Memo

TO: **RJ Development – Mr. Josh Snodgrass**

FROM: **Tristan T. Anderson, PE (WA)**
Kristopher T. Hauck, PE – Terracon

CC: **PCS Structural – Mr. Jeff Klein**

DATE: **3/3/2017**

REGISTERED PROFESSIONAL ENGINEER
78373PE
OREGON
JULY 11, 2006
KRISTOPHER T. HAUCK
3/3/17

RE: **Addendum Letter to Geotechnical Engineering Report Number 2**
Parkland Village Addition
3121 NW Cumulus Avenue
McMinnville, Oregon
Terracon Project No. 82165034

The purpose of this memo is to provide results of LPILE analyses requested by PCS Structural. Terracon prepared a Geotechnical Engineering Report (GER), dated June 22nd, 2016, and an Addendum to Geotechnical Engineering Report Memo (Addendum #1), dated February 7, 2017. This memo is a supplement to the GER and Addendum #1, and not intended to supersede the report. We still refer to the GER and Addendum #1 for other geotechnical related elements of the project.

To provide adequate occupancy for the development, it was proposed to reduce the recommended slope setback from 35 feet to 10 feet by utilizing deep foundations to improve slope stability and support the structure. Terracon Consultants, Inc. was previously contracted to complete additional slope stability analyses and provide lateral and drilled shaft parameters to the civil/structural engineer (PCS Structural) in Addendum #1. In this Addendum, however, our goal is to provide information regarding the magnitude of moment and deflections experienced by various drilled shafts under various loading scenarios.

Based on our understanding of the revised site development revisions since the Addendum #1, we understand that a small portion of the corner of the development is planned to extend into the 10-foot recommended setback from the top of the slope. Our analyses in the following section have taken into account this small section and provided an updated recommendation for the sections that extend into the 10-foot setback.

1.0 METHODOLOGY

The subsurface profile was previously developed in the GER and reiterated in Addendum #1, therefore, it is not included it here. The following table was previously provided in Addendum #1; however, we have included it here for the purposes of clarity in how the model was input into our analysis software.

SOIL PARAMETERS FOR AXIAL AND LATERAL CAPACITY

Soil Unit	Depth Interval (ft)	Soil Data[1]						Allowable Drilled Shaft Unit Capacity[6] (ksf)	
		Effective Unit Weight[2] (pcf)	Shear Strength Parameters[3]		Subgrade Modulus[4] k (pci)	Static Soil Strain[4] ε_{50} (%)	P-Y Curve Soil Model	Tip[5]	Skin
			$Ø_p$' (deg.)	C' (ksf)					
SOIL PROFILE ENCOUNTERED AND INTERPRETED IN SB-1									
Silt trace sand	0 – 5	110	26	0.20	N/R	N/R	11-Silt (Phi+C)	N/R	N/R
Silt trace sand	5 – 10	110	26	0.20	N/R	N/R	11-Silt (Phi+C)	4.80	0.27
Silt with sand	10 – 20	110	26	0.20	N/R	N/R	11-Silt (Phi+C)	7.97	0.32
Silt with sand	20 – 33	110	26	0.20	267.5	1.52	11-Silt (Phi+C)	14.1	0.42
Silt with sand (saturated)	33 – Undetermined	48	26	0.20	267.5	0.93	11-Silt (Phi+C)	22.1	0.55

Notes:
1. The Soil Data values presented herein are based on field and lab tests and correlations from SPT data and represent ultimate values, no factor of safety has been included. Drilled Shaft Unit Capacity values presented herein also represent ultimate values. The designer should incorporate appropriate factors of safety in his or her design.
2. From AllPile 7.15c using uncorrected Standard Penetration Blow Counts from SPT data. AllPile uses correlation tables based on compactness of granular soil and consistency of fine-grained from p.12 of the 1975 USS Steel Sheet Piling Design Manual.
3. Based on field tests and correlations with SPT data and lab strength data.
4. Values based on ranges presented in the L-Pile Manual for both static and cyclic conditions. N/R = Lateral support of soil should be neglected due to likely catastrophic event in these soil units.
5. Shaft tip capacity is based upon direct contact between concrete and medium stiff to stiff silt. These values are contingent upon a clean bottom following excavation. If loose or soft soil is left in the excavation bottom the tip capacity will be reduced significantly. Verification of a clean shaft bottom must be performed prior to placing reinforcing steel or concrete.
6. N/R = Shaft capacity should be neglected in the upper 5 feet of the profile due to soil effects associated with surficial slope stability.
7. Additional Drilled Shaft parameters including Passive Coefficient, Young's Modulus and Sliding Resistance are included in Boring Logs in Appendix C. Values calculated using reference documents: FHWA Report No. IF-02-034, Geotechnical Engineering Circular No. 5 and L-Pile Manual.

Our analyses used LPILE 2015.8.03 by Ensoft, Inc., rather than AllPile as described in the table. LPILE analyzes lateral deflections only, neglecting vertical loading, except for the purposes of determining p-δ secondary deflection of shafts.

Due to slope stability issues, the upper 20 feet of soils were neglected for lateral restraint, and all loadings were applied to a soil profile that started 20 feet below the existing ground surface. The lateral loads that were applied to the pile tops were 5 kips, 10 kips, 15 kips, and 20 kips. These loads were applied as a static load for evaluation, they do not consider any dynamic style of loading such as seismic, or flow slide impacts.

To develop shaft geometry, strength, and stiffness parameters, Terracon was provided the following data from PCS Structural:

Modulus of Elasticity: 3605ksi (for f_c' = 4ksi)

Shaft Diameter	Critical Moment	Gross Moment of Inertia	Cracked Moment of Inertia
Inches	Kip-ft	$Inch^4$	$Inch^4$
12	6.7	1018	509
14	10.6	1886	943
16	15.9	3217	1608

Analyses were performed assuming the section was uncracked initially (Gross Moment of Inertia). If the ultimate moment carried by the shaft exceeded the Critical Moment, then the analysis was rerun using the Cracked Moment of Inertia for the shaft. In all cases analyzed except one for the 16-inch diameter shaft, the moment conditions were sufficient to exceed the Gross Moment of Inertia. All piles were assumed to have a free-head fixity at the surface.

2.0 RESULTS OF ANALYSES & RECOMMENDATIONS

Each shaft was analyzed to determine the ultimate moment in the pile as well as moment and deflection diagrams with depth. We were also asked to provide an estimate for where lateral fixity occurs due to the proposed lateral loads. The following tables summarize analysis. Moment and deflection diagrams are attached at the end of this report in Figures 1 through 8.

12" Diameter Shaft Results Summary

Load	Cracked Section	Max Deflection	Maximum Moment	Depth to Maximum	Depth to Fixity
kips		inch	Kip-ft	ft	ft
5	Yes	0.10	11.0	3.5	10.0
10	Yes	0.29	26.6	4.0	12.5
15	Yes	0.70	52.8	5.0	12.5
20	Yes	1.52	89.9	6.0	15.0

14" Diameter Shaft Results Summary

Load	Cracked Section	Max Deflection	Maximum Moment	Depth to Maximum	Depth to Fixity
kips		inch	Kip-ft	ft	ft
5	Yes*	0.09	10.5	3.5	10.0
10	Yes	0.28	25.3	4.0	12.5
15	Yes	0.67	18.9	4.5	12.5
20	Yes	1.40	82.4	5.5	15.0

*Uncracked section run shows ultimate moment to be sufficient to crack section. After section cracks, moment carried by the section reduces.

16" Diameter Shaft Results Summary

Load	Cracked Section	Max Deflection	Maximum Moment	Depth to Maximum	Depth to Fixity
kips		inch	Kip-ft	ft	ft
5	No	0.04	11.5	3.5	12.5
10	Yes	0.19	25.9	4.0	12.5
15	Yes	0.37	44.2	4.5	12.5
20	Yes	0.69	70.5	5.0	15.0

These analyses were provided for lateral load conditions for the top of the piles using an assumed free-head condition. While the connections may be relatively flexible at the top of the piles, they are likely not a true free-head condition when poured into grade beams and integrated with the floors. Fixed- head conditions typically increase the depth to the maximum moment and depth to fixity. Therefore, since these piles are part of a stabilization protection measure and the pile heads not likely a true free-head condition, we recommend that the reinforcement within the piles should be extended in the pile a minimum depth of 5 feet beyond the 20-foot potential slide failure plane.

Lastly, the small corner of the structure that currently extends a few feet into the previous setback will increase the depth of the potential slide failure plane at the pile location. Therefore, we recommend for every foot into the 10-foot zone the piles extend, the depth of potential slide failure plane should be extended a foot.

3.0 GENERAL COMMENTS

The analyses and recommendations presented in this memo are based upon conversations with RJ Development and PCS Structural, and the data obtained from the borings performed at the indicated locations and from other information discussed in the GER for the Parkland Village Addition. This memo does not reflect variations that may occur between borings, across the site, or due to the modifying effects of construction or weather. The nature and extent of such variations may not become evident until during or after construction. If variations appear, we should be immediately notified so that further evaluation and supplemental recommendations can be provided.

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

12in Shaft, Cracked Section Moment Diagram

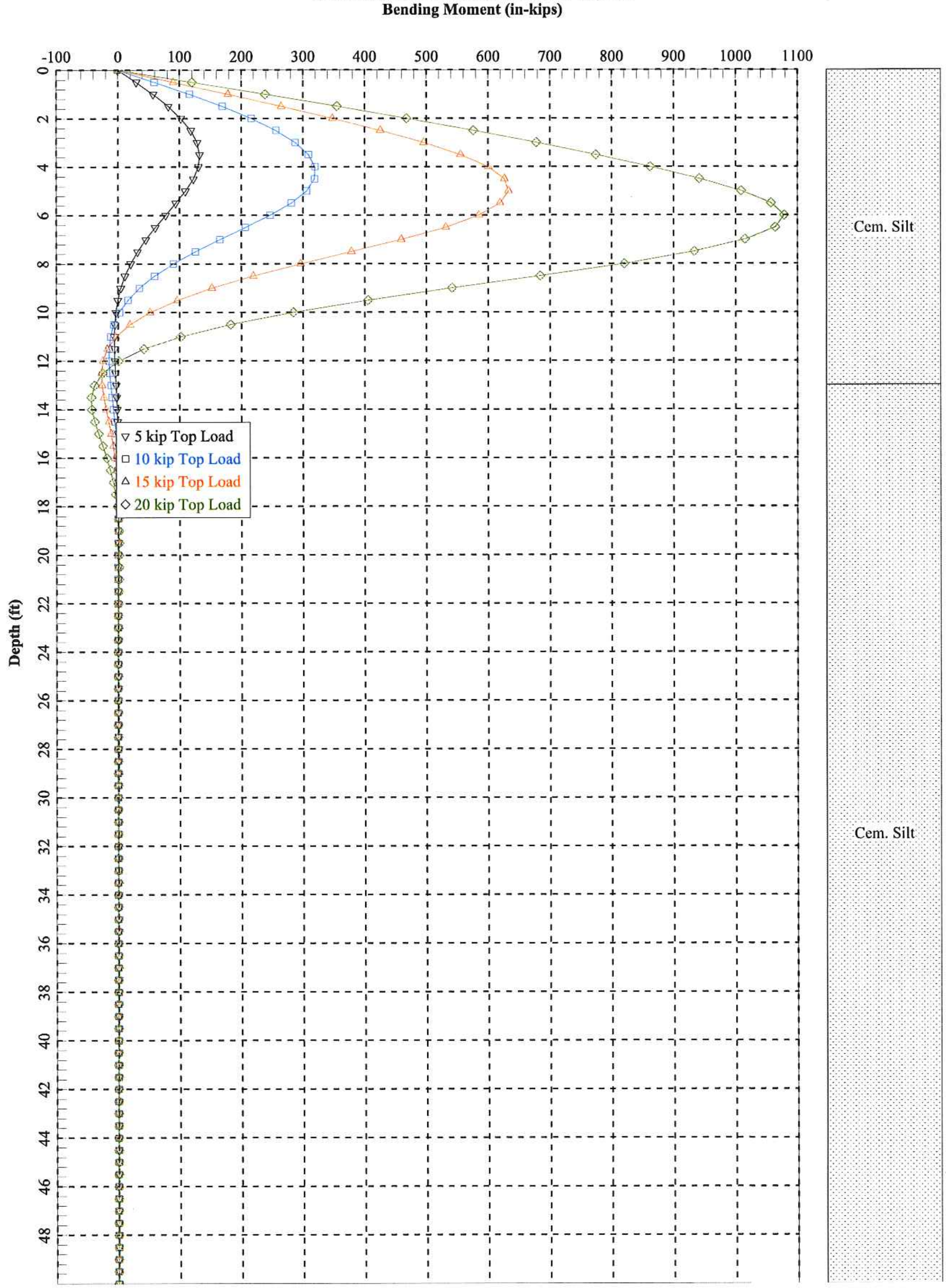


Figure 1

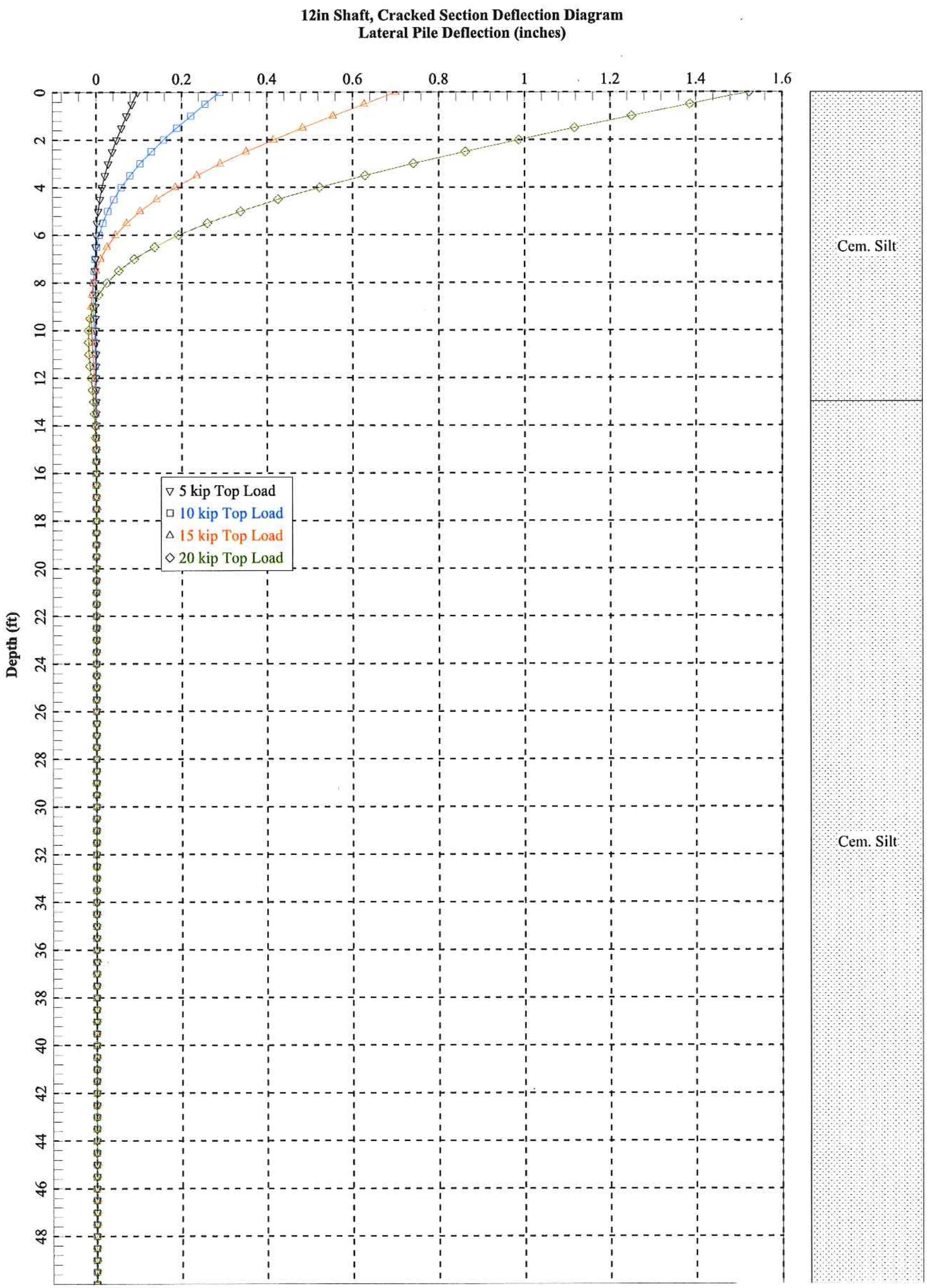
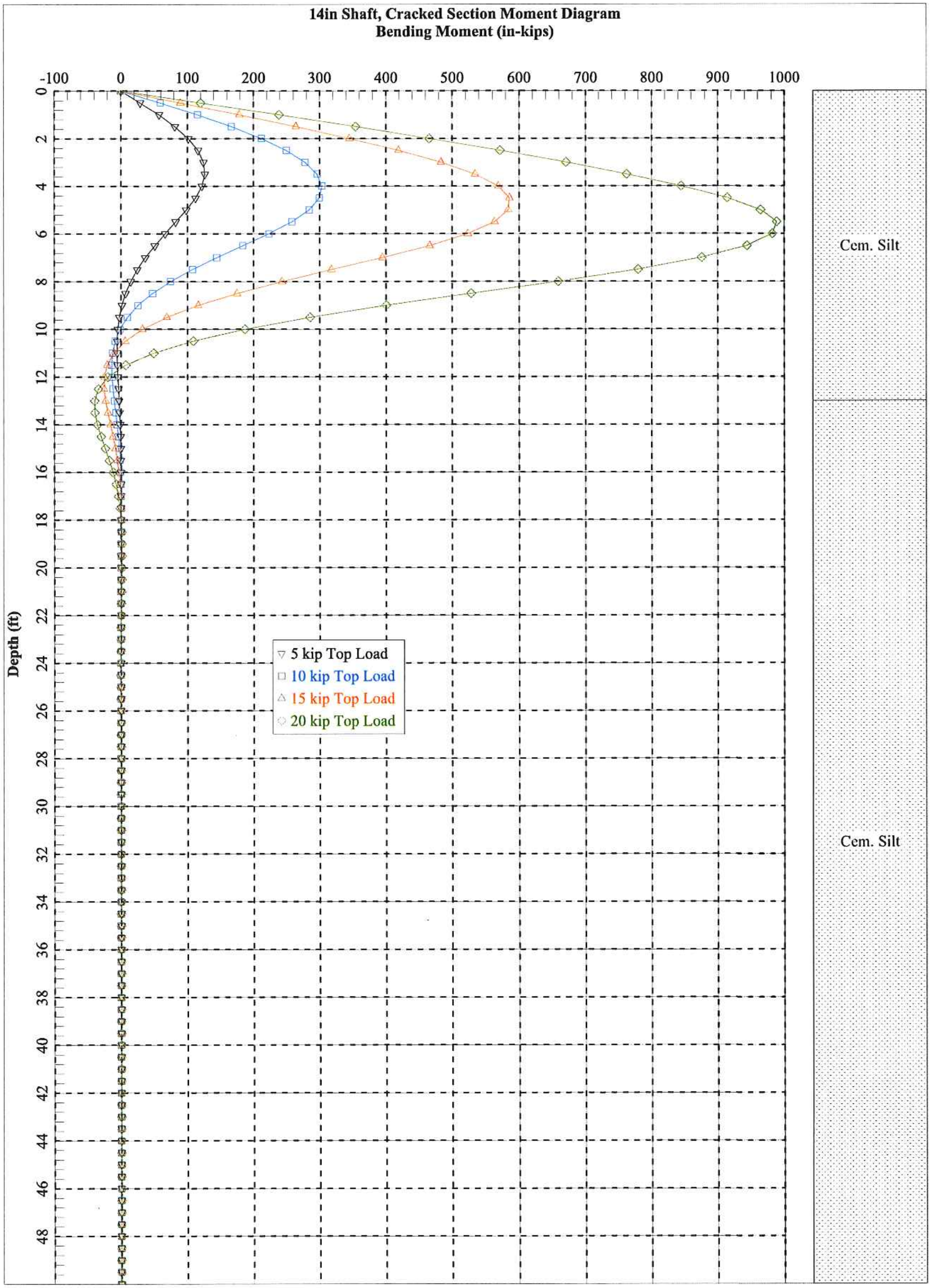

