAY VARY WITH TIME OF YEAR

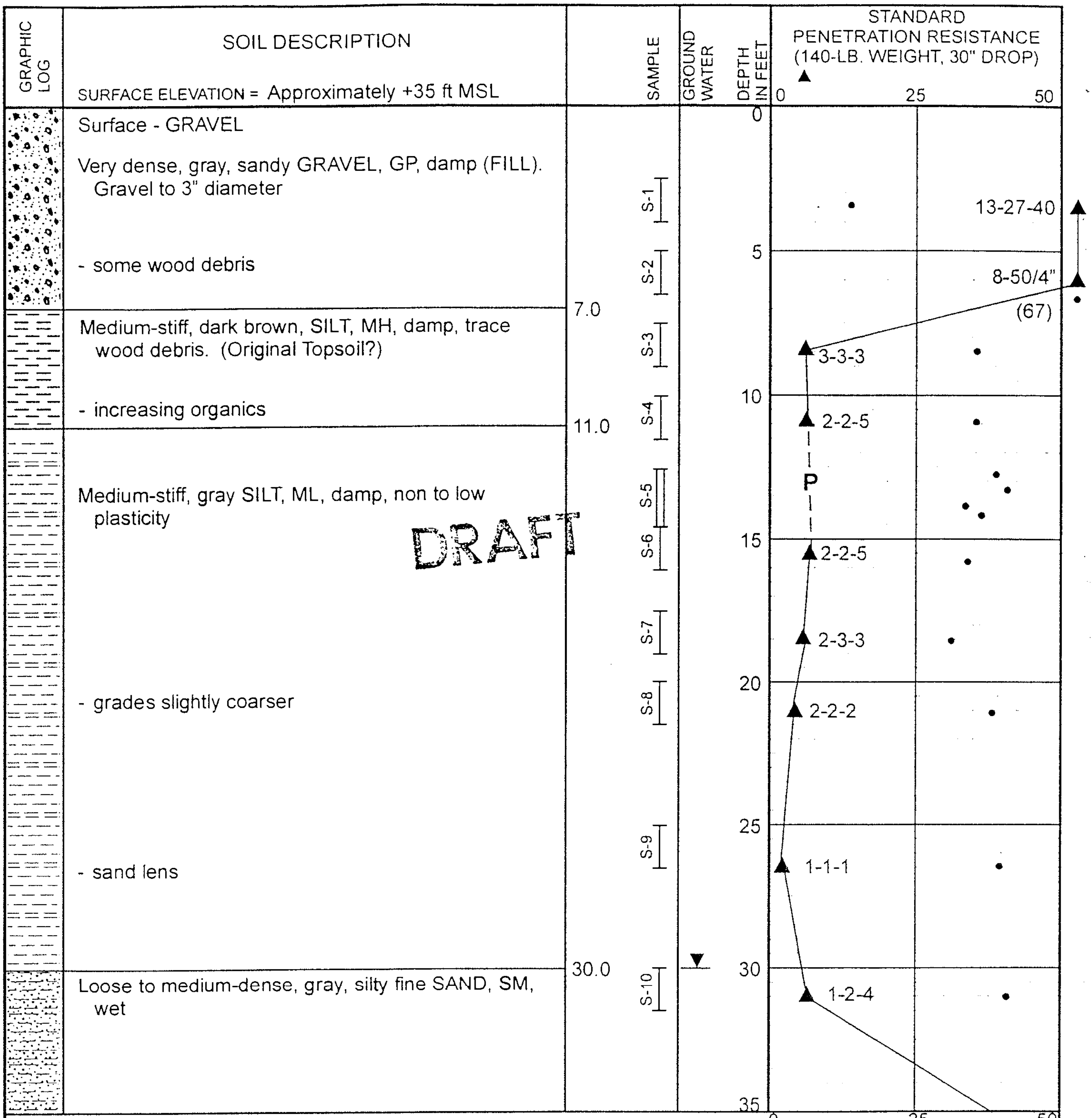
CDM

Log of Boring B-1

DEA/OR City/Amtrak Station
Oregon City, Oregon

FIGURE
A-2b

DRAWING NAME	CLIENT NO	PROJECT NO	TASK	DRAWN	DATE	APPROVED
Boring.cdr	20136	31297	INV FLD	BBP	05 Jan 00	



SOIL DESCRIPTION

SURFACE ELEVATION = Approximately +35 ft MSL

Surface - GRAVEL

Very dense, gray, sandy GRAVEL, GP, damp (FILL). Gravel to 3" diameter

- some wood debris

7.0

Medium-stiff, dark brown, SILT, MH, damp, trace wood debris. (Original Topsoil?)

