



**Geotechnical Investigation and Consultation Services**

**Proposed East Park Estates Development Site**

**Tax Lot No's. 100, 101, 200, 200, 201, 300 and 400**

**State Street and Cordon Road SE**

**Salem (Marion County), Oregon**

**for**

**Brownstone Development**



# REDMOND GEOTECHNICAL SERVICES

December 21, 2018

Mr. Randy Myers  
Brownstone Development  
P.O. Box 2375  
Lake Oswego, Oregon 97217

Dear Mr. Myers:

**Re: Geotechnical Investigation and Consultation Services, Proposed East Park Estates  
Development Site Tax Lot No's. 100, 101, 200, 200, 201, 300 and 400, State Street  
and Cordon Road SE, Salem (Marion County), Oregon**

Submitted herewith is our report entitled "Geotechnical Investigation and Consultation Services, Proposed East Park Estates Development Site, Tax Lot No's. 100, 101, 200, 200, 201, 300 and 400, State Street and Cordon Road SE, Salem (Marion County), Oregon". The scope of our services was outlined in our formal proposal to Mr. Randy Myers of Brownstone Development dated June 26, 2018. Written authorization of our services was provided by Mr. Randy Myers of Brownstone Development on September 14, 2018.

During the course of our investigation, we have kept you and/or others advised of our schedule and preliminary findings. We appreciate the opportunity to assist you with this phase of the project. Should you have any questions regarding this report, please do not hesitate to call.

Sincerely,

Daniel M. Redmond, P.E., G.E.  
President/Principal Engineer

Cc: Mr. Mark Grenz  
Multi/Tech Engineering Services, Inc.



571 12-31-18

## TABLE OF CONTENTS

	Page No.
INTRODUCTION	1
PROJECT DESCRIPTION	1
SCOPE OF WORK	2
SITE CONDITIONS	3
Site Geology	3
Surface Conditions	3
Subsurface Soil Conditions	4
Groundwater	5
INFILTRATION TESTING	5
LABORATORY TESTING	6
SEISMICITY AND EARTHQUAKE SOURCES	6
Liquefaction	7
Landslides	7
Surface Rupture	8
Tsunami and Seiche	8
Flooding and Erosion	8
CONCLUSIONS AND RECOMMENDATIONS	8
General	8
Site Preparation	9
Foundation Support	10
Shallow Foundations	11
Floor Slab Support	11

## **Table of Contents (continued)**

Retaining/Below Grade Walls	12
Pavements	13
Private Parking and Drive Areas	13
Local Residential Streets	14
Collector Streets	14
Pavement Subgrade, Base Course and Asphalt Materials	15
Wet Weather Grading and Soft Spot Mitigation	15
Shrink-Swell and Frost Heave	16
Excavations/Slopes	16
Surface Drainage/Groundwater	17
Design Infiltration Rates	17
Seismic Design Considerations	18
CONSTRUCTION MONITORING AND TESTING	18
CLOSURE AND LIMITATIONS	19
LEVEL OF CARE	19
REFERENCES	20
ATTACHMENTS	
Figure No. 1 - Site Vicinity Map	
Figure No. 2 - Site Exploration Plan	
Figure No. 3 - Typical Fill Slope Grading Detail	
Figure No. 4 - Perimeter Footing/Retaining Wall Drain Detail	
APPENDIX A	
Test Pit Logs and Laboratory Data	

**GEOTECHNICAL INVESTIGATION AND CONSULTATION SERVICES  
PROPOSED EAST PARK ESTATES DEVELOPMENT SITE  
TAX LOT NO'S. 100, 101, 200, 200, 201, 300 AND 400  
STATE STREET AND CORDON ROAD SE  
SALEM (MARION COUNTY) OREGON**

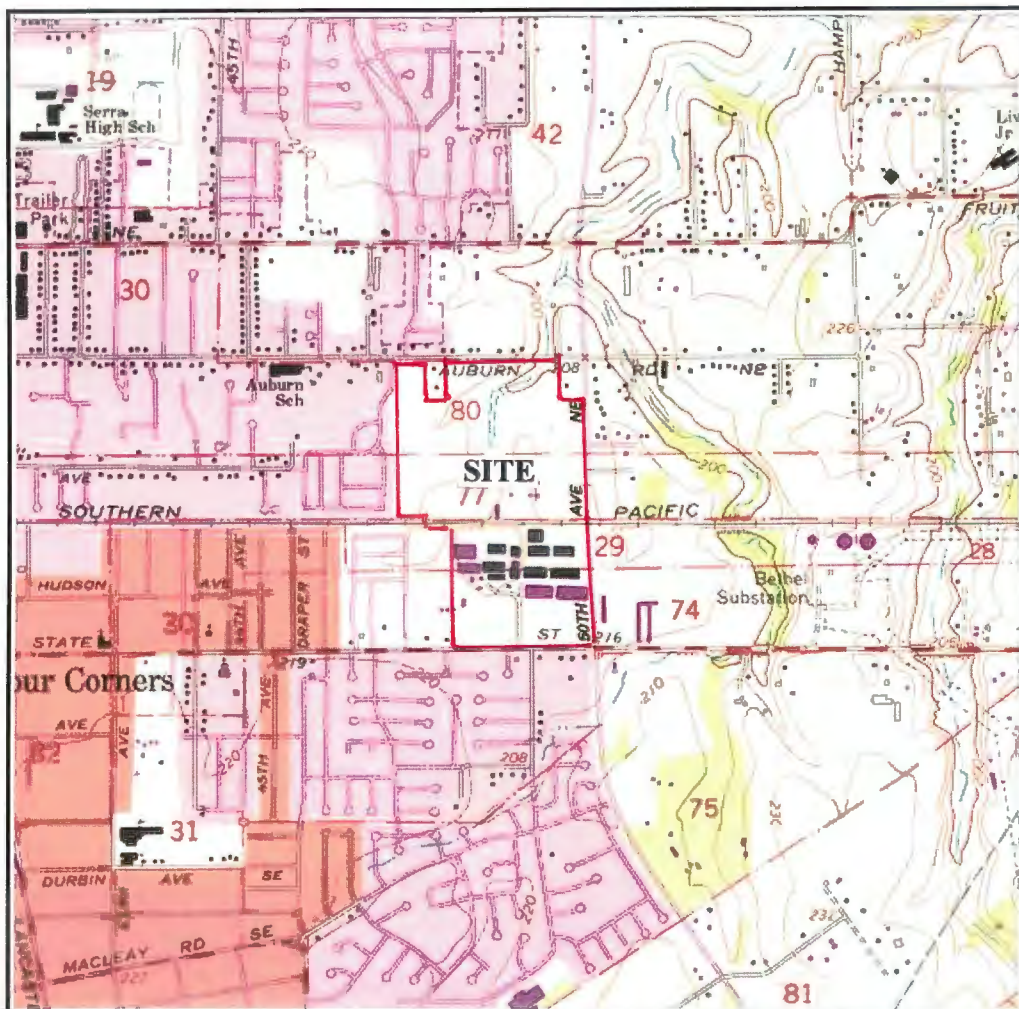
**INTRODUCTION**

Redmond Geotechnical Services, LLC is please to submit to you the results of our Geotechnical Investigation and Consultation services at the site of the proposed new East Park Estates development site located to the north of State Street and to the west of the intersection with Cordon Road SE in Salem (Marion County), Oregon. The general location of the subject site is shown on the Site Vicinity Map, Figure No. 1. The purpose of our geotechnical investigation and consultation services at this time was to explore the existing subsurface soils and/or groundwater conditions across the subject site and to evaluate any potential concerns at the site as well as to develop and/or provide appropriate geotechnical design and construction recommendations for the proposed new East Park Estates development project.

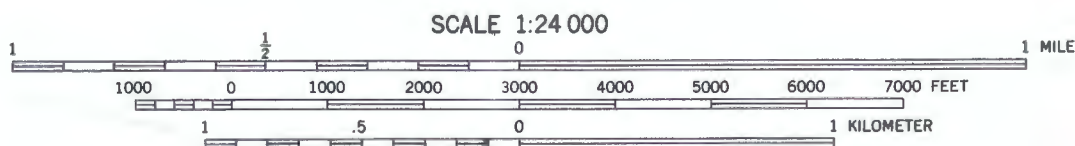
**PROJECT DESCRIPTION**

Although the East Park Estates project is still in the planning and design stages, we understand that present plans are to develop much and/or all of the subject property into new single-family residential lots. Specifically, we anticipate that full development of the subject property could result in the construction of approximately four hundred (400) to five hundred (500) new single-family residential lots ranging in size from about 5,000 to 10,000 square feet. However, we understand that the southerly portion of the subject property also allows for partial development under a wide range of scenarios. Specifically, based on a previous conceptual development plan prepared by WH Pacific, we understand that the southerly portion of the subject property may be developed with some multi-family residential (apartment) structures as well as some possible retail and/or commercial properties.

We envision that the new single-family residential homes will be of two- and/or three-story structures constructed with wood framing and wooden post and beam floors. However, development of the southerly portion of the site could include single- and/or three-story wood-frame apartment and/or commercial/retail structures with concrete slab-on-grade floors. Support of the new residential and/or commercial/retail structures is anticipated to consist primarily of conventional shallow strip (continuous) footings although some individual (column) footings may also be required.



SALEM EAST QUADRANGLE  
OREGON—MARION CO.  
7.5 MINUTE SERIES (TOPOGRAPHIC)  
NW/4 STAYTON 15' QUADRANGLE



CONTOUR INTERVAL 10 FEET  
NATIONAL GEODETIC VERTICAL DATUM OF 1929

### SITE VICINITY MAP

**EAST PARK ESTATES SITE**  
**STATE ST SE AND CORDON RD SE**

Project No. 1721.001.G

Figure No. 1



Additionally, due to the sloping nature of the central and/or northerly portion of the site, we envision that some of the single-family residential homes may include partial and/or below grade levels. As such, the use of some below grade retaining walls is also anticipated at the site and/or for the project.

Structural loading information, although unavailable at this time, is anticipated to be fairly typical for these types of single- and/or three-story wood-frame residential and/or commercial structures and are expected to result in maximum dead plus live continuous (strip) and individual (column) footing loads on the order of about 2.0 to 4.0 kips per lineal foot (klf) and 10 to 50 kips, respectively.

Although a site grading plan is not available at this time, we anticipate that both cuts and fills will be required for the project. In general, cuts and/or fills of between five (5) to ten (10) feet are generally anticipated. Other associated site improvements for the project will include construction of new paved public street improvements including the construction of new underground utility services as well as new concrete curbs and sidewalks. Additionally, we understand that storm water from hard and/or impervious areas (i.e., roofs and pavements) will be collected for on-site treatment and possible disposal primarily within the northeasterly portion of the subject property.

### **SCOPE OF WORK**

The purpose of our geotechnical investigation was to evaluate the overall subsurface soil and/or groundwater conditions underlying the subject site with regard to the proposed new residential and/or commercial development and construction at the site and any associated impacts or concerns with respect to development of the site as well as provide appropriate geotechnical design and construction recommendations for the project. Specifically, our geotechnical investigation included the following scope of work items:

1. Review of available and relevant geologic and/or geotechnical investigation reports for the subject site and/or area.
2. A detailed field reconnaissance and subsurface exploration program of the soil and ground water conditions underlying the site by means of twenty (20) exploratory test pit excavations. The exploratory test pits were excavated to depths ranging from about five (5) to eight (8) feet beneath existing site grades at the approximate locations as shown on the Site Exploration Plan, Figure No. 2. Additionally, field infiltration testing was also performed within various test pits excavated across the subject site.
3. Laboratory testing to evaluate and identify pertinent physical and engineering properties of the subsurface soils encountered relative to the planned site development and construction at the site. The laboratory testing program included tests to help evaluate the natural (field) moisture content and dry density, maximum dry density and optimum moisture content, gradational characteristics and Atterberg Limits as well as consolidation, (remolded) direct shear strength and "R"-value tests.

4. A literature review and engineering evaluation and assessment of the regional seismicity to evaluate the potential ground motion hazard(s) at the subject site. The evaluation and assessment included a review of the regional earthquake history and sources such as potential seismic sources, maximum credible earthquakes, and reoccurrence intervals as well as a discussion of the possible ground response to the selected design earthquake(s), fault rupture, landsliding, liquefaction, and tsunami and seiche flooding.
5. Engineering analyses utilizing the field and laboratory data as a basis for furnishing recommendations for foundation support of the proposed new residential and/or commercial structures. Recommendations include maximum design allowable contact bearing pressure(s), depth of footing embedment, estimates of foundation settlement, lateral soil resistance, and foundation subgrade preparation. Additionally, construction and/or permanent subsurface water drainage considerations have also been prepared. Further, our report includes recommendations regarding site preparation, placement and compaction of structural fill materials, suitability of the on-site soils for use as structural fill, criteria for import fill materials, and preparation of foundation, pavement and/or floor slab subgrades.
6. Flexible pavement design and construction recommendations for the proposed new public street improvements.

## **SITE CONDITIONS**

### **Site Geology**

Available geologic mapping of the area and/or subject site (Geologic Map of the Salem East 7.5 Minute Quadrangle Open-File Report 00-351) indicates that the near surface soils consist of lacustrine and fluvial (alluvium) sedimentary deposits of Pleistocene age. Characteristics include unconsolidated to semi-consolidated lacustrine clay, silt, sand and gravel; in places includes mudflow and related deposits of Piper (1942), Willamette Valley Silt (Allison, 1953; Wells and Peck, 1961), alluvial silt, sand and gravel that form terrace deposits of Wells and others (1983). These upper (surficial) unconsolidated to semi-consolidated alluvial sedimentary deposits are generally several tens of feet in thickness and are underlain at depth by semi-consolidated to well consolidated conglomerate gravels of Pleistocene age.

### **Surface Conditions**

The subject proposed new East Park Estates development property consists of seven (7) rectangular shaped tax lots (TL's 100, 101, 200, 200, 201, 300 and 400) which encompass a total plan area of approximately 117.34 acres. The proposed East Park Estates development property is roughly located to the north of State Street and to the west of the intersection with Cordon Road SE. The subject property is generally unimproved and consists of existing open land. However, the southerly portion of the subject property was previously development and contained several structures as well as various other site and/or surface improvements associated with the Pic Sweet mushroom development.