12in Shaft, Cracked Section Deflection Diagram
Lateral Pile Deflection (inches)
0
0.2
0.4
0.6
0.8
1
1.2
1.4
1.6
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
42
44
46
48
5 kip Top Load
10 kip Top Load
15 kip Top Load
20 kip Top Load
Cem. Silt
Cem. Silt

Figure 2



14in Shaft, Cracked Section Moment Diagram
Bending Moment (in-kips)
-100
0
100
200
300
400
500
600
700
800
900
1000
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
42
44
46
48
5 kip Top Load
10 kip Top Load
15 kip Top Load
20 kip Top Load
Cem. Silt
Cem. Silt

Figure 3

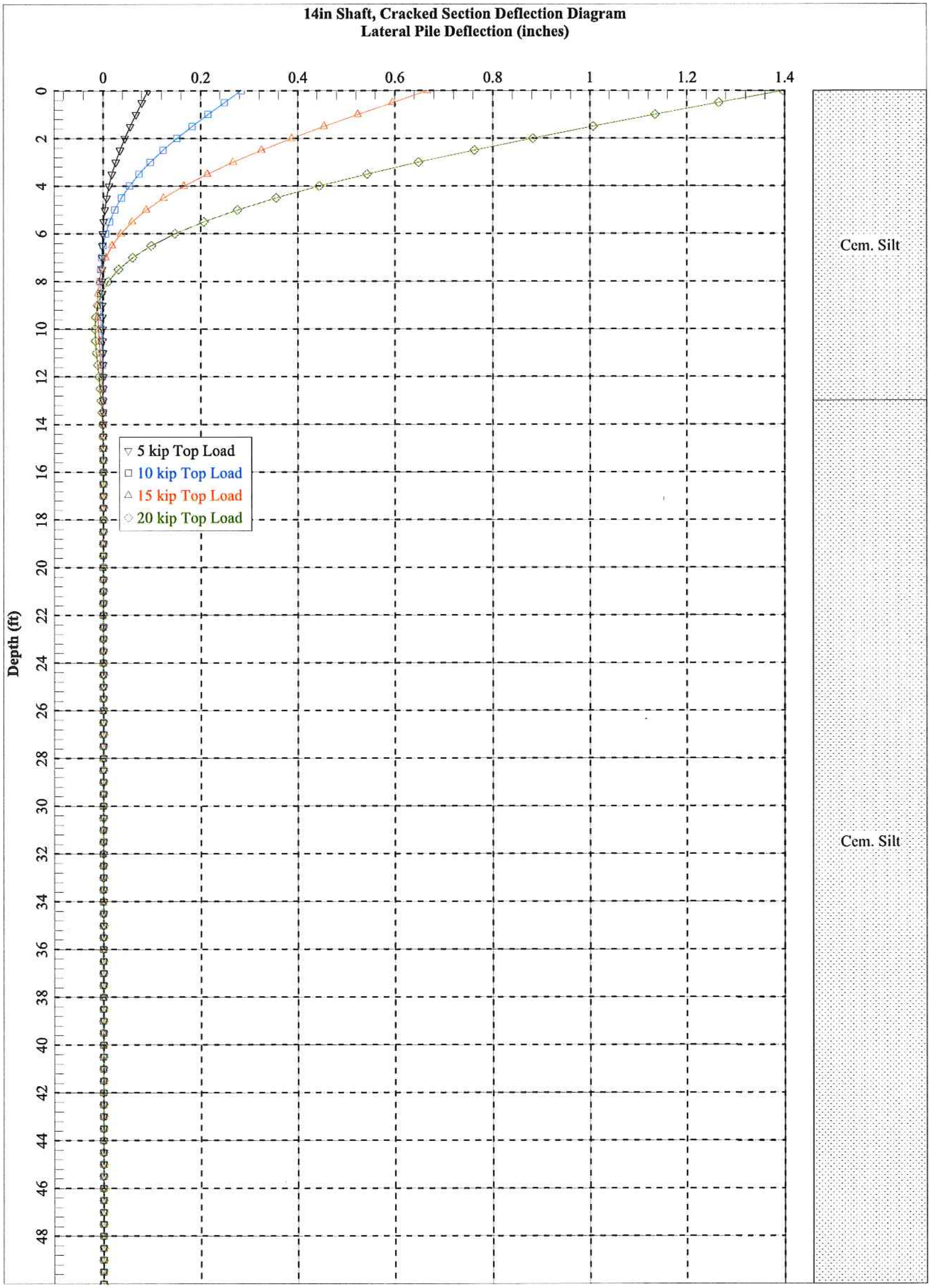

14in Shaft, Cracked Section Deflection Diagram
Lateral Pile Deflection (inches)
0
0.2
0.4
0.6
0.8
1
1.2
1.4
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
42
44
46
48
5 kip Top Load
10 kip Top Load
15 kip Top Load
20 kip Top Load
Cem. Silt
Cem. Silt

Figure 4

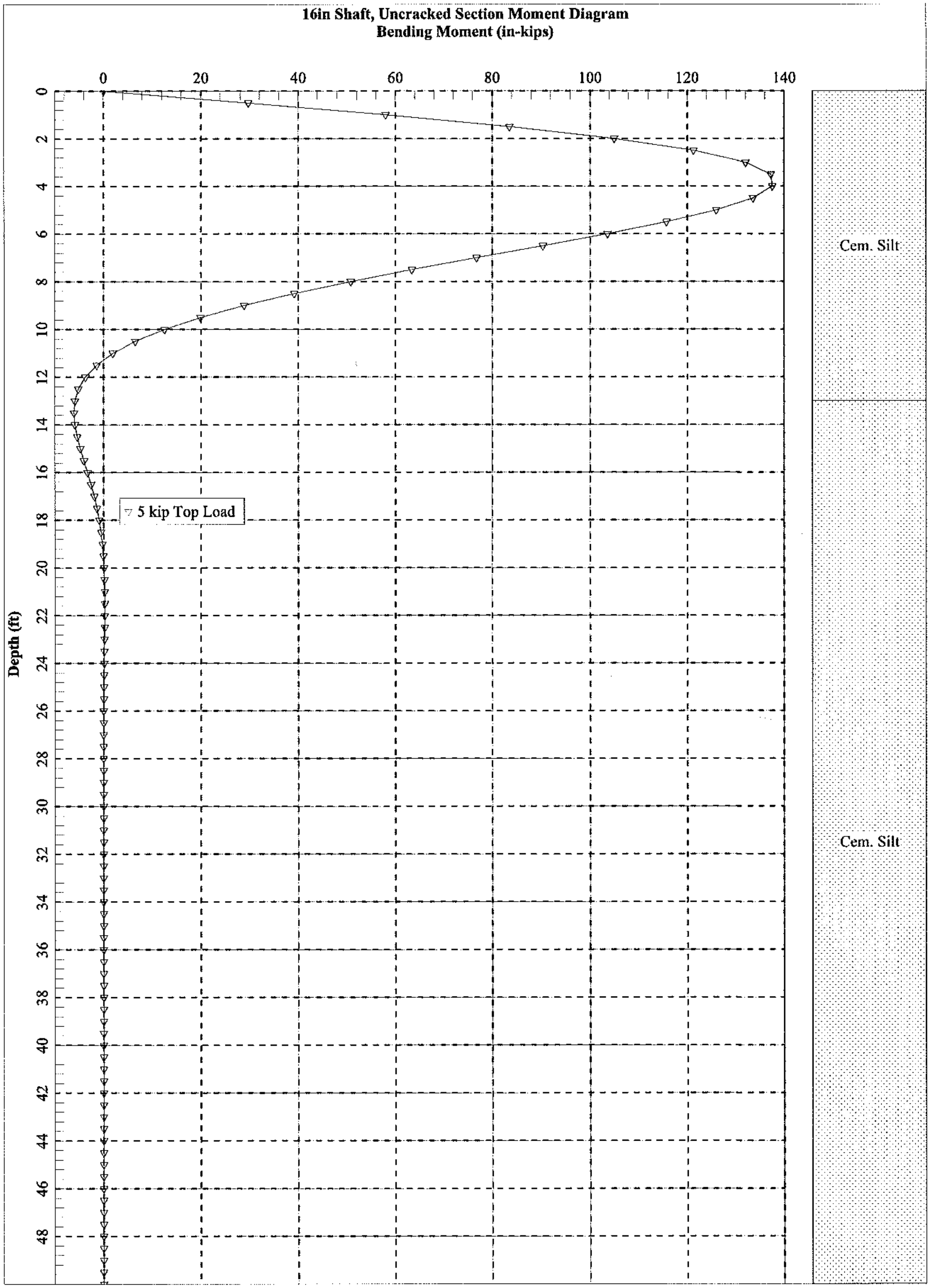
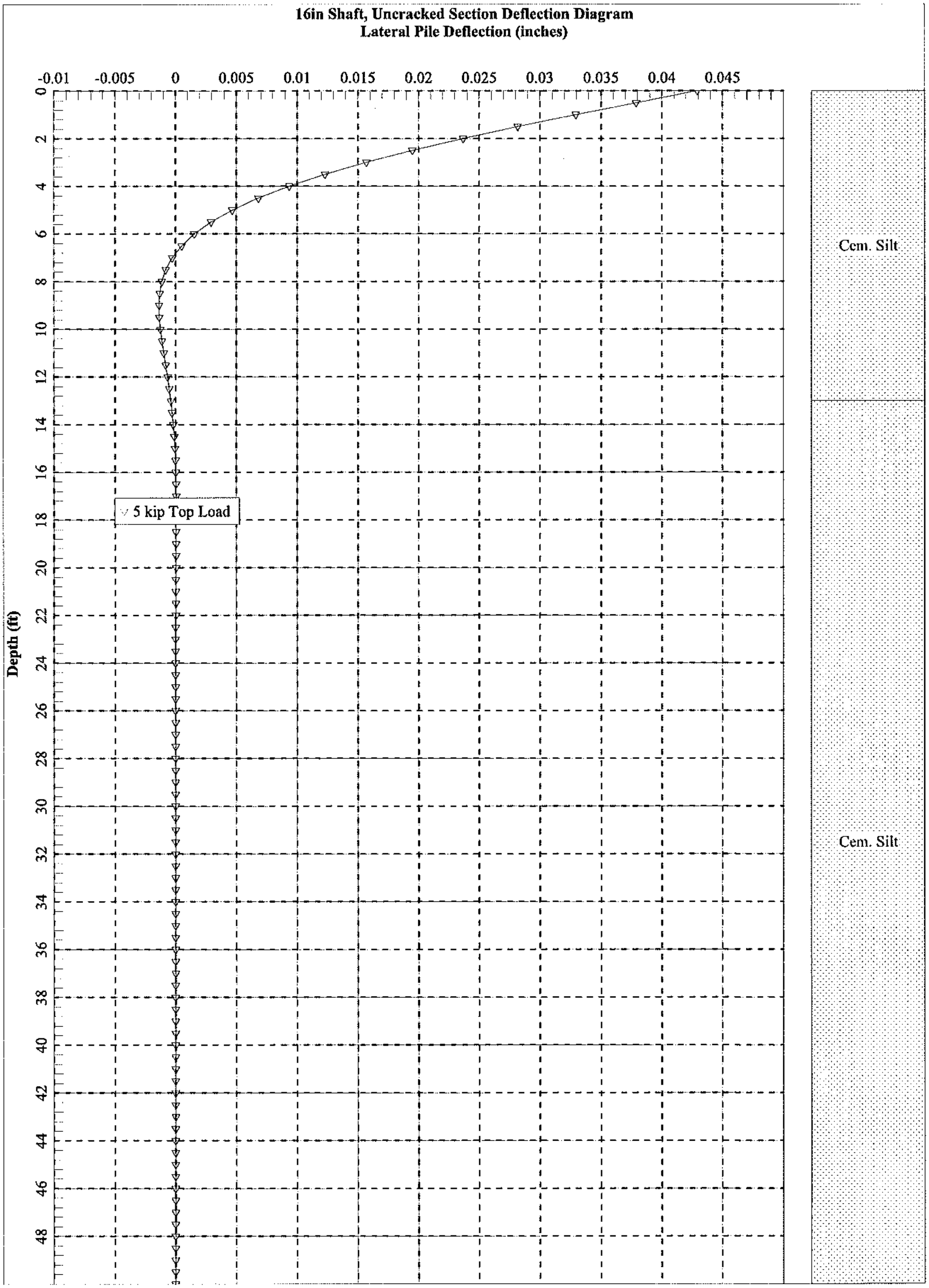
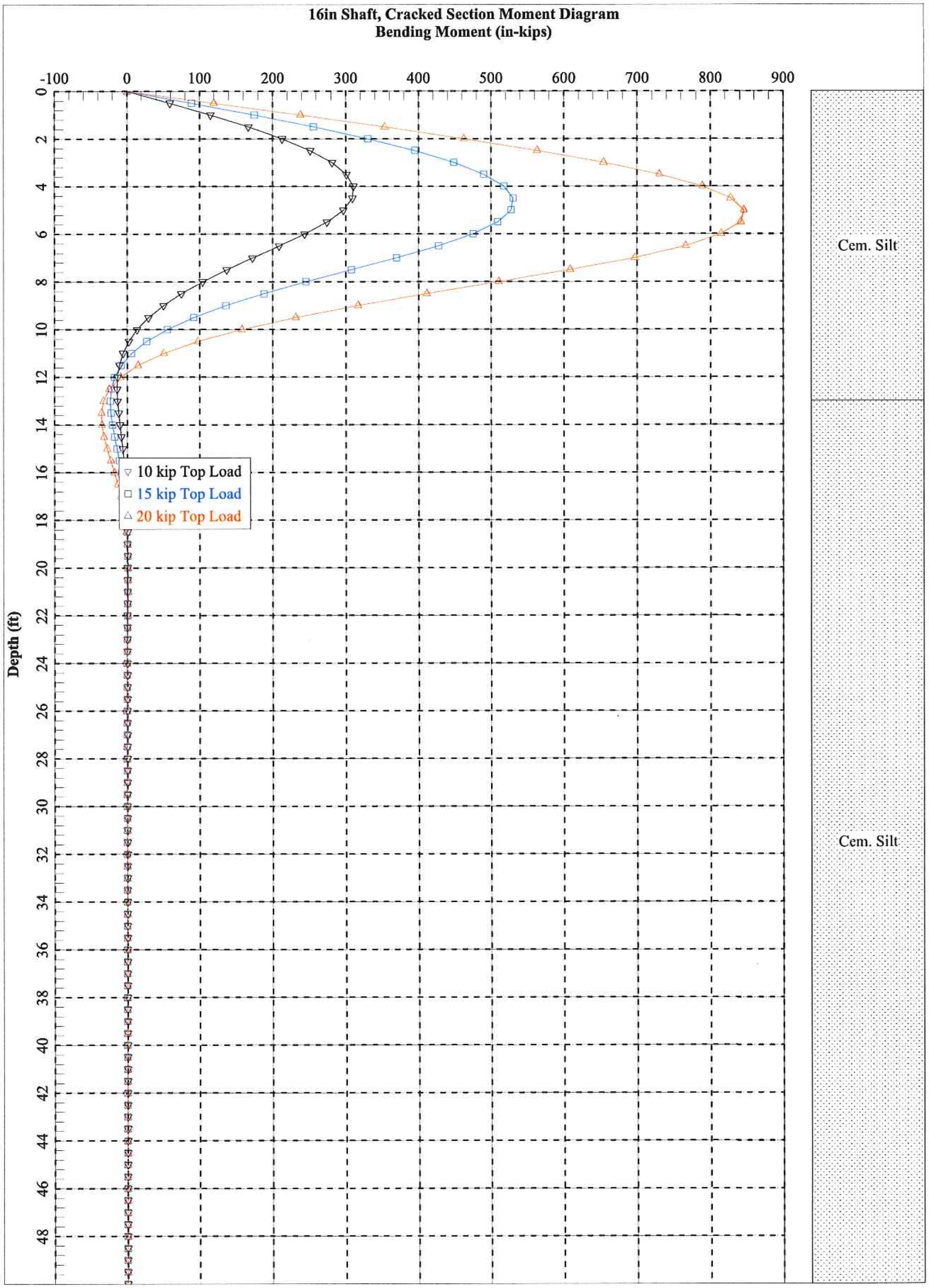

16in Shaft, Uncracked Section Moment Diagram
Bending Moment (in-kips)
0
20
40
60
80
100
120
140
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
42
44
46
48
5 kip Top Load
Cem. Silt
Cem. Silt

Figure 5



16in Shaft, Uncracked Section Deflection Diagram
Lateral Pile Deflection (inches)
-0.01
-0.005
0
0.005
0.01
0.015
0.02
0.025
0.03
0.035
0.04
0.045
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
42
44
46
48
5 kip Top Load
Cem. Silt
Cem. Silt

Figure 6



16in Shaft, Cracked Section Moment Diagram
Bending Moment (in-kips)
-100
0
100
200
300
400
500
600
700
800
900
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
42
44
46
48
10 kip Top Load
15 kip Top Load
20 kip Top Load
Cem. Silt
Cem. Silt