- increasing organics

11.0

Medium-stiff, gray SILT, ML, damp, non to low plasticity

DRAFT

- grades slightly coarser

- sand lens

30.0

Loose to medium-dense, gray, silty fine SAND, SM, wet

2.0" O.D. SPLIT SPOON SAMPLE

3.25" O.D. SPLIT SPOON SAMPLE

3.0" O.D. THIN WALL SAMPLE

CORE RUN SAMPLE

P SAMPLE PUSHED

★ SAMPLE NOT RECOVERED

ATTERBERG LIMITS

PLASTIC LIMIT, NATURAL WATER CONTENT, LIQUID LIMIT

IMPERVIOUS SEAL

▽ WATER LEVEL

OBSERVATION WELL TIP

• WATER CONTENT IN PERCENT

1 SOIL DESCRIPTIONS AND INTERFACES ARE INTERPRETIVE, ACTUAL CHANGES MAY BE GRADUAL

2 WATER LEVEL IS FOR DATE SHOWN AND MAY VARY WITH TIME OF YEAR

CDM

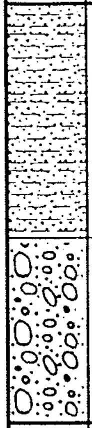
Log of Boring B-2

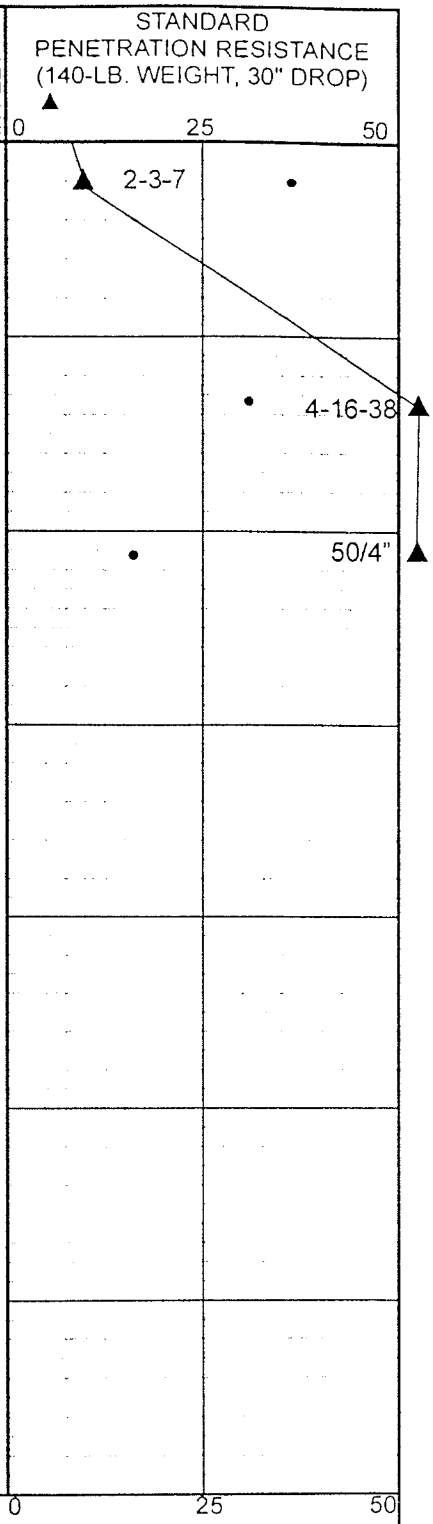
DEA/OR City/Amtrak Station
Oregon City, Oregon

FIGURE
A-3a

DRAWING NAME	CLIENT NO	PROJECT NO	TASK	DRAWN	DATE	APPROVED
Boring.cdr	20136	31297	INV FLD	BBP	05 Jan 00	

GRAPHIC LOG	SOIL DESCRIPTION SURFACE ELEVATION = Approximately +35 feet MSL		SAMPLE	GROUND WATER	DEPTH IN FEET
	Loose to medium-dense, gray, silty SAND, SM, wet.		S-11		35
		41.0	S-12		40
	- Driller reports 6" diameter cobbles Very dense, dark gray, GRAVEL with some sand, GW, wet	45.3	S-13		45
	Bottom of boring at 45.3 ft, completed 12/12/00 Groundwater at 30 feet bgs.				50
					55
					60
					65
					70





LAB DATA

Sample Number	Depth (ft)	γw (pcf)	γd (pcf)	wc (%)
S-5C	12.5 - 13.1	103.2	73.5	40.4
S-5B	13.2 - 13.8	113.3	82.5	37.4
S-5A	13.9 - 14.5	115.0	85.3	34.8

DRAFT

I 2.0" O.D. SPLIT SPOON SAMPLE
P SAMPLE PUSHED
IMPERVIOUS SEAL
• WATER CONTENT IN PERCENT
3.25" O.D. SPLIT SPOON SAMPLE
* SAMPLE NOT RECOVERED
▽ WATER LEVEL
3.0" O.D. THIN WALL SAMPLE
ATTERBERG LIMITS
LIQUID LIMIT
NATURAL WATER CONTENT
PLASTIC LIMIT
OBSERVATION WELL TIP
CORE RUN SAMPLE

1 SOIL DESCRIPTIONS AND INTERFACES ARE INTERPRETIVE, ACTUAL CHANGES MAY BE GRADUAL
2 WATER LEVEL IS FOR DATE SHOWN AND MAY VARY WITH TIME OF YEAR