As such, some remnants of the previous site development and/or surface improvements remain which generally include graveled drives and/or parking areas as well as abandoned underground utility services. Additionally, the northerly portion of the site contains an existing storm water pond. Surface vegetation across the site generally consists of a light to moderate growth of grass and weeds.

Topographically, the southerly portion of the site is characterized as relatively flat-lying terrain while the central and/or northerly portion of the site is characterized as relatively flat-lying to gently sloping terrain (10 to 20 percent) descending downward towards the north/northeast with overall topographic relief estimated at about twenty-one (21) feet and ranges from a low about Elevation 195 feet near the northeasterly portion of the subject site to a high of about Elevation 216 near the southwesterly portion of the site.

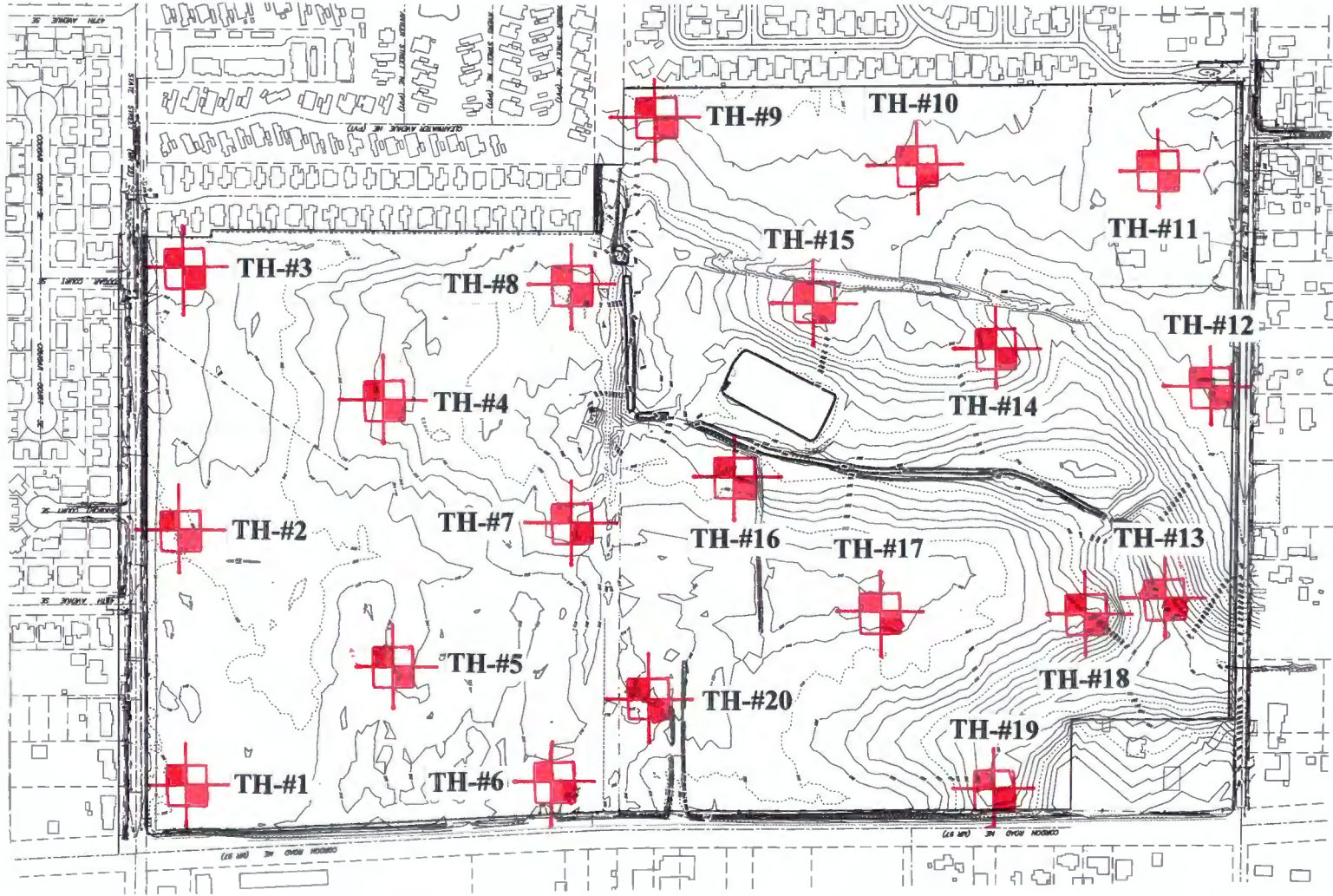
### **Subsurface Soil Conditions**

Our understanding of the subsurface soil conditions underlying the site was developed by means of twenty (20) exploratory test pits excavated to depths ranging from about five (5) to eight (8) feet beneath existing site grades on November 26, 2018 with a John Deere 200C track-mounted excavator. The location of the exploratory test pits were located in the field by marking off distances from existing and/or known site features and are shown in relation to the existing site improvements on the Site Exploration Plan, Figure No. 2. Detailed logs of the test pit explorations, presenting conditions encountered at each location explored, are presented in the Appendix, Figure No's. A-5 through A-14.

The exploratory test pit excavations were observed by staff from Redmond Geotechnical Services, LLC who logged each of the test pit explorations and obtained representative samples of the subsurface soils encountered across the site. Additionally, the elevation of the exploratory test pit excavations were referenced from an existing site conditions and/or topographic survey prepared by Multi/Tech Engineering Services, Inc. and should be considered as approximate. All subsurface soils encountered at the site and/or within the exploratory test pit excavations were logged and classified in general conformance with the Unified Soil Classification System (USCS) which is outlined on Figure No. A-4.

The test pit explorations revealed that the subject site is underlain by both native soil deposits and surficial fill materials. Specifically, the native soil deposits were composed of a surficial layer of dark brown, very moist to wet, soft to very soft, organic to highly organic, clayey, sandy silt topsoil materials to depths of about 8 to 12 inches. These surficial topsoil materials were in turn underlain by medium to olive-brown with gray mottling, moist to very moist becoming wet at depth, medium stiff to stiff, clayey, sandy silt to sandy, clayey silt to the maximum depth explored of about eight (8) feet beneath the existing site and/or surface grades. These clayey, sandy silt to sandy, clayey silt subgrade soil deposits are best characterized by relatively low to moderate strength and low to moderate compressibility. In addition to the above, fill materials were also encountered at and/or across the site.

**SITE EXPLORATION PLAN**  
**EAST PARK ESTATES SITE**  
**STATE ST SE AND CORDON RD SE**



**LEGEND**

TH-#20 Indicates approximate location of exploratory test hole



Specifically, surficial fill materials were encountered across much of the southerly portion of the site which generally consisted of about 12 to 18 inches of aggregate base rock materials. These surficial gravel fill materials were generally found to be poorly to moderately compacted and contained traces of organics. However, localized fill materials to depths of between three (3) and five (5) feet below the existing site and/or surface grades were encountered within test holes TH-#8, TH-#16 and TH-#18. These deeper fill materials, which were also found to be poorly to moderately compacted, contained organics as well as other miscellaneous construction debris including concrete, asphalt and brick rubble.

### **Groundwater**

Groundwater was generally not encountered within the exploratory test pit explorations at the time of excavations to depths of at least eight (8) feet beneath existing surface grades. However, groundwater and/or seepage was encountered within test hole TH-#13 at a depth of approximately three (3) feet beneath the existing site and/or surface grades. Additionally, the central and northeasterly portions of the subject property contain an existing seasonal drainage basin and/or surface drainage feature.

In this regard, groundwater elevations at the site may fluctuate seasonally in accordance with rainfall conditions and/or changes in site utilization as well as with runoff within the existing southeasterly drainage basin and may approach to near surface elevations and/or temporarily perch at the ground surface during periods of heavy rainfall.

### **INFILTRATION TESTING**

We performed four (4) field infiltration tests at the site on November 26, 2018. The infiltration tests were performed in test holes TH-#7, TH-#8, TH-#11 and TH-#19 at depths of between two (2) to five (5) feet beneath the existing site and/or surface grades. The subgrade soils encountered in the infiltration test hole consisted of clayey, sandy silt to sandy, clayey silt. The infiltration testing was performed in general conformance with current EPA and/or the City of Salem Encased Falling Head test method which consisted of advancing a 6-inch diameter PVC pipe approximately 6 inches into the exposed soil horizon at each test location. Using a steady water flow, water was discharged into the pipe and allowed to penetrate and saturate the subgrade soils. The water level was adjusted over a two (2) hour period and allowed to achieve a saturated subgrade soil condition consistent with the bottom elevation of the surrounding test pit excavation. Following the required saturating period, water was again added into the PVC pipe and the time and/or rate at which the water level dropped was monitored and recorded. Each measurable drop in the water level was recorded until a consistent infiltration rate was observed and/or repeated.

Based on the results of the field infiltration testing at the site (see Figure No's. A-21 through A-25), we have found that the native clayey, sandy silt to sandy, clayey silt subgrade soil deposits possess an ultimate infiltration rate on the order of about 0.2 to 0.6 inches per hour (in/hr).



## **LABORATORY TESTING**

Representative samples of the on-site subsurface soils were collected at selected depths and intervals from various test pit excavations and returned to our laboratory for further examination and testing and/or to aid in the classification of the subsurface soils as well as to help evaluate and identify their engineering strength and compressibility characteristics. The laboratory testing consisted of visual and textural sample inspection, moisture content and dry density determinations, maximum dry density and optimum moisture content, gradation analyses and Atterberg Limits as well as consolidation, (remolded) direct shear strength and "R"-value tests. Results of the various laboratory tests are presented in the Appendix, Figure No's. A-15 through A-21.

## **SEISMICITY AND EARTHQUAKE SOURCES**

The seismicity of the southwest Washington and northwest Oregon area, and hence the potential for ground shaking, is controlled by three separate fault mechanisms. These include the Cascadia Subduction Zone (CSZ), the mid-depth intraplate zone, and the relatively shallow crustal zone. Descriptions of these potential earthquake sources are presented below.

The CSZ is located offshore and extends from northern California to British Columbia. Within this zone, the oceanic Juan de Fuca Plate is being subducted beneath the continental North American Plate to the east. The interface between these two plates is located at a depth of approximately 15 to 20 kilometers (km). The seismicity of the CSZ is subject to several uncertainties, including the maximum earthquake magnitude and the recurrence intervals associated with various magnitude earthquakes. Anecdotal evidence of previous CSZ earthquakes has been observed within coastal marshes along the Washington and Oregon coastlines. Sequences of interlayered peat and sands have been interpreted to be the result of large Subduction zone earthquakes occurring at intervals on the order of 300 to 500 years, with the most recent event taking place approximately 300 years ago. A study by Geomatrix (1995) and/or USGS (2008) suggests that the maximum earthquake associated with the CSZ is moment magnitude (Mw) 8 to 9. This is based on an empirical expression relating moment magnitude to the area of fault rupture derived from earthquakes that have occurred within Subduction zones in other parts of the world. An Mw 9 earthquake would involve a rupture of the entire CSZ. As discussed by Geomatrix (1995) this has not occurred in other subduction zones that have exhibited much higher levels of historical seismicity than the CSZ. However, the 2008 USGS report has assigned a probability of 0.67 for a Mw 9 earthquake and a probability of 0.33 for a Mw 8.3 earthquake. For the purpose of this study an earthquake of Mw 9.0 was assumed to occur within the CSZ.

The intraplate zone encompasses the portion of the subducting Juan de Fuca Plate located at a depth of approximately 30 to 50 km below western Washington and western Oregon. Very low levels of seismicity have been observed within the intraplate zone in western Oregon and western Washington. However, much higher levels of seismicity within this zone have been recorded in Washington and California.

Several reasons for this seismic quiescence were suggested in the Geomatrix (1995) study and include changes in the direction of Subduction between Oregon, Washington, and British Columbia as well as the effects of volcanic activity along the Cascade Range. Historical activity associated with the intraplate zone includes the 1949 Olympia magnitude 7.1 and the 1965 Puget Sound magnitude 6.5 earthquakes. Based on the data presented within the Geomatrix (1995) report, an earthquake of magnitude 7.25 has been chosen to represent the seismic potential of the intraplate zone.

The third source of seismicity that can result in ground shaking within the Vancouver and southwest Washington area is near-surface crustal earthquakes occurring within the North American Plate. The historical seismicity of crustal earthquakes in this area is higher than the seismicity associated with the CSZ and the intraplate zone. The 1993 Scotts Mills (magnitude 5.6) and Klamath Falls (magnitude 6.0), Oregon earthquakes were crustal earthquakes.

### **Liquefaction**

Seismic induced soil liquefaction is a phenomenon in which loose, granular soils and some silty soils, located below the water table, develop high pore water pressures and lose strength due to ground vibrations induced by earthquakes. Soil liquefaction can result in lateral flow of material into river channels, ground settlements and increased lateral and uplift pressures on underground structures. Buildings supported on soils that have liquefied often settle and tilt and may displace laterally. Soils located above the ground water table cannot liquefy, but granular soils located above the water table may settle during the earthquake shaking.

Our review of the subsurface soil test pit logs from our exploratory field explorations (TH-#1 through TH-#20) and laboratory test results indicate that the site is generally underlain by medium stiff to stiff, clayey, sandy silt to sandy, clayey silt soils to depths of at least 8.0 feet beneath existing site grades. Additionally, groundwater was generally not encountered within any of the exploratory test pit excavations (TH-#1 through TH-#20) at the site during our field exploration work to depths of at least 8.0 feet except for groundwater seepage at a depth of three (3) feet in test hole TH-#13.

As such, due to the medium stiff to stiff and/or cohesive nature of the clayey, sandy silt to sandy, clayey silt subgrade soil deposits beneath the site, it is our opinion that the native clayey, sandy silt to sandy, clayey silt subgrade soil deposits located beneath the subject site have a very low potential for liquefaction during the design earthquake motions previously described.

