October 21, 2022

Mr. Jordan Sparks
Open Dental Software
3311 Marietta Street SE
Salem, Oregon 97317

**Geotechnical Plans Review Letter**
**Marietta - Building H**
**3365 Marietta Street SE**
**Salem, Oregon**

CGT Project Number G2205790

## 1.0 INTRODUCTION

Carlson Geotechnical (CGT), a division of Carlson Testing, Inc. (CTI), is pleased to submit this plans review letter for the proposed Building H at the Marietta Street Development project. The site is located at 3365 Marietta Street SE in Salem, Oregon. This letter was prepared following recent correspondence with the project general contractor, CD Redding, Inc., and is being provided as a continuation of services for the client on this project.

As indicated in their letter titled "B&S Plan Review" dated October 19, 2022, the local jurisdiction, City of Salem, listed the following as a requirement for the project under Item 5: *"Per Chapter 1803, a geotechnical report shall be provided. Please provide a geotechnical report to properly evaluate structural design information."* This letter was prepared to address this item.

## 2.0 BACKGROUND

CGT previously prepared a geology hazards report for the overall Marietta Street Development, the results of which were presented in our "Engineering Geology Report, Marietta Mass Grading Project – Phase 1," dated March 27, 2013 (CGT Project Number G1303960A). That report covered the parcels (Lots 2 and 3) that this current project (Building H) is located within.

GeoEngineers previously performed a geotechnical investigation of the overall Marietta Street Development, the results of which were presented in their report entitled "Geotechnical Engineering Report, Proposed Open Dental Software, Marietta Street Development," dated November 5, 2015. A copy of that report was provided to CGT a few years ago as part of work performed within Phase II of the Marietta Development. The GeoEngineers report covered the parcels (Lots 2 and 3) that this current project (Building H) is located within. For ease of reference, a copy of that report is attached to this report as Appendix A.

## 3.0 PROPOSED DEVELOPMENT

We were provided with a copy of the project civil plans, prepared by Westech Engineering, Inc., dated August 2022, and copies of the project architectural and structural plans, both prepared by Carlson Veit Junge Architects PC, and both dated September 30, 2022. Based on our review, we understand this phase of the project will include:

- Construction of a new, three-story, 13,500 square foot, commercial building within the west portion of Lot 3. The building will incorporate a slab-on-grade ground floor and finished floor elevation (FFE) will be established at 299.2 feet. Permanent grade changes within the building pad area will include cuts of up to about 8 feet and fills up to about 2 feet in depth. Based on review of the provided structural drawings, the building will be supported on conventional shallow foundations. Per Sheets S501 and S502, the foundations were designed based on a maximum allowable soil bearing pressure of 3,000 pounds per square foot (psf). This value is in conformance with that recommended by GeoEngineers in the referenced 2015 report.
- Along the west and south sides of the building, a concrete, cantilevered retaining wall will be constructed to retain site cuts. The retaining wall will be up to about 10 feet in height. Design of the retaining wall will rest with others.
- Construction of new parking lots to the west, east, and north of Building H. The west parking lot will extend off of the south side of the existing parking lot associated with Phase II construction. Permanent grade changes within the parking lot areas will be relatively minimal, with maximum cuts and fills on the order of about 2 feet. The exception to this includes the northeast corner of the east parking lot, where a fill embankment will be constructed to achieve design grades. The fill embankment will be up to about 5 feet in height relative to existing grades and constructed on a descending slope.
- Installation of new underground utilities to serve the new building.

## 4.0 SITE SURFACE CONDITIONS

As part of this assignment, CGT visited the site on October 12, 2022, in order to ascertain present-day surface conditions. The project area is bordered by Marietta Street SE to the south, an AC paved access road and parking lots to the north, Oregon Cryonics to the west, and undeveloped, moderately forested land to the east. At the time of our visit, the site was occupied by a metal barn with fenced storage area, gravel covered parking areas, and short grasses. With the exception of the access road that dissects the center of the area, and descends about 10 feet from Marietta Street SE, the project area sloped gently to the east. Photographs taken during our recent site visit are shown on the attached Figure 1.

## 5.0 GEOTECHNICAL REVIEW & DISCUSSION

### 5.1 Overview

Based on our review of the provided plans, the currently planned project within the aforementioned lots (Lots 2 and 3) is generally consistent with that understood by CGT in 2013 and GeoEngineers in 2015. While some alterations to existing ground surface conditions (e.g. removal of trees, vegetation, installation of gravel working surfaces, etc.) have occurred in Lots 2 and 3 following issuance of those reports, we do not interpret there have been appreciably thick fills placed at the site since that time. In our opinion, the conclusions and recommendations presented in the referenced 2015 geotechnical report may be used for design and construction of this project. Updated recommendations for seismic design of new structures at the site are presented in Section 6.0 of this report. Supplemental geotechnical recommendations for construction of the planned fill embankment near the northeast portion of the site are presented in Section 7.0 of this report.

### 5.2 Review of Slope Stability

Provided the recommendations contained in the referenced 2015 report regarding grading, drainage, and slope setback are incorporated into construction, the proposed project is not anticipated to have an appreciable effect on the hazard posed by slope stability. In addition, this judgment also assumes the planned fill embankment near the northeast portion of the site is constructed in general accordance with the recommendations presented in that report and within Section 7.0 of this report.

### 5.3 Supplemental Test Pits

Consistent with that recommended on page 4 of the referenced 2015 geotechnical report, we recommend supplemental geotechnical explorations (test pits) be performed in the general project area to refine near-surface conditions. In our opinion, the supplemental test pits may be performed at the onset of construction (i.e. once the earthwork contractor has mobilized to the site). As an alternative, the test pits may be performed prior to the onset of construction. In any case, the geotechnical engineer or his representative should be contacted to witness the excavation and backfilling of the supplemental test pits.

## 6.0 UPDATED GEOTECHNICAL RECOMMENDATIONS

We understand the project will be designed per the current (2019) Oregon Structural Specialty Code (OSSC). The above referenced 2015 GeoEngineers report was based on a previous (2014) version of the OSSC. The recommendations that follow are presented for design of new structures designed under the 2019 OSSC.

### 6.1 Seismic Site Class

Section 1613.2.2 of the 2019 Oregon Structural Specialty Code (2019 OSSC) requires that the determination of the seismic site class be in accordance with Chapter 20 of the American Society of Civil Engineers Minimum Design Loads for Buildings and Other Structures (ASCE 7-16). We have assigned the site as Site Class D ("Stiff Soil") based on geologic mapping and subsurface conditions encountered during the previous (2015) geotechnical investigation.

Earthquake ground motion parameters for the site were obtained in accordance with the 2019 OSSC using the Seismic Hazards by Location calculator on the ATC website[1]. The site Latitude 44.889668° North and Longitude 122.993825° West were input as the site location. The following table shows the recommended seismic design parameters for the site.

**Table 1 Seismic Ground Motion Values (2019 OSSC)**

| Parameter | | Value |
|---|---|---|
| Mapped Acceleration Parameters | Spectral Acceleration, 0.2 second ($S_S$) | 0.803g |
| | Spectral Acceleration, 1.0 second ($S_1$) | 0.405g |
| Coefficients (Site Class D) | Site Coefficient, 0.2 second ($F_A$) | 1.179 |
| | Site Coefficient, 1.0 second ($F_V$)[1] | 1.895 |
| Adjusted MCE Spectral Response Parameters | MCE Spectral Acceleration, 0.2 second ($S_{MS}$) | 0.947g |
| | MCE Spectral Acceleration, 1.0 second ($S_{M1}$) | 0.767g |
| Design Spectral Response Accelerations | Design Spectral Acceleration, 0.2 second ($S_{DS}$) | 0.631g |
| | Design Spectral Acceleration, 1.0 second ($S_{D1}$) | 0.511g |
| Seismic Design Category (Risk Category II) | | D |
| [1] Value determined from 2019 OSSC Table 1613.2.3(2). | | |

[1] Applied Technology Council (ATC), 2022. USGS seismic design parameters determined using "Seismic Hazards by Location," *accessed October 2022*, from the ATC website https://hazards.atcouncil.org/.

## 7.0 SUPPLEMENTAL RECOMMENDATIONS - PERMANENT SLOPES

### 7.1 Overview

Permanent cut or fill slopes constructed at the site should be graded at 2H:1V or flatter. Constructed slopes should be overbuilt by a few feet depending on their size and gradient so that they can be properly compacted prior to being cut to final grade. The surface of all slopes should be protected from erosion by seeding, sodding, or other acceptable means. Adjacent on-site and off-site structures should be located at least 5 feet from the top of slopes.

### 7.2 Placement of Fill on Slopes

New fill should be placed and compacted against horizontal surfaces. Where slopes exceed 5H:1V, the slopes should be keyed and benched prior to structural fill placement in general accordance with the attached Fill Slope Detail, Figure 2. If subdrains are needed on benches, subject to the review of the CGT geotechnical representative, they should be placed as shown on the attached Fill Slope Detail. In order to achieve well-compacted slope faces, slopes should be overbuilt by a few feet and then trimmed back to proposed final grades. The geotechnical engineer or his representative should observe the benches, keyways, and associated subdrains, if needed, prior to placement of structural fill.

## 8.0 LIMITATIONS & CLOSURE

This letter is subject to the same terms and conditions included in our agreement of geotechnical construction observation services GP9869, dated and authorized on October 6, 2022. Information contained herein is not to be reproduced, except in full, without prior authorization from this office. We appreciate the opportunity to work with you on this project. Please contact us at (503) 601-8250 if you have any questions regarding this report.