Figure 7

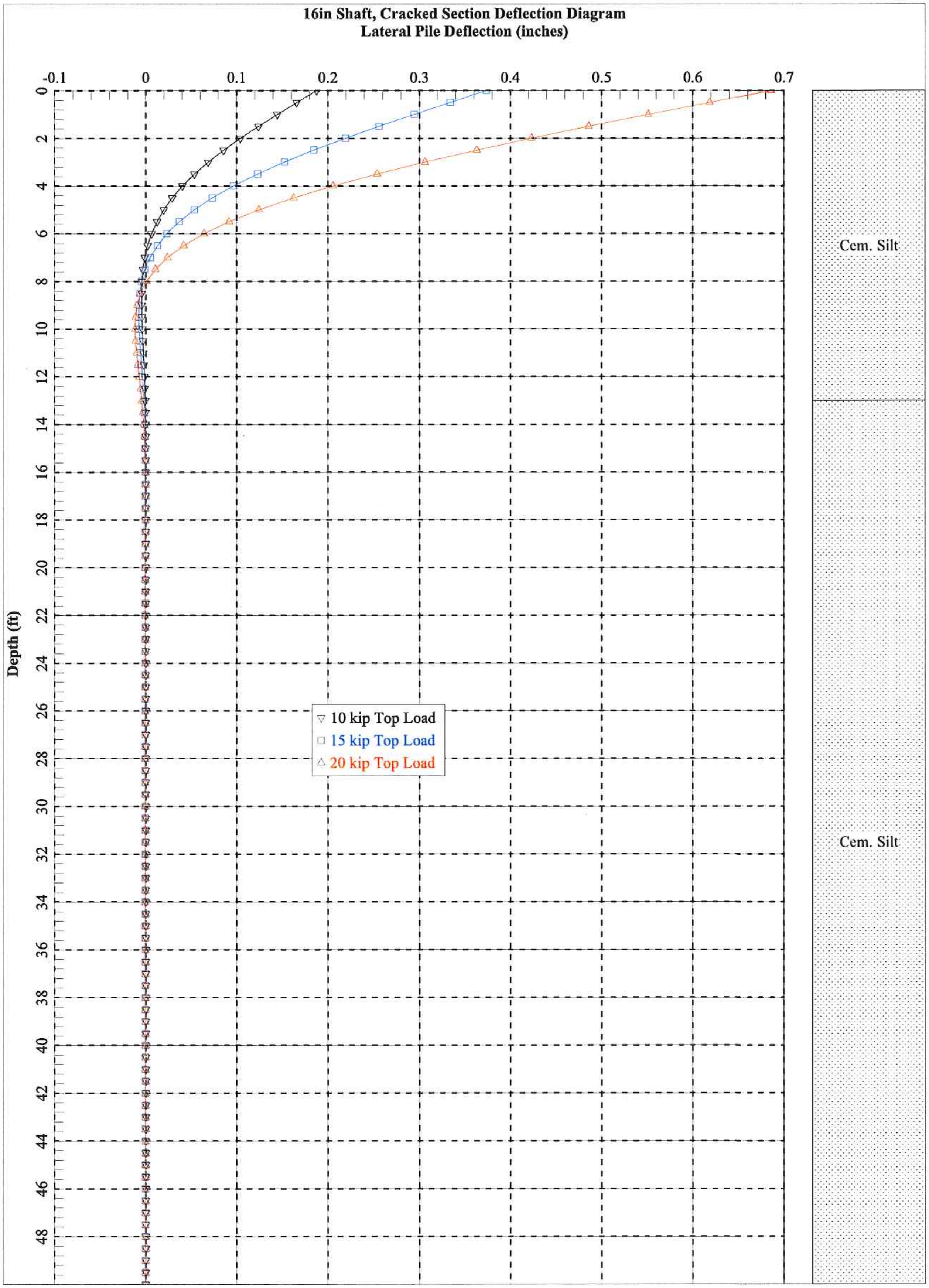

16in Shaft, Cracked Section Deflection Diagram
Lateral Pile Deflection (inches)
-0.1
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
42
44
46
48
10 kip Top Load
15 kip Top Load
20 kip Top Load
Cem. Silt
Cem. Silt

Figure 8

Geotechnical Engineering Report

Proposed Parkland Village Addition
3121 NE Cumulus Avenue
McMinnville, Oregon
June 22, 2016
Terracon Project No. 82165034

Prepared for:
RJ Development
Olympia, WA

Prepared by:
Terracon Consultants, Inc.
Portland, Oregon

June 22, 2016

RJ Development
401 Central Street SE
Olympia, Washington 98501

Attn: Mr. Joshua Snodgrass
P: (360) 528-3343 ext. 5
Josh@RJDevelopment.com

Re: Geotechnical Engineering Report
Proposed Parkland Village Addition
3121 NE Cumulus Avenue.
McMinnville, Oregon
Terracon Project No. 82165034

Dear Mr. Snodgrass:

Terracon Consultants, Inc. (Terracon) has completed the geotechnical engineering services for the above referenced project. These services were performed in general accordance with Terracon's Proposal P82165031, dated March 23, 2016. This geotechnical engineering report presents the results of the subsurface exploration and provides geotechnical recommendations concerning earthwork and the design and construction of foundations, floor slabs, and pavements for the proposed project.

We appreciate the opportunity to be of service to you on this project. If you have any questions concerning this report, or if we may be of further service, please contact us.

Sincerely,
Terracon Consultants, Inc.

REGISTERED PROFESSIONAL ENGINEER
78373PE
OREGON
JULY 11, 2006
KRISTOPHER T. HAUCK

6/22/16

Brice W. Plouse, EIT
Senior Staff Engineer

Kristopher T. Hauck, PE
Principal | Office Manager

TABLE OF CONTENTS

Page

EXECUTIVE SUMMARY i
1.0 INTRODUCTION 1
2.0 PROJECT INFORMATION 1
2.1 Project Description 1
2.2 Site Location and Description 2
3.0 SUBSURFACE CONDITIONS 2
3.1 Site Geology 2
3.1.1 Geologic Hazards 3
3.1.2 Seismic Hazards 3
3.2 Typical Subsurface Profile 4
3.3 Groundwater 4
4.0 RECOMMENDATIONS FOR DESIGN AND CONSTRUCTION 4
4.1 Geotechnical Considerations 4
4.2 Slope Stability 5
4.3 Earthwork 6
4.3.1 Site Preparation 6
4.3.2 Subgrade Preparation 7
4.3.3 Fill Material Types 8
4.3.4 Compaction Requirements 9
4.3.5 On-Site Soils 9
4.3.6 Wet-Weather Earthwork 9
4.3.7 Construction Considerations 10
4.4 Foundations 11
4.4.1 Footing Subgrade Preparation 11
4.4.2 Design Recommendations 12
4.3.2 Footing Drains 12
4.5 Floor Slabs 13
4.5.1 Design Recommendations 13
4.5.2 Construction Considerations 13
4.6 Pavements 14
4.6.1 Design Recommendations 14
4.6.2 Asphalt and Base Course Materials 15
4.6.3 Concrete Properties and Materials 15
4.6.4 Pavement Construction Considerations 15
4.7 Seismic Considerations 16
5.0 GENERAL COMMENTS 17

TABLE OF CONTENTS– continued

APPENDIX

Exhibit A-1	Site Location
Exhibit A-2	Exploration Plan
Exhibit A-3	Field Exploration Description
Exhibits A-4 to A-6	Boring logs B-1 to B-3

APPENDIX B – LABORATORY TESTING

Exhibit B-1	Laboratory Testing Description
Exhibit B-2	Atterberg Limits
Exhibit B-3 to B-5	Unconfined Compression Results
Exhibit B-6	Direct Shear Results

APPENDIX C – SUPPORTING DOCUMENTS

Exhibit C-1	General Notes
Exhibit C-2	Unified Soil Classification

APPENDIX D – SLOPE STABILITY ANALYSES RESULT

Exhibit D-1	Slope Stability Analysis
Exhibit D-2	Seismic Slope Stability Analysis

EXECUTIVE SUMMARY

Geotechnical explorations have been performed for the Proposed Parkland Addition located at 3121 NE Cumulus Avenue in McMinnville, Oregon. Terracon's geotechnical scope of work included the advancement of three geotechnical test borings to depths of up to 51½ feet below existing site grades (bgs) within the proposed development areas at the site.

The site appears suitable for the proposed construction based upon geotechnical conditions encountered in the borings and our current understanding of the proposed development. The following geotechnical considerations were identified:

- **Subsurface Conditions:** Geotechnical exploration borings B-1 through B-3 encountered native silt and sand soils throughout the depth of the borings. The native silt and sand soils are soft to stiff.
- **Structure Foundation Support:** Based on the subsurface conditions encountered at the site, the structures may be supported on conventional foundations bearing on a minimum of one foot of compacted select fill atop competent native soils. The compacted select fill is needed to limit static settlement.
- **Slope Stability:** Based on our analyses, the existing slope adjacent to the site is marginally stable. Therefore, in order to prevent adverse impacts to the existing slope and to protect the proposed development from potential slope instability, we recommend that the development incorporate a setback from the top of slope of at least 35 feet. In addition, due to the seismic risk of slope movement, the footings nearest the slope should be supported on a four foot thick geogrid-reinforced structural fill prism. If a reduced setback is desired, slope stabilization improvements would be necessary.
- Close monitoring of the construction operations discussed herein will be critical in achieving the design subgrade support. Therefore, we recommend that Terracon be retained to monitor this portion of the work.

This summary should be used in conjunction with the entire report for design purposes. It should be recognized that details were not included or fully developed in this section, and the report must be read in its entirety for a comprehensive understanding of the items contained herein. The section titled **GENERAL COMMENTS** should be read for an understanding of the report limitations.

GEOTECHNICAL ENGINEERING REPORT
PROPOSED PARKLAND VILLAGE ADDITION
3121 NE CUMULUS AVENUE
MCMINNVILLE, OREGON

Terracon Project No. 82165034
June 22, 2016

1.0 INTRODUCTION

This report presents the results of our geotechnical engineering services performed for the Proposed Parkland Village Addition to be located at 3121 NE Cumulus Avenue in McMinnville, Oregon. Our geotechnical engineering scope of work for this project included the proposed advancement of three geotechnical test borings to a maximum depth of 51½ feet below existing site grades (bgs). The purpose of our services is to provide information and geotechnical engineering recommendations relative to:

- subsurface soil conditions
- foundation settlement
- earthwork
- pavement design parameters
- slope stability
- foundation design and construction
- floor slab design and construction
- seismic site classification
- lateral earth pressure

2.0 PROJECT INFORMATION

2.1 Project Description

ITEM	DESCRIPTION
Site layout	We were provided with a site layout showing an addition to the current Parkland Assisted Living development. The development is located on the north side of the existing structure.
Structures	We understand that the expansion is expected to be a one story development in height with wood- or light gage metal-framed, with concrete slab on-grade floors.
Finish floor elevation	Not known at this time, but assumed to be near existing grades.
Maximum loads, assumed	Columns Footings: 50 to 75 kips maximum total loads (assumed) Walls: 1 to 4 kips/lf maximum total loads (assumed) Floor Slabs: 150 psf (assumed)
Maximum allowable settlement	Total: 1 inch over entire building shell footprint (assumed) Differential: ½ inch over 30 feet (assumed)

ITEM	DESCRIPTION
Grading	Undetermined at this time, but assumed to remain near existing grade.
Cut and fill slopes	None expected.
Pavements	Traffic loads undetermined, but we anticipate conventional asphalt concrete in the ground floor structure covered drive.

2.2 Site Location and Description

ITEM	DESCRIPTION
Location	The expansion site is located on the north side of the current development located at 3121 NE Cumulus Avenue in McMinnville, Oregon (Lat.: 45.203623, Long.: -123.156698).
Existing Improvements	**Site**: Developed with a single-story Senior and Assisted Living Facility encompassing the central portion of site and asphalt pavements on the remaining southern portion. **North**: Sloped to Southern Yamhill River tributary **South**: Residential developments **East**: Residential developments, then Southern Yamhill River tributary **West**: Empty field (different proposed development), then residential developments further west
Current ground cover	The ground is covered with grass and small trees.
Existing topography	The site is relatively flat. However, a steep slope approximately XX feet in overall relief is located immediately north and northeast of the site development. The slope is part of an overall drainage ravine for the Yamhill River.

3.0 SUBSURFACE CONDITIONS

A cursory review of historical aerial photographs from Google Earth shows the proposed development area has not been developed.

3.1 Site Geology

The *Oregon Department of Geology and Mineral Industries (DOGAMI) published Oregon Geologic Data Compilation-Release 5 (2009)* indicates the majority of the site is classified as the medium terrace Missoula Flood deposits (Qmt). Site geology is described as fine grained sediments. Based on our findings in the subsurface explorations, the site soils encountered are consistent with the above described Missoula Flood channel deposits.

3.1.1 Geologic Hazards

We reviewed the *Statewide Landslide Information Database for Oregon (SLIDO) published by the Oregon Department of Geology and Mineral Industries (DOGAMI) and updated* in 2014. The map also overlays *The Statewide Landslide Susceptibility Overview Map of Oregon (O-16-02) also published by DOGAMI in 2012.* The latter publication presents the landslide susceptibility in low (landsliding unlikely), moderate (landsliding possible), high (landsliding likely), and very high (existing landslide). The slope immediately north of the development is mapped as "high" landslide susceptibility.

3.1.2 Seismic Hazards

Seismic hazards resulting from earthquake motions can include slope instability, liquefaction, and surface rupture due to faulting or lateral spreading. Liquefaction is the phenomenon wherein soil strength is dramatically reduced when subjected to vibration or shaking.

We reviewed the *Relative Earthquake Hazard Maps for Selected Urban Areas in Western Oregon: McMinnville-Dayton-Lafayette (IMS-9) published by the Oregon Department of Geology and Mineral Industries (DOGAMI)* in 2000. The map evaluates the overall earthquake hazard rating based on three earthquake hazards including ground shaking amplification, liquefaction, and slope instability. The mapped categories range from Zone A for the highest overall relative earthquake hazard to Zone D for the lowest rating. Zone A indicates two or more individual earth quake hazards have a high relative hazard rating. Sites mapped as Zone B have a high rating from a single individual earthquake hazard. The subject site is mapped in an area categorized as Zone B due to a mapped high relative liquefaction hazard.

3.2 Typical Subsurface Profile

Based on the results of the borings, subsurface conditions on the project site can be generalized as follows:

Description	Approximate Depth to Bottom of Stratum	Material Encountered	Consistency/Density
Stratum 1 (Topsoil)	4 in.	Grass and root zone	N/A
Stratum 2 (Silt and Sand)	Undetermined; all borings terminated within this stratum at the planned exploration depth (maximum explored depth of 51½ feet)	Silt and Sand Mixtures	Soft to stiff and medium dense

Conditions encountered at each boring location are indicated on the individual boring logs found in Appendix A of this report. Stratification boundaries on the boring logs represent the approximate location of changes in soil types; in-situ, the transition between materials may be gradual. A discussion of field sampling procedures is included in Appendix and laboratory testing procedures and test results are presented in Appendix B.

3.3 Groundwater

Groundwater was observed from 19 to 34 feet bgs in the borings at the time of drilling and 30½ to 41¼ at completion of drilling. Groundwater level fluctuations occur due to seasonal variations in the amount of rainfall, runoff and other factors not evident at the time the borings were completed. Therefore, groundwater levels during construction or at other times may be higher or lower than the levels indicated on the boring logs. The possibility of groundwater level fluctuations should be considered when developing the design and construction plans for the project.

4.0 RECOMMENDATIONS FOR DESIGN AND CONSTRUCTION

4.1 Geotechnical Considerations

The subsurface conditions at the site were evaluated to develop geotechnical related design and construction recommendations for site development. In our opinion, the site is feasible for the proposed development provided the recommendations in this report are followed. Due to the risk of slope instability, we recommend a development (structure and grading) minimum setback of

35 feet from the top of the slope and a geogrid-reinforced fill prism for structure support be incorporated into the project details. The remaining portions of the structure could be supported on conventional spread and continuous footings bearing directly on one foot of compacted structural fill on the native stiff silt or re-compacted native soils.

The near surface native soils at the site are fine-grained and very moisture sensitive. Therefore, these soils will be difficult to reuse if overly moist (as they are in their current state) and should not be planned to be reused as structural fill. In addition, they should not be reused within the upper foot underneath floor slabs and/or footings. Recommendations for backfill are provided in the **Fill Material Types** and **Compaction** sections of this report.

4.2 Slope Stability

The existing slope below the expansion area is at an inclination that varies from approximately 25 to 80 percent and consists of silt soils with sand and organics, which we interpret to be native alluvial soils. The proposed expansion is planned to be constructed as close to the top of the slope as possible.

We evaluated the stability of the proposed slopes using the computer program Slope/W, Version 7.14, by Geo-Slope International. The Morgenstern-Price method with a rotational failure mechanism was selected since factors of safety for this method satisfy both moment and force equilibrium. Input parameters for the analysis consisted of slope geometry, geology, and ground water conditions of the slope, interpreted from our explorations, and available published information. The soil properties used in the slope stability analysis employ the Mohr-Coulomb model and are also shown on the Slope/W results sheets in Appendix D. The soil properties are based on soil strength parameters from laboratory strength testing, correlations to the index tests, SPT blow counts obtained from the borings, and our experience with similar type soils. The slope geometry was developed from plan sheet 1 developed by Civil West Engineering Services, Inc and based on aerial photographic and topographic data available from Google Earth.

In general, the calculated factor of safety is the ratio between the available soil shear resistance and the gravitational forces that tend to produce a slide. When the soil strength is equal to the slide-producing forces, a factor of safety of 1.0 would exist, and the slope would be in a state of incipient failure. An acceptable factor of safety would depend on the level of risk deemed acceptable by the owner and municipality. Typically, a static factor of safety of at least 1.5 is desired from a design standpoint for conditions where a failure could impact occupied structures and is considered acceptable for all slopes. During short-term seismic loading, a dynamic factor of safety of 1.1 is generally considered acceptable.

Seismic slope stability analyses were conducted using a horizontal seismic coefficient of 0.24g. This seismic coefficient is equal to approximately one-half of the peak ground acceleration of 0.47g, as determined for the site using 2010 ASCE 7-10 methods for a maximum considered earthquake return period of 2,475 years. The use of one-half of the site-specific peak ground acceleration (PGA) value is consistent with the standard of practice for evaluation of slope stability for non-liquefiable soils.

Our analyses indicate that the minimum factor of safety for significant slope failure landslides in the steep slopes area, occurring at or behind the assumed top of slope elevation of 96 feet extend beyond the top of the slope approximately 30 to 35 feet. Therefore, in order to not adversely impact the stability of the slope and to protect the structures from instability, we recommend all development remain a minimum of 35 feet setback from the top of the slope. In addition, we recommend incorporating a geogrid-reinforced fill prism underneath the footings closest to the top of the slope. This fill prism should consist of BX1200 geogrid (or equivalent biaxial strength geogrid) spaced 12 inches vertically within crushed aggregate base materials. The fill prism should have four layers of geogrid. The prism should extend at least 5 feet beyond the extents of the edge of footing in all directions.

Should the setback limits overly constrain site development and the client desire the development to extend closer to the top of the slope, then slope stabilization measures would need to be incorporated in the design of the development. These typically consist of buried piles extending through the potential slide failure plans and can be quite costly. The pile wall improvements would need to be designed to overcome the active or at-rest pressures, depending on foundation set back from improvements, during a static and seismic event.

4.3 Earthwork

The following sections present recommendations for site preparation, excavation, subgrade preparation, placement and compaction of structural fill, and grading. The recommendations presented for design and construction of earth supported elements are contingent upon following the recommendations outlined in this section.

4.3.1 Site Preparation

Site preparation and initial construction activities should be planned to reduce disturbance to the existing ground surface. Construction traffic should be restricted to dedicated driveway and laydown areas. Preparation should begin with procedures intended to drain ponded water and control surface water runoff.