### **Landslides**

No ancient and/or active landslides were observed or are known to be present on the subject site. Additionally, the site is generally characterized as relatively flat-lying to gently sloping terrain. As such, development of the subject site into the planned residential homes sites and/or commercial structures does not appear to present a potential geologic and/or landslide hazard provided that the site grading and development activities conform with the recommendations presented within this report.

### **Surface Rupture**

Although the site is generally located within a region of the country known for seismic activity, no known faults exist on and/or immediately adjacent to the subject site. As such, the risk of surface rupture due to faulting is considered negligible.

### **Tsunami and Seiche**

A tsunami, or seismic sea wave, is produced when a major fault under the ocean floor moves vertically and shifts the water column above it. A seiche is a periodic oscillation of a body of water resulting in changing water levels, sometimes caused by an earthquake. Tsunami and seiche are not considered a potential hazard at this site because the site is not near to the coast and/or there are no adjacent significant bodies of water.

### **Flooding and Erosion**

Stream flooding is a potential hazard that should be considered in lowland areas of Marion County and Salem. The FEMA (Federal Emergency Management Agency) flood maps should be reviewed as part of the design for the proposed new residential and/or commercial structures and site improvements. Elevations of structures on the site should be designed based upon consultants reports, FEMA (Federal Emergency Management Agency), and Marion County requirements for the 100-year flood levels of any nearby creeks, streams and/or drainage basins.

## **CONCLUSIONS AND RECOMMENDATIONS**

### **General**

Based on the results of our field explorations, laboratory testing, and engineering analyses, it is our opinion that the site is presently stable and suitable for the proposed new East Park Estates development and its associated site improvements provided that the recommendations contained within this report are properly incorporated into the design and construction of the project.

The primary features of concern at the site are 1) the presence of highly moisture sensitive clayey and silty subgrade soils across the site, 2) the presence of existing fill materials across the site, and 3) the relatively low infiltration rates anticipated within the near surface clayey and silty subgrade soils.

With regard to the moisture sensitive clayey and silty subgrade soils, we are generally of the opinion that all site grading and earthwork activities be scheduled for the drier summer months which is typically June through September. In regards to the presence of existing fill materials across the site, the results of our field and/or laboratory work indicate that the existing fill materials are presently only poorly to moderately compacted. Additionally, the existing fill materials were found to contain organics as well as miscellaneous construction debris. Further, the existing fill materials are believed to be undocumented.



In this regard, it is our professional opinion that the existing fill materials are unsuitable for direct support of the planned new site improvements. As such, we recommend that all of the existing fill materials present at and/or across the site be removed in their entirety down to an approved native subgrade soils. However, some of the existing fill materials may be suitable for use and/or re-use as structural fill provided that they are free of organics and/or deleterious materials and are placed and compacted to the requirements of structural fill. With regard to the relatively low infiltration rates anticipated within the clayey and silty subgrade soils beneath the site, we generally do not recommend any storm water infiltration within structural and/or embankment fills. However, some limited storm water infiltration may be feasible within the residential lots and/or areas of the site where the existing and/or finish slope gradients are no steeper than about 20 percent (1V:5H). In this regard, we recommend that all proposed storm water detention and/or infiltration systems for the project be reviewed and approved by Redmond Geotechnical Services, LLC.

The following sections of this report provide specific recommendations regarding subgrade preparation and grading as well as foundation and floor slab design and construction for the new East Park Estates development project.

### **Site Preparation**

As an initial step in site preparation, we recommend that the proposed new residential and/or commercial building sites as well as their associated structural and/or site improvement area(s) be stripped and cleared of all existing improvements, any existing unsuitable fill materials, surface debris, existing vegetation, topsoil materials, and/or any other deleterious materials present at the time of construction. In general, we envision that the site stripping to remove existing vegetation and topsoil materials will generally be about 8 to 12 inches. However, localized areas requiring deeper removals, such as the existing undocumented and/or unsuitable fill materials as well as any old foundation remnants, will likely be encountered and should be evaluated at the time of construction by the Geotechnical Engineer. The stripped and cleared materials should be properly disposed of as they are generally considered unsuitable for use/reuse as fill materials.

Following the completion of the site stripping and clearing work and prior to the placement of any required structural fill materials and/or structural improvements, the exposed subgrade soils within the planned structural improvement area(s) should be inspected and approved by the Geotechnical Engineer and possibly proof-rolled with a half and/or fully loaded dump truck. Areas found to be soft or otherwise unsuitable should be over-excavated and removed or scarified and recompacted as structural fill. During wet and/or inclement weather conditions, proof rolling and/or scarification and recompaction as noted above may not be appropriate.

The on-site native sandy, clayey silt subgrade soil materials are generally considered suitable for use/reuse as structural fill materials provided that they are free of organic materials, debris, and rock fragments in excess of about 6 inches in dimension. However, if site grading is performed during wet or inclement weather conditions, the use of some of the on-site native soil materials which contain significant silt and clay sized particles will be difficult at best.

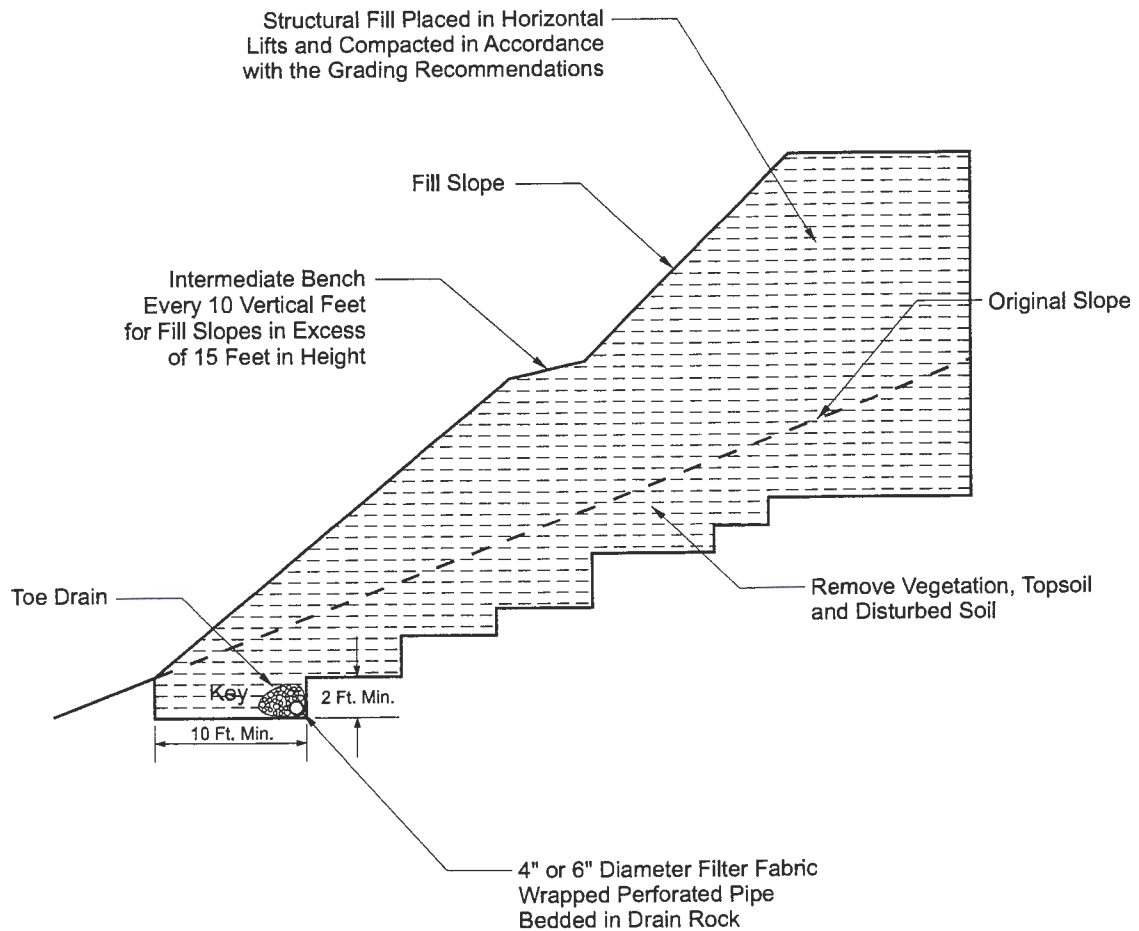
In this regard, during wet or inclement weather conditions, we recommend that an import structural fill material be utilized which should consist of a free-draining (clean) granular fill (sand & gravel) containing no more than about 5 percent fines. Representative samples of the materials which are to be used as structural fill materials should be submitted to the Geotechnical Engineer and/or laboratory for approval and determination of the maximum dry density and optimum moisture content for compaction.

In general, all site earthwork and grading activities should be scheduled for the drier summer months (late June through September) if possible. However, if wet weather site preparation and grading is required, it is generally recommended that the stripping of topsoil materials be accomplished with a tracked excavator utilizing a large smooth-toothed bucket working from areas yet to be excavated. Additionally, the loading of strippings into trucks and/or protection of moisture sensitive subgrade soils will also be required during wet weather grading and construction. In this regard, we recommend that areas in which construction equipment will be traveling be protected by covering the exposed subgrade soils with a woven geotextile fabric such as Mirafi FW404 followed by at least 12 inches or more of crushed aggregate base rock. Further, the geotextile fabric should have a minimum Mullen burst strength of at least 250 pounds per square inch for puncture resistance and an apparent opening size (AOS) between the U.S. Standard No. 70 and No. 100 sieves.

All structural fill materials placed within the new building and/or pavement areas should be moistened or dried as necessary to near (within 3 percent) optimum moisture conditions and compacted by mechanical means to a minimum of 92 percent of the maximum dry density as determined by the ASTM D-1557 (AASHTO T-180) test procedures. Structural fill materials should be placed in lifts (layers) such that when compacted do not exceed about 8 inches. Additionally, all fill materials placed within three (3) lineal feet of the perimeter (limits) of the proposed residential and/or commercial structures as well as pavement areas should be considered structural fill. Additionally, due to the sloping site conditions, we recommend that all structural fill materials planned in areas where existing surface and/or slope gradients exceed about 20 percent (1V:5H) be properly benched and/or keyed into the native (natural) slope subgrade soils. In general, a bench width of at least eight (8) feet and a keyway depth of at least one (1) foot is recommended. However, the actual bench width and keyway depth should be determined at the time of construction by the Geotechnical Engineer. Further, all fill slopes should be constructed with a finish slope surface gradient no steeper than about 2H:1V.

### **Foundation Support**

Based on the results of our investigation, it is our opinion that the site of the proposed new East Park Estates development is suitable for support of the one- and/or three-story wood-frame residential and/or commercial structures provided that the following foundation design recommendations are followed. The following sections of this report present specific foundation design and construction recommendations for the planned new residential and/or commercial structures.



## TYPICAL FILL SLOPE DETAIL

**EAST PARK ESTATES SITE  
STATE ST SE AND CORDON RD SE**

Project No. 1721.001.G

Figure No. 3

## **Shallow Foundations**

In general, conventional shallow continuous (strip) footings and individual (spread) column footings may be supported by approved native (untreated) subgrade soil materials based on an allowable contact bearing pressure of about 2,000 pounds per square foot (psf). However, structures supported by properly placed and compacted structural fill materials and/or large footings such as retaining walls, may be designed and/or sized based on an allowable contact bearing pressure of up to 2,500 pounds per square foot (psf). These recommended allowable contact bearing pressure is intended for dead loads and sustained live loads and may be increased by one-third for the total of all loads including short-term wind or seismic loads.

In general, continuous strip footings should have a minimum width of at least 16 inches and be embedded at least 18 inches below the lowest adjacent finish grade (includes frost protection). Individual column footings (where required) should be embedded at least 18 inches below grade and have a minimum width of at least 24 inches. Additionally, if foundation excavation and construction work is planned to be performed during wet and/or inclement weather conditions, we recommend that a 3 to 4 inch layer of compacted crushed rock be used to help protect the exposed foundation bearing surfaces until the placement of concrete.

Total and differential settlements of foundations constructed as recommended above and supported by approved native subgrade soils or by properly compacted structural fill materials are expected to be well within the tolerable limits for these types of single- and/or three-story residential and/or commercial wood-frame structures and should generally be less than about 1-inch and 1/2-inch, respectively.

Allowable lateral frictional resistance between the base of the footing element and the supporting subgrade bearing soil can be expressed as the applied vertical load multiplied by a coefficient of friction of 0.30 and 0.45 for native silty subgrade soils and/or import gravel fill materials, respectively. In addition, lateral loads may be resisted by passive earth pressures on footings poured "neat" against in-situ (native) subgrade soils or properly backfilled with structural fill materials based on an equivalent fluid density of 300 pounds per cubic foot (pcf). This recommended value includes a factor of safety of approximately 1.5 which is appropriate due to the amount of movement required to develop full passive resistance.

## **Floor Slab Support**

In order to provide uniform subgrade reaction beneath concrete slab-on-grade floors, we recommend that the floor slab area be underlain by a minimum of 6 inches of free-draining (less than 5 percent passing the No. 200 sieve), well-graded, crushed rock. The crushed rock should help provide a capillary break to prevent migration of moisture through the slab. However, additional moisture protection can be provided by using a 10-mil polyolefin geo-membrane sheet such as StegoWrap.

The base course materials should be compacted to at least 95 percent of the maximum dry density as determined by the ASTM D-1557 (AASHTO T-180) test procedures. Where floor slab subgrade materials are undisturbed, firm and stable and where the underslab aggregate base rock section has been prepared and compacted as recommended above, we recommend that a modulus of subgrade reaction of 150 pci be used for design.