Respectfully Submitted,
**CARLSON GEOTECHNICAL**

REGISTERED PROFESSIONAL ENGINEER
70773PE
OREGON
Jan. 10, 2007
BRAD MICHAEL WILCOX
EXPIRES: 6/30/2024

M. David Irish, CESCL
Geotechnical Project Manager
dirish@carlsontesting.com

Brad M. Wilcox, P.E., G.E.
Principal Geotechnical Engineer
bwilcox@carlsontesting.com

ATTACHMENTS: Figure 1, Site Photographs
Figure 2, Fill Slope Detail
Appendix A, Reproduced 2015 Geotechnical Report

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## *MARIETTA - BUILDING H - SALEM, OREGON*
## *Project Number G2205790*

**FIGURE 1**

**Site Photographs**

Facing northeast towards proposed building area

Facing southeast towards proposed building area

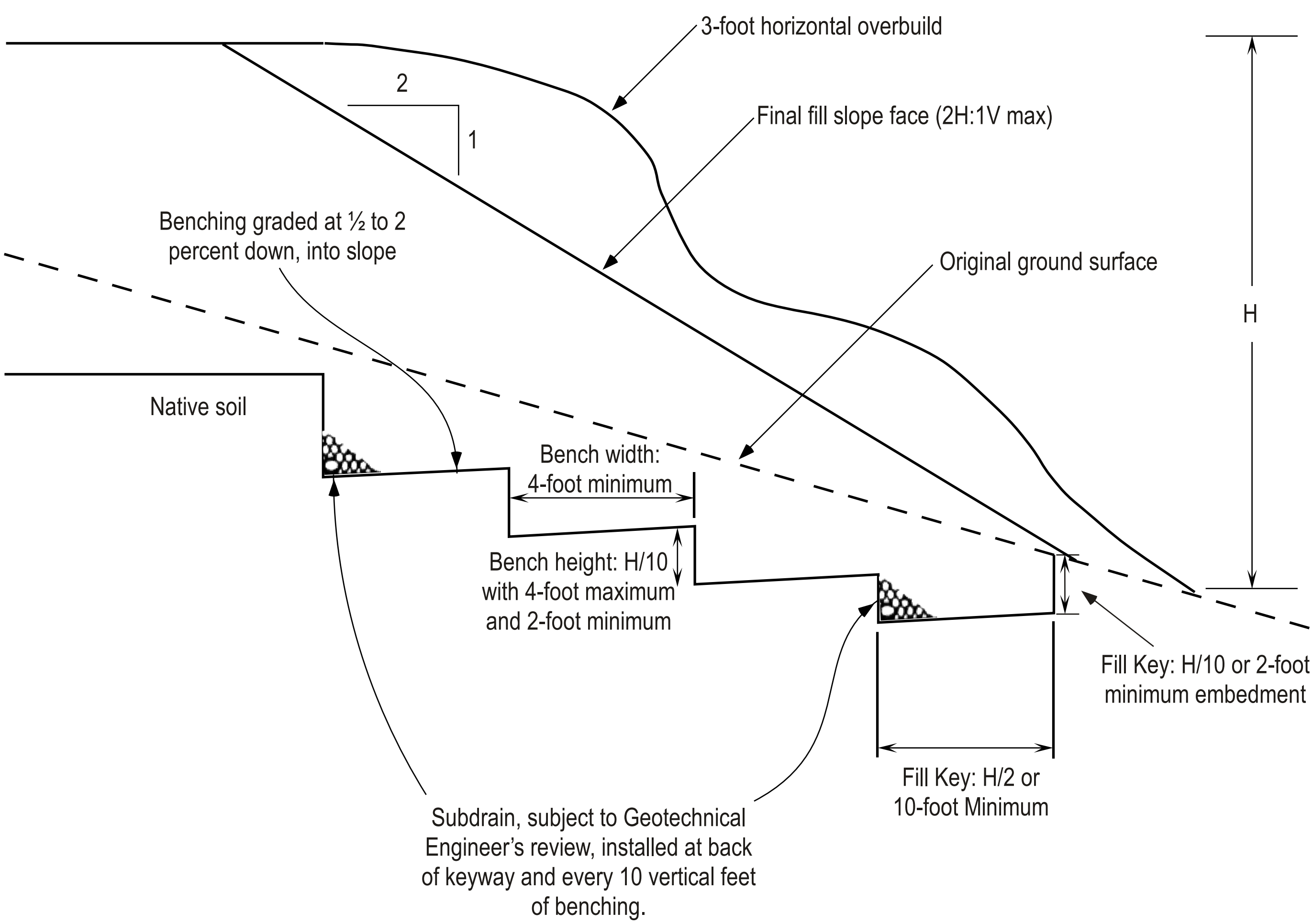
Facing southwest from northeast corner of proposed east parking lot

Facing west/southwest from access road on north side of project area

CARLSON CGT GEOTECHNICAL 503-601-8250

Drafted by: MDI

*Photographs were taken at the time of our site visit on October 12, 2022.*






NOTE: Surfaces to receive fill with slopes steeper than 5H:1V (horizontal:vertical) should be benched and keyed as shown.

# Appendix A: Reproduced 2015 Geotechnical Report

## Marietta - Building H
## 3365 Marietta Street SE
## Salem, Oregon

## CGT Project Number G2205790

October 21, 2022

*Prepared For:*

Mr. Jordan Sparks
Open Dental Software
3311 Marietta Street SE
Salem, Oregon 97317

*Prepared by*
**Carlson Geotechnical**

# Geotechnical Engineering Report

Proposed Open Dental Software
Marietta Street Development
Marietta Street SE and 32nd Avenue SE
Salem, Oregon

*for*

**Westech Engineering, Inc.**

November 5, 2015

GeoEngineers
Earth Science + Technology

# Geotechnical Engineering Report

Proposed Open Dental Software
Marietta Street Development
Marietta Street SE and 32nd Avenue SE
Salem, Oregon

*for*
**Westech Engineering, Inc.**

November 5, 2015

GeoEngineers

333 High Street NE, Suite 102
Salem, Oregon 97301
971.304.3078

# Geotechnical Engineering Report

# Proposed Open Dental Software Marietta Street Development
# Marietta Street SE and 32nd Avenue SE
# Salem, Oregon

**File No. 022126-001-00**

**November 6, 2015**

Westech Engineering, Inc.
3841 Fairview Industrial Drive SE, Suite 100
Salem, Oregon 97302

Attention: Steve Ward, PE

Prepared by:


GeoEngineers, Inc.
333 High Street NE, Suite 102
Salem, Oregon 97301
971.304.3078


REGISTERED PROFESSIONAL ENGINEER
00383
OREGON
FEBRUARY 8, 2000
JULIO C VELA
11/6/15

EXPIRES: 06/30/16


Julio C. Vela, PhD, PE, GE
Principal


TNG:JCV:cje:leh

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**APPENDICES**

## INTRODUCTION

GeoEngineers, Inc. (GeoEngineers) is pleased to submit this geotechnical engineering report for the proposed Open Dental development at the Marietta Street site located in Salem, Oregon. The site is located on the north side of Marietta Street, east of Interstate 5. The location of the site is shown on the Vicinity Map, Figure 1.

Base on the "Overall Site Plan" sheet (Drawing C2.0) provided by Westech Engineering, Inc. (Westech), dated June 19, 2015, we understand the proposed improvements consist of multiple phases of development that include an Open Dental Research and Development (R&D) structure and paved parking, an recreational vehicle (RV) phase of development including pull-through pads, an Open Dental Headquarters phase with a large footprint building and surrounding paved parking, and two future development pads along the south margin of the site. In addition, an existing steep natural grade is present on the east portion of the site. The steep natural grade is well outside of the proposed development areas and will not be impacted by fills or cuts as a part of this development.

## SCOPE OF SERVICES

The purpose of our geotechnical engineering evaluation was to explore the subsurface conditions at the site and provide geotechnical engineering recommendations for designing and constructing the project. We explored the subsurface by completing drilled boring explorations using a track-mounted drill rig. A detailed description of our exploration plan is provided in Appendix A. The specific scope of our services is outlined as follows:

1. Reviewed information regarding subsurface soil and groundwater at the site, including reports in our files, selected geologic maps, and other geotechnical engineering related information.
2. Conducted a site geologic reconnaissance walk-through of the slopes in the east portion of the site in accordance with Salem Revised Code Chapter 810 to provide an opinion of the geologic hazard of the existing slope.
3. Coordinated and managed the field investigation, including public utility notification and scheduling of subcontractors and GeoEngineers' field staff.
4. Completed the following explorations:
    - Seven borings in the proposed areas for the Open Dental R&D structure and paved parking, the RV phase of development, the Open Dental Headquarters phase and surrounding paved parking, and in the two future development pads along the south margin of the site to depths of 15 feet below ground surface (bgs). We encountered drilling refusal at two boring locations at approximately 10 to 12 feet bgs on large boulders.
    - Three borings were completed on Marietta Street SE where proposed partial-street improvements are to be completed as part of the project development. Borings were extended to 5 feet bgs.
5. Completed three dynamic cone penetration (DCP) tests along Marietta Street SE.

    Performed two infiltration tests at two locations along the west side of the site where the storm water detention area is proposed.

    Boring, DCP, and infiltration test locations are shown on Figure 2.

6. Obtained soil samples at representative intervals from the explorations, observed groundwater conditions and maintained detailed logs in general accordance with ASTM International (ASTM) Standard Practices Test Method D 2488.
7. Completed the following laboratory tests on select samples obtained from the explorations:
   - Ten moisture content determinations
   - Two percent fines determinations
   - One CBR test
8. Provided a geotechnical evaluation of the site and provided design recommendations in this geotechnical report to address the following geotechnical components:
   a. A general description of site topography, geology including geologic hazard relative to the existing slope, and subsurface conditions;

   b. An opinion as to the adequacy of the proposed development from a geotechnical engineering standpoint;

   c. Recommendations for site preparation measures, including disposition of undocumented fill and unsuitable native soils, recommendations for temporary cut slopes and constraints for wet weather construction;

   d. Recommendations for temporary excavation and temporary excavation protection, such as excavation sheeting and bracing;

   e. Recommendations for earthwork construction, including use of on-site and imported structural fill and fill placement and compaction requirements;

   f. Provide geotechnical engineering recommendations for use in designing conventional retaining walls, including backfill and drainage requirements and static and seismic lateral earth pressures;

   g. Recommendations for foundations to support the proposed structures, including minimum width and embedment, design soil bearing pressures, settlement estimates (total and differential), coefficient of friction and passive earth pressures for sliding resistance. We have assumed that shallow foundations can be used to adequately support the structures;

   h. Recommendations for supporting on-grade slabs, including base rock, capillary break, and modulus of subgrade reaction;

   i. Seismic design parameters, including soil site class evaluation in accordance with the current version of the International Building Code (IBC); and

   j. Recommendations for constructing on-site asphaltic concrete (AC) pavements and for half-street improvements on Marietta Street, including subgrade, drainage, base rock and pavement section. Our recommendations will be based on proposed traffic loads or loads provided by the project civil engineer and on subsurface and DCP data obtained as a part of this scope of work.

## SITE CONDITIONS

### Surface Conditions

The area proposed for development is located on the north side of Marietta Street, east of Interstate 5. This area is approximately 6.5 acres in size and the majority of the site is relatively level. The site surface consists of a brown silt which appears to have been recently hydroseeded. Slopes are present near the perimeter of the site that is generally flat as a result of significant cuts of up to 10 feet, conducted at the site to export material to an adjacent development. Marietta Street SE consists of a paved roadway running east to west just south of the proposed development as shown on the Site Plan, Figure 2. The "Overall Site Plan" sheet (Drawing C2.0) was provided by Westech, dated June 19, 2015, and explorations performed in the field were added to this baseline drawing.