Site preparation will require removing stripping and grubbing of the vegetative layer within the effective development areas. If existing facilities or utility lines are encountered during construction activities, existing features shall be removed within the building pad limits, they should be properly capped at the site perimeter, and the trenches should be backfilled in accordance with structural fill recommendations presented in Sections 4.3.3 and 4.3.4 of this report. If unexpected fills are encountered within proposed development areas, affected areas should be removed and the excavation thoroughly cleaned prior to backfill placement and/or construction unless evaluated and tested by an authorized Terracon representative.

In the event the exposed subgrade becomes unstable, yielding, or disturbed, we recommend that the materials be removed to a sufficient depth in order to develop stable subgrade soils that can be compacted to the minimum recommended levels. The severity of construction problems will be dependent, in part, on the precautions that are taken by the contractor to protect the subgrade soils.

4.3.2 Subgrade Preparation

Strip and remove existing vegetation, existing fill, topsoil, pavements, and other deleterious materials from the proposed development areas. Existing fill soils may remain within the non-building areas provided they are prepared according the following sections. Stripping depths to remove existing vegetation within the expansion pad are anticipated to be an average of about 4 to 6 inches, but may vary across the site and could be deeper. Areas where loose or soft surface soils exist, they should be compacted or removed and replaced to the depth of the disturbance as subsequently recommended for structural fill.

Excavations for footings should be completed to expose medium stiff silt materials and should be covered with the recommended granular select fill to prevent significant drying. The excavations should be observed for visual classification and T-probing by a representative of Terracon to confirm suitable subgrades for bearing support of foundations.
The upper one foot of pavement subgrades should be scarified and re-compacted to levels described in the **Compaction Requirements** section of this report after cutting to design subgrade elevation. We also recommend testing include proof-rolling to aid in the identification of weak or unstable areas within the near surface soils at the exposed subgrade level. Proof-rolling should be performed using heavy rubber-tired equipment, such as a fully-loaded dump truck, having a minimum gross weight of about 20 tons. Unsuitable areas observed at this time which are soft, yielding, or unable to be compacted to the specified criteria should be over-excavated and replaced with satisfactory fill material later described in section 4.3.3 of this report.

Based on the outcome of the proof-rolling operations, some undercutting or subgrade stabilization may be expected, especially during wet periods of the year. Methods of stabilization, which are outlined below, could include scarification and re-compaction and/or removal of unstable materials and replacement with granular fill (with or without geotextiles). The most suitable method of stabilization, if required, will be dependent upon factors such as schedule, weather, size of area to be stabilized and the nature of the instability.

- **Scarification and Re-compaction** - It may be feasible to scarify, dry, and re-compact the exposed sand soils at the site during periods of dry weather. The success of this procedure would depend primarily upon the extent of the disturbed area. Stable subgrades may not be achievable if the thickness of the soft soil is greater than about 1 to 1½ feet.

- **Granular Fill** - The use of crushed stone or gravel could be considered to improve subgrade stability. Typical undercut depths would range from about ½ foot to 2 feet. The use of high modulus geotextiles i.e., engineering fabric, should be limited to outside of the Building Ground Improvements area. The maximum particle size of granular material placed immediately over geotextile fabric or geogrid should not exceed 2 inches.
- **Chemical Stabilization** - Improvement of subgrades with Portland cement, lime kiln dust, or Class C fly ash could be considered for unstable and plastic soils. Chemical modification should be performed by a pre-qualified contractor having experience with successfully stabilizing subgrades in the project area on similar sized projects with similar soil conditions.

Over-excavations should be backfilled with structural fill material placed and compacted in accordance with sections 4.3.3 and 4.3.4 of this report. Subgrade preparation and selection, placement, and compaction of structural fill should be performed under engineering controlled conditions in accordance with the project specifications.

4.3.3 Fill Material Types

Engineered or structural fill should meet the following material property requirements:

Fill Type [1]	Specification	Acceptable for Placement
Common Fill[2,3]	2015 Oregon Standard Specification for Construction (OSSC) 00330.13 Selected General Backfill with the additional requirements of Liquid Limits < 40 and Plasticity Index < 10	All locations across the site, with the exception of floor and pavement base materials Dry Weather only.
Select Fill[2]	OSSC 00330.14 Selected Granular Backfill with exception of no more than 8% passing the No. 200 sieve by weight and reclaimed glass is not acceptable	All locations across the site, Wet Weather and Dry Weather acceptable.
Crushed Aggregate Base (CAB)	OSSC 02630.10 Dense Graded Aggregate (2"-0 to ¾"-0) with exception of no more than 8% passing the No. 200 sieve by weight	All locations across the site. Recommended for finished base course materials for floor slabs and pavements. Wet Weather and Dry Weather acceptable.

Fill Type [1]	Specification	Acceptable for Placement

1. Controlled, compacted fill should consist of approved materials that are free (free = less than 3% by weight) of organic matter and debris (i.e. wood sticks greater than ¾-inch in diameter). Frozen material should not be used, and fill should not be placed on a frozen subgrade. A sample of each material type should be submitted to the geotechnical engineer for evaluation.
2. Materials within 1-foot of floor slabs base, pavement base, and footings should have a maximum particle size of 3-inches.

If open-graded materials with large void spaces, such as quarry spalls, are used we recommend that the materials be placed over a geotextile fabric separator to prevent fines migration as well as to stabilize the subgrade. The geotextile fabric should be a woven product (Mirafi 500XT or equivalent).

4.3.4 Compaction Requirements

The following compaction requirements are recommended for the prepared subgrade and structural fill expected to be placed for this site:

Item	Description
Fill Lift Thickness	**Common Fill, Select Fill and CAB:** 10-inches or less in loose thickness when heavy, compaction equipment is used.
Compaction Requirements [1]	**Common Fill, Select Fill & CAB:** 95% of the material's maximum Proctor dry density (ASTM D1557) below building pad and upper two feet of site pavements. 92% of the materials maximum Proctor dry density (ASTM D1557) elsewhere.
Moisture Content	**Common Fill, Select Fill and CAB:** Within ±2 percent of optimum moisture content as determined by ASTM D1557.

1. We recommend that fill be tested for moisture content and compaction during placement. Should the results of the in-place density tests indicate the specified moisture or compaction limits have not been met, the area represented by the test should be reworked and retested as required until the specified moisture and compaction requirements are achieved.

4.3.5 On-Site Soils

Our explorations indicated that the on-site soils will likely consist of silt and sand soils. At the time of our exploration, moisture contents in the upper soils zone were found to generally range from approximately 21 to 34 percent, which we infer to be well above their optimum moisture content. Therefore, most on-site soils will likely be reusable only during dry weather if they can be adequately dried, but they will be difficult or impossible to reuse during wet weather. Any zones containing significant amounts of wood, asphalt, or other waste products should be excluded from reuse as structural fill.

4.3.6 Wet-Weather Earthwork

As discussed above, the on-site fine-grained native soils would be difficult to reuse as structural fill during wet weather and are likely precluded from use within the building pad over-excavation.

Consequently, the project specifications should include provisions for using imported, clean, granular fill. As a general structural fill material, we recommend using the crushed aggregate base courses meeting the Oregon Standard Specifications section 02630.10, which are readily available in the region, although some local sources of pit-run or bank-run may be available. The use of high modulus geotextiles (i.e., engineering fabric such as Mirafi HP370) may be used to aid in stabilization of the subgrade. To reduce the potential for subgrade disturbance during wet-weather periods, contractor should install haul roads consisting of clean, crushed rock at a minimum depth of 18 inches. Haul roads install and intended to be incorporated into final pavement section shall be evaluated for conformance with sections 4.3.2 thru 4.3.4 prior to placement of crushed rock.

4.3.7 Construction Considerations

Native fine grained soils were encountered near the surface across the site and were observed to consist of silt and fine sands and in a moisture condition much greater than about 2 percent over optimum moisture content. Therefore, the fine grained site soils are considered to be moisture sensitive and will be difficult or impossible to compact as structural fill. Accordingly, the fine-grained soils from site excavations are not considered suitable as granular fill in footing areas, their use in non-footing areas will depend on their moisture content at the time of earthwork, the prevailing weather conditions when site grading activities take place, and the proposed location for reuse. The onsite granular soils may be suitable for reuse as structural fill in building areas if the material is in accordance with the **Fill Material Types** section of this report.

Even if stable subgrades are exposed during construction, unstable subgrade conditions could develop during general construction operations, particularly if the soils are wetted and/or subjected to repetitive construction traffic. The use of light construction equipment would aid in reducing subgrade disturbance. The use of remotely operated equipment, such as a backhoe, would be beneficial to perform cuts and reduce subgrade disturbance. If the subgrade should become frozen, desiccated, saturated, or disturbed, stabilization measures will need to be employed.

The contractor is responsible for designing and constructing stable, temporary excavations (including utility trenches) as required to maintain stability of both the excavation sides and bottom. Excavations should be sloped or shored in the interest of safety following local and federal regulations, including current OSHA excavation and trench safety standards. Care should be taken when excavating near adjacent structures or right-of-ways. If excavations will encroach below a 1H:1V plane below the foundations of adjacent structures or right-of-ways, the contractor should be prepared to provide temporary shoring designed to resist the structure or traffic surcharge loads.

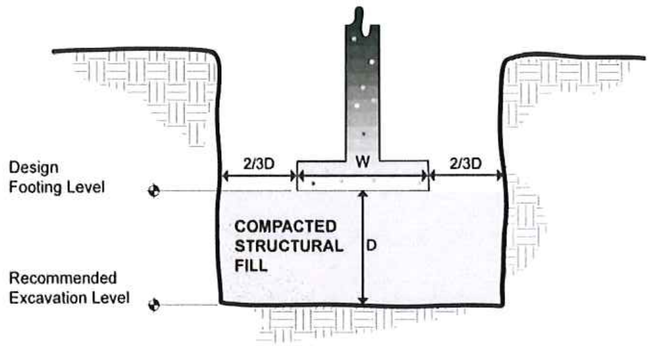
The geotechnical engineer should be retained during the construction phase of the project to observe earthwork and to perform necessary tests and observations during subgrade preparation, probing, placement and compaction of controlled compacted fills, and backfilling of excavations to the completed subgrade.

4.4 Foundations

The proposed structures may be supported by isolated spread footings and continuous footings bearing on one foot of granular structural fill (Select Fill) over the medium stiff native silts or re-compacted silt with sand subgrade soils. As discussed in the Slope Stability section of the report, the footings closest to the top of the slope should be supported on the geogrid-reinforced structural fill prism. Design recommendations for foundations for the proposed structures and related structural elements are presented in the following sections.

4.4.1 Footing Subgrade Preparation

Unsuitable bearing soils were encountered in the near surface of our explorations. The footing excavation should be extended one foot in depth and be replaced with compacted structural fill. The footings should bear on properly compacted structural backfill extending down to the stiff native soils or scarified and re-compacted subgrade soils to a depth of one foot. Foundations should not be supported on soft or loose soils or existing fill soils. Over-excavation for compacted backfill placement below footings should extend laterally beyond all edges of the footings at least 8 inches per foot of over-excavation depth below footing base elevation. Zones of loose, soft or otherwise unsuitable soil encountered in or below the footing subgrade should be over-excavated and replaced with properly compacted Select Fill.



Overexcavation / Backfill

NOTE: Excavations in sketches shown vertical for convenience. Excavations should be sloped as necessary for safety.

The compactive effort should be in accordance with recommendations provided in the **4.33 Earthwork** section of this report.

4.4.2 Design Recommendations

DESCRIPTION	Column	Wall
Net allowable bearing pressure [1] One foot of granular structural fill placed directly above the competent native soils	2,500 psf	2,500 psf
Minimum dimensions	2 feet	12 inches
Minimum embedment below finished grade for frost protection [2]	12 inches	12 inches
Approximate total static settlement [3]	<1 inch	<1 inch
Estimated differential settlement [3]	<¾ inch between columns	<½ inch over 30 feet
Allowable passive pressure [4]	230 psf/ft	
Allowable coefficient of sliding friction [4]	0.33	

1. The recommended net allowable bearing pressure is the pressure in excess of the minimum surrounding overburden pressure at the footing base elevation. Assumes any unsuitable fill or soft soils, if encountered, will be undercut and replaced with structural fill. Assumes native soils will be undercut 1 foot and replaced with structural fill.
2. And to reduce the effects of seasonal moisture variations in the subgrade soils. For exterior footings and footings beneath unheated areas.
3. The foundation settlement will depend upon the variations within the subsurface soil profile, the structural loading conditions, the embedment depth of the footings, the thickness of compacted fill, and the quality of the earthwork operations. The above settlement estimates have assumed that the maximum footing size is 4 feet for column footings and 1.5 foot for continuous footings.
4. The value presented is an equivalent fluid pressure. The sides of the excavation for the spread footing foundation must be nearly vertical and the concrete should be placed neat against these vertical faces for the passive earth pressure values to be valid. Passive resistance in the upper 12 inches of the soil profile should be neglected.

The net allowable bearing pressures presented in the table above may be increased by one-third to resist transient, dynamic loads such as wind or seismic forces. Please note that lateral resistance to footings should be ignored in the upper 12-inches from finish grade.

4.3.2 Footing Drains

We recommend that footings drains be installed around the perimeter of the proposed building at the base of the foundations. Footing drains should consist of a minimum 4-inch diameter, Schedule 40, rigid, perforated PVC pipe placed at the base of the heel of the footing with the perforations facing down. The pipe should be surrounded by a minimum of 4 inches of clean free-draining granular material. We recommend enveloping the drain rock with a non-woven geotextile, such as Mirafi 140N, or equivalent. Footing drains should be directed toward appropriate storm water drainage facilities. Water from downspouts and surface water should be independently collected and routed to a suitable discharge location.

4.5 Floor Slabs

We understand that the structures typically include construction of slabs-on-grade floors. The following design recommendations are provided for newly constructed concrete slabs.

4.5.1 Design Recommendations

ITEM	DESCRIPTION
Interior floor system	Concrete slab-on-grade.
Base / Capillary Break	6-inches of CAB material (¾"-0)
Modulus of subgrade reaction	125 pci for point load conditions

1. The concrete slab design should include a capillary break, comprised of free-draining, compacted, granular material, at least 6 inches thick. Free-draining granular material should have less than 5 percent fines (material passing the #200 sieve).

Where appropriate, saw-cut control joints should be placed in the slab to help control the location and extent of cracking. For additional recommendations refer to the ACI Design Manual. Joints or any cracks in pavement areas that develop should be sealed with a water-proof, non-extruding compressible compound specifically recommended for heavy duty concrete pavement and wet environments.

The use of a vapor retarder or barrier should be considered beneath concrete slabs on grade that will be covered with wood, tile, carpet or other moisture sensitive or impervious coverings, or when the slab will support equipment sensitive to moisture. When conditions warrant the use of a vapor retarder, the slab designer and slab contractor should refer to ACI 302 and ACI 360 for procedures and cautions regarding the use and placement of a vapor retarder/barrier.

4.5.2 Construction Considerations

On most project sites, the site grading is accomplished relatively early in the construction phase. Fills are placed and compacted in a uniform manner. However, as construction proceeds, excavations for utilities are made into these areas, rainfall and surface water saturates some areas, heavy traffic from concrete trucks and other delivery vehicles disturbs the subgrade and many surface irregularities are filled in with loose soils to improve trafficability temporarily. As a result, the floor slab subgrades, initially prepared early in the project, should be carefully re-evaluated as the time for floor construction approaches.

4.6 Pavements

4.6.1 Design Recommendations

Traffic patterns and anticipated loading conditions were not available at the time this report was prepared. We anticipate that traffic loads will be produced primarily by automobile traffic and occasional delivery trucks. The thickness of pavements subjected to heavy truck traffic should be determined using expected traffic volumes, vehicle types, and vehicle loads and should be in accordance with local, city or county ordinances.

Pavement thickness can be determined using AASHTO, Asphalt Institute and/or other methods if specific wheel loads, axle configurations, frequencies, and desired pavement life are provided. Terracon can provide thickness recommendations for pavements for loads other than personal vehicles and occasional delivery truck if provided.

Listed below are minimum pavement component thicknesses, which may be used as a guide for pavement systems at the site for typical commercial building traffic patterns. It should be noted that these systems were derived based on general characterization of the subgrade as predominantly fine-grained. No specific testing (such as CBR, resilient modulus test, etc.) was performed for this project to evaluate the support characteristics of the subgrade.

MINIMUM PAVEMENT THICKNESSES		
COMPONENT	Material Thickness, Inches	
	Automobile Parking Areas	Drive Lanes
Asphalt Concrete	3	4
Crushed Aggregate Base (CAB)	8	8

Prior to placement of the CAB the pavement subgrades should be prepared as per the recommendations in the **Earthwork** section of this report. Long term pavement performance will be dependent upon several factors, including maintaining subgrade moisture levels and providing for preventive maintenance. The following recommendations should be considered the minimum:

- The subgrade and the pavement surface have a minimum ¼-inch per foot slope to promote proper surface drainage;
- Consider appropriate edge drainage and pavement under drain systems;
- Install joint sealant and seal cracks immediately;
- Seal all landscaped areas in, or adjacent to pavements to minimize or prevent moisture migration to subgrade soils;
- Placing compacted, low permeability backfill against the exterior side of curb and gutter.

Preventive maintenance should be planned and provided for through an on-going pavement management program. Preventive maintenance activities are intended to slow the rate of pavement deterioration, and to preserve the pavement investment. Preventive maintenance consists of both localized maintenance (e.g. crack and joint sealing and patching) and global maintenance (e.g. surface sealing). Preventive maintenance is usually the first priority when implementing a planned pavement maintenance program and provides the highest return on investment for pavements. Prior to implementing any maintenance, additional engineering observation is recommended to determine the type and extent of preventive maintenance.

As previously stated, haul roads and laydown areas should be included in project planning to provide access to the building area during construction.

4.6.2 Asphalt and Base Course Materials

Specifications for manufacturing and placement of pavements and crushed aggregate base course should conform to specifications presented in Section 00745, of the 2015 Oregon Standard Specifications for Construction. All base course materials should be compacted to at least 95 percent of the maximum dry density determined in accordance with ASTM D1557. We recommend that all base courses be proofrolled with a loaded dump truck prior to placing the following lift of material. We recommend that asphalt be compacted to a minimum of 92 percent of the Rice (theoretical maximum) density.

4.6.3 Concrete Properties and Materials

Concrete pavement design recommendations are based on an assumed modulus of rupture of 580 psi and a minimum 28-day compressive strength of 4,000 psi for the concrete. It is our opinion that concrete pavements should be reinforced and have relatively closely spaced control joints on the order of 15 to 20 feet. We recommend that minimum reinforcement consist of 6x6-W2.0xW2.0 welded wire or equivalent. The welded wire reinforcement should be terminated 3 inches on either side of all construction, contraction and expansion joints. Construction Considerations

4.6.4 Pavement Construction Considerations

On most project sites, the site grading is accomplished relatively early in the construction phase. Fills are placed and compacted in a uniform manner. However, as construction proceeds, excavations are made into these areas, rainfall and surface water saturates some areas, heavy traffic from concrete trucks and other delivery vehicles disturbs the subgrade and many surface irregularities are filled in with loose soils to improve trafficability temporarily. As a result, the pavement subgrades, initially prepared early in the project, should be carefully evaluated as the time for pavement construction approaches.