### **Retaining/Below Grade Walls**

Retaining and/or below grade walls should be designed to resist lateral earth pressures imposed by native soils or granular backfill materials as well as any adjacent surcharge loads. For walls which are unrestrained at the top and free to rotate about their base, we recommend that active earth pressures be computed on the basis of the following equivalent fluid densities:

#### **Non-Restrained Retaining Wall Pressure Design Recommendations**

<b>Slope Backfill (Horizontal/Vertical)</b>	<b>Equivalent Fluid Density/Silt (pcf)</b>	<b>Equivalent Fluid Density/Gravel (pcf)</b>
Level	35	30
3H:1V	60	50
2H:1V	90	80

For walls which are fully restrained at the top and prevented from rotation about their base, we recommend that at-rest earth pressures be computed on the basis of the following equivalent fluid densities:

#### **Restrained Retaining Wall Pressure Design Recommendations**

<b>Slope Backfill (Horizontal/Vertical)</b>	<b>Equivalent Fluid Density/Silt (pcf)</b>	<b>Equivalent Fluid Density/Gravel (pcf)</b>
Level	45	35
3H:1V	65	60
2H:1V	95	90

The above recommended values assume that the walls will be adequately drained to prevent the buildup of hydrostatic pressures. Where wall drainage will not be present and/or if adjacent surcharge loading is present, the above recommended values will be significantly higher.

Backfill materials behind walls should be compacted to 90 percent of the maximum dry density as determined by the ASTM D-1557 (AASHTO T-180) test procedures. Special care should be taken to avoid over-compaction near the walls which could result in higher lateral earth pressures than those indicated herein. In areas within three (3) to five (5) feet behind walls, we recommend the use of hand-operated compaction equipment.



## **Pavements**

Flexible pavement design for the proposed new street improvements for the East Park Estates development project was determined in accordance with the City of Salem Department of Public Works Administrative Rules Chapter 109-006 (Street Design Standards) Section 6 dated January 1, 2014.

Specifically, on November 26, 2018, samples of the subgrade soils from the areas of the proposed new public streets were collected by means of test hole excavations. The subgrade soils encountered in the test holes located across the proposed East Park Estates development site generally consisted of native soils comprised of medium to olive-brown, medium stiff to stiff, clayey, sandy silt to sandy, clayey SILT (ML).

The subgrade soil samples collected at the site were tested in the laboratory in accordance with the ASTM Vol. 4.08 Part D-2844-69 (AASHTO T-190-93) test method for the determination of the subgrade soil "R"-value and expansion pressure. The results of the "R"-value testing was then converted to an equivalent Resilient Modulus ( $M_{RSG}$ ) in accordance with current AASHTO methodology. The results of the laboratory "R"-value tests revealed that the subgrade soils have an apparent "R"-value of between 25 and 29 with an average "R"-value of 27 (see Figure No's. A-20 and A-21). Using the current AASHTO methodology for converting "R"-value to Resilient Modulus ( $M_{RSG}$ ), the subgrade soils have a Resilient Modulus ( $M_{RSG}$ ) of about 5,291 psi which is classified a "Fair" ( $M_{RSG}$  = 5,000 psi to 10,000 psi).

In addition to the above, Dynamic Cone Penetration (DCP) tests were performed along the proposed new interior public street alignments at approximate 200-foot intervals. The results of the DCP tests found that the underlying native clayey, sandy silt to sandy, clayey silt subgrade soils have a DCP value of between 2 to 3 blows per 2-inches which correlates to a California Bearing Ratio (CBR) of between 5 and 12. Using current AASHTO methodology for converting CBR to Resilient Modulus ( $M_{RSG}$ ), the subgrade soils have a Resilient Modulus ( $M_{RSG}$ ) of between 5,842 and 10,637 psi with an average  $M_{RSG}$  of 7,150 psi which is classified as "Fair" ( $M_{RSG}$  = 5,000 psi to 10,000 psi).

## **Private Parking and Drive Areas**

	<b><u>Asphaltic Concrete Thickness (inches)</u></b>	<b><u>Crushed Base Rock Thickness (inches)</u></b>
Automobile Parking Areas	3.0	8.0
Automobile Drive Areas	3.5	10.0

Note: Where heavy vehicle traffic is anticipated such as those required for fire and/or garbage trucks, we recommend that the automobile drive area pavement section be increased by adding 0.5 inches of asphaltic concrete and 2.0 inches of aggregate base rock. Additionally, the above recommended flexible pavement section(s) assumes a design life of 20 years.



### **Local Residential Streets**

The following documents and/or design input parameters were used to help determine the flexible pavement section design for any new and/or existing local residential streets:

- . **Street Classification:** Local Residential Street
- . **Design Life:** 25 years
- . **Serviceability:** 4.2 initial, 2.5 terminal
- . **Traffic Loading Data:** 100,000 18-kip EAL's
- . **Reliability Level:** 90%
- . **Drainage Coefficient:** 1.0 (asphalt), 0.8 (aggregate)
- . **Asphalt Structural Coefficient:** 0.41
- . **Aggregate Structural Coefficient:** 0.10

Based on the above design input parameters and using the design procedures contained within the AASHTO 1993 Design of Pavement Structures Manual, a Structural Number (SN) of 2.6 was determined.

In this regard, we recommend the following flexible pavement section for the construction of new Local Residential Streets:

<b><u>Material Type</u></b>	<b><u>Pavement Section (inches)</u></b>
Asphaltic Concrete	4.0
Aggregate Base Rock	10.0

### **Collector Streets**

The following documents and/or design input parameters were used to help determine the flexible pavement section design for improvements to new and/or existing Collector Streets:

- . **Street Classification:** Collector Street
- . **Design Life:** 20 years
- . **Serviceability:** 4.2 initial, 2.5 terminal
- . **Traffic Loading Data:** 1,000,000 18-kip EAL's
- . **Reliability Level:** 90%
- . **Drainage Coefficient:** 1.0 (asphalt), 0.8 (aggregate)
- . **Asphalt Structural Coefficient:** 0.41
- . **Aggregate Structural Coefficient:** 0.10

Based on the above design input parameters and using the design procedures contained within the AASHTO 1993 Design of Pavement Structures Manual, a Structural Number (SN) of 4.1 was determined.

In this regard, we recommend the following flexible pavement section for the new improvements to new and/or existing Collector Streets:

<u>Material Type</u>	<u>Pavement Section (inches)</u>
Asphaltic Concrete	5.0
Aggregate Base Rock	14.0

#### **Pavement Subgrade, Base Course & Asphalt Materials**

The above recommended pavement section(s) were based on the design assumptions listed herein and on the assumption that construction of the pavement section(s) will be completed during an extended period of reasonably dry weather. All thicknesses given are intended to be the minimum acceptable. Increased base rock sections and the use of a woven geotextile fabric may be required during wet and/or inclement weather conditions and/or in order to adequately support construction traffic and protect the subgrade during construction. Additionally, the above recommended pavement section(s) assume that the subgrade will be prepared as recommended herein, that the exposed subgrade soils will be properly protected from rain and construction traffic, and that the subgrade is firm and unyielding at the time of paving. Further, it assumes that the subgrade is graded to prevent any ponding of water which may tend to accumulate in the base course.

Pavement base course materials should consist of well-graded 1-1/2 inch and/or 3/4-inch minus crushed base rock having less than 5 percent fine materials passing the No. 200 sieve. The base course and asphaltic concrete materials should conform to the requirements set forth in the latest edition of the Oregon Department of Transportation, Standard Specifications for Highway Construction. The base course materials should be compacted to at least 95 percent of the maximum dry density as determined by the ASTM D-1557 (AASHTO T-180) test procedures. The asphaltic concrete paving materials should be compacted to at least 92 percent of the theoretical maximum density as determined by the ASTM D-2041 (Rice Gravity) test method.

#### **Wet Weather Grading and Soft Spot Mitigation**

Construction of the proposed new public street improvements as well as any private access drives and/or parking areas is generally recommended during dry weather. However, during wet weather grading and construction, excavation to subgrade can proceed during periods of light to moderate rainfall provided that the subgrade remains covered with aggregate. A total aggregate thickness of 8-inches may be necessary to protect the subgrade soils from heavy construction traffic. Construction traffic should not be allowed directly on the exposed subgrade but only atop a sufficient compacted base rock thickness to help mitigate subgrade pumping.

If the subgrade becomes wet and pumps, no construction traffic shall be allowed on the road alignment. Positive site drainage away from the street shall be maintained if site paving will not occur before the on-set of the wet season.

Depending on the timing for the project, any soft subgrade found during proof-rolling or by visual observations can either be removed and replaced with properly dried and compacted fill soils or removed and replaced with compacted crushed aggregate. However, and where approved by the Geotechnical Engineer, the soft area may be covered with a bi-axial geogrid and covered with compacted crushed aggregate.

#### **Soil Shrink-Swell and Frost Heave**

The results of the laboratory "R"-value tests indicate that the native subgrade soils possess a low to moderate expansion potential. As such, the exposed subgrade soils should not be allowed to completely dry and should be moistened to near optimum moisture content (plus or minus 3 percent) at the time of the placement of the crushed aggregate base rock materials. Additionally, exposure of the subgrade soils to freezing weather may result in frost heave and softening of the subgrade. As such, all subgrade soils exposed to freezing weather should be evaluated and approved by the Geotechnical Engineer prior to the placement of the crushed aggregate base rock materials.

#### **Excavation/Slopes**

Temporary excavations of up to about four (4) feet in depth may be constructed with near vertical inclinations. Temporary excavations greater than about four (4) feet but less than eight (8) feet should be excavated with inclinations of at least 1 to 1 (horizontal to vertical) or properly braced/shored. Where excavations are planned to exceed about eight (8) feet, this office should be consulted. All shoring systems and/or temporary excavation bracing for the project should be the responsibility of the excavation contractor. Permanent slopes should be constructed no steeper than about 2H to 1V unless approved by the Geotechnical Engineer.

Depending on the time of year in which trench excavations occur, trench dewatering may be required in order to maintain dry working conditions if the invert elevations of the proposed utilities are located at and/or below the groundwater level. If groundwater is encountered during utility excavation work, we recommend placing trench stabilization materials along the base of the excavation.

Trench stabilization materials should consist of 1-foot of well-graded gravel, crushed gravel, or crushed rock with a maximum particle size of 4 inches and less than 5 percent fines passing the No. 200 sieve. The material should be free of organic matter and other deleterious material and placed in a single lift and compacted until well keyed.

### **Surface Drainage/Groundwater**

We recommend that positive measures be taken to properly finish grade the site so that drainage waters from the residential and/or commercial structures and landscaping areas as well as adjacent properties or buildings are directed away from the new residential and/or commercial structures foundations and/or floor slabs. All roof drainage should be directed into conduits that carry runoff water away from the residential and/or commercial structures to a suitable outfall. Roof downspouts should not be connected to foundation drains. A minimum ground slope of about 2 percent is generally recommended in unpaved areas around the proposed new residential and/or commercial structures.

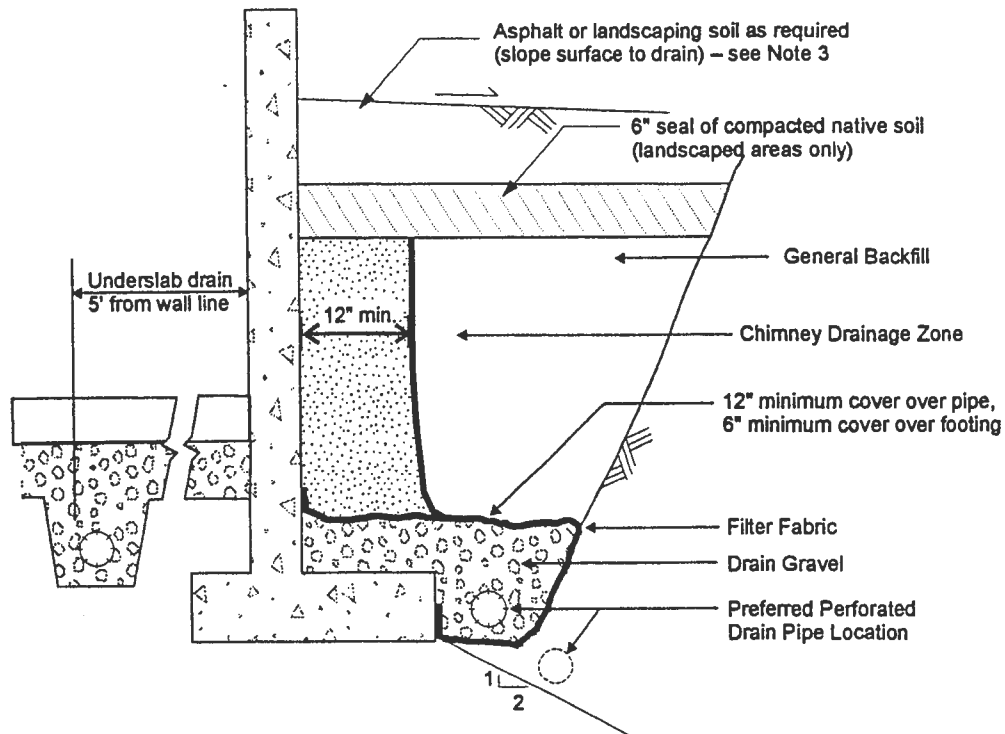
Groundwater was generally encountered at the site in any of the exploratory test pits (TH-#1 through TH-#20) at the time of excavation to depths of at least eight (8) feet beneath existing site grades. However, groundwater seepage was encountered in test hole TH-#13 at a depth of three (3) feet at the time of our field explorations. Additionally, the central and/or northeasterly portion of the site contains an existing seasonal drainage basin feature.

As such, groundwater elevations in the area and/or across the subject property may fluctuate seasonally and may temporarily pond/perch near the ground surface during periods of prolonged rainfall. In this regard, based on our current understanding of the possible site grading required to bring the subject site to finish design grade(s), we are of the opinion that an underslab drainage system is likely not required for the proposed commercial structures. However, a perimeter foundation drain is recommended for any perimeter footings and/or below grade retaining walls. Additionally, residential homes and/or commercial structures constructed at the site with a partial and/or below grade level should consider the use of an underslab drainage system. A typical recommended perimeter footing/retaining wall drain detail is shown on Figure No. 4. Further, due to our understanding that various surface infiltration ditches and/or swales may be utilized for the project as well as the relatively low infiltration rates of the near surface sandy, clayey silt and/or silty clay subgrade soils anticipated within and/or near to the foundation bearing level of the proposed residential and/or commercial structures, we are generally of the opinion that storm water detention and/or disposal systems should not be utilized within the residential lots and/or around the proposed commercial structures unless approved by the Geotechnical Engineer.