### Site Geology

The site is located in the Willamette Valley physiographic province, which extends approximately from Cottage Grove, Oregon, in the south to the Columbia River in the north (Orr and Orr, 1999). The Willamette Valley province is a part of the larger lowland that is a tectonically active forearc basin located along the convergent Cascadia margin. In general, the lowland is an elongated alluvial/fluvial plain that is bounded by the Coast Range on the west and by the Cascade Mountains on the east. The Mill Creek Valley, where the site is located, has been cut between Columbia River Basalt highlands set within the Willamette Valley province, consistent with the exposed weathered basalt slopes along the site margins.

Published geologic maps of the area (Tolan and Beeson, 2000) indicate that most of the area adjacent to the project site is underlain by unconsolidated alluvium comprised of intercalated silt, sand, and gravel with a maximum thickness of approximately 15 feet. These are underlain by older alluvial deposits, including moderately indurated siltstones, sandstones, and conglomerates, as well as poorly indurated clay and silt. The older alluvial deposits may be over 100 feet thick and overlie basement rocks of the Columbia River Basalt Group and possibly older volcanic rocks.

Both the unconsolidated alluvium and the older alluvium were deposited in stream channels and adjacent floodplains of the drainage system flowing through the Mill Creek Valley. These alluvial deposits typically occur in flat, moderately to poorly drained areas characterized by low relief and shallow groundwater.

The highlands present in the Mill Creek Valley such as where the site is situated, consist of several formations of the Columbia River Basalt Group, which have been weathered to varying extent. Based on geologic mapping and consistent with our experience in the area, sandstone and siltstone, with a maximum thickness of approximately 10 feet, are locally interlayered with the basalt.

Our review of the site geology, together with on-site observations, suggests that the site geology is generally consistent with published geologic mapping.

### Subsurface Conditions

We completed field explorations at the site on October 20$^{th}$ and 21$^{st}$, 2015. Our explorations included seven drilled borings to depths ranging between 10 and 15 feet bgs, two infiltration test borings to depths between 3 and 5 feet bgs, and three dynamic cone penetrometer soundings and drilled borings to a depth of 5 feet bgs. The approximate locations of the explorations are shown on Figure 2. Appendix A summarizes our exploration methods and presents our exploration logs. Laboratory test results are provided on the exploration logs and described in Appendix A.

In general, subsurface conditions consist of a stiff to very hard brown to reddish brown silt with varying amounts of sand and gravel. The silt in our borings ranged from a silt with trace fine sand and clay, to a sandy silt with gravel. The native site soils are a residual soil from severely weathered basalt formation that comprise the highland area consistent with the geology described above. All borings terminated in this deposit except for B-5 and B-9 where boulders were encountered at 9.5 and 12 feet bgs, respectively.

The upper 15 feet of soil encountered in the area of future development in the southeast portion of the site (B-10) are classified as fill. The fill soil encountered in the area are generally very stiff, based on in-place sampling. Depending on the extent of future development in the area, additional explorations should be conducted to confirm that the soil in the fill area is adequate for structural support.

#### *Asphalt Concrete and Base*

At the ground surface on Marietta Street at each boring location, we encountered approximately 2.5 inches of AC pavement. Below the pavement, we encountered approximately 6 to 11 inches of sandy coarse gravel or sandy coarse gravel with silt, which we identified as the aggregate base course.

### Groundwater

Groundwater was not encountered in any of our explorations, but may be present at shallow depths in a perched condition during wet times of the year or during extended periods of wet weather. Dewatering of trenches and excavations may be required when groundwater seepage and/or perched groundwater are encountered. Groundwater conditions at the site are expected to vary seasonally due to rainfall events and other factors not observed in our explorations.

## CONCLUSIONS

### General

Based on our explorations, testing, and analyses, it is our opinion that the site is suitable for the proposed project from a geotechnical standpoint, provided the recommendations in this report are included in design and construction. We offer the following conclusions regarding geotechnical design at the site.

- Where present, organic-rich surface material to below the root zone should be stripped from all areas to be improved.
- Because of the fine-grained composition of the site surface soils, they will become significantly disturbed if earthwork or construction traffic over the site occurs during periods of wet weather or when the moisture content of the soil is more than a few percentage points above optimum. Wet weather construction practices will be required over exposed native soils, except during the dry summer months.
- Proposed structures can be satisfactorily supported on continuous and isolated shallow foundations supported on the stiff native soils or on structural fill that extends to native soil.
- Based on proposed development, we estimate maximum anticipated loads of 100 kips or less for columns, 4 kips per lineal foot (klf) or less for walls, and floor loads of 150 pounds per square foot (psf)

or less. Based on these assumed design loads, we estimate total settlement to be less than 1 inch. If larger structural loads are anticipated, we should review and reassess the estimated settlement.

- Groundwater was not encountered during our explorations, but based on our experience and our observations, perched groundwater may be present during periods of persistent rainfall.
- Slabs on grade will be satisfactorily supported on stiff native soils with a minimum 6 inch capillary break overlying approved subgrade or on structural fill over stiff native soils.
- Standard pavement sections prepared as described in this report will suitably support estimated traffic loads.
- Based on our testing, infiltration rates are very near zero. The upper soils in the area tested are very stiff and predominantly fine-grained. Two tests were conducted in the native soils at the west side of the project site. Additional tests were attempted in the area proposed for stormwater detention on the east side of the site, but the soils were so stiff that hand augers could not be advanced beyond a few inches from the surface. The final grading plan should account for adequate drainage away from structural elements and roadways. The design of the stormwater detention area should also take into account the very low infiltration rates.
- Based on a geologic reconnaissance of the site, the proposed project and associated earthwork does not present a hazard for increasing landslide potential.

## EARTHWORK RECOMMENDATIONS

### Site Preparation

On-site areas have generally been cut and graded as a part of material excavation and export for use as fill at the adjacent site. Existing vegetation, including hydroseeded areas from surface soil stabilization efforts should be stripped and removed from all proposed improvement areas. Generally, site preparation and earthwork operations will include, removing existing pavements within the improvement area, stripping and grubbing, grading the site, and excavating for utilities and foundations.

If present, existing utilities in proposed building areas should be identified prior to excavation. Live utility lines identified beneath proposed structures should be relocated. Abandoned utility lines beneath structures should be completely removed or filled with grout in order to reduce potential settlement of new structures. Soft or loose soil encountered in utility line excavations should be removed and replaced with structural fill where it is located within structural areas.

Materials generated during demolition of existing improvements should be transported off site for disposal. Existing voids and new depressions created during site preparation, and resulting from removal of existing utilities, or other subsurface elements, should be cleaned of loose soil or debris down to firm soil and backfilled with compacted structural fill. Disturbance to a greater depth should be expected if site preparation and earthwork are conducted during periods of wet weather.

#### *Stripping and Clearing*

Based on our observations at the site, we estimate that the depth of stripping should generally be on the order of about 2 to 3 inches. Greater stripping depths may be required to remove localized zones of loose or organic soil, and in areas where moderate to heavy vegetation may be present near site margins, or where surface disturbance has occurred. In addition, if present in the areas of development, the primary root systems of shrubs and trees should be completely removed. Stripped material should be transported off site for disposal or processed and used as fill in landscaping areas.

We encountered boulders or less decomposed rock during our subsurface explorations approximately 10 feet bgs, and they could be present near the surface in the project area. Accordingly, the contractor should be prepared to remove boulders, if encountered during grading or utility excavations. Boulders may be removed from the site or buried in landscape areas. Voids caused by boulder removal should be backfilled with structural fill.

### Excavation

Based on the materials encountered in our subsurface exploration, it is our opinion that conventional earthmoving equipment in proper working condition should be capable of making necessary general excavations.

The earthwork contractor should be responsible for reviewing this report, including the boring logs, providing their own assessments, and providing equipment and methods needed to excavate the site soils while protecting subgrades.

### Dewatering

As discussed in the "Groundwater" section of this report, groundwater was not encountered in our explorations, and we do not expect groundwater to be a major factor during shallow excavations and earthwork. Excavations that extend into saturated/wet soils or excavations that extend into perched groundwater should be dewatered. Sump pumps are expected to adequately address groundwater encountered in shallow excavations. In addition to groundwater seepage, surface water inflow to the excavations during the wet season can be problematic. Provisions for surface water control during earthwork and excavations should be included in the project plans and should be installed prior to commencing earthwork.

### Trench Cuts and Trench Shoring

All trench excavations should be made in accordance with applicable Occupational Safety and Health Administration (OSHA) and state regulations. In our opinion, native soils are generally OSHA Type B. Excavations deeper than 4 feet should be shored or laid back at an inclination of 1H:1V (horizontal to vertical) or flatter if workers are required to enter. Excavations made to construct footings or other structural elements should be laid back or shored at the surface as necessary to prevent soil from falling into excavations.

Shoring for trenches less than 6 feet deep that are above the effects of groundwater should be possible with a conventional box system. Moderate sloughing should be expected outside the box. Shoring deeper than 6 feet or below the groundwater table should be designed by a registered engineer before installation. Further, the shoring design engineer should be provided with a copy of this report.

It should be expected that unsupported cut slopes will experience some sloughing and raveling if exposed to water. Plastic sheeting, placed over the exposed slope and directing water away from the slope, will reduce the potential for sloughing and erosion of cut slopes during wet weather.

In our opinion, the contractor will be in the best position to observe subsurface conditions continuously throughout the construction process and to respond to the soil and groundwater conditions. Construction site safety is generally the sole responsibility of the contractor, who also is solely responsible for the means, methods, and sequencing of the construction operations and choices regarding excavations and shoring. Under no circumstances should the information provided by GeoEngineers be interpreted to mean that GeoEngineers is assuming responsibility for construction site safety or the contractor's activities; such responsibility is not being implied and should not be inferred.

### Subgrade Preparation and Evaluation

Disturbed material may be present after site stripping is complete. Subgrade areas should be prepared to be in a uniformly stiff and unyielding condition prior to placement of structural fill or structural elements. We recommend that prepared subgrades be observed by a member of our firm, who will evaluate the suitability of the subgrade and identify any areas of yielding which are indicative of soft or loose soil. The exposed subgrade soil should be proof-rolled with heavy rubber-tired equipment and/or probed with a ½-inch-diameter steel rod, as appropriate depending on prevailing conditions. If soft, yielding or otherwise unsuitable areas revealed during probing or proof-rolling cannot be compacted to a stable and uniformly firm condition, we recommend that: (1) the subgrade soils be scarified (e.g., with a ripper or a farmer's disc), aerated and recompacted; or (2) the unsuitable soils be removed and replaced with structural fill, as needed.