We recommend the entire pavement subgrade should be scarified and re-compacted as recommended in 4.3 of this report to provide a uniform subgrade for pavement construction. Areas that appear severely desiccated following site stripping may require further undercutting and moisture conditioning.

After scarification and re-compaction of subgrade soils, moisture content and density of the top 12 inches of the subgrade soils be evaluated and the pavement subgrades be proof-rolled prior to commencement of crushed aggregate base placement. Areas not in compliance with the required ranges of moisture or density should be moisture conditioned and re-compacted. Particular attention should be paid to high traffic areas that were rutted and disturbed earlier and to areas where backfilled trenches are located. Proof-roll testing should be performed by a qualified representative of Terracon at time of subgrade completion. Subgrade soils subjected to proof-roll testing should not exhibit pumping, yielding or deflection of greater than 1 inch in magnitude. Areas where unsuitable conditions are located should be repaired by removing and replacing the materials with properly compacted fills.

If a significant precipitation event occurs after the evaluation of subgrade soils or if the surface becomes disturbed, the subgrade should be reviewed by qualified personnel immediately prior to paving. The subgrade should be in its finished form at the time of the final review.

4.7 Seismic Considerations

DESCRIPTION	VALUE
2012 International Building Code Site Classification (IBC) [1]	D[2]
Site Latitude	N 45.203623
Site Longitude	W 123.156698
S_s Spectral Acceleration for a Short Period	0.991
S_1 Spectral Acceleration for a 1-Second Period	0.466
F_a site coefficient	1.104
F_v site coefficient	1.534
Peak Ground Acceleration (PGA)	0.452
Site Specific Coefficient (F_{PGA})	1.048

1. In general accordance with the *2012 International Building Code,* Table 1613.5.2. IBC Site Class is based on the average characteristics of the upper 100 feet of the subsurface profile.
2. The 2012 International Building Code (IBC) requires a site soil profile determination extending to a depth of 100 feet for seismic site classification. The current scope does not include the required 100 foot soil profile determination. Borings extended to a maximum depth of about 51½ feet, and this seismic site class definition considers that dense soil as noted on the published geologic mapping continues below the maximum depth of the subsurface exploration. Additional exploration to deeper depths would be required to confirm the conditions below the current depth of exploration. Therefore, we would interpret that site soils encountered at the site are representative of the soils to a depth of 100 feet.

Earthquake-Induced Soil Liquefaction

Liquefaction is the phenomenon wherein soil strength is dramatically reduced when subjected to vibration or shaking. Liquefaction generally occurs in saturated, loose sand and soft to medium stiff, low plasticity silt deposits. Based on the soft to medium stiff non-plastic silt with sand soils encountered from approximately 20 to 31½ feet bgs in the borings and depth to groundwater (between 19 and 34 feet), it is our opinion that the risk of liquefaction at the site is low due the moderate plasticity of the remaining site soils and we have therefore classified the site as a Site Class D.

5.0 GENERAL COMMENTS

Terracon should be retained to review the final design plans and specifications so comments can be made regarding interpretation and implementation of our geotechnical recommendations in the design and specifications. Terracon also should be retained to provide observation and testing services during grading, excavation, foundation construction and other earth-related construction phases of the project.

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

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

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

APPENDIX A

FIELD EXPLORATION

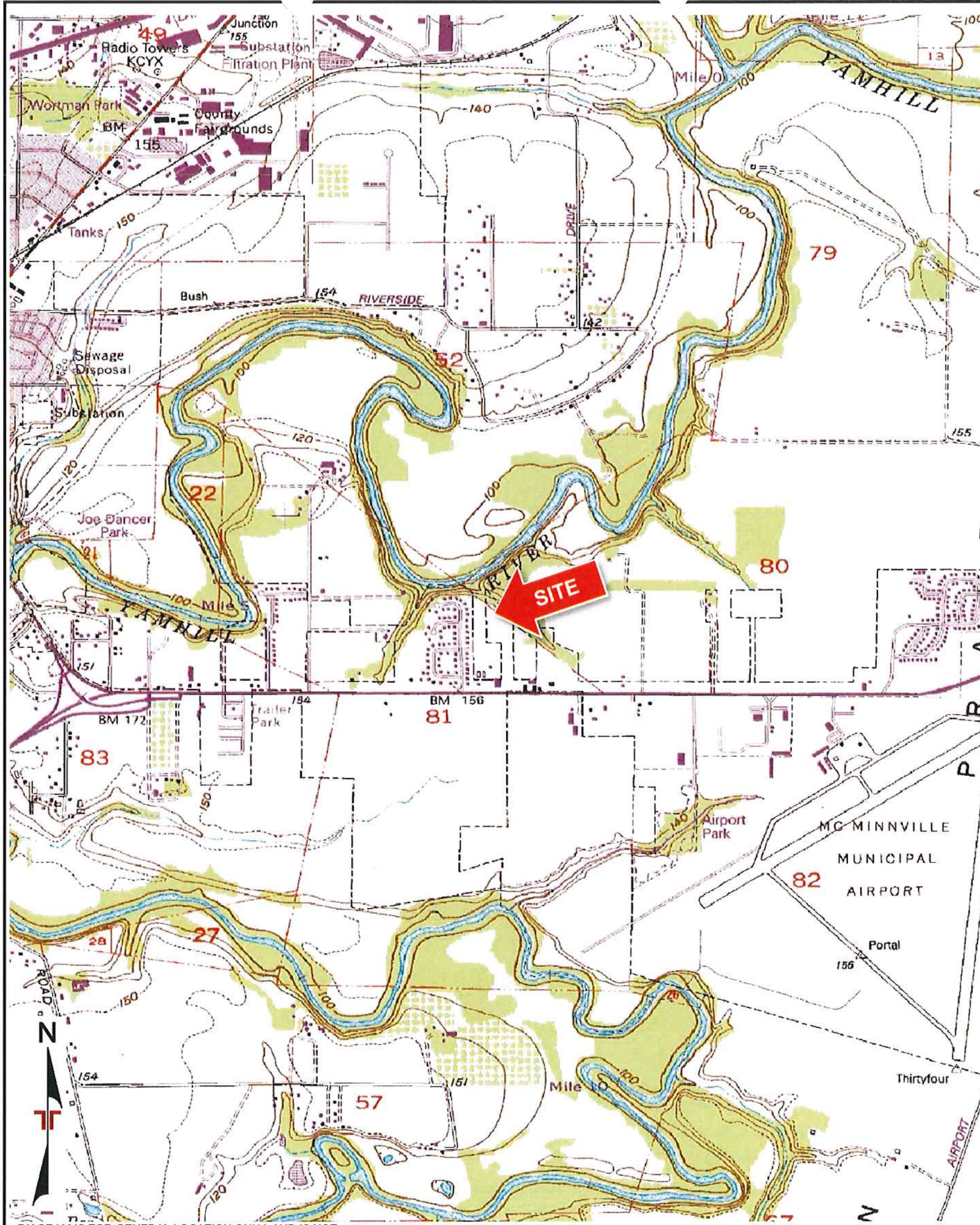


DIAGRAM IS FOR GENERAL LOCATION ONLY, AND IS NOT INTENDED FOR CONSTRUCTION PURPOSES

Project Manager: KTH	Project No. 82165034	Terracon 4103 SE International Way, Suite 300 Portland, Oregon 97222	SITE LOCATION	Exhibit
Drawn by: JAE	Scale: N.T.S		PARKLAND EXPANSION 3123 NE CUMULUS AVENUE MCMINNVILLE, OR	A-1
Checked by: BWP	File Name: EXH. A-1			
Approved by: KTH	Date: APRIL 2016			

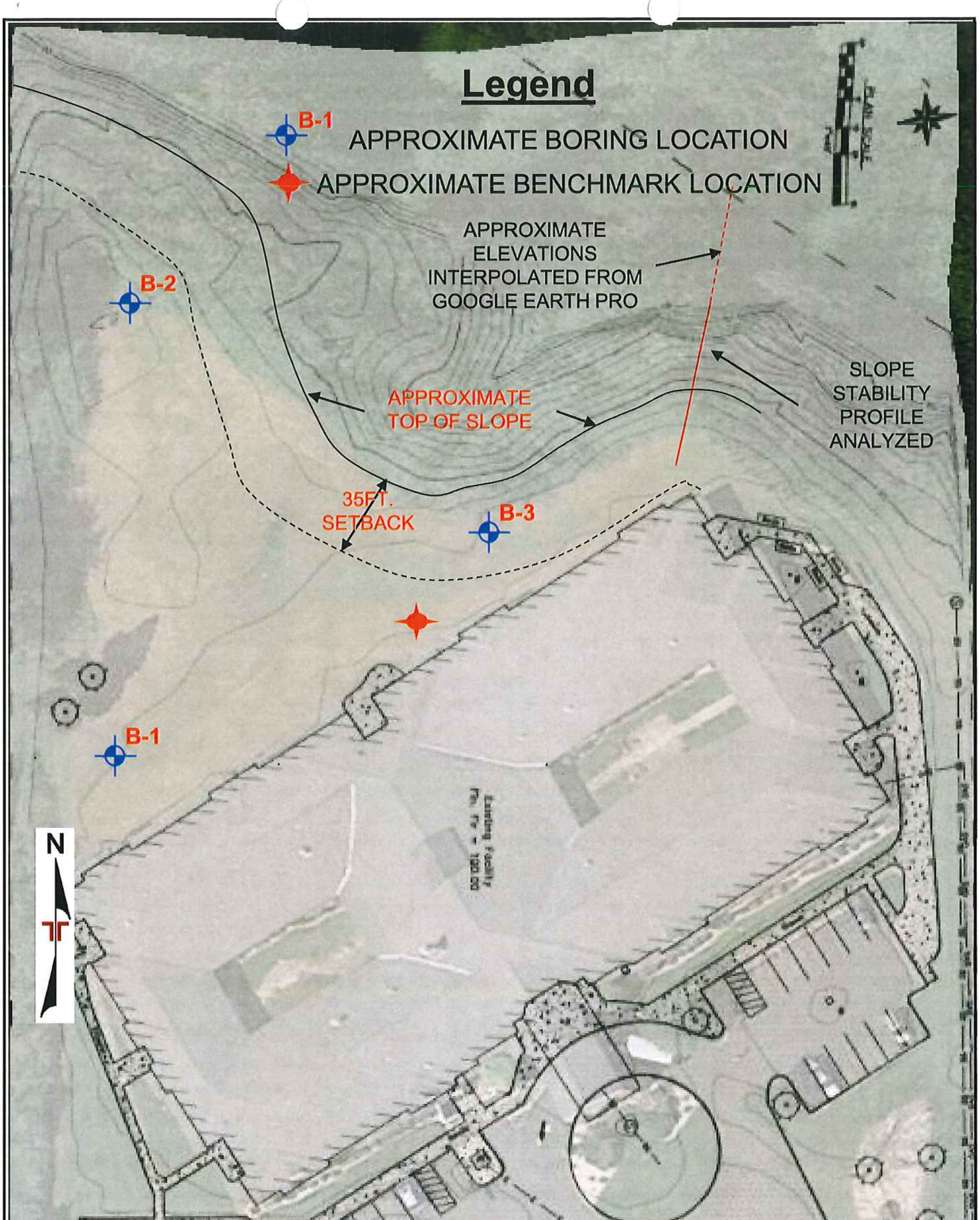


DIAGRAM IS FOR GENERAL LOCATION ONLY, AND IS NOT INTENDED FOR CONSTRUCTION PURPOSES

Project Manager: KTH	Project No. 82165034	Terracon	SITE AND EXPLORATION PLAN	Exhibit
Drawn by: JAE	Scale: N.T.S	4103 SE International Way, Suite 300	PARKLAND EXPANSION	A-2
Checked by: BWP	File Name: EXH. A-2	Portland, Oregon 97222	3123 NE CUMULUS AVENUE	
Approved by: KTH	Date: JUNE 2016		MCMINNVILLE, OR	

Field Exploration Description

The boring locations were located in the field by Terracon personnel based on estimated dimensions from site features and the provided site plan by RJ Development. Terracon personnel estimated ground surface elevations of the borings (based on a site specific assumed elevation of 100 feet at the irrigation control valve on the north side of the property; see attached exhibit A-2) by using an engineer's level and rod. The locations and elevations of the borings should be considered accurate only to the degree implied by the means and methods used to define them.

The borings were drilled with a track mounted hollow-stem auger drill rig under subcontract to Terracon. A field engineer from our firm continuously observed the borings, logged the subsurface conditions, and obtained representative soil samples. Samples of the soil encountered in the borings were obtained using the split barrel and thin-walled tube sampling procedures. The samples were stored in moisture tight containers and transported to our laboratory for further visual classification and testing. After we logged each boring, the operator backfilled each boring in general conformance with local regulations and patched the surface to match the existing ground surface.

In the split-barrel sampling procedure, the number of blows required to advance a standard 2-inch O.D. split-barrel sampler the last 12 inches of the typical total 18-inch penetration by means of a 140-pound auto-hammer with a free fall of 30 inches, is the standard penetration resistance value (SPT-N). This value is used to estimate the in-situ relative density of cohesionless soils and consistency of cohesive soils. An automatic safety hammer used to advance the split-barrel sampler in the borings performed on this site.

In the thin-walled tube sampling procedure, a thin-walled, seamless steel tube with a sharp cutting edge is pushed hydraulically into the soil to obtain a relatively undisturbed sample. The samples were tagged for identification, sealed to reduce moisture loss, and taken to our laboratory for further examination, testing, and classification. Information provided on the boring logs attached to this report includes soil descriptions, consistency evaluations, boring depths, sampling intervals, and groundwater conditions.

A field log of each boring was prepared by the field engineer. These logs included visual classifications of the materials encountered during drilling as well as the driller's interpretation of the subsurface conditions between samples. Final boring logs included with this report represent the engineer's interpretation of the field logs and include modifications based on laboratory observation and tests of the samples.

BORING LOG NO. B-1

PROJECT: Parkland Assited Living Expansion

CLIENT:

SITE:
McMinnville, Oregon

LOCATION See Exhibit A-2

Latitude: 45.20384° Longitude: -123.15731°

DEPTH	Description	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	RECOVERY (In.)	FIELD TEST RESULTS	STRENGTH TEST: TEST TYPE	COMPRESSIVE STRENGTH (psf)	STRAIN (%)	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
0.3	**TOPSOIL**, 3-inch Grass and Root Zone												
	SILT WITH SAND (ML), brown with orange mottling, stiff			SPT	8	2-5-8 N=13				22			
2.5													
	SANDY SILT (ML), gray with orange mottling, medium stiff			SPT	12	2-2-3 N=5				26			
	stiff	5		SPT	15	3-4-6 N=10				34			
	brownish gray			SPT	18	4-5-6 N=11				32			
		10		Tube						35	86		
				SPT	18	3-4-5 N=9				34			
	light brown, medium stiff, trace mica	15		SPT	15	3-3-3 N=6				35			
			▽										
		20		Tube						35	85		
	medium stiff to stiff			SPT	15	2-2-6 N=8				37			
		25		SPT	15	2-3-4 N=7				43			
26.5	***Boring Terminated at 26.5 Feet***												

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic SPT Hammer

Advancement Method:
Hollow stem auger

Abandonment Method:
Borings backfilled with bentonite chips upon completion

See Exhibit A-3 for description of field procedures.
See Appendix B for description of laboratory procedures and additional data (if any).
See Appendix C for explanation of symbols and abbreviations.

Notes:

WATER LEVEL OBSERVATIONS

▽ *While drilling*

Terracon
4103 SE International Way Ste 300
Portland, OR

Boring Started: 4/5/2016	Boring Completed: 4/5/2016
Drill Rig: D-50 track	Driller: Terracon
Project No.: 82165034	Exhibit: A-4

BORING LOG NO. B-2

PROJECT: Parkland Assited Living Expansion

CLIENT:

SITE:
McMinnville, Oregon

LOCATION See Exhibit A-2
Latitude: 45.20433° Longitude: -123.1572°

GRAPHIC LOG / DEPTH	Description	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	RECOVERY (In.)	FIELD TEST RESULTS	STRENGTH TEST: TEST TYPE	COMPRESSIVE STRENGTH (psf)	STRAIN (%)	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
0.3	**TOPSOIL**, 3-inch Grass and Root Zone												
	SILT WITH SAND (ML), brown, soft				8	1-1-2 N=3				28			
	medium stiff				15	1-2-3 N=5				28		32-23-9	
	soft, trace mica	5			10	1-1-2 N=3				35			
7.5	**SANDY SILT (ML)**, brown with orange mottling, medium stiff to stiff, low plasticity				18	2-3-5 N=8				37			
		10					UC	1032	0.9	36	84		
					15	2-2-4 N=6				37			
	soft	15			18	1-2-2 N=4				36			
	medium stiff	20			12	1-2-3 N=5				38			
		25			12	2-2-4 N=6				37			

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic SPT Hammer

Advancement Method:
Hollow stem auger

Abandonment Method:
Borings backfilled with bentonite chips upon completion

See Exhibit A-3 for description of field procedures.
See Appendix B for description of laboratory procedures and additional data (if any).
See Appendix C for explanation of symbols and abbreviations.

Notes:

WATER LEVEL OBSERVATIONS
While drilling
At completion of drilling

Terracon
4103 SE International Way Ste 300
Portland, OR

Boring Started: 4/4/2016	Boring Completed: 4/5/2016
Drill Rig: D-50 track	Driller: Terracon
Project No.: 82165034	Exhibit: A-5

BORING LOG NO. B-2

PROJECT: Parkland Assited Living Expansion

CLIENT:

SITE: **McMinnville, Oregon**

LOCATION See Exhibit A-2

Latitude: 45.20433° Longitude: -123.1572°

DEPTH	Description	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	RECOVERY (In.)	FIELD TEST RESULTS	STRENGTH TEST: TEST TYPE	STRENGTH TEST: COMPRESSIVE STRENGTH (psf)	STRENGTH TEST: STRAIN (%)	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
	SANDY SILT (ML), brown with orange mottling, medium stiff to stiff, low plasticity *(continued)*												
31.5		30						1743	13	35	87	NP	
	SILTY SAND (SM), fine grained, gray, medium dense, trace mica		▽		10	4-5-8 N=13				33			
		35			15	2-6-9 N=15				38			97
45.0		40	▼		12	3-4-6 N=10				39			
	SANDY SILT (ML), gray, stiff	45			15	2-4-5 N=9				36			
51.5	medium stiff to stiff	50			15	4-4-4 N=8				43			
	Boring Terminated at 51.5 Feet												

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic SPT Hammer

Advancement Method:
Hollow stem auger

Abandonment Method:
Borings backfilled with bentonite chips upon completion

See Exhibit A-3 for description of field procedures.
See Appendix B for description of laboratory procedures and additional data (if any).
See Appendix C for explanation of symbols and abbreviations.