### **Design Infiltration Rates**

Based on the results of our field infiltration testing, we recommend using the following infiltration rate to design any on-site near surface storm water infiltration and/or disposal systems for the project:

<b>Subgrade Soil Type</b>	<b>Recommended Infiltration Rate</b>
sandy, clayey SILT (ML)	0.1 to 0.3 inches per hour (in/hr)



**SCHEMATIC - NOT TO SCALE**

**NOTES:**

1. Filter Fabric to be non-woven geotextile (Amoco 4545, Mirafi 140N, or equivalent)
2. Lay perforated drain pipe on minimum 0.5% gradient, widening excavation as required. Maintain pipe above 2:1 slope, as shown.
3. All-granular backfill is recommended for support of slabs, pavements, etc. (see text for structural fill).
4. Drain gravel to be clean, washed  $\frac{3}{4}$ " to  $1\frac{1}{2}$ " gravel.
5. General backfill to be on-site gravels, or  $\frac{3}{4}$ "-0 or  $1\frac{1}{2}$ "-0 crushed rock compacted to 92% Modified Proctor (AASHTO T-180).
6. Chimney drainage zone to be 12" wide (minimum) zone of clean washed, medium to coarse sand or drain gravel if protected with filter fabric. Alternatively, prefabricated drainage structures (Miradrain 6000 or similar) may be used.

**PERIMETER FOOTING/RETAINING WALL DRAIN DETAIL**

Project No. 1721.001.G

**EAST PARK ESTATES SITE  
STATE ST SE AND CORDON RD SE**

Figure No. 4



Note: A safety factor of two (2) was used to calculate the above recommended design infiltration rate. Additionally, given the gradational variability of the on-site sandy, clayey sit subgrade soils beneath the site as well as the anticipation of some site grading for the project, it is generally recommended that field testing be performed during and/or following construction of any on-site storm water infiltration system(s) in order to confirm that the above recommended design infiltration rates are appropriate.

### **Seismic Design Considerations**

Structures at the site should be designed to resist earthquake loading in accordance with the methodology described in the 2014 and/or latest edition of the State of Oregon Structural Specialty Code (OSSC) and/or Amendments to the 2015 International Building Code (IBC). The maximum considered earthquake ground motion for short period and 1.0 period spectral response may be determined from the Oregon Structural Specialty Code and/or from the National Earthquake Hazard Reduction Program (NEHRP) "Recommended Provisions for Seismic Regulations for New Buildings and Other Structures" published by the Building Seismic Safety Council. We recommend Site Class "D" be used for design. Using this information, the structural engineer can select the appropriate site coefficient values ( $F_a$  and  $F_v$ ) from the 2015 IBC to determine the maximum considered earthquake spectral response acceleration for the project. However, we have assumed the following response spectrum for the project:

**Table 1. Recommended Seismic Design Parameters**

Site Class	$S_s$	$S_1$	$F_a$	$F_v$	$S_{M5}$	$S_{M1}$	$S_{D5}$	$S_{D1}$
D	0.903	0.423	1.139	1.577	1.028	0.668	0.686	0.445

Notes: 1.  $S_s$  and  $S_1$  were established based on the IBC 2015 mapped maximum considered earthquake spectral acceleration maps for 2% probability of exceedence in 50 years.

2.  $F_a$  and  $F_v$  were established based on IBC 2015 tables using the selected  $S_s$  and  $S_1$  values.

### **CONSTRUCTION MONITORING AND TESTING**

We recommend that **Redmond Geotechnical Services, LLC** be retained to provide construction monitoring and testing services during all earthwork operations for the proposed new East Park Estates development. The purpose of our monitoring services would be to confirm that the site conditions reported herein are as anticipated, provide field recommendations as required based on the actual conditions encountered, document the activities of the grading contractor and assess his/her compliance with the project specifications and recommendations.



It is important that our representative meet with the contractor prior to any site grading to help establish a plan that will minimize costly over-excavation and site preparation work. Of primary importance will be observations made during site preparation and stripping, structural fill placement, footing excavations and construction as well as retaining wall backfill.

### **CLOSURE AND LIMITATIONS**

This report is intended for the exclusive use of the addressee and/or their representative(s) to use to design and construct the proposed new residential and/or commercial structures and their associated site improvements described herein as well as to prepare any related construction documents. The conclusions and recommendations contained in this report are based on site conditions as they presently exist and assume that the explorations are representative of the subsurface conditions between the explorations and/or at other locations across the study area. The data, analyses, and recommendations herein may not be appropriate for other structures and/or purposes. We recommend that parties contemplating other structures and/or purposes contact our office. In the absence of our written approval, we make no representation and assume no responsibility to other parties regarding this report. Additionally, the above recommendations are contingent on Redmond Geotechnical Services, LLC being retained to provide all site inspections and construction monitoring services for this project. Redmond Geotechnical Services, LLC will not assume any responsibility and/or liability for any engineering judgment, inspection and/or testing services performed by others.

It is the owners/developers responsibility for insuring that the project designers and/or contractors involved with this project implement our recommendations into the final design plans, specifications and/or construction activities for the project. Further, in order to avoid delays during construction, we recommend that the final design plans and specifications for the project be reviewed by our office to evaluate as to whether our recommendations have been properly interpreted and incorporated into the project.

If during any future site grading and construction, subsurface conditions different from those encountered in the explorations are observed or appear to be present beneath excavations, we should be advised immediately so that we may review these conditions and evaluate whether modifications of the design criteria are required. We also should be advised if significant modifications of the proposed site development are anticipated so that we may review our conclusions and recommendations.

### **LEVEL OF CARE**

The services performed by the Geotechnical Engineer for this project have been conducted with that level of care and skill ordinarily exercised by members of the profession currently practicing in the area under similar budget and time restraints. No warranty or other conditions, either expressed or implied, is made.

## REFERENCES

- Adams, John, 1984, Active Deformation of the Pacific Northwest Continental Margin: *Tectonics*, v.3, no. 4, p. 449-472.
- Applied Technology Council, ATC-13, 1985, Earthquake Damage Evaluation Data for California.
- Atwater, B.F., 1992, Geologic evidence for earthquakes during the past 2000 years along the Copalis River, southern coastal Washington: *Journal of Geophysical Research*, v. 97, p. 1901-1919.
- Atwater, B.F., 1987a, A periodic Holocene recurrence of widespread, probably coseismic Subsidence in southwestern Washington: *EOS*, v. 68, no. 44.
- Atwater, B.F., 1987b, Evidence for great Holocene earthquakes along the outer coast of Washington State: *Science*, v. 236, no. 4804, pp. 942-944.
- Campbell, K.W., 1990, Empirical prediction of near-surface soil and soft-rock ground motion for the Diablo Canyon Power Plant site, San Luis Obispo County, California: Dames & Moore report to Lawrence Livermore National Laboratory.
- Carver, G.A., and Burke, R.M., 1987, Late Holocene paleoseismicity of the southern end of the Cascadia Subduction zone [abs.]: *EOS*, v. 68, no. 44, p. 1240.
- Chase, R.L., Tiffin, D.L., Murray, J.W., 1975, The western Canadian continental margin: In Yorath, C.J., Parker, E.R., Glass, D.J., editors, *Canada's continental margins and offshore petroleum exploration: Canadian Society of Petroleum Geologists Memoir 4*, p. 701-721.
- Crouse, C.B., 1991a, Ground motion attenuation equations for earthquakes on the Cascadia Subduction Zone: *Earthquake Spectra*, v. 7, no. 2, pp. 201-236.
- Crouse, C.B., 1991b, Errata to Crouse (1991a), *Earthquake Spectra*, v. 7, no. 3, p. 506.
- Dariento, M.E., and Peterson, C.D., 1987, Episodic tectonic subsidence recorded in late Holocene salt marshes, northern Oregon central Cascadia margin: *Tectonics*, v. 9, p. 1-22.
- Dariento, M.E., and Peterson, C.D., 1987, Episodic tectonic subsidence recorded in late Holocene salt marshes northwest Oregon [abs]: *EOS*, v. 68, no. 44, p. 1469.
- EERI (Earthquake Engineering Research Institute), 1993, The March 25, 1993, Scotts Mill Earthquake, Western Oregon's Wake-Up Call: *EERI Newsletter*, Vol. 27, No. 5, May.
- Geomatrix, 1995 Seismic Design Mapping, State of Oregon: Final Report to Oregon Department of Transportation, January.

Geologic Map Series (GMS-49), Map of Oregon Seismicity, 1841-1986 dated 1986.

Geologic Map of the Salem East Quadrangle, Oregon by James Bela dated 1981.

Grant, W.C., and McLaren, D.D., 1987, Evidence for Holocene Subduction earthquakes along the northern Oregon coast [abs]: EOS v. 68, no. 44, p. 1239.

Grant, W.C., Atwater, B.F., Carver, G.A., Darienzo, M.E., Nelson, A.R., Peterson, C.D., and Vick, G.S., 1989, Radiocarbon dating of late Holocene coastal subsidence above the Cascadia Subduction zone-compilation for Washington, Oregon, and northern California, [abs]: EOS Transactions of the American Geophysical Union, v. 70, p. 1331.

International Conference of Building Officials (ICBO), 1994, Uniform Building Code: 1994 Edition, Whittier, CA. 1994.

Joyner, W.B., and Boore, D.M., 1998, Measurement, characterization and prediction of strong ground motion: Earthquake Engineering and Soil Dynamics II – Recent Advances in Ground Motion Evaluation, ASCE Geotech. Special Publ. No. 20, p. 43-102.

Riddihough, R.P., 1984, Recent movements of the Juan de Fuca plate system: Journal of Geophysical Research, v. 89, no. B8, p. 6980-6994.

Youngs, R.R., Day, S.M., and Stevens, J.L., 1998, Near field ground motions on rock for large Subduction earthquakes: Earthquake Engineering and Soil Dynamics II – Recent Advances in Ground Motion Evaluation, ASCE Geotech. Special Publ. No. 20, p. 445-462.

# Appendix "A"

---

Test Pit Logs and Laboratory Test Data

## **APPENDIX**

### **FIELD EXPLORATIONS AND LABORATORY TESTING**

#### **FIELD EXPLORATION**

Subsurface conditions at the site were explored by excavating twenty (20) exploratory test pits (TH-#1 through TH-#20) on November 26, 2018. The approximate location of the test pit explorations are shown in relation to the existing site features on the Site Exploration Plan, Figure No. 2.

The test pits were excavated using track-mounted excavating equipment in general conformance with ASTM Methods in Vol. 4.08, D-1586-94 and D-1587-83. The test pits were excavated to depths ranging from about 5.0 to 8.0 feet beneath existing site grades. Detailed logs of the test pits are presented on the Log of Test Pits, Figure No's. A-5 through A-14. The soils were classified in accordance with the Unified Soil Classification System (USCS), which is outlined on Figure No. A-4.

The exploration program was coordinated by a field engineer who monitored the excavating and exploration activity, obtained representative samples of the subsurface soils encountered, classified the soils by visual and textural examination, and maintained continuous logs of the subsurface conditions. Disturbed and/or undisturbed samples of the subsurface soils were obtained at appropriate depths and/or intervals and placed in plastic bags and/or with a thin walled ring sample.

Groundwater was generally not encountered in any of the exploratory test pits (TH-#1 through TH-#20) at the time of excavating to depths of at least 8.0 feet beneath existing surface grades except for seepage encountered at a depth of approximately 3 feet in test hole TH-#13.

#### **LABORATORY TESTING**

Pertinent physical and engineering characteristics of the soils encountered during our subsurface investigation were evaluated by a laboratory testing program to be used as a basis for selection of soil design parameters and for correlation purposes. Selected tests were conducted on representative soil samples. The program consisted of tests to evaluate the existing (in-situ) moisture-density, maximum dry density and optimum moisture content, gradational characteristics, and Atterberg Limits as well as consolidation, direct shear strength and "R"-value tests.

##### **Dry Density and Moisture Content Determinations**

Density and moisture content determinations were performed on both disturbed and relatively undisturbed samples from the test pit explorations in general conformance with ASTM Vol. 4.08 Part D-216. The results of these tests were used to calculate existing overburden pressures and to correlate strength and compressibility characteristics of the soils. Test results are shown on the test pit logs at the appropriate sample depths.

### **Maximum Dry Density**

Two (2) Maximum Dry Density and Optimum Moisture Content tests were performed on representative samples of the on-site sandy, clayey silt subgrade soils in accordance with ASTM Vol. 4.08 Part D-1557. This test was conducted to help establish various engineering properties for use as structural fill. The test results are presented on Figure No. A-15.

### **Atterberg Limits**

Three (3) Liquid Limit (LL) and Plastic Limit (PL) tests were performed on representative samples of the sandy, clayey silt and/or silty clay subgrade soils in accordance with ASTM Vol. 4.08 Part D-4318-85. These tests were conducted to facilitate classification of the soils and for correlation purposes. The test results appear on Figure No. A-16.

### **Gradation Analysis**

Two (2) Gradation analyses were performed on representative samples of the subsurface soils in accordance with ASTM Vol. 4.08 Part D-422. The test results were used to classify the soil in accordance with the Unified Soil Classification System (USCS). The test results are shown graphically on Figure No. A-17.

### **Consolidation Test**

One (1) Consolidation test was performed on a representative sample of the sandy, clayey silt subgrade soil to assess the compressibility characteristics of the underlying subgrade soils in accordance with ASTM Vol. 4.08 Part D-2435-80.

Conventional loading increments of 100, 200, 400, ... 12,800 psf were applied after the 100 percent time of primary consolidation was identified for each loading increment. The samples were unloaded and allowed to rebound after the completion of the loading sequence. Deflection versus time readings were recorded for all load increments from 100 through 12,800 psf. The deflection corresponding to 100 percent primary consolidation was plotted on the consolidation strain versus consolidation pressure curve, which is presented on Figure No. A-18.