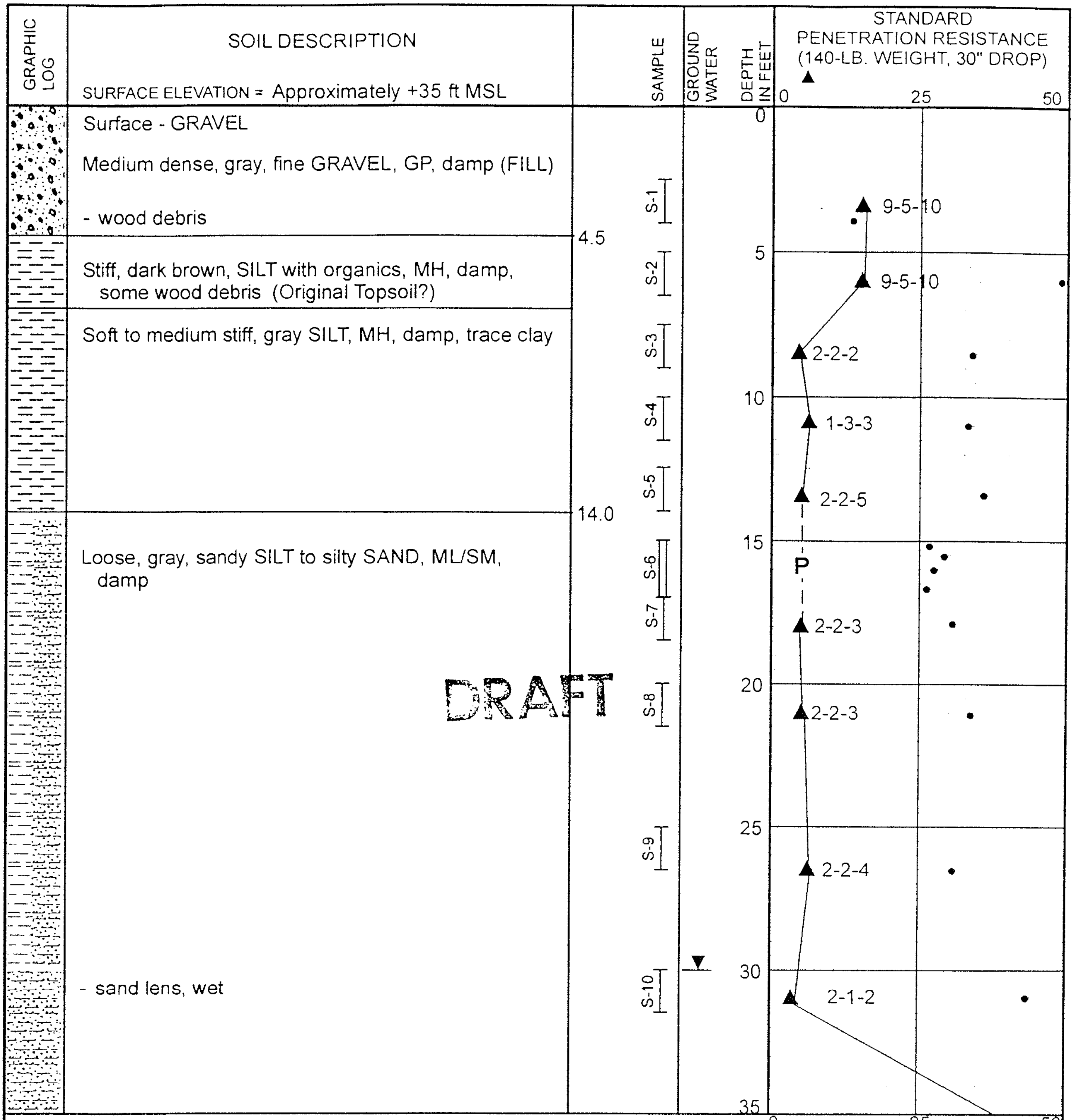
CDM

Log of Boring B-2

DEA/OR City/Amtrak Station
Oregon City, Oregon

FIGURE
A-3b

DRAWING NAME	CLIENT NO	PROJECT NO	TASK	DRAWN	DATE	APPROVED
Boring cdr	20136	31297	INV FLD	BBP	05 Jan 00	



SOIL DESCRIPTION

SURFACE ELEVATION = Approximately +35 ft MSL

Surface - GRAVEL

Medium dense, gray, fine GRAVEL, GP, damp (FILL)

- wood debris

4.5

Stiff, dark brown, SILT with organics, MH, damp, some wood debris (Original Topsoil?)

Soft to medium stiff, gray SILT, MH, damp, trace clay

14.0

Loose, gray, sandy SILT to silty SAND, ML/SM, damp

- sand lens, wet

2.0" O.D. SPLIT SPOON SAMPLE
3.25" O.D. SPLIT SPOON SAMPLE
3.0" O.D. THIN WALL SAMPLE
CORE RUN SAMPLE
P SAMPLE PUSHED
* SAMPLE NOT RECOVERED
ATTERBERG LIMITS
LIQUID LIMIT
NATURAL WATER CONTENT
PLASTIC LIMIT
IMPERVIOUS SEAL
▽ WATER LEVEL
OBSERVATION WELL TIP
• WATER CONTENT IN PERCENT

1 SOIL DESCRIPTIONS AND INTERFACES ARE INTERPRETIVE. ACTUAL CHANGES MAY BE GRADUAL

2 WATER LEVEL IS FOR DATE SHOWN AND MAY VARY WITH TIME OF YEAR

CDM

Log of Boring B-3

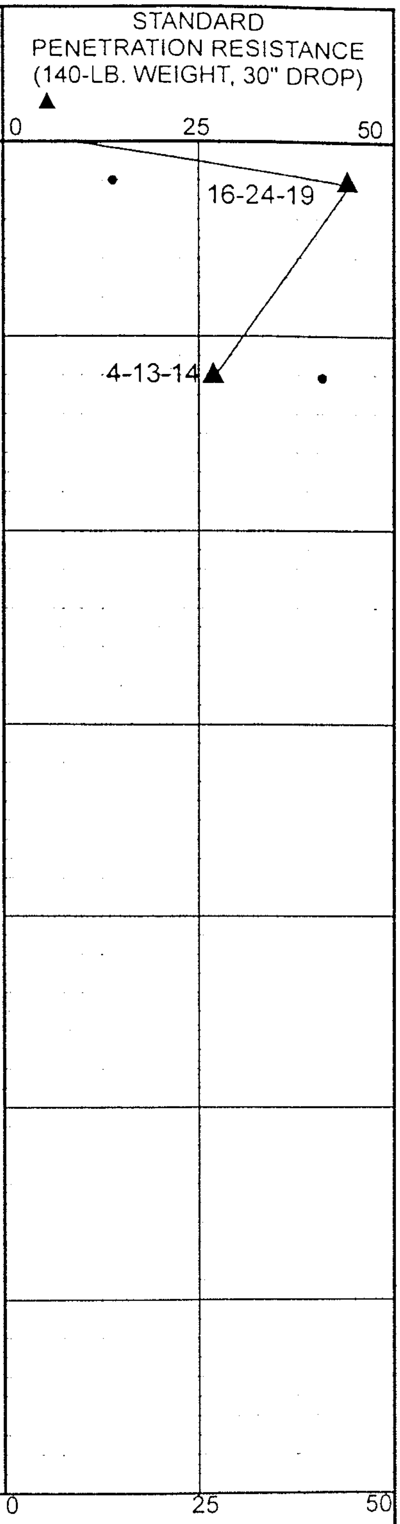
DEA/OR City/Amtrak Station
Oregon City, Oregon