Notes:

WATER LEVEL OBSERVATIONS

▽ *While drilling*

▼ *At completion of drilling*

Terracon
4103 SE International Way Ste 300
Portland, OR

Boring Started: 4/4/2016	Boring Completed: 4/5/2016
Drill Rig: D-50 track	Driller: Terracon
Project No.: 82165034	Exhibit: A-5

BORING LOG NO. B-3

PROJECT: Parkland Assited Living Expansion

CLIENT:

SITE: **McMinnville, Oregon**

LOCATION See Exhibit A-2

Latitude: 45.20404° Longitude: -123.15669°

DEPTH	Description	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	RECOVERY (In.)	FIELD TEST RESULTS	STRENGTH TEST: TEST TYPE	STRENGTH TEST: COMPRESSIVE STRENGTH (psf)	STRENGTH TEST: STRAIN (%)	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
0.3	**TOPSOIL**, 3-inch Grass and Root Zone												
	SILT WITH SAND (ML), brown, stiff					2-7-7 N=14				22			
	medium stiff					2-3-3 N=6				22			
	trace sand, brownish gray	5				1-2-3 N=5				25			
							UC						
9.0	**SANDY SILT (ML)**, grayish brown, stiff	10				2-4-5 N=9				33			
	gray to brown, medium stiff, trace mica	15				2-2-3 N=5				39			
	gray, soft	20				3-2-2 N=4				35			
	grayish brown with orange mottling	25				2-2-3 N=5				37			

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic SPT Hammer

Advancement Method:
Hollow stem auger

Abandonment Method:
Borings backfilled with bentonite chips upon completion

See Exhibit A-3 for description of field procedures.
See Appendix B for description of laboratory procedures and additional data (if any).
See Appendix C for explanation of symbols and abbreviations.

Notes:

WATER LEVEL OBSERVATIONS

While drilling

At completion of drilling

Terracon
4103 SE International Way Ste 300
Portland, OR

Boring Started: 4/4/2016	Boring Completed: 4/4/2016
Drill Rig: D-50 track	Driller: Terracon
Project No.: 82165034	Exhibit: A-6

BORING LOG NO. B-3

PROJECT: Parkland Assited Living Expansion

CLIENT:

SITE:
McMinnville, Oregon

LOCATION See Exhibit A-2

Latitude: 45.20404° Longitude: -123.15669°

GRAPHIC LOG	DEPTH / DESCRIPTION	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	RECOVERY (In.)	FIELD TEST RESULTS	STRENGTH TEST: TEST TYPE	STRENGTH TEST: COMPRESSIVE STRENGTH (psf)	STRENGTH TEST: STRAIN (%)	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
	SANDY SILT (ML), grayish brown, stiff *(continued)*												
		30	▽ (While drilling)	X		2-2-3 N=5				36			
			▽ (At completion of drilling)										
		35		X		2-2-3 N=5				39			
	gray	40		X		2-2-3 N=5				37		39-25-14	
		45		X		4-7-6 N=13				34			
	50.0 **SILTY SAND (SM)**, fine grained, dark gray, medium dense	50		X		5-5-9 N=14				36			
	51.5 ***Boring Terminated at 51.5 Feet***												

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic SPT Hammer

Advancement Method: Hollow stem auger Abandonment Method: Borings backfilled with bentonite chips upon completion	See Exhibit A-3 for description of field procedures. See Appendix B for description of laboratory procedures and additional data (if any). See Appendix C for explanation of symbols and abbreviations.	Notes:

WATER LEVEL OBSERVATIONS

▽ *While drilling*

▽ *At completion of drilling*

Terracon
4103 SE International Way Ste 300
Portland, OR

Boring Started: 4/4/2016	Boring Completed: 4/4/2016
Drill Rig: D-50 track	Driller: Terracon
Project No.: 82165034	Exhibit: A-6

APPENDIX B

LABORATORY TESTING

Laboratory Testing

As part of the testing program, all samples were examined in the laboratory by experienced personnel and classified in accordance with the attached General Notes and the Unified Soil Classification System based on the texture and plasticity of the soils. The group symbol for the Unified Soil Classification System is shown in the appropriate column on the boring logs and a brief description of the classification system is included with this report in the Appendix.

At that time, the field descriptions were confirmed or modified as necessary and an applicable laboratory testing program was formulated to determine engineering properties of the subsurface materials.

Laboratory tests were conducted on selected soil samples and the test results are presented in this appendix. The laboratory test results were used for the geotechnical engineering analyses, and the development of foundation and earthwork recommendations. Laboratory tests were performed in general accordance with the applicable ASTM, local or other accepted standards.

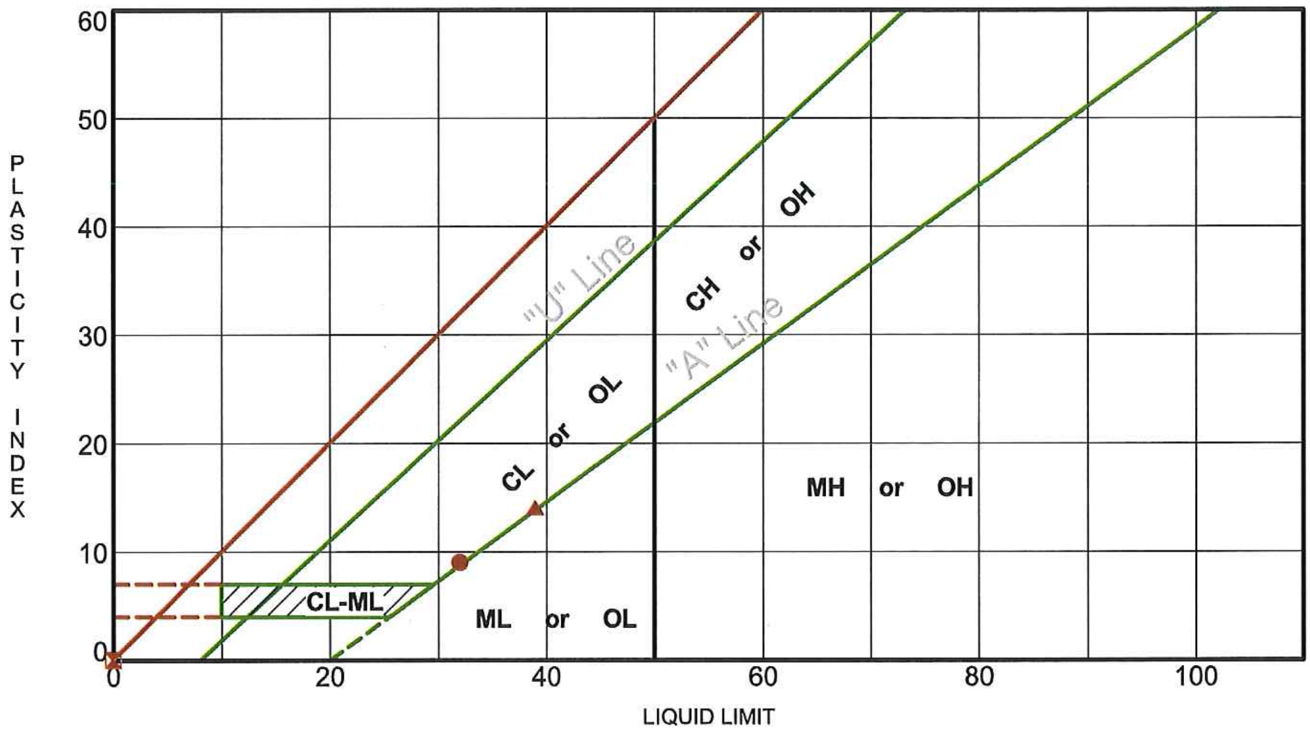
Selected soil samples obtained from the site were tested for the following engineering properties:

- In-situ Water Content (ASTM D 2216)
- Atterberg Limits (ASTM D 4318)
- Fines Content (passing No. 200 sieve) Determination (ASTM D 1140)
- Unconfined Compression Results (ASTM D 2166)
- Direct Shear Results (ASTM D 3080)

It is important to note that the site soils generally contain particles larger than 2 inches in diameter. Due to the sampling equipment being limited in diameter (1.8-inches), the grain size analyses are completed on materials that were able to be sampled. Therefore, the grain size analyses should be considered to be the materials passing a 2-inch sieve and not necessarily representative of the entire subsurface materials matrix.

ATTERBERG LIMITS RESULTS

ASTM D4318



	Boring ID	Depth	LL	PL	PI	Fines	USCS	Description
●	B-2	2.5 - 4	32	23	9			
⊠	B-2	30 - 31.5	NP	NP	NP			
▲	B-3	40 - 41.5	39	25	14			

PROJECT: Parkland Assited Living Expansion

SITE: McMinnville, Oregon

Terracon
4103 SE International Way Ste 300
Portland, OR

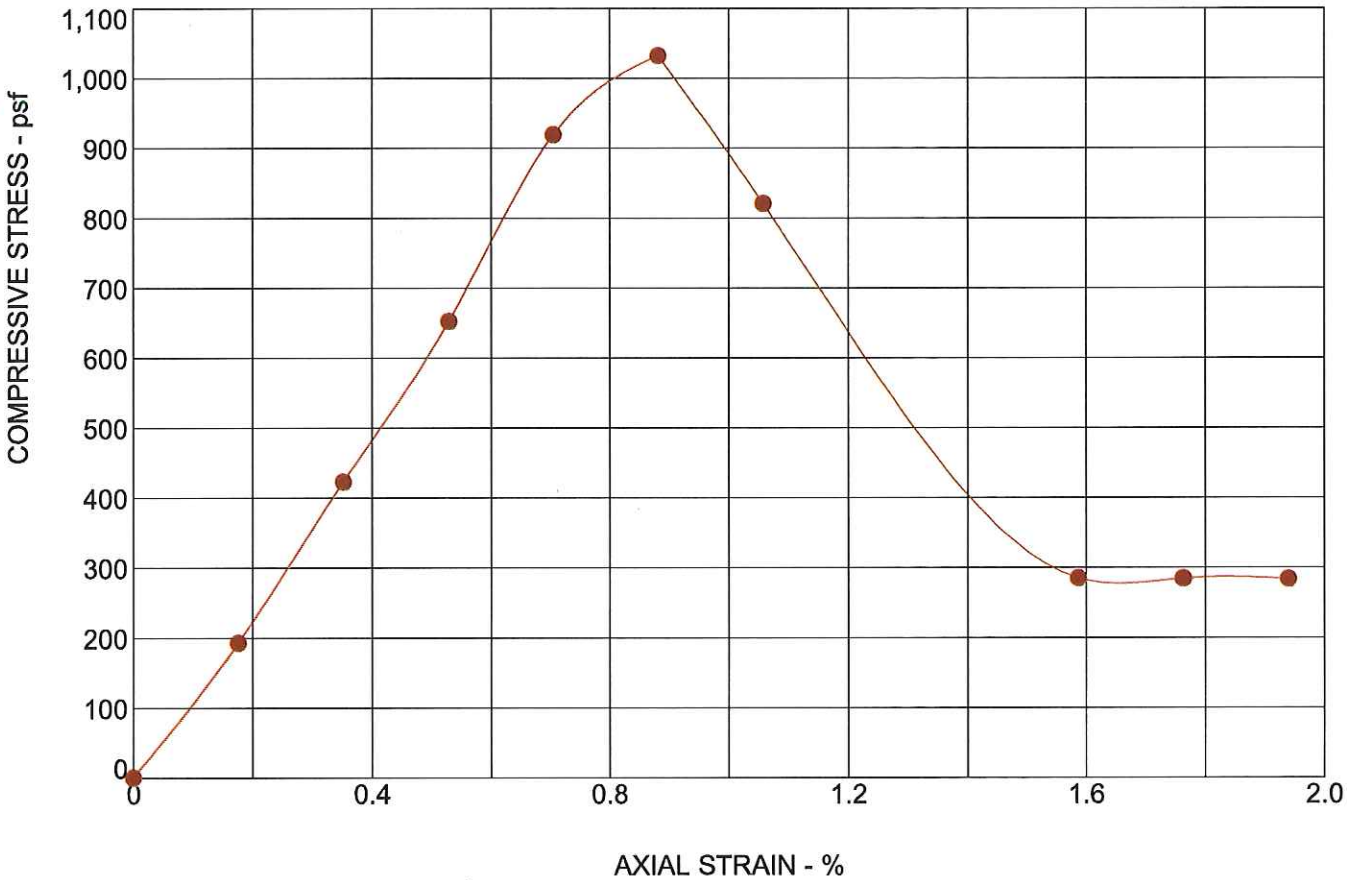
PROJECT NUMBER: 82165034

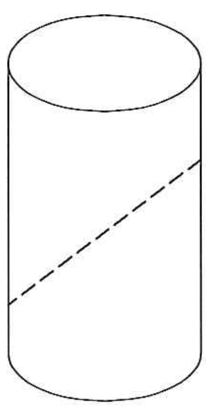
CLIENT:

EXHIBIT: B-2

UNCONFINED COMPRESSION TEST

ASTM D2166



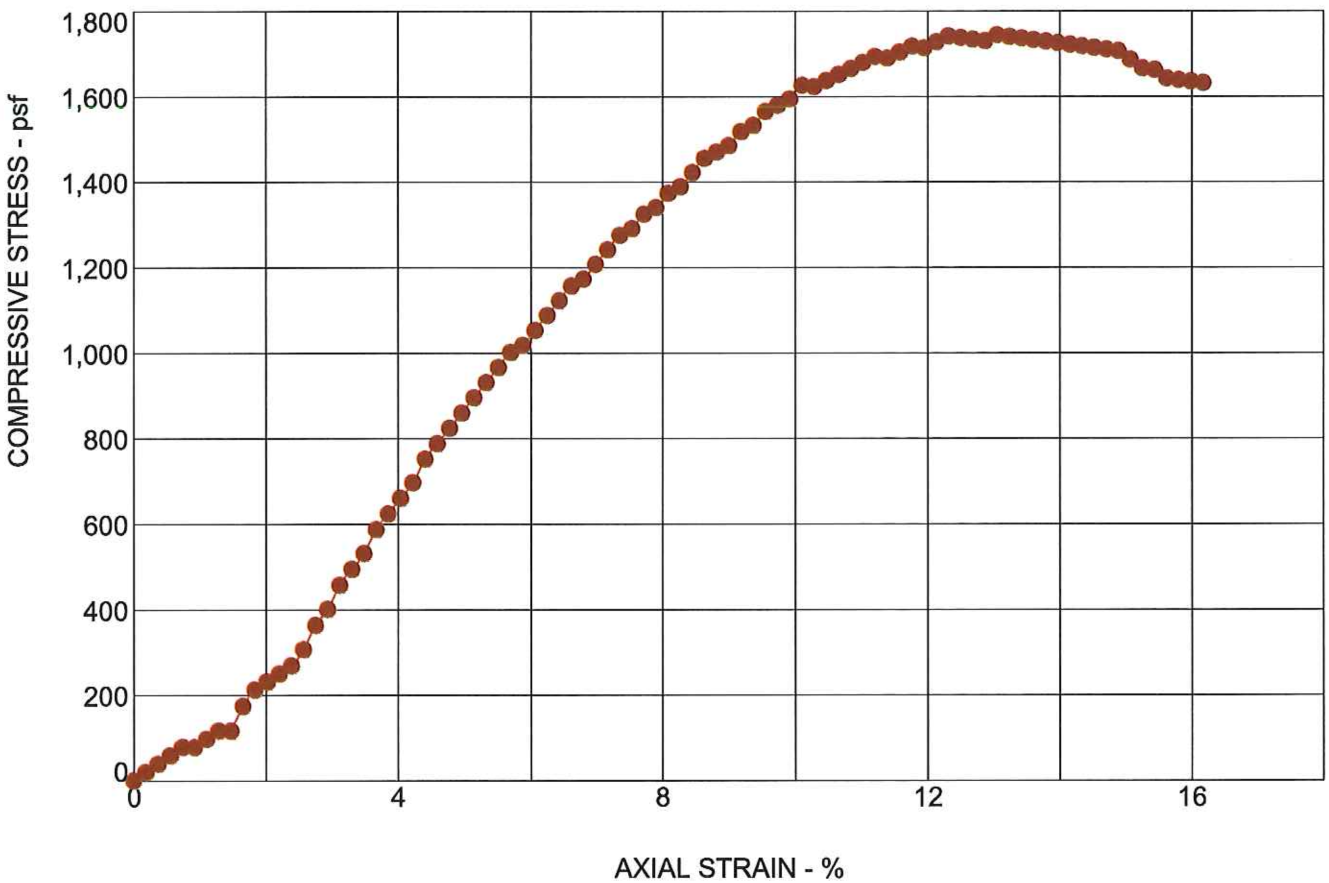
SPECIMEN FAILURE MODE	SPECIMEN TEST DATA		
 Failure Mode: Shear (dashed)	Moisture Content:	%	36
	Dry Density:	pcf	83
	Diameter:	in.	2.87
	Height:	in.	5.67
	Height / Diameter Ratio:		1.98
	Calculated Saturation:	%	
	Calculated Void Ratio:		
	Assumed Specific Gravity:		
	Failure Strain:	%	0.88
	Unconfined Compressive Strength	(psf)	1032
	Undrained Shear Strength:	(psf)	516
	Strain Rate:	in/min	0.0800
	Remarks:		

SAMPLE TYPE: Shelby Tube	SAMPLE LOCATION: B-2 @ 10 - 11.5 feet			
DESCRIPTION:	LL	PL	PI	Percent < #200 Sieve

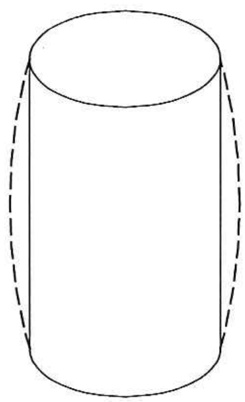
PROJECT: Parkland Assited Living Expansion	Terracon	PROJECT NUMBER: 82165034
SITE: McMinnville, Oregon	4103 SE International Way Ste 300 Portland, OR	CLIENT:
		EXHIBIT: B-3

UNCONFINED COMPRESSION TEST

ASTM D2166



SPECIMEN FAILURE MODE	SPECIMEN TEST DATA		
	Moisture Content:	%	35
	Dry Density:	pcf	87
	Diameter:	in.	2.84
	Height:	in.	5.44
	Height / Diameter Ratio:		1.92
	Calculated Saturation:	%	
	Calculated Void Ratio:		
	Assumed Specific Gravity:		
	Failure Strain:	%	13.05
	Unconfined Compressive Strength	(psf)	1743
	Undrained Shear Strength:	(psf)	872
	Strain Rate:	in/min	0.0857
	Remarks:		