### **Direct Shear Strength Test**

One (1) Direct Shear Strength test was performed on an undisturbed and/or remolded sample at a continuous rate of shearing deflection (0.02 inches per minute) in accordance with ASTM Vol. 4.08 Part D-3080-79. The test results were used to determine engineering strength properties and are shown graphically on Figure No. A-19.



**"R"-Value Tests**

Four (4) "R"-value tests were performed on a remolded subgrade soil sample in accordance with ASTM Vol. 4.08 Part D-2844. The test results were used to help evaluate the subgrade soils supporting and performance capabilities when subjected to traffic loading. The test results are shown on Figure No's. A-20 and A-21.

The following figures are attached and complete the Appendix:

Figure No. A-4	Key To Exploratory Test Pit Logs
Figure No's. A-5 through A-14	Log of Test Pits
Figure No. A-15	Maximum Dry Density
Figure No. A-16	Atterberg Limits Test Results
Figure No. A-17	Gradation Test Results
Figure No. A-18	Consolidation Test Results
Figure No. A-19	Direct Shear Strength Test Results
Figure No's. A-20 and A-21	Results of "R"-Value Tests
Figure No's. A-22 through A-25	Field Infiltration Test Results

PRIMARY DIVISIONS			GROUP SYMBOL	SECONDARY DIVISIONS
COARSE GRAINED SOILS MORE THAN HALF OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVELS MORE THAN HALF OF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE	CLEAN GRAVELS (LESS THAN 5% FINES)	GW	Well graded gravels, gravel-sand mixtures, little or no fines.
			GP	Poorly graded gravels or gravel-sand mixtures, little or no fines.
		GRAVEL WITH FINES	GM	Silty gravels, gravel-sand-silt mixtures, non-plastic fines.
			GC	Clayey gravels, gravel-sand-clay mixtures, plastic fines.
	SANDS MORE THAN HALF OF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE	CLEAN SANDS (LESS THAN 5% FINES)	SW	Well graded sands, gravelly sands, little or no fines.
			SP	Poorly graded sands or gravelly sands, little or no fines.
		SANDS WITH FINES	SM	Silty sands, sand-silt mixtures, non-plastic fines.
			SC	Clayey sands, sand-clay mixtures, plastic fines.
FINE GRAINED SOILS MORE THAN HALF OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS  LIQUID LIMIT IS LESS THAN 50%		ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity.
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.
			OL	Organic silts and organic silty clays of low plasticity.
	SILTS AND CLAYS  LIQUID LIMIT IS GREATER THAN 50%		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
			CH	Inorganic clays of high plasticity, fat clays.
			OH	Organic clays of medium to high plasticity, organic silts.
HIGHLY ORGANIC SOILS			Pt	Peat and other highly organic soils.

#### DEFINITION OF TERMS

U.S. STANDARD SERIES SIEVE					CLEAR SQUARE SIEVE OPENINGS		
200	40	10	4		3/4"	3"	12"
SILTS AND CLAYS	SAND			GRAVEL		COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE		

#### GRAIN SIZES

SANDS, GRAVELS AND NON-PLASTIC SILTS	BLOWS/FOOT <sup>†</sup>
VERY LOOSE	0 - 4
LOOSE	4 - 10
MEDIUM DENSE	10 - 30
DENSE	30 - 50
VERY DENSE	OVER 50

CLAYS AND PLASTIC SILTS	STRENGTH <sup>‡</sup>	BLOWS/FOOT <sup>†</sup>
VERY SOFT	0 - 1/4	0 - 2
SOFT	1/4 - 1/2	2 - 4
FIRM	1/2 - 1	4 - 8
STIFF	1 - 2	8 - 16
VERY STIFF	2 - 4	16 - 32
HARD	OVER 4	OVER 32

#### RELATIVE DENSITY

<sup>†</sup> Number of blows of 140 pound hammer falling 30 inches to drive a 2 inch O.D. (1-3/8 inch I.D.) split spoon (ASTM D-1586).

<sup>‡</sup> Unconfined compressive strength in tons/sq. ft. as determined by laboratory testing or approximated by the standard penetration test (ASTM D-1586), pocket penetrometer, torvane, or visual observation.

#### CONSISTENCY

#### KEY TO EXPLORATORY TEST PIT LOGS Unified Soil Classification System (ASTM D-2487)

EAST PARK ESTATES SITE

Salem, Oregon

PROJECT NO.

DATE

Figure A-4

1721.001.G

12/21/18



PO Box 20547 • PORTLAND, OREGON 97294

BACKHOE COMPANY: I &amp; E Construction

BUCKET SIZE: 24 inches

DATE: 11/26/18

DEPTH (FEET)	BAG SAMPLE	DENSITY TEST	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	SOIL DESCRIPTION
						TEST PIT NO. TH-#1 ELEVATION 215'±
0					ML	Dark brown, wet, soft, highly organic, clayey, sandy SILT (Topsoil)
					ML	Medium to olive-brown with gray mottling, moist to very moist, medium stiff to stiff, clayey, sandy SILT to sandy, clayey SILT
5						Becomes very moist to wet at 7 feet
10						Total Depth = 8.0 feet No groundwater encountered at time of exploration
15						

TEST PIT NO. TH-#2 ELEVATION 215'±						
0					ML	Dark brown, wet, soft, highly organic, clayey, sandy SILT (Topsoil)
	X			18.8	ML	Medium to olive-brown, with gray mottling, moist to very moist, medium stiff to stiff, clayey, sandy SILT to sandy, clayey SILT
5	X			21.3		
10						Total Depth = 6.0 feet No groundwater encountered at time of exploration
15						

**LOG OF TEST PITS**

PROJECT NO. 1721.001.G

EAST PARK ESTATES SITE

FIGURE NO. A-5

BACKHOE COMPANY: I &amp; E Construction

BUCKET SIZE: 24 inches

DATE: 11/26/18

DEPTH (FEET)	BAG SAMPLE	DENSITY TEST	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	SOIL DESCRIPTION
						TEST PIT NO. TH-#3 ELEVATION 216'±
0					ML	Dark brown, wet, soft, highly organic, clayey, sandy SILT (Topsoil)
					ML	Medium to olive-brown with gray mottling, moist to very moist, medium stiff to stiff, clayey, sandy SILT to sandy, clayey SILT
5						Becomes very moist to wet at 7 feet
10						Total Depth = 8.0 feet No groundwater encountered at time of exploration
15						

TEST PIT NO. TH-#4 ELEVATION 212'±						
0					ML/GM	<u>FILL</u> : Medium to gray-brown, very moist, moderately compacted, clayey and sandy SILT to crushed aggregate base rock with organics
	X			20.4	ML	<u>NATIVE GROUND</u> : Medium to olive-brown with gray mottling, moist to very moist, medium stiff to stiff, clayey, sandy SILT to sandy clayey SILT
5						Total Depth = 6.0 feet No groundwater encountered at time of exploration
10						
15						

**LOG OF TEST PITS**

PROJECT NO. 1721.001.G

EAST PARK ESTATES SITE

FIGURE NO. A-6

BACKHOE COMPANY: I &amp; E Construction

BUCKET SIZE: 24 inches

DATE: 11/26/18

DEPTH (FEET)	BAG SAMPLE	DENSITY TEST	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	SOIL DESCRIPTION
						TEST PIT NO. TH-#5 ELEVATION 214'±
0					GM	<u>FILL</u> : Medium to gray-brown, very moist, moderately compacted, crushed aggregate base rock with organics
5					ML	<u>NATIVE GROUND</u> : Medium to olive-brown with gray mottling, moist to very moist, medium stiff to stiff, clayey, sandy SILT to sandy clayey SILT
10						Total Depth = 6.0 feet No groundwater encountered at time of exploration
15						

TEST PIT NO. TH-#6 ELEVATION 212'±						
0					GM	<u>FILL</u> : Medium to gray-brown, very moist, moderately compacted, crushed aggregate base rock with organics
5	X			23.9	ML	<u>NATIVE GROUND</u> : Medium to olive-brown with gray mottling, moist to very moist, medium stiff to stiff, clayey, sandy SILT to sandy, clayey SILT
	X			26.6		Becomes very moist to wet at 6 feet
10						Total Depth = 7.0 feet No groundwater encountered at time of exploration
15						

**LOG OF TEST PITS**

PROJECT NO. 1721.001.G

EAST PARK ESTATES SITE

FIGURE NO. A-7



BACKHOE COMPANY: I &amp; E Construction

BUCKET SIZE: 24 inches

DATE: 11/26/18

DEPTH (FEET)	BAG SAMPLE	DENSITY TEST	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	SOIL DESCRIPTION
						TEST PIT NO. TH-#7 ELEVATION 211'±
0					GM	FILL: Medium to gray-brown, very moist, moderately compacted, crushed aggregate base rock with organics
5					ML	NATIVE GROUND: Medium to olive-brown, moist to very moist, medium stiff to stiff, clayey sandy SILT to sandy, clayey SILT  Becomes very moist to wet at 6 feet
10						Total Depth = 7.0 feet No groundwater encountered at time of exploration
15						

TEST PIT NO. TH-#8 ELEVATION 211'±						
0					GM	FILL: Gray-brown, very moist to wet, poorly to moderately compacted, crushed aggregate base rock with organics
5					GM ML	FILL: Gray-brown, wet, poorly compacted, mixture of pit run rock and clayey, sandy SILT with organics and wood debris
5					ML	NATIVE GROUND: Dark gray-brown, wet, soft, sandy, clayey SILT (OLD Topsoil Zone)
10					ML	Medium to olive-brown, very moist, medium stiff to stiff, clayey, sandy SILT to sandy, clayey SILT
15						Total Depth = 7.0 feet No groundwater encountered at time of exploration

**LOG OF TEST PITS**

PROJECT NO. 1721.001.G

EAST PARK ESTATES SITE

FIGURE NO. A-8

BACKHOE COMPANY: I &amp; E Construction

BUCKET SIZE: 24 inches

DATE: 11/26/18

DEPTH (FEET)	BAG SAMPLE	DENSITY TEST	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	SOIL DESCRIPTION
						TEST PIT NO. TH-#9 ELEVATION 215'±
0	X			19.5	ML	Dark brown, wet, soft, highly organic, clayey, sandy SILT (Topsoil)
					ML	Medium to olive-brown with gray mottling, very moist, medium stiff to stiff, clayey, sandy SILT to sandy, clayey SILT
5						Total Depth = 5.0 feet No groundwater encountered at time of exploration
10						
15						

TEST PIT NO. TH-#10 ELEVATION 214'±						
0					ML	Dark brown, wet, soft, highly organic, clayey, sandy SILT (Topsoil)
					ML	Medium to olive-brown with gray mottling, very moist, medium stiff to stiff, clayey, sandy SILT to sandy, clayey SILT
5						Becomes very moist to wet at 6 feet
						Total Depth = 7.0 feet No groundwater encountered at time of exploration
10						
15						

**LOG OF TEST PITS**

PROJECT NO. 1721.001.G

EAST PARK ESTATES SITE

FIGURE NO. A-9

BACKHOE COMPANY: I &amp; E Construction

BUCKET SIZE: 24 inches

DATE: 11/26/18

DEPTH (FEET)	BAG SAMPLE	DENSITY TEST	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	SOIL DESCRIPTION
TEST PIT NO. TH-#11 ELEVATION 212'±						
0					ML	Dark brown, wet, very soft, highly organic, sandy, clayey SILT (Topsoil)
	X			24.1	ML	Medium to olive-brown with gray mottling, very moist, medium stiff, clayey, sandy SILT to sandy, clayey SILT
	X			27.7		
5						Total Depth = 5.0 feet No groundwater encountered at time of exploration
10						
15						

TEST PIT NO. TH-#12 ELEVATION 212'±						
0					ML	Dark brown, very moist, soft, organic, clayey, sandy SILT (Topsoil)
					ML	Medium to olive-brown with gray mottling, moist to very moist, medium stiff to stiff, clayey, sandy SILT to sandy, clayey SILT
5						Total Depth = 6.0 feet No groundwater encountered at time of exploration
10						
15						

**LOG OF TEST PITS**

PROJECT NO. 1721.001.G

EAST PARK ESTATES SITE

FIGURE NO. A-10

BACKHOE COMPANY: I &amp; E Construction

BUCKET SIZE: 24 inches

DATE: 11/26/18

DEPTH (FEET)	BAG SAMPLE	DENSITY TEST	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	SOIL DESCRIPTION
						TEST PIT NO. TH-#13 ELEVATION 196'±
0					ML	Medium to gray-brown, very moist to wet, very soft, organic, sandy, clayey SILT (Topsoil/Possible Fill)
	X			33.7	ML/CL	Light gray, wet to saturated, very soft, slightly sandy, clayey SILT to silty CLAY
5	X			44.1	CL	Bluish-gray, saturated, very soft, slightly sandy, silty CLAY
						Total Depth = 7.0 feet Groundwater seepage encountered at 3 feet at time of exploration
10						
15						

						TEST PIT NO. TH-#14 ELEVATION 210'±
0					ML	Dark brown, very moist to wet, very soft, highly organic, clayey, sandy SILT (Topsoil)
	X			20.7	ML	Medium to olive-brown with gray mottling, very moist, medium stiff to stiff, clayey, sandy SILT to sandy, clayey SILT
5						Total Depth = 5.0 feet No groundwater encountered at time of exploration
10						
15						

**LOG OF TEST PITS**

PROJECT NO. 1721.001.G

EAST PARK ESTATES SITE

FIGURE NO. A-11



BACKHOE COMPANY: I &amp; E Construction

BUCKET SIZE: 24 inches

DATE: 11/26/18

DEPTH (FEET)	BAG SAMPLE	DENSITY TEST	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	SOIL DESCRIPTION
						TEST PIT NO. TH-#15 ELEVATION 213'±
0					ML	Dark brown, very moist to wet, very soft, highly organic, clayey, sandy SILT (Topsoil)
					ML	Medium to olive-brown with gray mottling, very moist, medium stiff to stiff, clayey, sandy SILT to sandy, clayey SILT
5						
						Total Depth = 6.0 feet No groundwater encountered at time of exploration
10						
15						