FIGURE
A-4a

DRAWING NAME	CLIENT NO	PROJECT NO	TASK	DRAWN	DATE	APPROVED
Boring.cdr	20136	31297	INV FLD	LJR	05 Jan 00	

GRAPHIC LOG	SOIL DESCRIPTION SURFACE ELEVATION = Approximately +35 feet MSL		SAMPLE	GROUND WATER	DEPTH IN FEET
	Dense, gravelly SAND, SP, wet		S-11		35
	Medium dense, dark gray, silty GRAVEL, wet - driller reports large gravel and cobbles	41.0	S-12		40
	Bottom of boring at 45.0 ft, completed 12/12/00 Groundwater at 30 feet bgs.	45.0			45
					50
					55
					60
					65
					70

STANDARD PENETRATION RESISTANCE (140-LB. WEIGHT, 30" DROP)



LAB DATA

Sample Number	Depth (ft)	γw (pcf)	γd (pcf)	wc (%)
S-6C	15.2 - 15.5	97.3	76.3	27.6
S-6B	15.6 - 16.2	101.9	79.4	28.5
S-6A	16.3 - 16.9	103.0	81.2	26.9

DRAFT

I 2.0" O.D. SPLIT SPOON SAMPLE

3.25" O.D. SPLIT SPOON SAMPLE

3.0" O.D. THIN WALL SAMPLE

▯ CORE RUN SAMPLE

P SAMPLE PUSHED

* SAMPLE NOT RECOVERED

ATTERBERG LIMITS

LIQUID LIMIT

NATURAL WATER CONTENT

PLASTIC LIMIT

IMPERVIOUS SEAL

▽ WATER LEVEL

OBSERVATION WELL TIP

• WATER CONTENT IN PERCENT

1 SOIL DESCRIPTIONS AND INTERFACES ARE INTERPRETIVE, ACTUAL CHANGES MAY BE GRADUAL

2 WATER LEVEL IS FOR DATE SHOWN AND MAY VARY WITH TIME OF YEAR

CDM

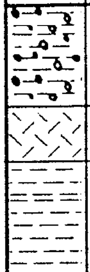
Log of Boring B-3

DEA/OR City/Amtrak Station
Oregon City, Oregon

FIGURE
A-4b

DRAWING NAME	CLIENT NO	PROJECT NO	TASK	DRAWN	DATE	APPROVED
Boring cdr	20136	31297	INV FLD	LJR	05 Jan 00	

GRAPHIC LOG	SOIL DESCRIPTION SURFACE ELEVATION = Approximately +35 ft MSL		SAMPLE	GROUND WATER	DEPTH IN FEET	STANDARD PENETRATION RESISTANCE (140-LB. WEIGHT, 30" DROP) ▲ 0 25 50
	Surface - Asphalt Loose sandy GRAVEL				0	
	Wood Chips	2.5				
	sandy SILT	4.0			5	
					10	
					15	
					20	
					25	
					30	
					35	0 25 50



DRAFT

- 2.0" O.D. SPLIT SPOON SAMPLE
- 3.25" O.D. SPLIT SPOON SAMPLE
- 3.0" O.D. THIN WALL SAMPLE
- CORE RUN SAMPLE
- P SAMPLE PUSHED
- ★ SAMPLE NOT RECOVERED
- ATTERBERG LIMITS: PLASTIC LIMIT, NATURAL WATER CONTENT, LIQUID LIMIT
- IMPERVIOUS SEAL
- ▽ WATER LEVEL
- OBSERVATION WELL TIP
- • WATER CONTENT IN PERCENT

1. SOIL DESCRIPTIONS AND INTERFACES ARE INTERPRETIVE. ACTUAL CHANGES MAY BE GRADUAL.
2. WATER LEVEL IS FOR DATE SHOWN AND MAY VARY WITH TIME OF YEAR

CDM

Log of Boring PT-1

DEA/OR City/Amtrak Station
Oregon City, Oregon

FIGURE **A-5**

DRAWING NAME	CLIENT NO	PROJECT NO	TASK	DRAWN	DATE	APPROVED
Boring cdr	20136	31297	INV FLD	BBP	05 Jan 00	

GRAPHIC LOG	SOIL DESCRIPTION SURFACE ELEVATION = Approximately +35 ft MSL		SAMPLE	GROUND WATER	DEPTH IN FEET	STANDARD PENETRATION RESISTANCE (140-LB. WEIGHT, 30" DROP) ▲ 0 25 50
	Surface - Asphalt Loose sandy GRAVEL	2.5			0	
	Wood Chips	4.0				
	sandy SILT	5.5	S-1		5	
	Bottom of boring at 5.5 feet. Completed 12/13/00.				10 15 20 25 30 35	0 25 50