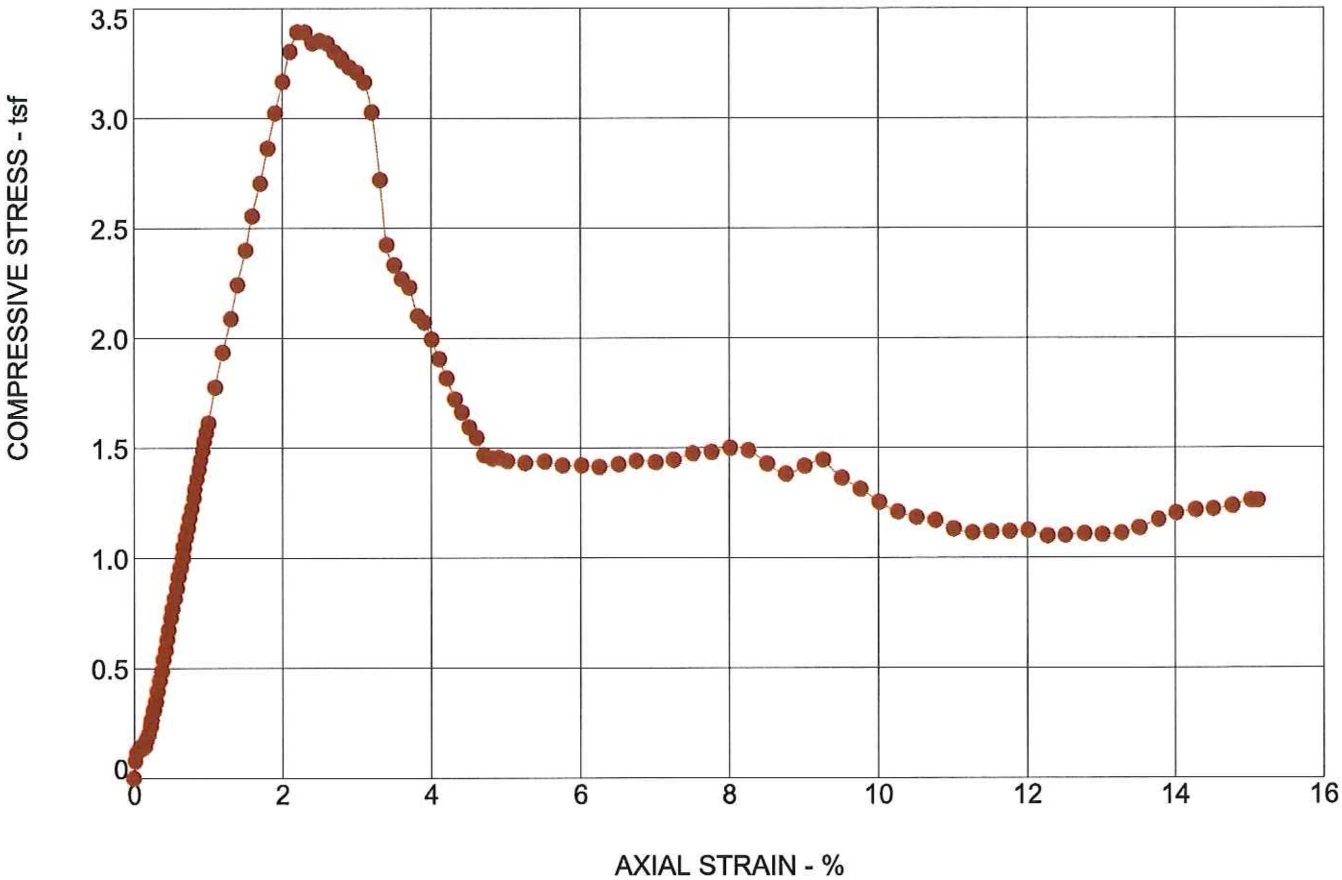
Failure Mode: Bulge (dashed)

SAMPLE TYPE: Shelby Tube	SAMPLE LOCATION: B-2 @ 30 - 31.5 feet			
DESCRIPTION:	LL NP	PL NP	PI NP	Percent < #200 Sieve

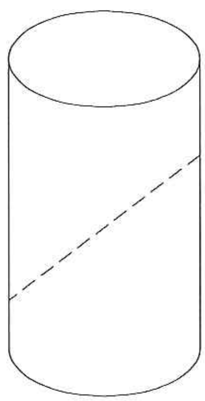
PROJECT: Parkland Assited Living Expansion	Terracon 4103 SE International Way Ste 300 Portland, OR	PROJECT NUMBER: 82165034
SITE: McMinnville, Oregon		CLIENT:
		EXHIBIT: B-4

UNCONFINED COMPRESSION TEST

ASTM D2166



SPECIMEN FAILURE MODE	SPECIMEN TEST DATA		
	Moisture Content:	%	31
	Dry Density:	pcf	91
	Diameter:	in.	2.86
	Height:	in.	5.58
	Height / Diameter Ratio:		1.95
	Calculated Saturation:	%	96.68
	Calculated Void Ratio:		0.88
	Assumed Specific Gravity:		2.75
	Failure Strain:	%	2.21
	Unconfined Compressive Strength	(tsf)	3.39
	Undrained Shear Strength:	(tsf)	1.70
	Strain Rate:	in/min	0.0560
	Remarks:		

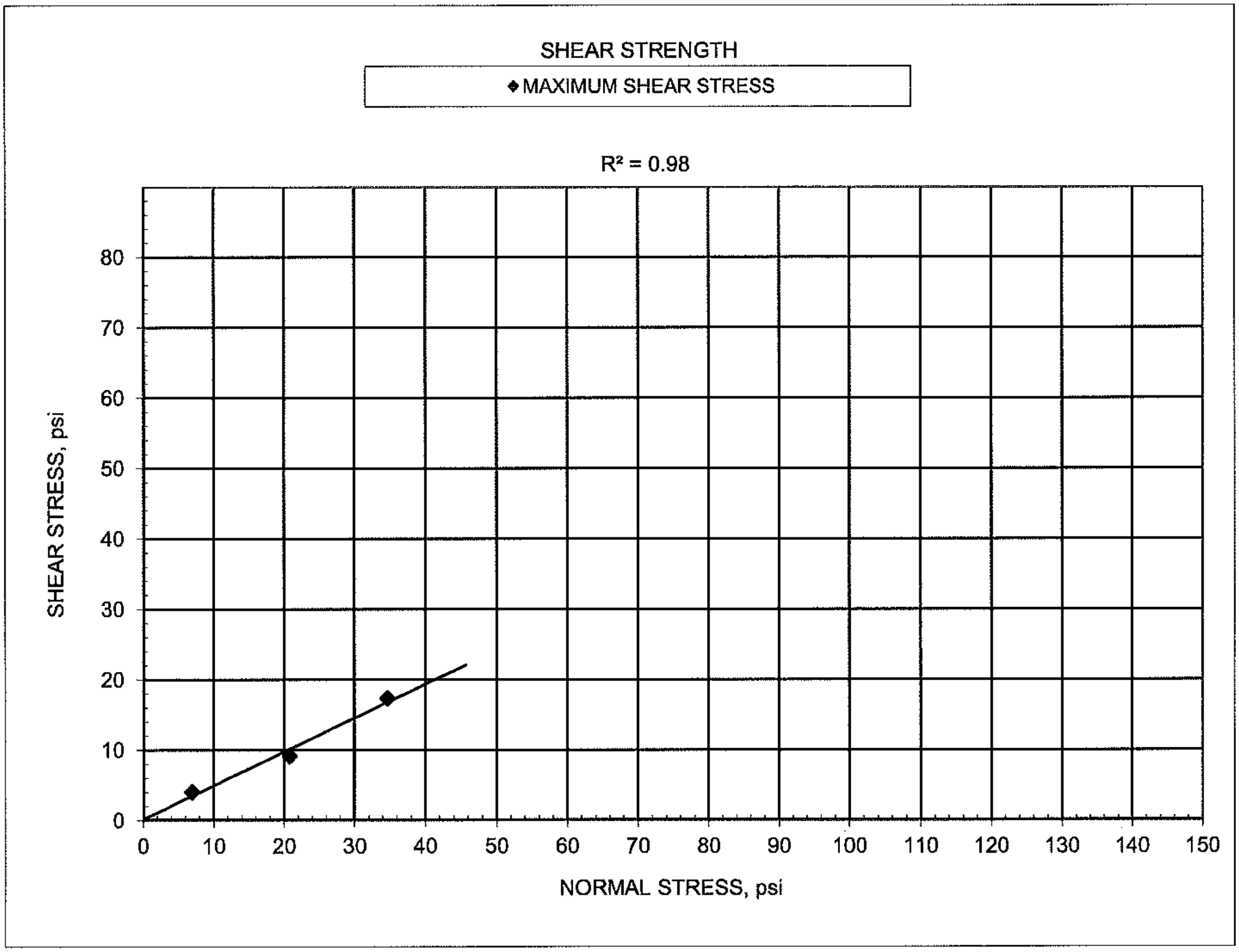


Failure Mode: Shear (dashed)

SAMPLE TYPE: Shelby Tube	SAMPLE LOCATION: B-3 @ 7.5 - 9 feet			
DESCRIPTION: Gray and Brown Silty Clay	LL	PL	PI	Percent < #200 Sieve

PROJECT: Parkland Expansion	Terracon	PROJECT NUMBER: 82165034
SITE: McMinnville, OR	51 Lost Mound Dr Ste 135 Chattanooga, TN	CLIENT: RJ Development Services
		B-5

DIRECT SHEAR TEST OF SOILS UNDER CONSOLIDATED DRAINED CONDITIONS
ASTM D3080



The reported cohesion may be apparent cohesion.

		FRICTION ANGLE	COHESION	NORMAL STRESS, psi	NORMAL STRESS, psi	NORMAL STRESS, psi
AT MAXIMUM SHEAR STRESS		25.6 deg	0.2 psi	6.9	20.8	34.7
INITIAL AREA, mm2	3166.9	INITIAL MOISTURE, %		33.8	34.3	32.9
INITIAL LENGTH, mm	25.40	INITIAL DRY DENSITY, pcf		83.2	78.5	85.6
SPECIFIC GRAVITY	2.70	INITIAL SATURATION, %		89	81	92
SG TESTED		INITIAL VOID RATIO		1.03	1.15	0.97
SG ASSUMED	X	FINAL MOISTURE, %		38.6	40.7	26.5
LIQUID LIMIT	X	FINAL SATURATION, %		100	99	99
PLASTIC LIMIT	X	FINAL VOID RATIO		1.04	1.11	0.72
PLASTICITY INDEX	X	MAXIMUM SHEAR STRESS, psi		4.03	9.12	17.34
SAMPLE TYPE	SHELBY TUBE	RATE OF LOADING, in/min		0.0018	0.0018	0.0018
DESCRIPTION	Gray and Brown Silty Clay					

PROJECT NAME: Parkland Expansion

BORING NO. B-3

LOCATION: McMinnville, OR

SAMPLE NO. S-4

JOB NO.: 82165034

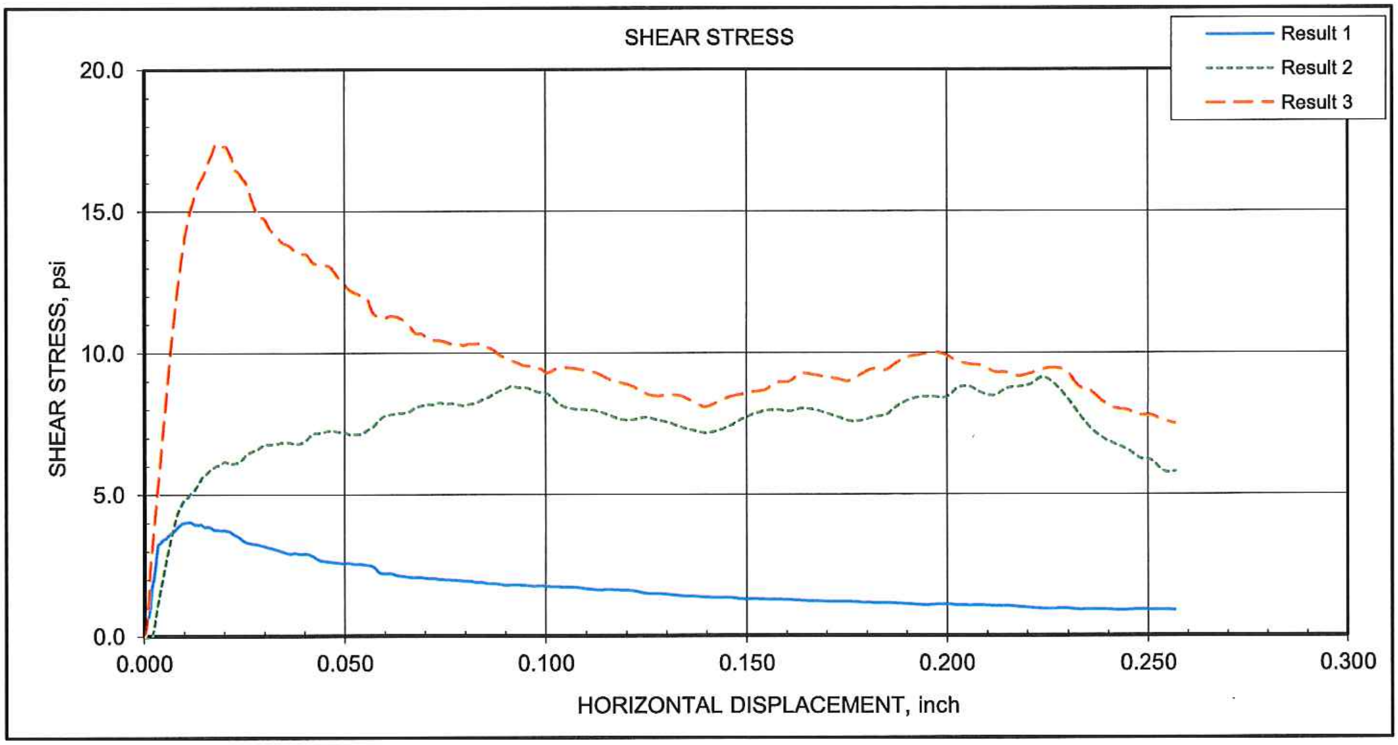
DEPTH, feet 7.5 TO 9

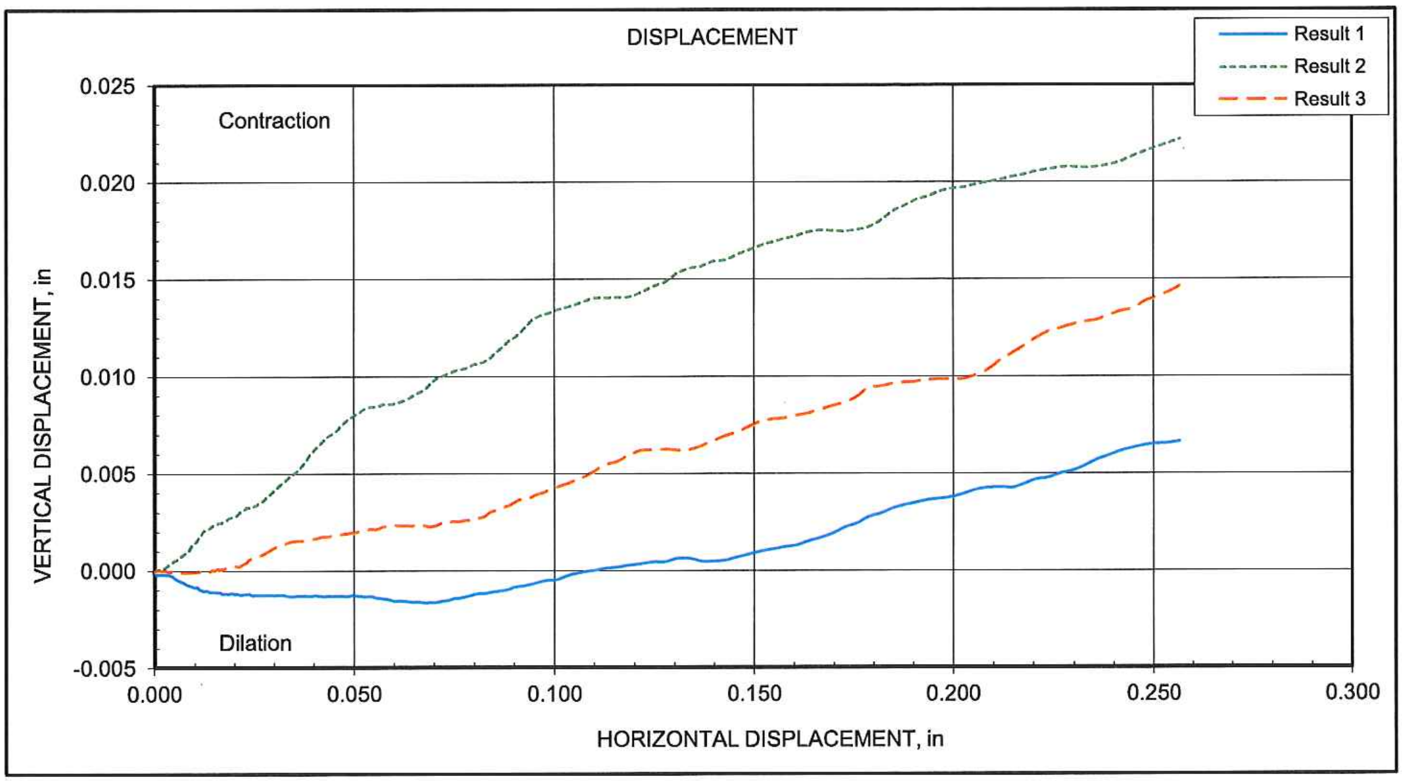
DATE: 5/3/2016

Terracon

Parkland Expansion
McMinnville, OR
82165034
5/3/2016

BORING NO.	B-3
SAMPLE NO.	S-4
DEPTH, feet	7.5 TO 9





 Terracon

APPENDIX C

SUPPORTING DOCUMENT

GENERAL NOTES

DESCRIPTION OF SYMBOLS AND ABBREVIATIONS

SAMPLING	Shelby Tube; Standard Penetration Test	WATER LEVEL	▽ Water Initially Encountered ▽ Water Level After a Specified Period of Time ▼ Water Level After a Specified Period of Time Water levels indicated on the soil boring logs are the levels measured in the borehole at the times indicated. Groundwater level variations will occur over time. In low permeability soils, accurate determination of groundwater levels is not possible with short term water level observations.	FIELD TESTS	**N** Standard Penetration Test Resistance (Blows/Ft.) **(HP)** Hand Penetrometer **(T)** Torvane **(DCP)** Dynamic Cone Penetrometer **(PID)** Photo-Ionization Detector **(OVA)** Organic Vapor Analyzer

DESCRIPTIVE SOIL CLASSIFICATION

Soil classification is based on the Unified Soil Classification System. Coarse Grained Soils have more than 50% of their dry weight retained on a #200 sieve; their principal descriptors are: boulders, cobbles, gravel or sand. Fine Grained Soils have less than 50% of their dry weight retained on a #200 sieve; they are principally described as clays if they are plastic, and silts if they are slightly plastic or non-plastic. Major constituents may be added as modifiers and minor constituents may be added according to the relative proportions based on grain size. In addition to gradation, coarse-grained soils are defined on the basis of their in-place relative density and fine-grained soils on the basis of their consistency.

LOCATION AND ELEVATION NOTES

Unless otherwise noted, Latitude and Longitude are approximately determined using a hand-held GPS device. The accuracy of such devices is variable. Surface elevation data annotated with +/- indicates that no actual topographical survey was conducted to confirm the surface elevation. Instead, the surface elevation was approximately determined from topographic maps of the area.