TEST PIT NO. TH-#16 ELEVATION 209'±						
0					ML/ GM	<u>FILL</u> : Medium to gray-brown, very moist to wet, poorly to moderately compacted, mixture of clayey, sandy SILT and crushed aggregate base rock with trace of organics
5					ML	<u>NATIVE GROUND</u> : Dark gray-brown, very moist to wet, soft to medium stiff, sandy, clayey SILT with trace of organics
						Total Depth = 7.0 feet No groundwater encountered at time of exploration
10						
15						

**LOG OF TEST PITS**

PROJECT NO. 1721.001.G

EAST PARK ESTATES SITE

FIGURE NO. A-12



BACKHOE COMPANY: I &amp; E Construction

BUCKET SIZE: 24 inches

DATE: 11/26/18

DEPTH (FEET)	BAG SAMPLE	DENSITY TEST	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	SOIL DESCRIPTION
						TEST PIT NO. TH-#17 ELEVATION 212'±
0					ML	Dark brown, very moist to wet, very soft, highly organic, clayey, sandy SILT (Topsoil)
	X			19.6	ML	Medium to olive-brown with gray mottling, very moist, medium stiff, clayey, sandy SILT to sandy, clayey SILT
5						Total Depth = 5.0 feet No groundwater encountered at time of exploration
10						
15						

TEST PIT NO. TH-#18 ELEVATION 207'±						
0					ML/ GM	<u>FILL</u> : Gray-brown, moist, poorly compacted, clayey, sandy SILT and crushed aggregate base rock with asphalt debris, concrete rubble and brick
					ML	<u>NATIVE GROUND</u> : Dary gray to olive-brown, very moist, soft to medium stiff, organic, sandy, clayey SILT (Old Topsoil Zone)
5					ML	Medium to olive-brown with gray mottling, very moist, medium stiff, clayey, sandy SILT to sandy, clayey SILT
10						Total Depth = 6.0 feet No groundwater encountered at time of exploration
15						

**LOG OF TEST PITS**

PROJECT NO. 1721.001.G

EAST PARK ESTATES SITE

FIGURE NO. A-13

BACKHOE COMPANY: I &amp; E Construction

BUCKET SIZE: 24 inches

DATE: 11/26/18

DEPTH (FEET)	BAG SAMPLE	DENSITY TEST	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	SOIL DESCRIPTION
						TEST PIT NO. TH-#19 ELEVATION 201'±
0					ML	Dark brown, wet, very soft, highly organic, sandy, clayey SILT (Topsoil)
	X			18.8	ML	Medium to olive-brown with gray mottling, very moist, medium stiff, clayey, sandy SILT to sandy, clayey SILT
5	X			26.8		Becomes wet at 5 feet
						Total Depth = 6.0 feet No groundwater encountered at time of exploration
10						
15						

TEST PIT NO. TH-#20 ELEVATION 213'±						
0					GM	<u>FILL</u> : Gray-brown, very moist, moderately compacted, crushed aggregate base rock with organics
					ML	<u>NATIVE GROUND</u> : Dark gray-brown, very moist, medium stiff, clayey, sandy SILT with trace of organics (Old Topsoil Zone)
5					ML	Medium to olive-brown with gray mottling, very moist, medium stiff, clayey, sandy SILT to sandy, clayey SILT
						Total Depth = 6.0 feet No groundwater encountered at time of exploration
10						
15						

**LOG OF TEST PITS**

PROJECT NO. 1721.001.G

EAST PARK ESTATES SITE

FIGURE NO. A-14

### MAXIMUM DENSITY TEST RESULTS

SAMPLE LOCATION	SOIL DESCRIPTION	MAXIMUM DRY DENSITY (pcf)	OPTIMUM MOISTURE CONTENT (%)
TH-#2 @ 2.0'	Medium to olive-brown, clayey, sandy SILT (ML)	110.0	16.0
TH-#14 @ 2.0'	Medium to olive-brown, clayey, sandy SILT (ML)	108.0	17.0

### EXPANSION INDEX TEST RESULTS

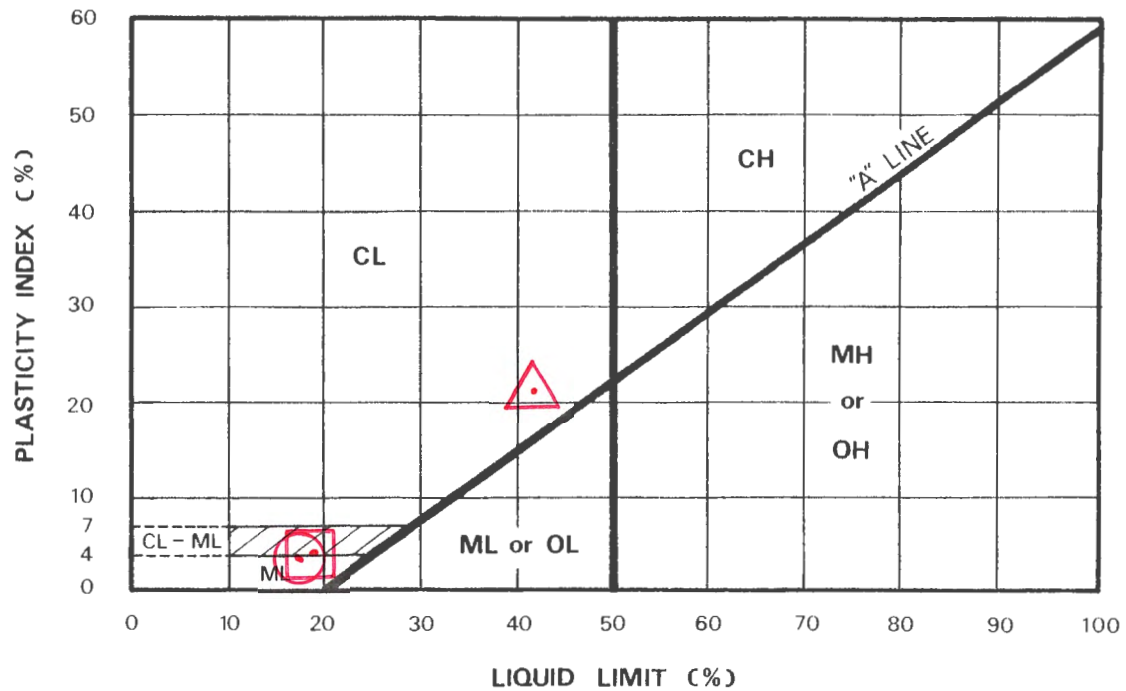
SAMPLE LOCATION	INITIAL MOISTURE (%)	COMPACTED DRY DENSITY (pcf)	FINAL MOISTURE (%)	VOLUMETRIC SWELL (%)	EXPANSION INDEX	EXPANSIVE CLASS.

### MAXIMUM DENSITY & EXPANSION INDEX TEST RESULTS

PROJECT NO.: 1721.001.G

EAST PARK ESTATES SITE

FIGURE NO.: A-15



KEY SYMBOL	BORING NO.	SAMPLE DEPTH (feet)	NATURAL WATER CONTENT %	LIQUID LIMIT %	PLASTICITY INDEX %	PASSING NO. 200 SIEVE %	LIQUIDITY INDEX	UNIFIED SOIL CLASSIFICATION SYMBOL
○	TH-#2	2.0	18.8	17.1	3.9	74.1		ML
△	TH-#13	5.5	44.1	41.8	20.7	----		CL
□	TH-#14	2.0	20.7	18.9	4.2	79.2		ML



PO Box 20547 • PORTLAND, OREGON 97294

### PLASTICITY CHART AND DATA

EAST PARK ESTATES SITE  
Salem, Oregon

PROJECT NO.

DATE

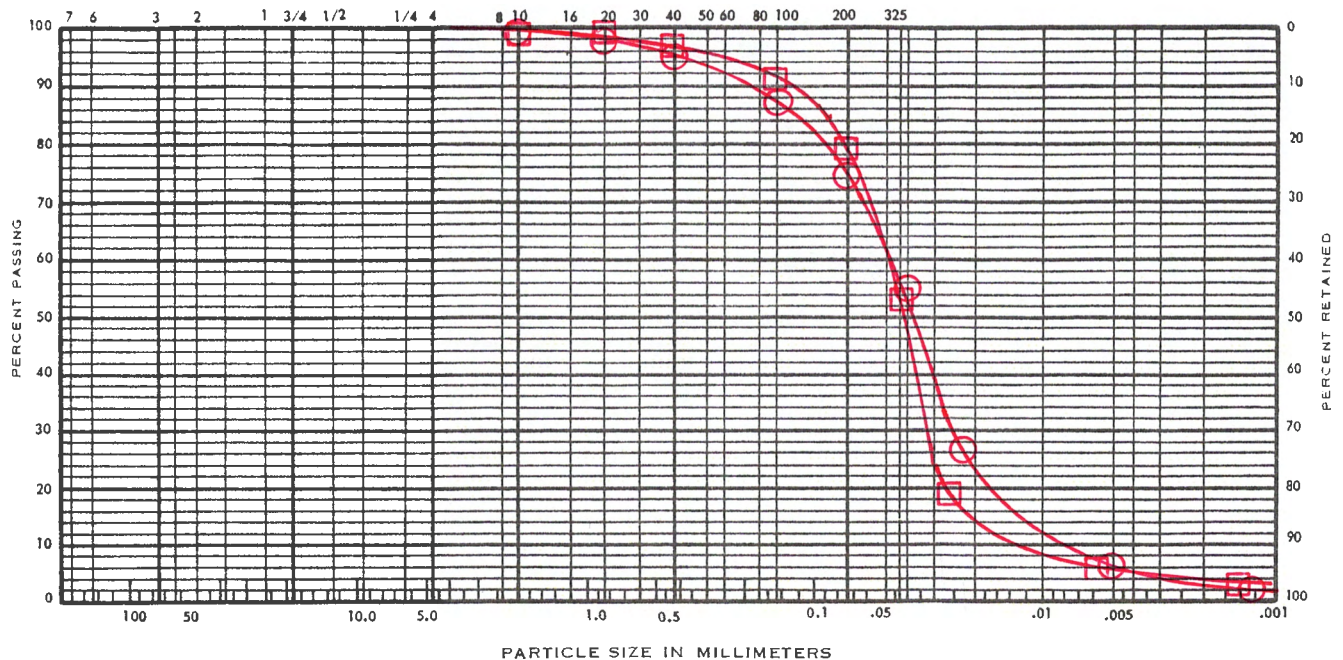
1721.001.G

12/21/18



Figure A-16



U. S. STANDARD SIEVE SIZES



COBBLES	GRAVEL		SAND			SILT AND CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

KEY SYMBOL	BORING NO.	SAMPLE DEPTH (feet)	ELEV. (feet)	UNIFIED SOIL CLASSIFICATION SYMBOL	SAMPLE DESCRIPTION
	TH-#2	2.0		ML	Medium to olive-brown, clayey, sandy SILT
	TH-#14	2.0		ML	Medium to olive-brown, clayey, sandy SILT

## GRADATION TEST DATA

EAST PARK ESTATES SITE  
Salem, Oregon

PROJECT NO.

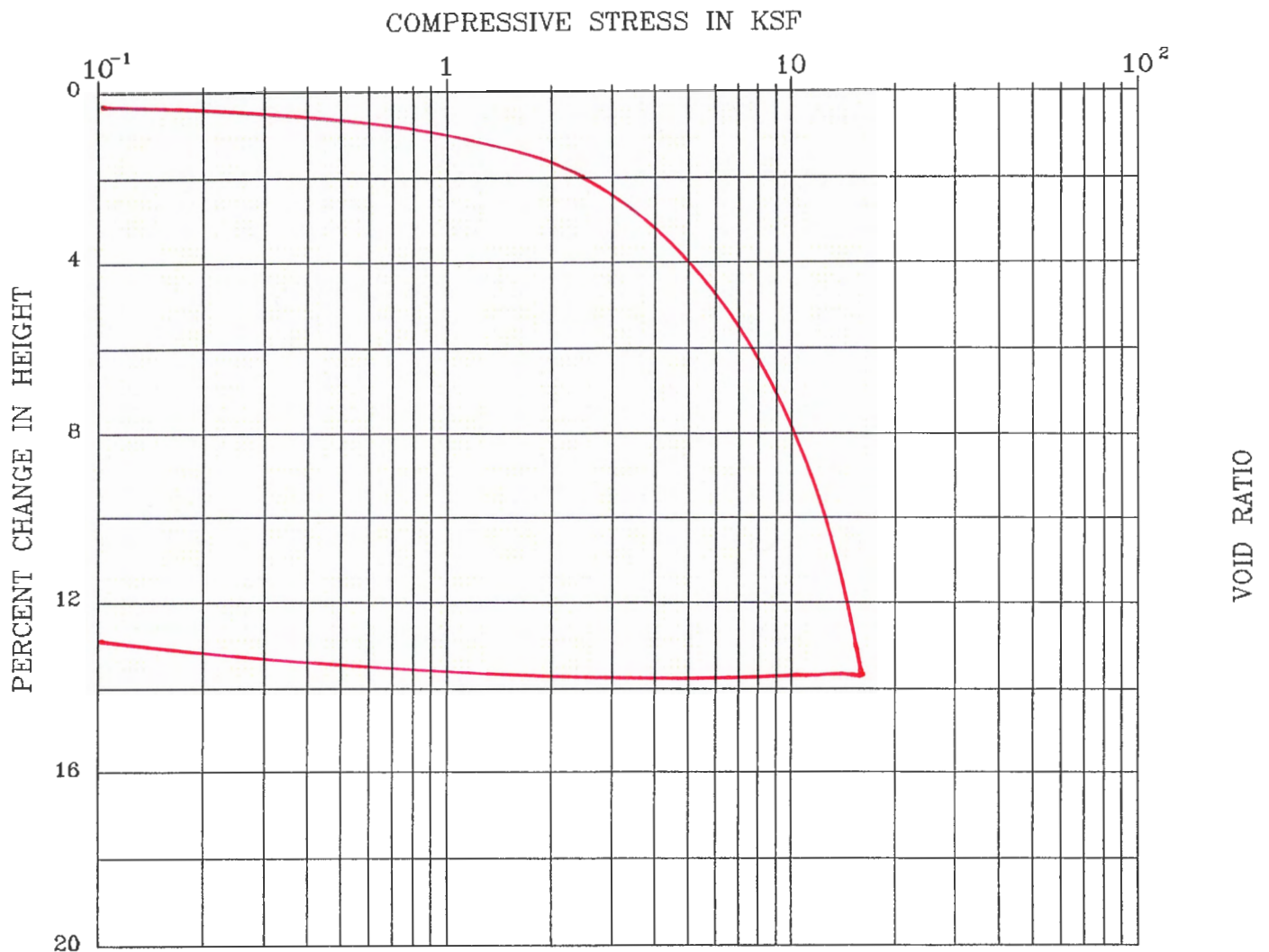
DATE \_\_\_\_\_

FIGURE A-17

1721.001.G

12/21/18





BORING : TH-#2                      DESCRIPTION : clayey, sandy SILT (ML)  
 DEPTH (ft) : 2.0                      LIQUID LIMIT : 17.1  
 SPEC. GRAVITY : 2.5 (assumed)      PLASTIC LIMIT : 13.2