### Subgrade Protection and Wet Weather Considerations

The fine-grained soils at the site are highly susceptible to moisture. Wet weather construction practices will be necessary if work is performed during periods of wet weather. If site grading will occur during wet weather conditions, it will be necessary to use track-mounted equipment, load removed material into trucks supported on existing gravel surfacing, use gravel working pads, and employ other methods to reduce ground disturbance. The contractor should be responsible to protect the subgrade during construction.

Earthwork planning should include considerations for minimizing subgrade disturbance. We provide the following recommendations if wet weather construction is considered:

- The ground surface in and around the work area should be sloped so that surface water is directed to a sump or discharge location. The ground surface should be graded such that areas of ponded water do not develop. Measures should be taken by the contractor to prevent surface water from collecting in excavations and trenches. Measures should be implemented to remove surface water from the work area.
- Earthwork activities should not take place during periods of heavy precipitation.
- Slopes with exposed soils should be covered with plastic sheeting or similar means.
- The site soils should not be left uncompacted and exposed to moisture. Sealing the surficial soils by rolling with a smooth-drum roller prior to periods of precipitation will reduce the extent to which these soils become wet or unstable.
- Construction activities should be scheduled so that the length of time that soils are left exposed to moisture is reduced to the extent practicable.

- Construction traffic should be restricted to specific areas of the site, preferably areas that are surfaced with working pad materials not susceptible to wet weather disturbance such as haul roads and rocked staging areas.
- When on-site fine-grained soils are wet of optimum, they are easily disturbed and will not provide adequate support for construction traffic or the proposed development. The use of granular haul roads and staging areas will be necessary for support of construction traffic. Generally a 12- to 16-inch thick mat of imported granular base rock aggregate material is sufficient for light staging areas for the building pad and light staging activities, but is not expected to be adequate to support repeated heavy equipment or truck traffic. The granular mat for haul roads and areas with repeated heavy construction traffic should be increased to between 18 and 24 inches. The actual thickness of haul roads and staging areas should be based on the contractor's approach to site development and the amount and type of construction traffic.
- The base rock thickness described in the "Pavement Recommendations" section of this report is intended to support post-construction design traffic loads. The design base rock thickness may not support construction traffic or pavement construction. A thicker base rock section may be required to support construction traffic as noted above.
- During periods of wet weather, concrete should be placed as soon as practical after preparation of the footing excavations. Foundation bearing surfaces should not be exposed to standing water. Should water infiltrate and pool in the excavation, it should be removed before placing structural fill or reinforcing steel. Subgrade protection for foundations consisting of a lean concrete mat may be necessary if footing excavations are exposed to extended wet weather conditions.

During wet weather, or when the exposed subgrade is wet or unsuitable for proof-rolling, the prepared subgrade should be evaluated by observing excavation activity and probing with a steel foundation probe. Observations, probing, and compaction testing should be performed by a member of our staff. Wet soil that has been disturbed due to site preparation activities or soft or loose zones identified during probing should be removed and replaced with compacted structural fill.

### Soil Amendment with Cement

As an alternative to the use of imported granular material for wet weather structural fill, an experienced contractor may be able to amend the on-site soil with Portland cement or with limekiln dust and Portland cement to obtain suitable support properties. Successful use of soil amendment depends on the use of correct mixing techniques, soil moisture content, and amendment quantities. Specific recommendations, based on exposed site conditions, for soil amending can be provided if necessary. However, for preliminary planning purposes, it may be assumed that a minimum of 5 percent cement (by dry weight, assuming a unit weight of 100 pounds per cubic foot [pcf]) will be sufficient for subgrade and general fill amendment. Treatment depths of 12 to 16 inches for roadway subgrades are typical (assuming a seven-day unconfined compressive strength of at least 80 pounds per square inch [psi]), though they may be adjusted in the field depending on site conditions. Soil amending should be conducted in accordance with the specifications provided in Oregon Structural Specialty Code (OSSC) 00344 (Treated Subgrade).

Portland cement-amended soil is hard and has low permeability; therefore, this soil does not drain well nor is it suitable for planting. Future landscape areas should not be cement amended, if practical, or accommodations should be planned for drainage and planting. Cement amendment should not be used if runoff during construction cannot be directed away from adjacent low-lying, wet areas and active waterways and drainage paths.

We recommend a target strength for cement-amended soils of 80 psi. The amount of cement used to achieve this target generally varies with moisture content and soil type. It is difficult to predict field performance of soil to cement amendment due to variability in soil response, and we recommend laboratory testing to confirm expectations. However, for preliminary design purposes, 4 to 5 percent cement by weight of dry soil can generally be used when the soil moisture content does not exceed approximately 20 percent. If the soil moisture content is in the range of 20 to 35 percent, 5 to 7 percent by weight of dry soil is recommended. The amount of cement added to the soil may need to be adjusted based on field observations and performance.

When used for construction of pavement, staging, or haul road subgrades, the amended surface should be protected from abrasion by placing a minimum 4-inch thickness of crushed rock. To prevent strength loss during curing, cement-amended soil should be allowed to cure for a minimum of four days prior to placing the crushed rock. The crushed rock may typically become contaminated with soil during construction. Contaminated base rock should be removed and replaced with clean rock in pavement areas such that the minimum thickness of free-draining base at the surface is 4 inches.

It is not possible to amend soil during heavy or continuous rainfall. Work should be completed during suitable conditions.

## Erosion Control

Erosion control plans are required on construction projects located within Marion County in accordance with Oregon Administrative Rules (OAR) 340-41-006 and 340-41-455 and City of Salem (City) regulations. Measures that can be employed to reduce erosion include the use of silt fences, hay bales, buffer zones of natural growth, sedimentation ponds, and granular haul roads.

## Fill Materials

### *General*

Structural areas include areas beneath foundations, floor slabs, pavements, and any other areas intended to support structures or within the influence zone of structures should generally meet the criteria for structural fill presented below. All structural fill soils should be free of debris, clay balls, roots, organic matter, frozen soil, man-made contaminants, particles with greatest dimension exceeding 4 inches (3 inch maximum particle size in building footprints), and other deleterious materials. The suitability of soil for use as structural fill will depend on the gradation and moisture content of the soil. As the amount of fines in the soil matrix increases, the soil becomes increasingly more sensitive to small changes in moisture content and achieving the required degree of compaction becomes more difficult or impossible. Recommendations for suitable fill material are provided in the following sections.

### *On-Site Soils*

On-site soils generally consist of a native brown to red silt to a sandy silt with trace gravel. On-site soils are suitable for reuse as structural fill provided the maximum particle size is less than 4 inches in diameter, the soil is adequately moisture conditioned, and free of organic debris. As observed during placement on the adjacent site as fill, the on-site soils are generally friable and can be well processed and compacted provided they are properly moisture conditioned. When used as structural fill, the on-site soil should be placed in lifts with a maximum uncompacted thickness of 8 inches. The silt should be compacted to not less than 92 percent of the maximum dry density (MDD), as determined by ASTM D 1557. The site soil is sensitive to small changes in moisture content and highly susceptible to disturbance when wet. Use of the on-site fine-grained soils as structural fill will be very difficult or may not be possible during wet weather (See "Wet Weather Construction" section of this report).

#### *Imported Select Structural Fill*

Select imported granular material may be used as structural fill. The imported material should consist of pit or quarry run rock, crushed rock, or crushed gravel and sand that is fairly well-graded between coarse and fine sizes (approximately 25 to 65 percent passing the U.S. No. 4 sieve). It should have less than 5 percent passing the U.S. No. 200 sieve and have a minimum of 75 percent fractured particles according to American Association of State Highway and Transportation Officials (AASHTO) TP-61.

#### *Aggregate Base*

Aggregate base material located under floor slabs and pavements, crushed rock used in footing overexcavations and retaining wall backfill should consist of imported clean, durable, crushed angular rock. Such rock should be well-graded, have a maximum particle size of 1 inch, have less than 5 percent passing the U.S. No. 200 sieve (3 percent for retaining walls), and meet the gradation requirements in Table 1. In addition, aggregate base shall have a minimum of 75 percent fractured particles according to AASHTO TP-61 and a sand equivalent of not less than 30 percent based on AASHTO T-176.

**TABLE 1. RECOMMENDED GRADATION FOR AGGREGATE BASE**

| Sieve size | Percent Passing (by weight) |
|---|---|
| 1 inch | 100 |
| 1/2 inch | 50 to 65 |
| No. 4 | 40 to 60 |
| No. 40 | 5 to 15 |
| No. 200 | 0 to 5 |

#### *Trench Backfill*

Backfill for pipe bedding and in the pipe zone should consist of well-graded granular material with a maximum particle size of 3/4 inch and less than 5 percent passing the U.S. No. 200 sieve. The material should be free of organic matter and other deleterious materials. Further, the backfill should meet the pipe manufacturer's recommendations. Above the pipe zone backfill, Imported Select Structural Fill may be used as described above.

### Fill Placement and Compaction

Structural fill should be compacted at moisture contents that are within 3 percent of the optimum moisture content as determined by ASTM Standard Practices Test Method D 1557 (Modified Proctor). The optimum moisture content varies with gradation and should be evaluated during construction. Fill material that is not near the optimum moisture content should be moisture conditioned prior to compaction.

Fill and backfill material should be placed in uniform, horizontal lifts, and compacted with appropriate equipment. The appropriate lift thickness will vary depending on the material and compaction equipment used. Fill material should be compacted in accordance with Table 2, below. It is the contractor's responsibility to select appropriate compaction equipment and place the material in lifts that are thin enough to meet these criteria. However, in no case should the loose lift thickness exceed 18 inches.

**TABLE 2. COMPACTION CRITERIA**

| Fill Type | Compaction Requirements | | |
|---|---|---|---|
| | Percent Maximum Dry Density Determined by ASTM Test Method D 1557 at ± 3% of Optimum Moisture | | |
| | 0 to 2 Feet Below Subgrade | > 2 Feet Below Subgrade | Pipe Zone |
| Fine-grained soils (non-expansive) | 95 | 92 | ---- |
| Imported Granular, maximum particle size < 1-1/4-inch | 95 | 95 | ---- |
| Imported Granular, maximum particle size 1-1/4-inch to 6-inch (3-inch maximum under building footprints) | n/a (proof-roll) | n/a (proof-roll) | ---- |
| Retaining Wall Backfill* | 92 | 92 | ---- |
| Nonstructural Zones | 90 | 90 | 90 |
| Trench Backfill | 95 | 90 | 90 |

Notes:

* Measures should be taken to prevent over-compaction of the backfill behind retaining walls. We recommend placing the zone of backfill located within 5 feet of the wall in lifts not exceeding about 6 inches in loose thickness and compacting this zone with hand-operated equipment such as a vibrating plate compactor and a jumping jack.