DRAFT

I 2.0" O.D. SPLIT SPOON SAMPLE

3.25" O.D. SPLIT SPOON SAMPLE

3.0" O.D. THIN WALL SAMPLE

CORE RUN SAMPLE

P SAMPLE PUSHED

* SAMPLE NOT RECOVERED

ATTERBERG LIMITS

LIQUID LIMIT

NATURAL WATER CONTENT

PLASTIC LIMIT

IMPERVIOUS SEAL

▽ WATER LEVEL

OBSERVATION WELL TIP

• WATER CONTENT IN PERCENT

1 SOIL DESCRIPTIONS AND INTERFACES ARE INTERPRETIVE. ACTUAL CHANGES MAY BE GRADUAL

2 WATER LEVEL IS FOR DATE SHOWN AND MAY VARY WITH TIME OF YEAR

CDM

Log of Boring PT-2

DEA/OR City/Amtrak Station
Oregon City, Oregon

FIGURE
A-6

DRAWING NAME	CLIENT NO	PROJECT NO	TASK	DRAWN	DATE	APPROVED
Boring cdr	20136	31297	INV FLD	BBP	05 Jan 00	

Appendix B
Laboratory Testing

David Evans & Associates/Amtrak Station
Oregon City, Oregon

Natural Water Contents

All jar samples were visually classified to refine, when necessary, the field soil classification. In additions, natural moisture contents were taken on all samples in accordance with ASTM D 2216. The moisture contents are expressed as a percentage of free water lost by evaporation compared to the dry weight of the soil. These results are presented graphically on the boring logs (**Figures A-2** and **A-4**).

Undisturbed Tube Samples

All thin-walled, steel tube samples and Dames & Moore ring samples were extruded, classified, tested for relative strength with a Torvane and/or Pocket Penetrometer device, and tested for natural moisture content and unit weight. The unit weight determinations are tabulated on the respective logs.

Atterberg Limits

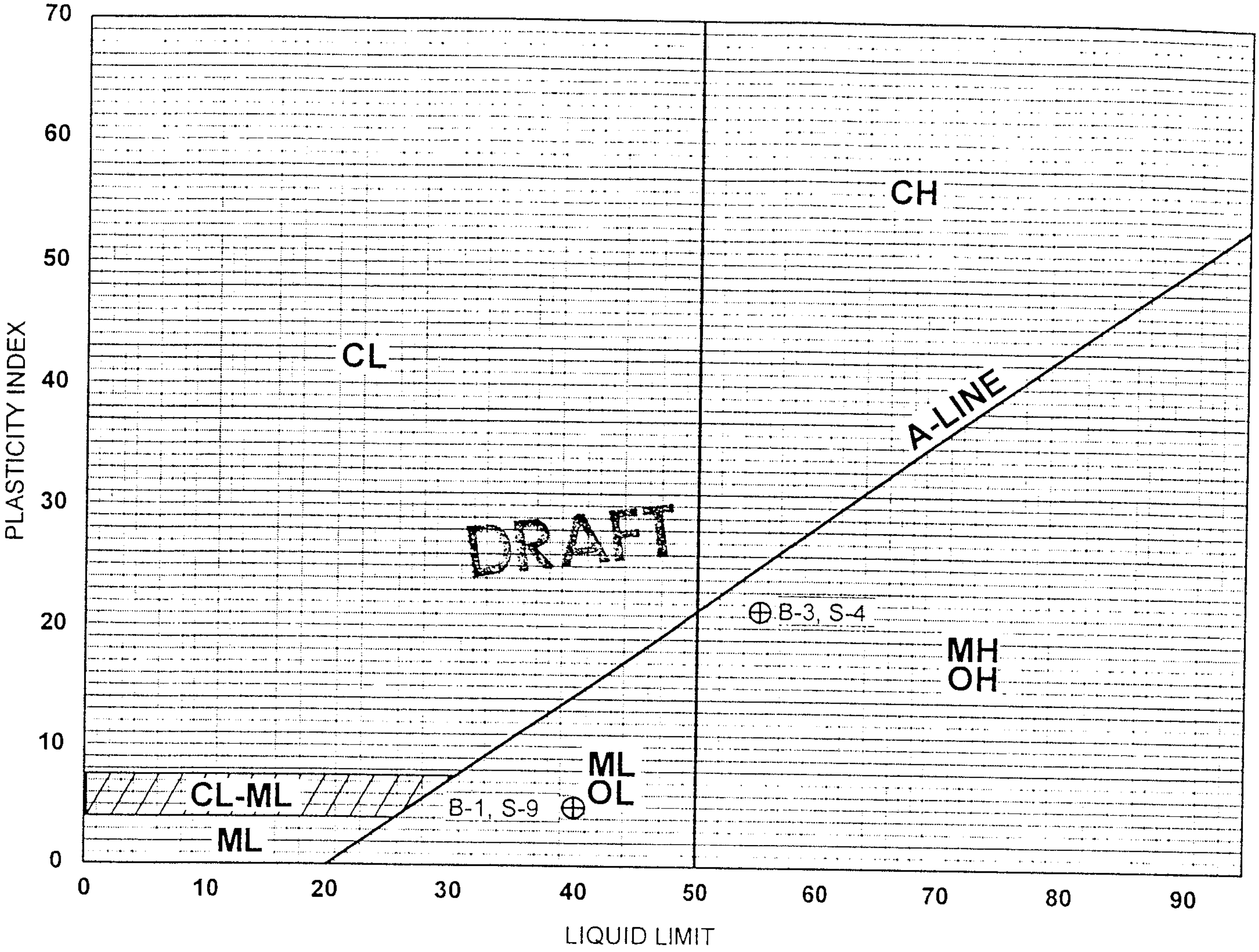
Atterberg Limits (ASTM D 423 for the liquid limit and ASTM D 424 for the plastic limit) were performed on six samples to evaluate the relative plasticity and assist in the classification of the fine-grained soils. Atterberg Limits are a quick index test that provides and indication of relative swell potential. The results are plotted on the Plasticity Charts, **Figure B-1**.