	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	PERCENT SATURATION	VOID RATIO
INITIAL	18.8	89.4	82.3	
FINAL	12.4	97.8	93.5	

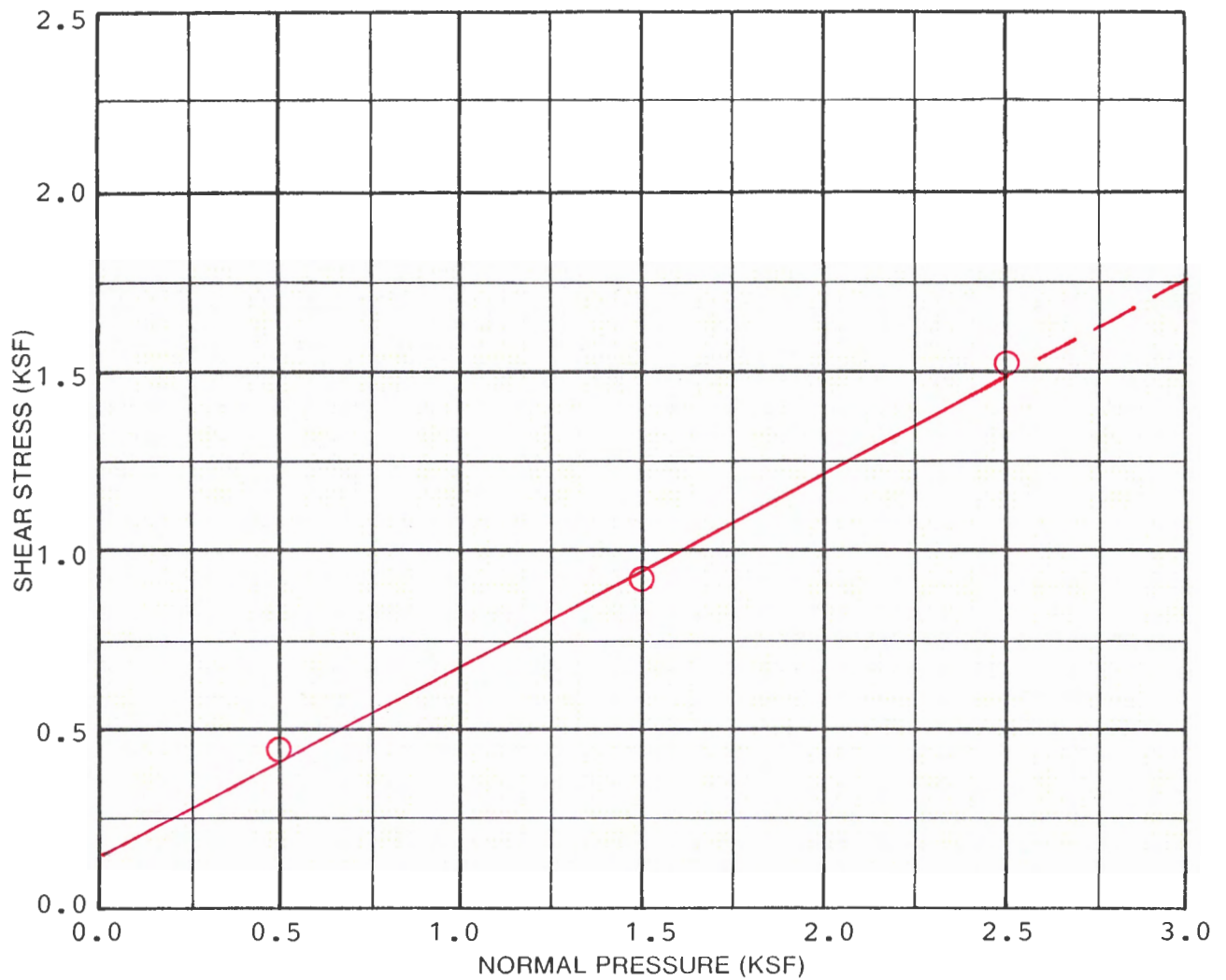


PO Box 20547 • PORTLAND, OREGON 97294

### CONSOLIDATION TEST DATA

EAST PARK ESTATES SITE  
Salem, Oregon

PROJECT NO.	DATE	Figure A-18
1721.001.G	12/21/18	



SAMPLE DATA	
DESCRIPTION: Medium to olive-brown, clayey, sandy SILT (ML) (Remolded)	
BORING NO.: TH-#2	
DEPTH (ft): 2.0	ELEVATION (ft):
TEST RESULTS	
APPARENT COHESION (C): 150 psf	
APPARENT ANGLE OF INTERNAL FRICTION ( $\phi$ ): 28°	

TEST DATA				
TEST NUMBER	1	2	3	4
NORMAL PRESSURE (KSF)	0.5	1.5	2.5	
SHEAR STRENGTH (KSF)	0.4	0.9	1.5	
INITIAL H <sub>2</sub> O CONTENT (%)	17.0	17.0	17.0	
FINAL H <sub>2</sub> O CONTENT (%)	17.6	13.2	8.7	
INITIAL DRY DENSITY (PCF)	98.0	98.0	98.0	
FINAL DRY DENSITY (PCF)	98.6	103.3	108.4	
STRAIN RATE: 0.02 inches per minute				

## **RESULTS OF R (RESISTANCE) VALUE TESTS**

**SAMPLE LOCATION: TH-#2**

**SAMPLE DEPTH: 2.0 feet bgs**

Specimen	A	B	C
Exudation Pressure (psi)	219	329	431
Expansion Dial (0.0001")	0	1	2
Expansion Pressure (psf)	0	3	8
Moisture Content (%)	18.6	15.4	11.1
Dry Density (pcf)	99.4	103.2	108.6
Resistance Value, "R"	15	26	36
"R"-Value at 300 psi Exudation Pressure = 25			

**SAMPLE LOCATION: TH-#6**

**SAMPLE DEPTH: 3.0 feet bgs**

Specimen	A	B	C
Exudation Pressure (psi)	208	326	439
Expansion Dial (0.0001")	0	1	2
Expansion Pressure (psf)	0	3	8
Moisture Content (%)	18.3	15.1	10.7
Dry Density (pcf)	99.9	104.1	109.7
Resistance Value "R"	16	30	40
"R"-Value at 300 psi Exudation Pressure = 29			

## **RESULTS OF R (RESISTANCE) VALUE TESTS**

**SAMPLE LOCATION: TH-#11**

**SAMPLE DEPTH: 2.0 feet bgs**

Specimen	A	B	C
Exudation Pressure (psi)	209	319	421
Expansion Dial (0.0001")	0	1	2
Expansion Pressure (psf)	0	3	8
Moisture Content (%)	18.6	15.6	11.1
Dry Density (pcf)	99.3	103.4	108.7
Resistance Value, "R"	15	27	37
"R"-Value at 300 psi Exudation Pressure = 26			

**SAMPLE LOCATION: TH-#17**

**SAMPLE DEPTH: 2.5 feet bgs**

Specimen	A	B	C
Exudation Pressure (psi)	218	316	429
Expansion Dial (0.0001")	0	1	2
Expansion Pressure (psf)	0	3	8
Moisture Content (%)	18.3	15.2	10.7
Dry Density (pcf)	99.8	104.3	109.2
Resistance Value "R"	16	29	40
"R"-Value at 300 psi Exudation Pressure = 28			

## Division 004 Appendix C - Infiltration Testing

<b>Location:</b> East Park Estates	<b>Date:</b> November 26, 2018	<b>Test Hole:</b> TH-#7
<b>Depth to Bottom of Hole:</b> 4.0 feet	<b>Hole Diameter:</b> 6 inches	<b>Test Method:</b> Encased Falling Head
<b>Tester's Name:</b> Daniel M. Redmond, P.E., G.E.		
<b>Tester's Company:</b> Redmond Geotechnical Services, LLC		<b>Tester's Contact Number:</b> 503-285-0598
<b>Depth (feet)</b>	<b>Soil Characteristics</b>	
0-1.5	Fill: Gray-brown, crushed aggregate (GM)	
1.5-4.0	Medium to olive-brown, clayey, sandy SILT (ML)	

Time	Time Interval (Minutes)	Measurement (inches)	Drop in Water (inches)	Infiltration Rate (inches/hour)	Remarks
11:00	0	36.00	----		Filled w/12" water
11:20	20	36.44	0.44	1.32	
11:40	20	36.79	0.35	1.05	
12:00	20	37.09	0.30	0.90	
12:20	20	37.35	0.26	0.78	
12:40	20	37.58	0.23	0.69	
1:00	20	37.79	0.21	0.63	
1:20	20	37.99	0.20	0.60	
1:40	20	38.19	0.20	0.60	

Infiltration Test Data Table



## Division 004 Appendix C - Infiltration Testing

<b>Location:</b> East Park Estates	<b>Date:</b> November 26, 2018	<b>Test Hole:</b> TH-#8
<b>Depth to Bottom of Hole:</b> 5.0 feet	<b>Hole Diameter:</b> 6 inches	<b>Test Method:</b> Encased Falling Head
<b>Tester's Name:</b> Daniel M. Redmond, P.E., G.E.		
<b>Tester's Company:</b> Redmond Geotechnical Services, LLC		<b>Tester's Contact Number:</b> 503-285-0598
<b>Depth (feet)</b>	<b>Soil Characteristics</b>	
0-4.0	Fill: Gray-brown, crushed aggregate (GM)	
4.0-5.0	Dark gray-brown, sandy, clayey SILT (ML)	

Time	Time Interval (Minutes)	Measurement (inches)	Drop in Water (inches)	Infiltration Rate (inches/hour)	Remarks
11:30	0	48.00	----		Filled w/12" water
11:50	20	48.20	0.20	0.60	
12:10	20	48.35	0.15	0.45	
12:30	20	48.47	0.12	0.36	
12:50	20	48.57	0.10	0.30	
1:10	20	48.66	0.09	0.27	
1:30	20	48.74	0.08	0.24	
1:50	20	48.81	0.07	0.20	
2:10	20	48.88	0.07	0.20	

Infiltration Test Data Table

## Division 004 Appendix C - Infiltration Testing

<b>Location:</b> East Park Estates	<b>Date:</b> November 26, 2018	<b>Test Hole:</b> TH-#11
<b>Depth to Bottom of Hole:</b> 3.0 feet	<b>Hole Diameter:</b> 6 inches	<b>Test Method:</b> Encased Falling Head
<b>Tester's Name:</b> Daniel M. Redmond, P.E., G.E.		
<b>Tester's Company:</b> Redmond Geotechnical Services, LLC		<b>Tester's Contact Number:</b> 503-285-0598
<b>Depth (feet)</b>	<b>Soil Characteristics</b>	
0-1.0	Dark brown Topsoil (ML)	
1.0-3.0	Medium to olive-brown, sandy, clayey SILT (ML)	

Time	Time Interval (Minutes)	Measurement (inches)	Drop in Water (inches)	Infiltration Rate (inches/hour)	Remarks
12:00	0	24.00	----		Filled w/12" water
12:20	20	24.40	0.40	1.20	
12:40	20	24.73	0.33	0.99	
1:00	20	24.98	0.25	0.75	
1:20	20	25.18	0.20	0.60	
1:40	20	25.34	0.16	0.48	
2:00	20	25.48	0.14	0.42	
2:20	20	25.61	0.13	0.40	
2:40	20	25.74	0.13	0.40	

Infiltration Test Data Table

## Division 004 Appendix C - Infiltration Testing

<b>Location:</b> East Park Estates	<b>Date:</b> November 26, 2018	<b>Test Hole:</b> TH-#19
<b>Depth to Bottom of Hole:</b> 4.0 feet	<b>Hole Diameter:</b> 6 inches	<b>Test Method:</b> Encased Falling Head
<b>Tester's Name:</b> Daniel M. Redmond, P.E., G.E.		
<b>Tester's Company:</b> Redmond Geotechnical Services, LLC		<b>Tester's Contact Number:</b> 503-285-0598
<b>Depth (feet)</b>	<b>Soil Characteristics</b>	
0-1.0	Dark brown Topsoil (ML)	
1.0-4.0	Medium to olive-brown, sandy, clayey SILT (ML)	

Time	Time Interval (Minutes)	Measurement (inches)	Drop in Water (inches)	Infiltration Rate (inches/hour)	Remarks
12:30	0	36.00	----		Filled w/12" water
12:50	20	36.40	0.40	1.20	
1:10	20	36.73	0.33	0.99	
1:30	20	37.00	0.27	0.81	
1:50	20	37.22	0.22	0.66	
2:10	20	37.40	0.18	0.54	
2:30	20	37.56	0.16	0.48	
2:50	20	37.71	0.15	0.45	
3:10	20	37.86	0.15	0.45	

Infiltration Test Data Table