A representative from GeoEngineers should evaluate compaction of each lift of fill. Compaction should be evaluated by compaction testing, unless other methods are proposed for oversized materials and are approved by GeoEngineers during construction. These other methods typically involve procedural placement and compaction specifications together with verifying requirements such as proof-rolling.

## PAVEMENT RECOMMENDATIONS

### Dynamic Cone Penetrometer Testing

We conducted DCP tests in general accordance with ASTM D6951 to estimate the subgrade resilient modulus ($M_R$) at each test location. We recorded penetration depth of the cone versus hammer blow count and terminated testing when at a depth of approximately 5 feet below the existing pavement surface on Marietta Street. We plotted depth of penetration versus blow count and visually assessed regions where slopes of the data were relatively constant using equation from the ODOT Pavement Design Guide to estimate the moduli using a conversion coefficient, $C_f$ = 0.35. Table 3 lists our estimate of the subgrade resilient modulus at each test location based on data obtained in the upper 18 inches below the proposed pavement surface. Field data are summarized in Figures A-12 through A-14.

**TABLE 3: ESTIMATED SUBGRADE RESILIENT MODULI BASED ON DCP TESTING**

| Boring Number | Estimated Resilient Modulus (psi) |
|---|---|
| DCP-1 | 5,000 |
| DCP-2 | 7,400 |
| DCP-3 | 7,300 |

### Laboratory Testing

We conducted laboratory testing on soil samples obtained during our subsurface explorations to determine in situ moisture contents and the California Bearing Ratio (CBR) value. Details of the laboratory results are presented in Appendix A.

### On-site AC Pavement

Pavement subgrades should be prepared in accordance with the "Earthworks Recommendations" section of this report. Our pavement recommendations assume that traffic at the site will consist of occasional truck traffic and passenger cars. We do not have specific information on the frequency and type of vehicles that will use the area; however, we have based our design analysis on traffic consisting of two to four heavy trucks per day to account for delivery- and service-type vehicles and passenger car traffic for the heavy-duty pavement sections, and passenger car traffic only for the light-duty pavement sections.

Our pavement recommendations are based on the following assumptions:

- The on-site soil subgrade below proposed fill placed to raise site grades or below aggregate base sections has been prepared as described in the "Site Preparation" section of this report, and observations indicate that subgrade is in a firm and unyielding condition.
- A resilient modulus of 20,000 psi was estimated for base rock prepared and compacted as recommended.
- A resilient modulus of 5,800 psi was estimated for subgrade prepared and compacted as recommended.
- Initial and terminal serviceability indices of 4.2 and 2.5, respectively.
- Reliability and standard deviations of 90 percent and 0.45, respectively.
- Structural coefficients of 0.41 and 0.10 for the asphalt and base rock, respectively.
- A 20-year design life.
- Truck traffic consists of an even distribution of two-axle service trucks/vans and no more than four large, four-axle trucks per day.

Drive and parking areas around proposed building footprints for passenger and patrol cars are considered light traffic areas, and site access roads, RV phase area, and on-site routes used for heavier truck traffic volumes are considered heavy traffic areas. Service trucks (assumed to be two-axle trucks) and service van routes are considered moderate traffic areas. The recommended pavement sections are provided in Table 4. If any of the noted assumptions vary from project design use, our office should be contacted with the appropriate information so that the pavement designs can be revised or confirmed adequate.

**TABLE 4. MINIMUM PAVEMENT DESIGN REQUIREMENTS**

| | Minimum Asphalt Thickness (inches) | Minimum Base Thickness (inches) |
|---|---|---|
| Light-Duty | 2.5 | 6.0 |
| Heavy-Duty | 3.5 | 6.0 |

If the soil subgrade is cement amended as part of site development to a minimum depth of 12 inches, the minimum base thickness in Table 4 above can be reduced to 4.0 inches. The reduced base thickness assumes subgrade is cement treated as described in the "Earthwork Recommendations" section of this report and has a minimum seven-day compressive strength of 80 psi. We recommend assuming a minimum cement ration of 5 percent by dry weight. In addition, to prevent strength loss during curing, cement-amended areas should be allowed to cure without construction traffic for at least four days.

The aggregate base course should conform to the "Aggregate Base" section of this report and be compacted to at least 95 percent of the MDD determined in accordance with AASHTO T-180/ASTM Test Method D 1557.

The AC pavement should conform to Section 00745 of the most current edition of the Oregon Department of Transportation (ODOT) Standard Specifications for Highway Construction. The Job Mix Formula should meet the requirements for a ½-inch Dense Graded Level 2 Mix. The AC should be PG 64-22 grade meeting the ODOT Standard Specifications for Asphalt Materials. AC pavement should be compacted to 91.0 percent at Maximum Theoretical Unit Weight (Rice Gravity) of AASHTO T-209.

The recommended pavement sections assume that final improvements surrounding the pavement will be designed and constructed such that stormwater or excess irrigation water from landscape areas does not infiltrate below the pavement section into the crushed base.

### Off-site Pavement

We understand that a half-street improvement is proposed for Marietta Street SE as a part of project development. The existing pavement section on the public roadway is approximately 2.5 inches of AC over 6 to 11 inches of aggregate base based on our explorations. The existing roadway surface is only lightly to moderately worn and showing minimal signs of fatigue from its current light use. Based on the light pavement section observed of 2.5 inches the pavement will likely require reconstruction as a part of project development to support increased traffic. Our recommended AC pavement section is thicker than what is currently in place based on the design values used in our evaluation.

Our interpretations of the subgrade resilient modulus are based on subsurface explorations, DCP testing, and on-site observations. Traffic loading is based on default values provided in the City design standards as noted below.

Design traffic loads were not provided at the time this report was prepared. We used the prescriptive pavement thickness design method as described in Division 006 of City of Salem Department of Public Works Administrative Rules Design Standards (COSDS) dated July 2014, the ODOT Pavement Design Guide dated August 2011, and the AASHTO Guide for Design of Pavement Structures dated 1993 in developing our recommendations. Descriptions of our input parameters and recommended pavement designs are summarized below.

The prescriptive pavement thickness design method in COSDS requires an estimate of traffic loading (ESALs) and classification of the subgrade soil. Section 6.24(d) of COSDS provides default traffic loads for various street classifications. We used a default ESAL value of 100,000 for a local street classification per COSDS. Our interpretation of the subgrade class is based on results of our subsurface explorations, laboratory testing, and our experience with similar soil. Although the majority of the existing roadway proposed for the half-street improvement is constructed over dense gravel fill that could be classified as "Good," we selected the soil type that will provide the weakest support, as per Section 6.24(c) of COSDS to classify the subgrade as "Fair." This subgrade class was determined by the results of DCP and CBR testing.

Section 6.24(f) of COSDS provides minimum pavement thickness for various traffic loads and subgrade classifications. Using the ESAL value and subgrade classification described above, COSDS specifies a minimum of 4.5 inches of AC over 10.0 inches of aggregate base rock. Using the structural layer coefficients provided in Section 6.24(e) of COSDS, this section provides a structural number of 2.65, and we estimate it will carry at least 170,000 ESALs according to the pavement design method in the AASHTO Guide for Design of Pavement Structures (1993).

Our recommended AC pavement section for the half-street improvement on the portion of Marietta Street SE south of the development site is:

- 2.0 inches of ½-inch, Level 3 HMAC wearing course (one lift)
- 2.5 inches of ½-inch, Level 3 HMAC (one lift)
- 10.0 inches of aggregate base

Construction traffic can damage prepared subgrade and should not be allowed to operate on unprotected subgrade. If required by the construction sequencing, the subgrade should be protected from damage by construction traffic as described in the "Earthwork Recommendations" section of this report.

Our pavement design recommendations have assumed that the subgrade will be consistent across the roadway. The design pavement sections may not be sufficient in soft subgrade areas if present. Removal of soft subgrade may be necessary as discussed in the "Earthwork Recommendations" section of this report.

## INFILTRATION TESTING

As requested by the project team, we conducted infiltration tests on site to assist in the evaluation of the site for design of stormwater detention area. We conducted two infiltration tests as requested at depths ranging from approximately 3 to 5 feet bgs at the selected locations I-1 and I-2 as shown in Figure 2. Testing was conducted using the encased falling head procedures. A 6-inch layer of pea gravel was placed in the pipe prior to adding water to diminish disturbance from flowing water at the base of the pipe interior. The test area was pre-soaked over a 4-hour period by repeated addition of water into the pipe when necessary. A good seal was present between the base of the pipe and the underlying soil in our opinion.

After the saturation period, the pipe was filled with clean water to at least 1 foot above the bottom of the pipe placed in the boring. The drop in water level was measured over a period of time after the soak period. In the case where the water level falls during the time-measured testing, infiltration rates diminish as a result of less head from the water column in the test. We observed only negligible drops in the water level during the testing period. The field test results are summarized in Table 5. The data and incremental infiltration rate over time are included in the infiltration test data summary in Appendix A, Figures A-15 and A-16. Also noted in Appendix A is the average rate over the full time of the test from placement of water.

**TABLE 5. INFILTRATION RESULTS**

| Infiltration Test No. | Location | Depth (feet) | USCS Material Type | Field Measured Infiltration Rate[1] (inches/hour) |
|---|---|---|---|---|
| I-1 | West Side of test site (see Site Plan) | 5 | ML | 0.01 |
| I-2 | West Side of test site (see Site Plan) | 3'-2.5" | ML | 0.02 |

Notes:

[1] Appropriate factors should be applied to the field measured infiltration rate, based on the design methodology and specific system used.

USCS = Unified Soil Classification System

The infiltration rates shown in Table 1 are field-measured infiltration rates. These represent a relatively short-term measured rate, and factors of safety have not been applied for the type of infiltration system being considered, or for variability that may be present in the on-site soil. In our opinion, and consistent with the state of the practice, correction factors should be applied to this measured rate to reflect the small area of testing and the number of tests conducted.