Sieve Analysis

Grain size analyses were performed on a sample from PT-2. The sample was analyzed in general accordance with the mechanical sieve analysis method, ASTM D 421. In general, the sieve method consists of shaking a washed and oven-dried sample through a set of varied sieve sizes. The material retained on each sieve is recorded as a percent of total sample weight and a graphic plot of the weight percent versus grain-size is generated. The results of this test are presented on **Figure B-2**.

Consolidation Test

Consolidation tests were performed on two sample from boring B-1. Consolidation tests result in a plot of strain response versus the logarithm of applied normal stress. The test is performed by adding increasing stress increments to a 1-inch thick specimen and allowing sufficient time for primary consolidation to occur. The final plot represents the total strain under each load increment and is used to estimate settlement of soils under structural or embankment loads. The final logarithm of stress versus strain plot for the consolidation tests are presented on **Figure B-3** and **B-4.**



B-/TP-NUMBER	SAMPLE NUMBER	U.S.C.	WATER CONTENT IN PERCENT	LIQUID LIMIT IN PERCENT	PLASTIC LIMIT IN PERCENT	PLASTICITY INDEX
B-2	S-9	ML	39.0	40.0	35.0	5.0
B-3	S-4	MH	32.9	55.3	33.7	21.6

Sample Descriptions:

B-1, S-9: Gray mottled rust, SILT, ML; damp, low plasticity
B-3, S-4: Gray SILT, MH; damp, trace clay, medium to high plasticity

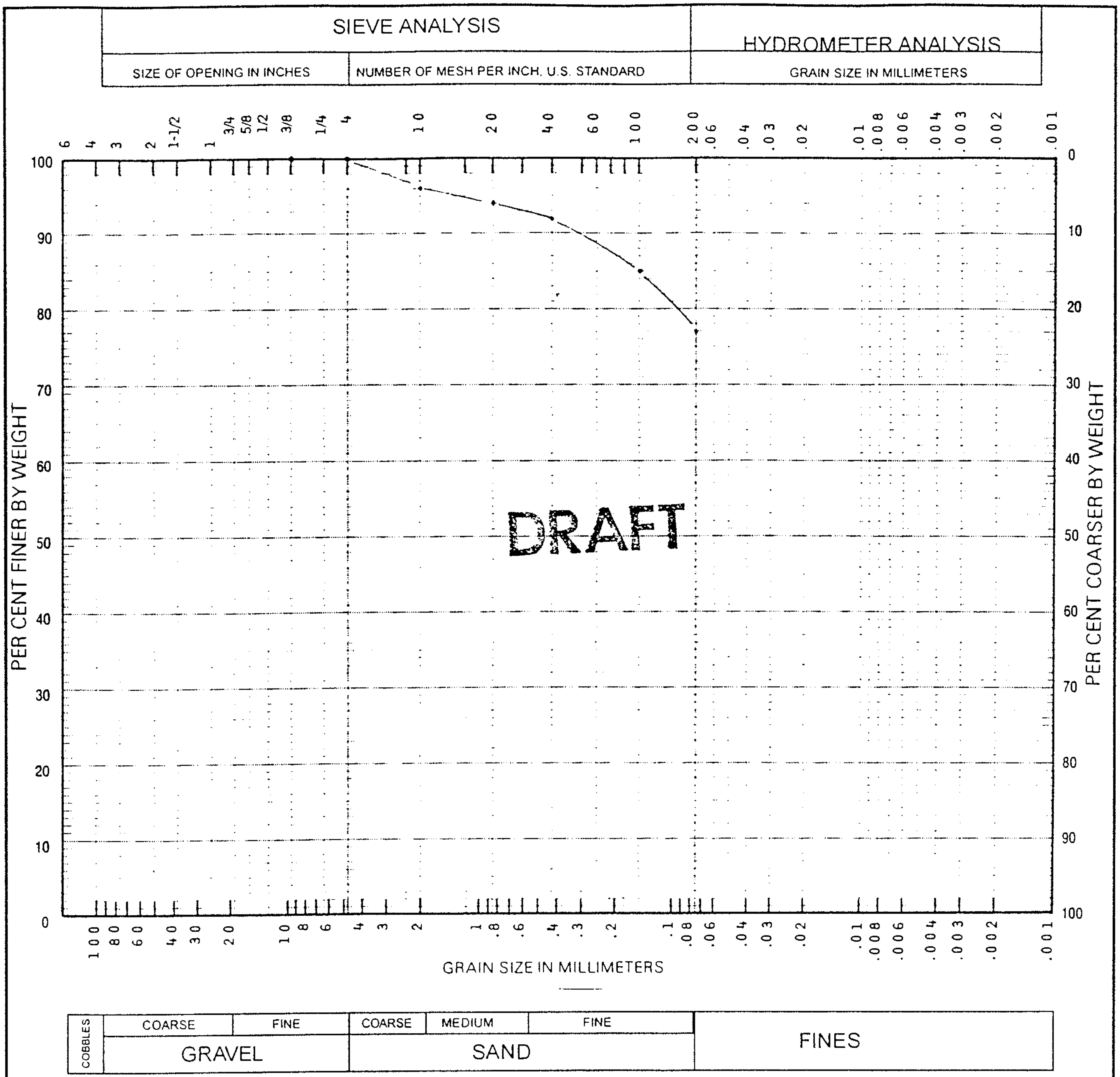
CDM

Atterberg Limits

DEA/ OR City/ Amtrak Station
Oregon City, Oregon

FIGURE
B-1

DRAWING NAME	CLIENT NO	PROJECT NO	TASK	DRAWN	DATE	APPROVED
Boring cdr	20136	31297	--	LJR	24 Jan 01	



DRAFT

SAMPLE NUMBER	DEPTH IN FEET	USC/ AASHTO	CLASSIFICATION	NATURAL WATER CONTENT IN PERCENT	LL	PL	PI
PT-2,	4.0	ML	Sandy SILT	39.1			