STRENGTH TERMS	RELATIVE DENSITY OF COARSE-GRAINED SOILS (More than 50% retained on No. 200 sieve.) Density determined by Standard Penetration Resistance		CONSISTENCY OF FINE-GRAINED SOILS (50% or more passing the No. 200 sieve.) Consistency determined by laboratory shear strength testing, field visual-manual procedures or standard penetration resistance		
	Descriptive Term (Density)	**Standard Penetration or N-Value Blows/Ft.**	**Descriptive Term (Consistency)**	**Unconfined Compressive Strength Qu, (psf)**	**Standard Penetration or N-Value Blows/Ft.**
	Very Loose	0 - 3	Very Soft	less than 500	0 - 1
	Loose	4 - 9	Soft	500 to 1,000	2 - 4
	Medium Dense	10 - 29	Medium Stiff	1,000 to 2,000	4 - 8
	Dense	30 - 50	Stiff	2,000 to 4,000	8 - 15
	Very Dense	> 50	Very Stiff	4,000 to 8,000	15 - 30
			Hard	> 8,000	> 30

RELATIVE PROPORTIONS OF SAND AND GRAVEL

Descriptive Term(s) of other constituents	Percent of Dry Weight
Trace	< 15
With	15 - 29
Modifier	> 30

GRAIN SIZE TERMINOLOGY

Major Component of Sample	Particle Size
Boulders	Over 12 in. (300 mm)
Cobbles	12 in. to 3 in. (300mm to 75mm)
Gravel	3 in. to #4 sieve (75mm to 4.75 mm)
Sand	#4 to #200 sieve (4.75mm to 0.075mm
Silt or Clay	Passing #200 sieve (0.075mm)

RELATIVE PROPORTIONS OF FINES

Descriptive Term(s) of other constituents	Percent of Dry Weight
Trace	< 5
With	5 - 12
Modifier	> 12

PLASTICITY DESCRIPTION

Term	Plasticity Index
Non-plastic	0
Low	1 - 10
Medium	11 - 30
High	> 30

Terracon

UNIFIED SOIL CLASSIFICATION SYSTEM

Criteria for Assigning Group Symbols and Group Names Using Laboratory Tests [A]				Soil Classification: Group Symbol	Soil Classification: Group Name [B]
Coarse Grained Soils: More than 50% retained on No. 200 sieve	**Gravels:** More than 50% of coarse fraction retained on No. 4 sieve	**Clean Gravels:** Less than 5% fines [C]	$Cu \geq 4$ and $1 \leq Cc \leq 3$ [E]	GW	Well-graded gravel [F]
			$Cu < 4$ and/or $1 > Cc > 3$ [E]	GP	Poorly graded gravel [F]
		Gravels with Fines: More than 12% fines [C]	Fines classify as ML or MH	GM	Silty gravel [F,G,H]
			Fines classify as CL or CH	GC	Clayey gravel [F,G,H]
	Sands: 50% or more of coarse fraction passes No. 4 sieve	**Clean Sands:** Less than 5% fines [D]	$Cu \geq 6$ and $1 \leq Cc \leq 3$ [E]	SW	Well-graded sand [I]
			$Cu < 6$ and/or $1 > Cc > 3$ [E]	SP	Poorly graded sand [I]
		Sands with Fines: More than 12% fines [D]	Fines classify as ML or MH	SM	Silty sand [G,H,I]
			Fines classify as CL or CH	SC	Clayey sand [G,H,I]
Fine-Grained Soils: 50% or more passes the No. 200 sieve	**Silts and Clays:** Liquid limit less than 50	**Inorganic:**	PI > 7 and plots on or above "A" line [J]	CL	Lean clay [K,L,M]
			PI < 4 or plots below "A" line [J]	ML	Silt [K,L,M]
		Organic:	Liquid limit - oven dried / Liquid limit - not dried < 0.75	OL	Organic clay [K,L,M,N]
					Organic silt [K,L,M,O]
	Silts and Clays: Liquid limit 50 or more	**Inorganic:**	PI plots on or above "A" line	CH	Fat clay [K,L,M]
			PI plots below "A" line	MH	Elastic Silt [K,L,M]
		Organic:	Liquid limit - oven dried / Liquid limit - not dried < 0.75	OH	Organic clay [K,L,M,P]
					Organic silt [K,L,M,Q]
Highly organic soils:	Primarily organic matter, dark in color, and organic odor			PT	Peat

[A] Based on the material passing the 3-inch (75-mm) sieve

[B] If field sample contained cobbles or boulders, or both, add "with cobbles or boulders, or both" to group name.

[C] Gravels with 5 to 12% fines require dual symbols: GW-GM well-graded gravel with silt, GW-GC well-graded gravel with clay, GP-GM poorly graded gravel with silt, GP-GC poorly graded gravel with clay.

[D] Sands with 5 to 12% fines require dual symbols: SW-SM well-graded sand with silt, SW-SC well-graded sand with clay, SP-SM poorly graded sand with silt, SP-SC poorly graded sand with clay

[E] $Cu = D_{60}/D_{10} \quad Cc = \dfrac{(D_{30})^2}{D_{10} \times D_{60}}$

[F] If soil contains ≥ 15% sand, add "with sand" to group name.

[G] If fines classify as CL-ML, use dual symbol GC-GM, or SC-SM.

[H] If fines are organic, add "with organic fines" to group name.

[I] If soil contains ≥ 15% gravel, add "with gravel" to group name.

[J] If Atterberg limits plot in shaded area, soil is a CL-ML, silty clay.

[K] If soil contains 15 to 29% plus No. 200, add "with sand" or "with gravel," whichever is predominant.

[L] If soil contains ≥ 30% plus No. 200 predominantly sand, add "sandy" to group name.

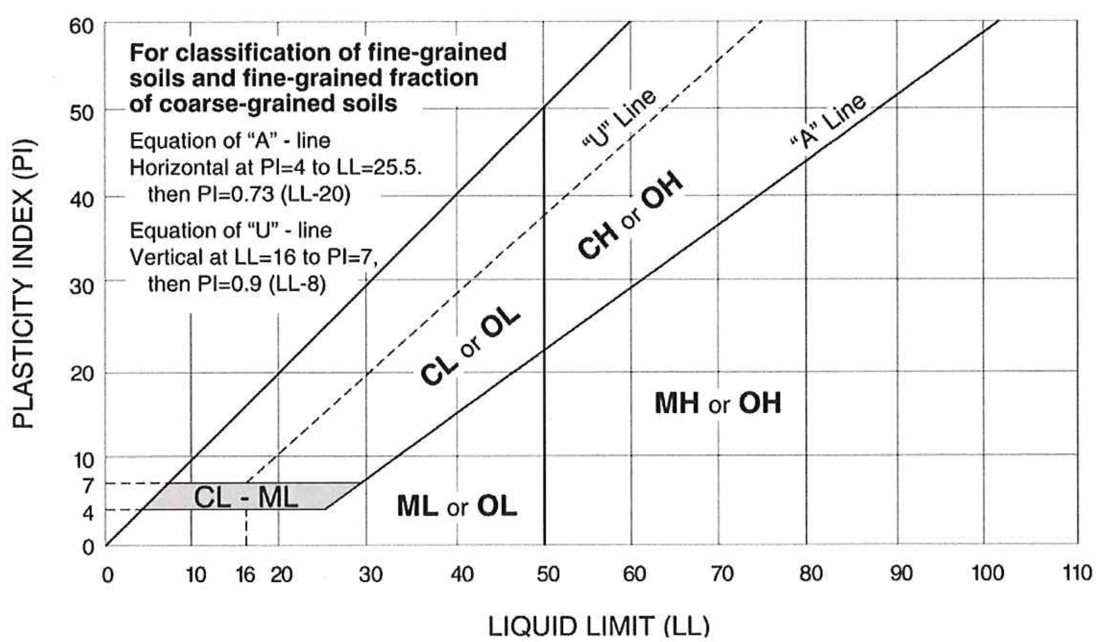
[M] If soil contains ≥ 30% plus No. 200, predominantly gravel, add "gravelly" to group name.

[N] PI ≥ 4 and plots on or above "A" line.

[O] PI < 4 or plots below "A" line.

[P] PI plots on or above "A" line.

[Q] PI plots below "A" line.



DESCRIPTION OF ROCK PROPERTIES

WEATHERING	
Term	**Description**
Unweathered	No visible sign of rock material weathering, perhaps slight discoloration on major discontinuity surfaces.
Slightly weathered	Discoloration indicates weathering of rock material and discontinuity surfaces. All the rock material may be discolored by weathering and may be somewhat weaker externally than in its fresh condition.
Moderately weathered	Less than half of the rock material is decomposed and/or disintegrated to a soil. Fresh or discolored rock is present either as a continuous framework or as corestones.
Highly weathered	More than half of the rock material is decomposed and/or disintegrated to a soil. Fresh or discolored rock is present either as a discontinuous framework or as corestones.
Completely weathered	All rock material is decomposed and/or disintegrated to soil. The original mass structure is still largely intact.
Residual soil	All rock material is converted to soil. The mass structure and material fabric are destroyed. There is a large change in volume, but the soil has not been significantly transported.

STRENGTH OR HARDNESS		
Description	**Field Identification**	**Uniaxial Compressive Strength, PSI (MPa)**
Extremely weak	Indented by thumbnail	40-150 (0.3-1)
Very weak	Crumbles under firm blows with point of geological hammer, can be peeled by a pocket knife	150-700 (1-5)
Weak rock	Can be peeled by a pocket knife with difficulty, shallow indentations made by firm blow with point of geological hammer	700-4,000 (5-30)
Medium strong	Cannot be scraped or peeled with a pocket knife, specimen can be fractured with single firm blow of geological hammer	4,000-7,000 (30-50)
Strong rock	Specimen requires more than one blow of geological hammer to fracture it	7,000-15,000 (50-100)
Very strong	Specimen requires many blows of geological hammer to fracture it	15,000-36,000 (100-250)
Extremely strong	Specimen can only be chipped with geological hammer	>36,000 (>250)

DISCONTINUITY DESCRIPTION			
Fracture Spacing (Joints, Faults, Other Fractures)		**Bedding Spacing (May Include Foliation or Banding)**	
Description	**Spacing**	**Description**	**Spacing**
Extremely close	< ¾ in (<19 mm)	**Laminated**	< ½ in (<12 mm)
Very close	¾ in – 2-1/2 in (19 - 60 mm)	**Very thin**	½ in – 2 in (12 – 50 mm)
Close	2-1/2 in – 8 in (60 – 200 mm)	**Thin**	2 in – 1 ft (50 – 300 mm)
Moderate	8 in – 2 ft (200 – 600 mm)	**Medium**	1 ft – 3 ft (300 – 900 mm)
Wide	2 ft – 6 ft (600 mm – 2.0 m)	**Thick**	3 ft – 10 ft (900 mm – 3 m)
Very Wide	6 ft – 20 ft (2.0 – 6 m)	**Massive**	> 10 ft (3 m)

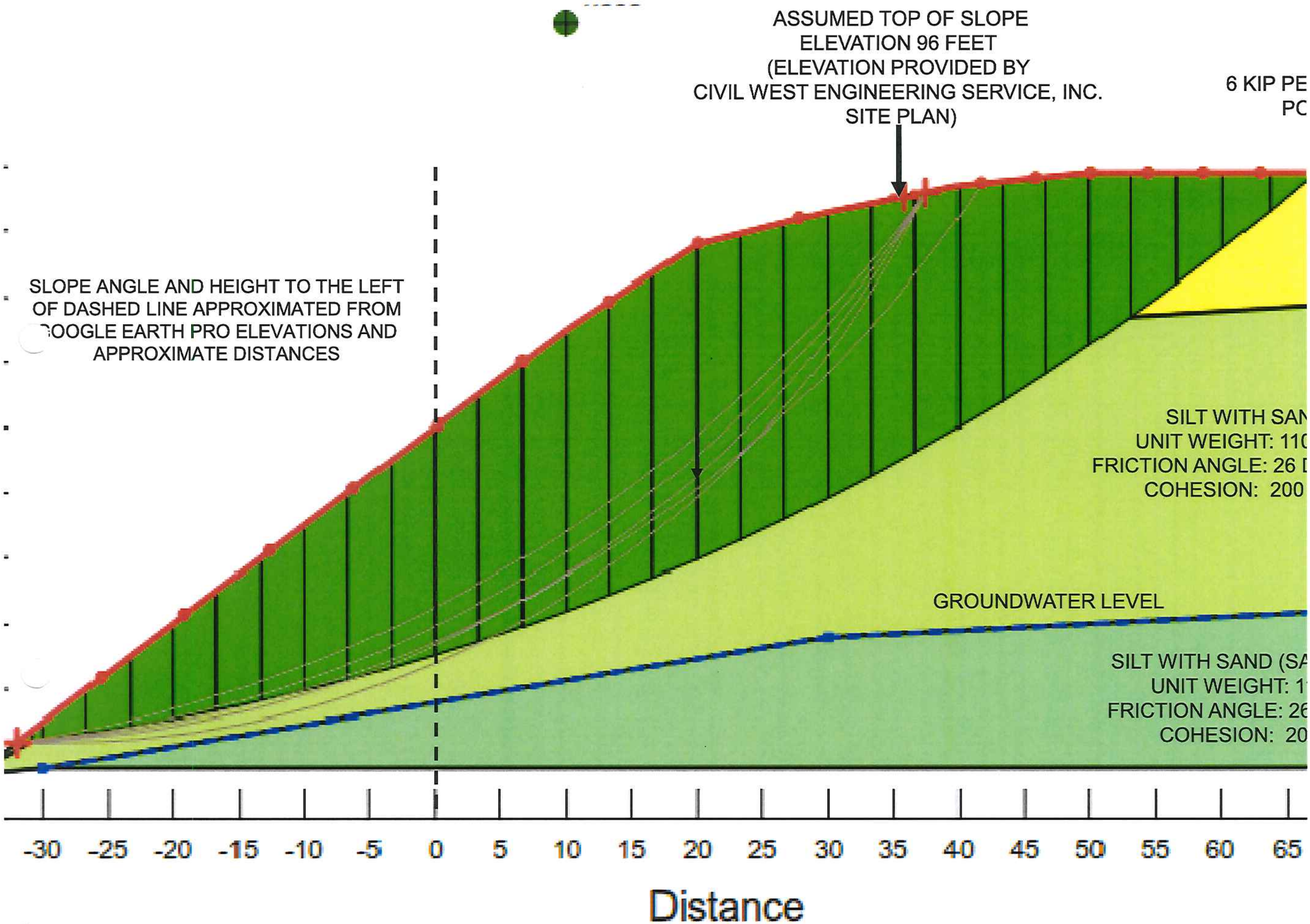
Discontinuity Orientation (Angle): Measure the angle of discontinuity relative to a plane perpendicular to the longitudinal axis of the core. (For most cases, the core axis is vertical; therefore, the plane perpendicular to the core axis is horizontal.) For example, a horizontal bedding plane would have a 0 degree angle.

ROCK QUALITY DESIGNATION (RQD*)	
Description	**RQD Value (%)**
Very Poor	0 - 25
Poor	25 – 50
Fair	50 – 75
Good	75 – 90
Excellent	90 - 100

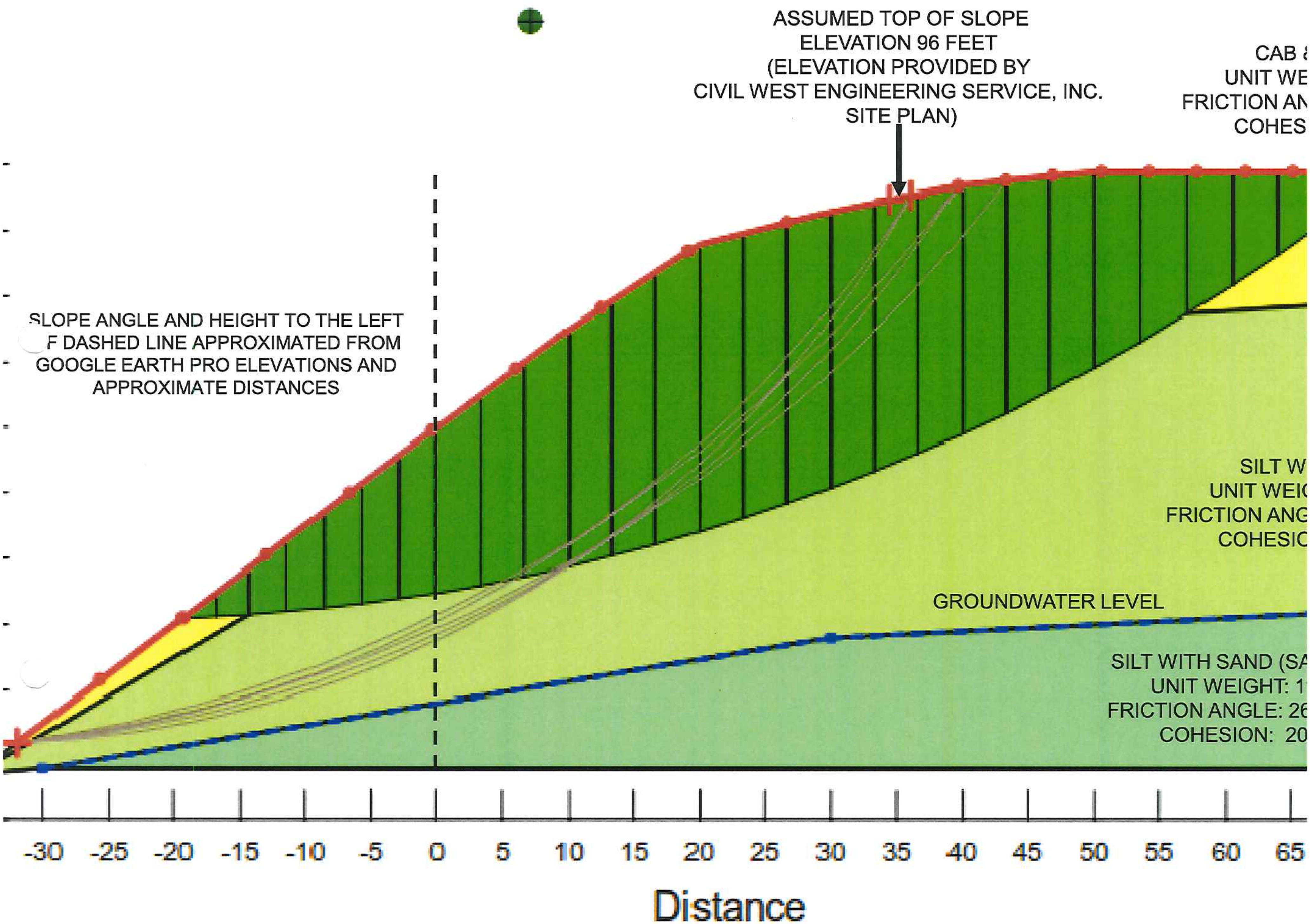
*The combined length of all sound and intact core segments equal to or greater than 4 inches in length, expressed as a percentage of the total core run length.

Reference: U.S. Department of Transportation, Federal Highway Administration, Publication No FHWA-NHI-10-034, December 2009 Technical Manual for Design and Construction of Road Tunnels – Civil Elements

APPENDIX D
SLOPE STABILITY ANALYSES RESULTS



Project Manager:	Project No.		SLOPE STA



Project Manager: | Project No. | SEISMIC SLOP