Infiltration testing resulted in observation of very minimal to negligible infiltration rates in the fine-grained soil. We attempted to advance test locations at the east end of the site using hand auger equipment where large drilling equipment could not access proposed test locations. Upper soils were very stiff silt and silt with clay and we could not advance hand augers deeper than a few inches from existing ground surface. Based on the consistency of the soil observed and the fine-grained soils encountered, it is our opinion that infiltration rates at the east side of the site will be negligible.

If the very low infiltration rates noted above are used for design, appropriate correction factors should also be applied by the project civil engineer to account for long-term infiltration parameters. From a geotechnical perspective, we recommend a factor of safety (correction factor) of at least 2 be applied to the field infiltration values to account for potential soil variability with depth and location within the area tested. In addition, the stormwater system design engineer should determine and apply appropriate remaining correction factor values, or factors of safety, to account for repeated wetting and drying that occur in this area, degree of in-system filtration, frequency and type of system maintenance, vegetation, potential for siltation and bio-fouling, etc., as well as system design correction factors for overflow or redundancy, and base and facility size.

The actual depths, lateral extent, and estimated infiltration rates can vary from the values presented above. Field testing/confirmation during construction is often required in large or long systems or other situations where soil conditions may vary within the area where the system is constructed. The results of this field testing might necessitate that the infiltration locations be modified to achieve the design infiltration rate.

Also, infiltration flow rate of a focused stormwater system typically diminishes over time as suspended solids and precipitates in the stormwater further clog the void spaces between the soil particles or cake on the infiltration surface. The serviceable life of an infiltration media in a stormwater system can be extended by pre-filtering or with on-going accessible maintenance. Eventually, most systems will fail and will need to be replaced or have media regenerated or replaced. We recommend that infiltration systems include an overflow that is connected to a suitable discharge point. Also, infiltration systems can cause localized high groundwater levels and should not be located near basement walls, retaining walls, or other embedded structures unless these are specifically designed to account for the resulting hydrostatic pressure. Infiltration locations should not be located on sloping ground, unless it is approved by a geotechnical engineer, and should not be infiltrated at a location that allows for flow to travel laterally toward a slope face, such as a mounded water condition or too close to a slope face.

#### *Suitability of Infiltration System*

Successful design and implementation of stormwater infiltration systems and whether a system is suitable for a development depend on several site-specific factors. Stormwater infiltration systems are generally best suited for sites having sandy or gravelly soil with saturated hydraulic conductivities greater than 2 inches per hour. Sites with silty or clayey soil such as encountered at this site, and sites with fine sand, silty sand, or gravel that has a high percentage of silt or clay in the matrix, or sites with relatively shallow underlying decomposed rock (residual soil) are generally not well suited for stormwater infiltration. Soil that has fine-grained matrices is susceptible to volumetric change and softening during wetting and drying cycles. Fine-grained soil also has large variations in the magnitude of infiltration rates because of bedding and stratification that occurs during alluvial deposition and often have thin layers of less permeable or impermeable soil within a larger layer.

As a result of stiff fine-grained soil conditions and very low to negligible measured infiltration rates, we recommend infiltration of stormwater not be used as the sole method of stormwater management at this site unless those design factors can be otherwise accounted for.

## STRUCTURAL DESIGN RECOMMENDATIONS

### Foundation Support Recommendations

Proposed structures can be satisfactorily founded on continuous wall or isolated column footings supported on firm native soils, or on structural fill placed over native soils. Exterior footings should be established at least 18 inches below the lowest adjacent grade. The recommended minimum footing depth is greater than the anticipated frost depth. Interior footings can be founded a minimum of 12 inches below the top of the floor slab. Isolated column and continuous wall footings should have minimum widths of 24 and 18 inches, respectively. We have assumed that the column loads will be 100 kips, wall loads will be 4 klf or less, and floor loads for slabs on grade will be 150 psf or less for the single story housing buildings. If design loads exceed these values, our recommendations may need to be revised.

#### *Foundation Subgrade Preparation*

Fill material beneath proposed structural elements should be prepared as described below and in the “Site Preparation” section. We recommend loose or disturbed soils resulting from foundation excavation be removed before placing reinforcing steel and concrete. Foundation bearing surfaces should not be exposed to standing water. If water infiltrates and pools in the excavation, the water, along with any disturbed soil, should be removed before placing reinforcing steel. A thin layer of crushed rock can be used to provide protection to the subgrade from weather and light foot traffic. Compaction should be performed as described in the "Fill Placement and Compaction" section.

We recommend GeoEngineers observe all foundation excavations before placing concrete forms and reinforcing steel in order to determine that bearing surfaces have been adequately prepared and the soil conditions are consistent with those observed during our explorations.

#### *Bearing Capacity – Spread Footings*

We recommend conventional footings be proportioned using a maximum allowable bearing pressure of 3,000 psf if supported on stiff native soils or on structural fill placed over native soils. This bearing pressure applies to the total of dead and long-term live loads and may be increased by one-third when considering earthquake or wind loads. This is a net bearing pressure. The weight of the footing and overlying backfill can be ignored in calculating footing sizes.

#### *Foundation Settlement*

Foundations designed and constructed as recommended are expected to experience settlements of less than 1 inch. Differential settlements of up to one half of the total settlement magnitude can be expected between adjacent footings supporting comparable loads.

#### *Lateral Resistance*

The ability of the soil to resist lateral loads is a function of frictional resistance, which can develop on the base of footings and slabs, and the passive resistance, which can develop on the face of below-grade elements of the structure as these elements tend to move into the soil. For footings and floor slabs founded in accordance with the recommendations presented above, the allowable frictional resistance may be computed using a coefficient of friction of 0.30 applied to vertical dead-load forces. Our analysis indicates that the available passive earth pressure for footings confined by on-site soil and structural fill is 350 pcf, modeled as an equivalent fluid pressure. Typically, the movement required to develop the available passive resistance may be relatively large; therefore, we recommend using a reduced passive pressure of 250 pcf equivalent fluid pressure. Adjacent floor slabs, pavements, or the upper 12-inch depth of adjacent unpaved areas should not be considered when calculating passive resistance. In addition, in order to rely on passive resistance, a minimum of 10 feet of horizontal clearance must exist between the face of the footings and any adjacent downslopes.

The passive earth pressure and friction components may be combined provided that the passive component does not exceed two-thirds of the total. The passive earth pressure value is based on the assumptions that the adjacent grade is level and that groundwater remains below the base of the footing throughout the year. The top foot of soil should be neglected when calculating passive lateral earth pressures unless the foundation area is covered with pavement or slab-on-grade. The lateral resistance values include a safety factor of approximately 1.5.

### Drainage Considerations

We recommend the ground surface be sloped away from the buildings at least 2 percent. All downspouts should be tightlined away from the building foundation areas and should also be discharged into a stormwater disposal system. Downspouts should not be connected to footing drains.

Although not required based on groundwater depths observed on our explorations, if perimeter footing drains are used for below-grade structural elements or crawlspaces, they should be installed at the base of the exterior footings. The perimeter footing drains should be provided with cleanouts and should consist of at least 4-inch-diameter perforated pipe placed on a 3 inch bed of, and surrounded by, 6 inches of drainage material enclosed in a non-woven geotextile such as Mirafi 140N (or approved equivalent) to prevent fine soil from migrating into the drain material. We recommend against using flexible tubing for footing drainpipes. The perimeter drains should be sloped to drain by gravity to a suitable discharge point, preferably a storm drain. We recommend that the cleanouts be covered and placed in flush mounted utility boxes. Water collected in roof downspout lines must not be routed to the footing drain lines.

### Floor Slabs

Satisfactory subgrade support for floor slabs supporting the planned 150 psf floor loads can be obtained provided the floor slab subgrade is described in the "Site Preparation" section of this report. Slabs should be reinforced according to their proposed use and per the structural engineer's recommendations. Subgrade support for concrete slabs can be obtained from the stiff/medium dense or firmer native soils. Floor slabs may overly existing fill provided it is scarified and recompacted in accordance with the recommendations presented in this report. We recommend that on-grade slabs be underlain by a minimum 6-inch-thick capillary break layer to reduce the potential for moisture migration into the slab. The capillary break material should consist of Aggregate Base material as described "Fill Materials" section of this report. The material should be placed as recommended in the "Fill Placement and Compaction" section.

If dry slabs are required (e.g., where adhesives are used to anchor carpet or tile to the slab), a waterproof liner may be placed as a vapor barrier below the slab. The vapor barrier should be selected by the structural engineer and should be accounted for in the design floor section and mix design selection for the concrete, to accommodate the effect of the vapor barrier on concrete slab curing. Load-bearing concrete slabs should be designed assuming a modulus of subgrade reaction (k) of 150 psi per inch. We estimate that concrete slabs constructed as recommended will settle less than ½ inch. We recommend that the floor slab subgrade be evaluated by proof-rolling prior to placing concrete.

### Conventional Retaining Walls

#### *Drainage*

Positive drainage is imperative behind any retaining structure. This can be accomplished by providing a drainage zone of free-draining material behind the wall with perforated pipes to discharge the collected water. The drainage material should consist of coarse sand and gravel containing less than 5 percent fines based on the fraction of material passing the 3/4-inch sieve. The wall drainage zone should extend horizontally at least 18 inches from the back of the wall.

A perforated smooth-walled rigid polyvinyl chloride (PVC) pipe having a minimum diameter of 4 inches should be placed at the bottom of the drainage zone along the entire length of the wall, with the pipe invert at or below the base of the wall footing. The drainpipes should discharge to a tightline leading to an appropriate collection and disposal system. An adequate number of cleanouts should be incorporated into the design of the drains in order to provide access for regular maintenance. In general, roof downspouts, perimeter drains or other types of drainage systems should not be connected to retaining wall drain systems.

***Design Parameters***

The pressures presented assume that backfill placed within 2 feet of the wall is compacted by hand-operated equipment to a density of 90 percent of the MDD and that wall drainage measures are included as previously recommended. For walls constructed as described above, we recommend using an active lateral earth pressure corresponding to an equivalent fluid density of 35 pcf for the level backfill condition. For walls with backfill sloping upward behind the wall at 2H:1V, an equivalent fluid density of 55 pcf should be used. This assumes that the tops of the walls are not structurally restrained and are free to rotate. For the at-rest condition (walls restrained from movement at the top) an equivalent fluid density of 55 pcf should be used for design. For seismic conditions, we recommend a uniform lateral pressure of 4H (where H is the height of the wall) psf be added to these lateral pressures. If the retaining system is designed as a braced system but is expected to yield a small amount during a seismic event, an active earth pressure condition may be assumed and combined with the uniform seismic surcharge pressure.