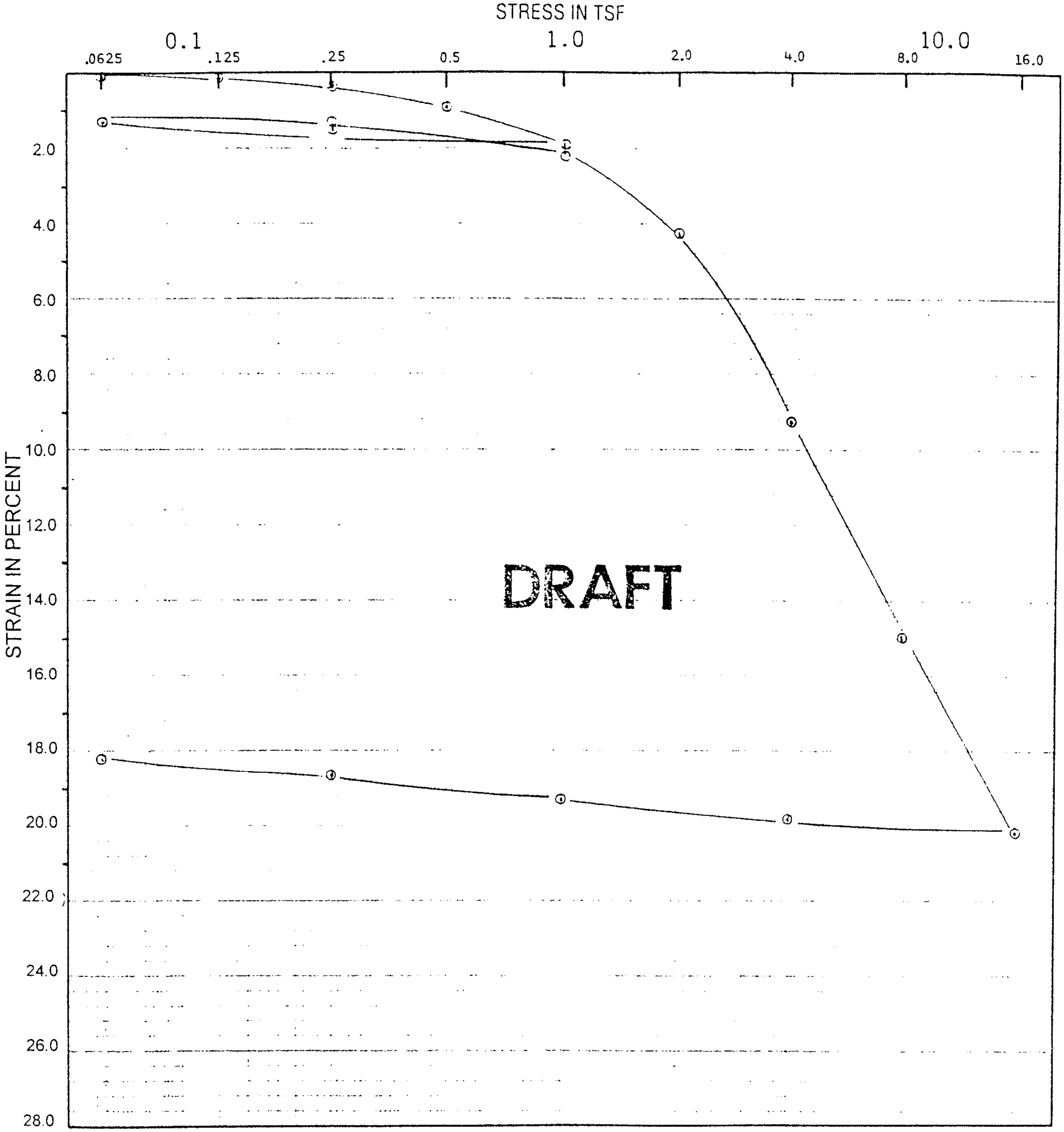
CDM

Grain Size Analysis

DEA/ OR City/ Amtrak Station
Oregon City, Oregon

FIGURE
B-2

	PROJECT NO	DRAWN	DATE	APPROVED	REVISED	DATE
Grain Size Analysis cdr	31297	LJR	23 Jan 01			



DRAFT

* PERCENT SWELL WITH ADDITION OF WATER

BORING NUMBER	SAMPLE NUMBER	DEPTH	WATER CONTENT IN PERCENT		INITIAL UNIT WEIGHT IN PCF		CLASSIFICATION
			NATURAL	FINAL	WET	DRY	
B-1	S-4	10.0-12.0	33.3	30.1	102.2	76.7	Med. stiff, dark brown, mottled, SILT, damp, trace fine sand, low plasticity.