The recommended pressures do not include the effects of surcharges from surface loads. If vehicles will be operated within one-half the height of the wall, a traffic surcharge should be added to the wall pressure. The traffic surcharge can be approximated by the equivalent weight of an additional 2 feet of backfill behind the wall. Additional surcharge loading conditions should also be considered on a case-by-case basis.

Retaining walls founded on native soil or structural fill extending to these materials may be designed using the allowable soil bearing values and lateral resistance values presented above in the "Shallow Foundations" section of this report. We estimate settlement of retaining structures will be similar to the values previously presented for building foundations.

## Seismic Design

We recommend seismic design be performed using the procedure outlined in the 2012 IBC and the 2014 OSSC. The parameters provided in Table 6 are based on the conditions encountered during our subsurface exploration program and should be used in preparation of response spectra for the proposed structures.

**TABLE 6. SEISMIC DESIGN PARAMETERS**

| Parameter | Value |
|---|---|
| Site Class | D |
| Spectral Response Acceleration, $S_s$ | 0.90 g |
| Spectral Response Acceleration, $S_1$ | 0.43 g |
| Site Coefficient, $F_a$ | 1.14 |
| Site Coefficient, $F_v$ | 1.58 |
| Spectral Response Acceleration (Short Period), $S_{DS}$ | 0.68 g |
| Spectral Response Acceleration (1-Second Period) $S_{D1}$ | 0.45 g |

***Liquefaction Potential***

Liquefaction is a phenomenon caused by a rapid increase in pore water pressure that reduces the effective stress between soil particles to near zero. The excessive buildup of pore water pressure results in the sudden loss of shear strength in a soil. Granular soil, which relies on interparticle friction for strength, is susceptible to liquefaction until the excess pore pressures can dissipate. Sand boils and flows observed at the ground surface after an earthquake are the result of excess pore pressures dissipating upwards, carrying soil particles with the draining water. In general, loose, saturated sand soil with low silt and clay contents is the most susceptible to liquefaction. Low plasticity, silty sand may be moderately susceptible to liquefaction under relatively higher levels of ground shaking.

Based on our analysis, the site soils are not prone to liquefaction during the design level earthquake. Accordingly, lateral spreading or liquefaction induced deformations are not expected.

## GEOLOGIC HAZARD

Based on the procedures prescribed in the City of Salem's landslide hazard ordinance (Salem Revised Code, Chapter 810), a portion of the east margin of the site may be identified as a moderate landslide hazard risk. We reviewed Oregon Department of Geology and Mineral Industries (DOGAMI) maps that serve as a basis for the City of Salem's Landslide Hazard Susceptibility Map. A portion of the steep slope located in the southeast portion of the site is mapped as having a potential for rapidly moving landslides on IMS-22 (Hofmeister et al., 2002). The slope height is approximately 65 feet between approximate elevations 230 and 295 feet above mean sea level. The steep portion of the slope noted on IMS-22 is directly east of Marietta Street SE. The slope extends to the north on the site, but is not mapped as an area of landslide potential.

The mapped slope is a portion of the hillside that is the highland area of the site, composed of Columbia River Basalt (Tolan and Beeson, 2000). Exposures along the hillside consist of weathered basalt and decomposed saprolite consisting of silt and clay, consistent with the subsurface explorations on site. The hillside is generally well vegetated with semi-mature and mature trees and dense undergrowth. Considering the height of the slope and the density of vegetative cover grown onto weathered in-place rock, and with no grading proposed in the area of the slope, it is our opinion that the steep slope presents a very low potential for rapidly moving landslide instability impacting the proposed development.

Also, the adjacent project development along the downslope side to the east included placing several feet of fill to raise site grades. Fill removed from the uphill portion of this site was placed as fill on the adjacent site. Based on current plans, no earthwork or disruption of the slope vegetation is planned on the slope. It is our opinion that the proposed earthwork for the project does not present a hazard for increasing landslide potential.

## OTHER CONSIDERATIONS

### Frost Penetration

The near-surface soils are slightly too moderately susceptible to frost heave. However, foundation and floor slab elements are expected to bear on compacted granular fill. We anticipate that the depth of frost penetration in this region is approximately 12 inches. The recommended exterior and interior footing embedment depths provided above should allow adequate frost protection. Frost susceptibility in pavement areas is also expected to be low if they are constructed and supported as recommended.

### Expansive Soils

Based on our laboratory test results and experience with similar soils in the area, we do not consider the soils encountered in our borings to be expansive.

## DESIGN REVIEW AND CONSTRUCTION SERVICES

Recommendations provided in this report are based on the assumptions and preliminary design information stated herein. We welcome the opportunity to review and discuss construction plans and specifications for this project as they are being developed. In addition, GeoEngineers should be retained to review the geotechnical-related portions of the plans and specifications to evaluate whether they are in conformance with the recommendations provided in this report.

Satisfactory foundation and earthwork performance depends to a large degree on quality of construction. Sufficient monitoring of the contractor's activities is a key part of determining that the work is completed in accordance with the construction drawings and specifications. Subsurface conditions observed during construction should be compared with those encountered during the subsurface explorations. Recognition of changed conditions often requires experience; therefore, qualified personnel should visit the site with sufficient frequency to detect whether subsurface conditions change significantly from those anticipated.

We recommend that GeoEngineers be retained to observe construction at the site to confirm that subsurface conditions are consistent with the site explorations, and to confirm that the intent of project plans and specifications relating to earthwork, pavement and foundation construction are being met.

## LIMITATIONS

We have prepared this report for the exclusive use of Westech Engineering, Inc. and their authorized agents and/or regulatory agencies for the proposed development at the Marietta Street site in Salem, Oregon.

This report is not intended for use by others, and the information contained herein is not applicable to other sites. No other party may rely on the product of our services unless we agree in advance and in writing to such reliance.

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

Please refer to Appendix B titled "Report Limitations and Guidelines for Use" for additional information pertaining to use of this report.

## REFERENCES

International Code Council, 2012. 2012 International Building Code.

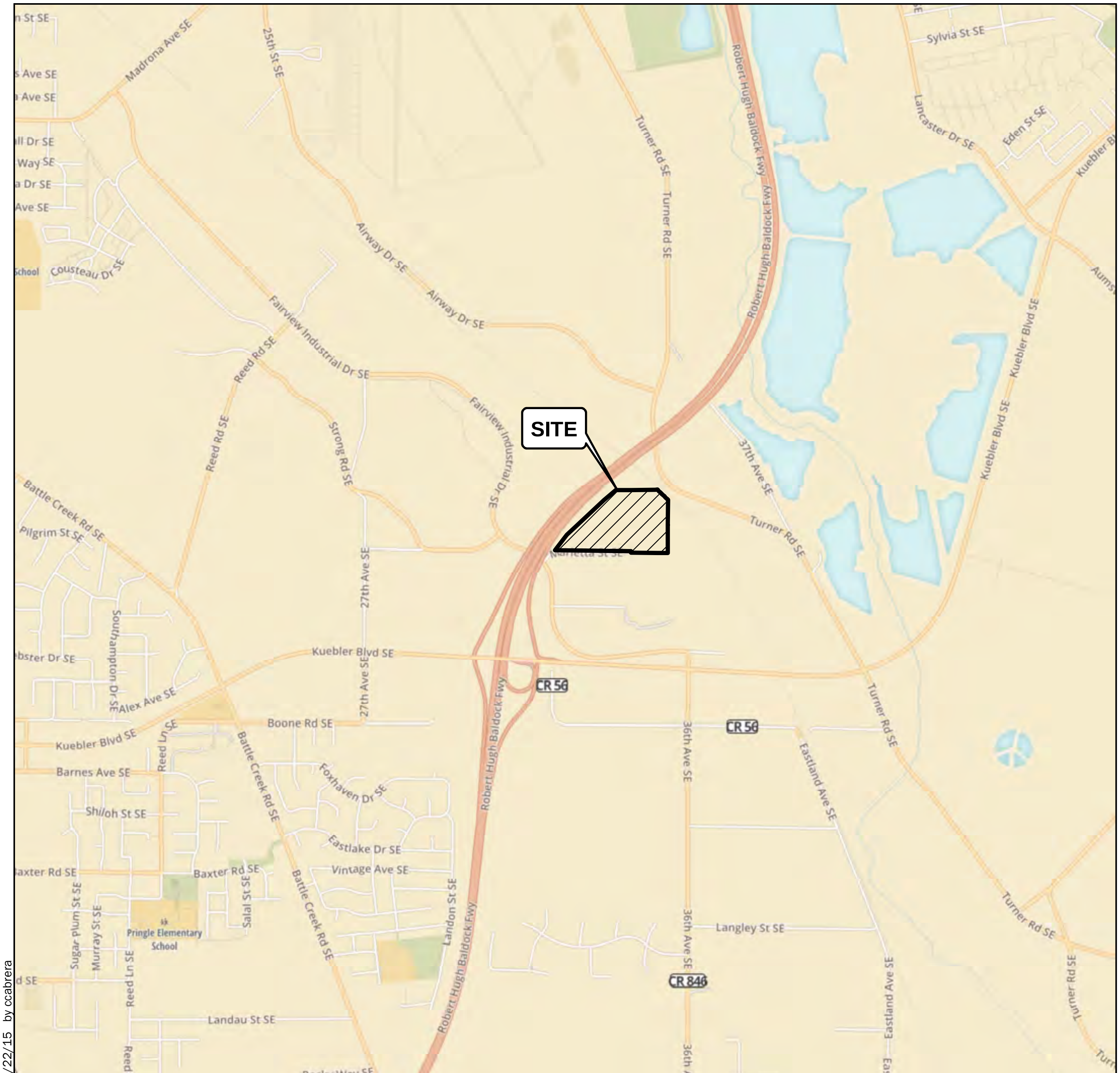
International Code Council, 2014. 2014 Oregon Structural Specialty Code.

Occupational Safety and Health Administration (OSHA) Technical Manual Section V: Chapter 2, Excavations: Hazard Recognition in Trenching and Shoring: http://www.osha.gov/dts/osta/otm/otm_v/otm_v_2.html.

Orr, E.L., and Orr, W.N., 1999, Geology of Oregon, 5th ed., Kendall/Hunt Publishing Company, Dubuque, Iowa, 254p.

Tolan,T.L., and Beeson, M.H., 2000, Geologic Map of the Salem East 7.5 Minute Quadrangle, Marion County, Oregon: U.S. Geological Survey, Open-File Report 00-351, scale 1:24,000.