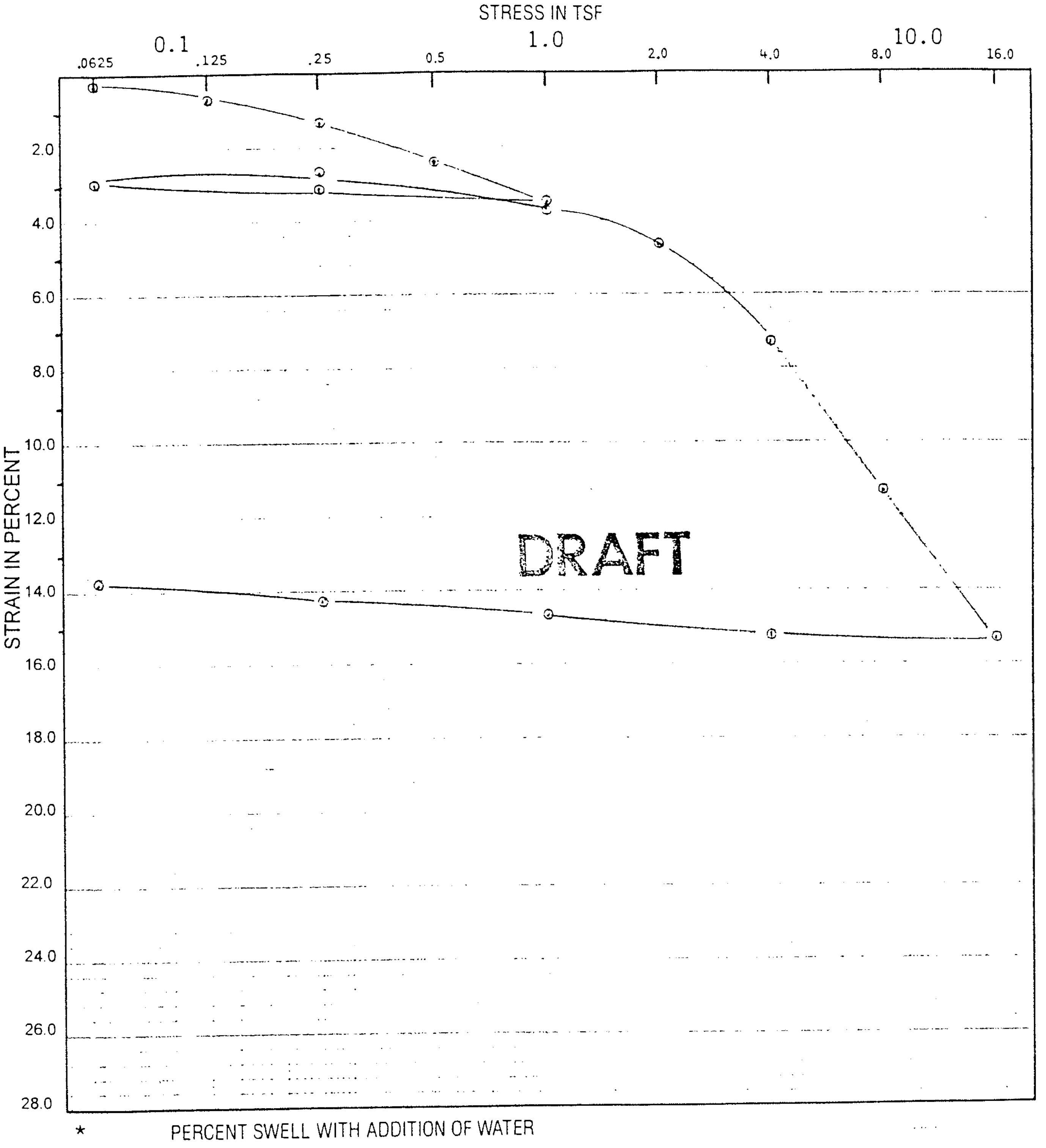
CDM

Consolidation Test: B-1, S-4

DEA/ OR City/ Amtrak Station
Oregon City, Oregon

FIGURE
B-3

DRAWING NAME	PROJECT NO	DRAWN	DATE	APPROVED
Consol.cdr	31297	LJR	23 Jan 01	



BORING NUMBER	SAMPLE NUMBER	DEPTH	WATER CONTENT IN PERCENT		INITIAL UNIT WEIGHT IN PCF		CLASSIFICATION
			NATURAL	FINAL	WET	DRY	
B-1	S-9	25.0-27.0	41.4	31.1	113.6	80.3	Med. stiff, dark brown, mottled gray, SILT, damp, trace clay and fine sand, low plasticity.

CDM

Consolidation Test: B-1, S-9

DEA/ OR City/ Amtrak Station
Oregon City, Oregon

FIGURE
B-4

DRAWING NAME	PROJECT NO	DRAWN	DATE	APPROVED
Consol cdr	31297	LJR	23 Jan 01	

ACRONYMS AND ABBREVIATIONS

ACRONYMS AND ABBREVIATIONS

AC	asphalt concrete
ACP	Asphalt Concrete Pavement
ASTM	American Society for Testing and Materials
BGS	below ground surface
CLSM	controlled low strength material
FEMA	Federal Emergency Management Agency
g	gravitational acceleration (32.2 feet/second^2)
H:V	horizontal to vertical
IBC	International Building Code
MCE	maximum considered earthquake
NAVD	North American Vertical Datum
OSSC	Oregon Standard Specifications for Construction (2015)
pcf	pounds per cubic foot
pci	pounds per cubic inch
PG	performance grade
psf	pounds per square foot
psi	pounds per square inch
RFP	Request for Proposal
ROW	right-of-way
SOSSC	State of Oregon Structural Specialty Code
SPT	standard penetration test