FIGURES





N
W E
S

2,000 0 2,000
Feet

## Notes:

1. The locations of all features shown are approximate.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.

Data Source: Mapbox Open Street Map, 2015

Projection: NAD 1983 UTM Zone 10N

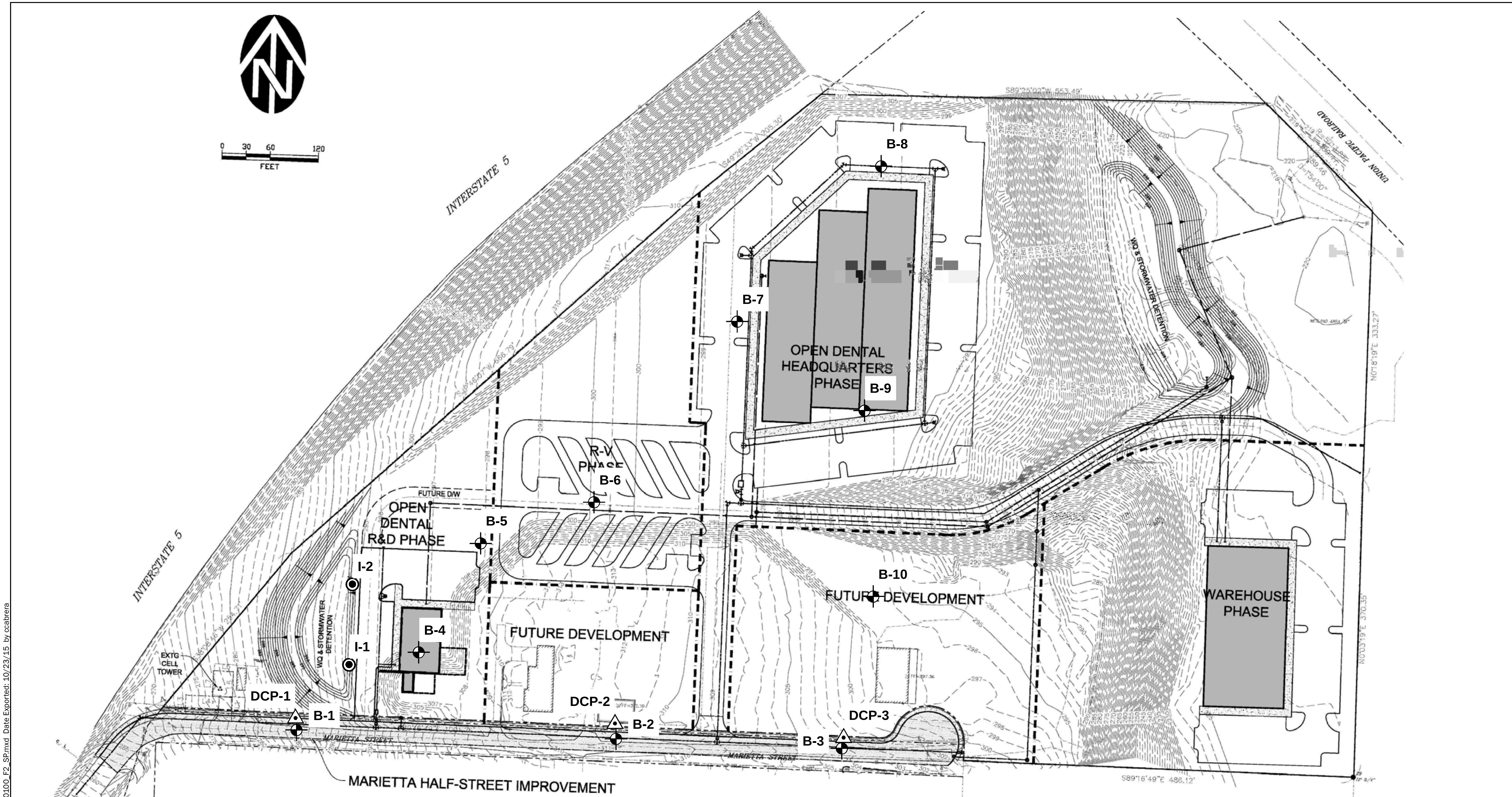
**Vicinity Map**

Open Dental Software – Marietta Street Development
Salem, Oregon

GEOENGINEERS

**Figure 1**

\\pdx\projects\2\2216001\GIS\MXD\221600100_F1_VM.mxd Date Exported: 10/22/15 by ccabrera




Notes:

1. The locations of all features shown are approximate.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.

Data Source: Base drawing from Westech Engineering, Inc, Drawing C2.0, Overall Site Plan, June 2015.

Projection: NAD 1983 StatePlane Oregon North FIPS 3601 Feet

**Legend**

B-1 Boring Number and Approximate Location

DCP-1 Dynamic Cone Penetrometer Number and Approximate Location

I-1 Infiltration Number and Approximate Location

N
W E
S

120 0 120

Feet

**Site Plan**

Open Dental Software – Marietta Street Development
Salem, Oregon

GEOENGINEERS

**Figure 2**

APPENDICES

# APPENDIX A
## Field Explorations and Laboratory Testing

## APPENDIX A
## FIELD EXPLORATIONS AND LABORATORY TESTING

### Field Explorations

Soil and groundwater conditions at the proposed Open Dental development were explored on October 20th and 21st, 2015 by completing ten borings (B-1 through B-10), three dynamic cone penetrometer soundings to a depth of 5 feet bgs, and two infiltration tests at the approximate locations shown in Figure 2. The borings were advanced using a hollow-stem auger to depths ranging from approximately 3.5 to 15 feet bgs using a track mounted drill rig owned and operated by Western States Soil Conservation.

The drilling was continuously monitored by a staff engineer from our office who maintained a detailed log of subsurface explorations, visually classified the soil encountered and obtained representative soil samples from the borings. Representative soil samples were obtained from each boring at approximate 2½- to 5-foot-depth intervals using a standard split spoon (SPT) sampler. The samplers were driven into the soil using an automatic 140-pound hammer, free-falling 30 inches on each blow. The number of blows required to drive the sampler each of three, 6-inch increments of penetration were recorded in the field. The sum of the blow counts for the last two, 6-inch increments of penetration is reported on the boring logs as the ASTM D 1556 Standard Penetration Test (SPT) N-value.

DCP soundings were performed by a staff geotechnical engineer from our office who recorded blow count versus cumulative penetration depth. This penetration resistance data was compared to the adjacent borings (B-1 through B-3) where a detailed log of subsurface explorations were maintained, the soils encountered were visually classified, representative soil samples from the borings were obtained. Representative soil samples were obtained from each of these borings at approximately 2.5 and 5 feet bgs.

Recovered soil samples were visually classified in the field in general accordance with ASTM D 2488 and the classification chart listed in Key to Exploration Logs, Figure A-1. Logs of the borings are presented in Figures A-2 and A-11. The logs are based on interpretation of the field and laboratory data, and indicate the depth at which subsurface materials or their characteristics change, although these changes might actually be gradual.

### Laboratory Testing

Soil samples obtained from the explorations were visually classified in the field and in our laboratory using the Unified Soil Classification System (USCS) and ASTM classification methods. ASTM Test Method D 2488 was used to visually classify the soil samples, while ASTM D 2487 was used to classify the soils based on laboratory tests results. Moisture content tests were performed in general accordance with ASTM method D2216-05. Percent fines (silt- and clay-sized particles passing the U.S. No. 200 sieve) tests (ASTM D1140) were completed on representative soil samples. Results of the moisture content and percent fines testing are presented on the appropriate exploration logs at the respective sample depths.

## SOIL CLASSIFICATION CHART

| MAJOR DIVISIONS | | | SYMBOLS GRAPH | SYMBOLS LETTER | TYPICAL DESCRIPTIONS |
|---|---|---|---|---|---|
| COARSE GRAINED SOILS<br>MORE THAN 50% RETAINED ON NO. 200 SIEVE | GRAVEL AND GRAVELLY SOILS<br>MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE | CLEAN GRAVELS<br>(LITTLE OR NO FINES) | | GW | WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES |
| | | | | GP | POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES |
| | | GRAVELS WITH FINES<br>(APPRECIABLE AMOUNT OF FINES) | | GM | SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES |
| | | | | GC | CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES |
| | SAND AND SANDY SOILS<br>MORE THAN 50% OF COARSE FRACTION PASSING NO. 4 SIEVE | CLEAN SANDS<br>(LITTLE OR NO FINES) | | SW | WELL-GRADED SANDS, GRAVELLY SANDS |
| | | | | SP | POORLY-GRADED SANDS, GRAVELLY SAND |
| | | SANDS WITH FINES<br>(APPRECIABLE AMOUNT OF FINES) | | SM | SILTY SANDS, SAND - SILT MIXTURES |
| | | | | SC | CLAYEY SANDS, SAND - CLAY MIXTURES |
| FINE GRAINED SOILS<br>MORE THAN 50% PASSING NO. 200 SIEVE | SILTS AND CLAYS | LIQUID LIMIT LESS THAN 50 | | ML | INORGANIC SILTS, ROCK FLOUR, 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 GREATER THAN 50 | | MH | INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS SILTY SOILS |
| | | | | CH | INORGANIC CLAYS OF HIGH PLASTICITY |
| | | | | OH | ORGANIC CLAYS AND SILTS OF MEDIUM TO HIGH PLASTICITY |
| HIGHLY ORGANIC SOILS | | | | PT | PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS |

NOTE: Multiple symbols are used to indicate borderline or dual soil classifications

## ADDITIONAL MATERIAL SYMBOLS

| SYMBOLS GRAPH | SYMBOLS LETTER | TYPICAL DESCRIPTIONS |
|---|---|---|
| | AC | Asphalt Concrete |
| | CC | Cement Concrete |
| | CR | Crushed Rock/ Quarry Spalls |
| | TS | Topsoil/ Forest Duff/Sod |

### Groundwater Contact

**Measured groundwater level in exploration, well, or piezometer**

**Measured free product in well or piezometer**

### Graphic Log Contact

**Distinct contact between soil strata or geologic units**

**Approximate location of soil strata change within a geologic soil unit**

### Material Description Contact

**Distinct contact between soil strata or geologic units**

**Approximate location of soil strata change within a geologic soil unit**

### Sampler Symbol Descriptions

- **2.4-inch I.D. split barrel**
- **Standard Penetration Test (SPT)**
- **Shelby tube**
- **Piston**
- **Direct-Push**
- **Bulk or grab**
- **Continuous Coring**

**Blowcount is recorded for driven samplers as the number of blows required to advance sampler 12 inches (or distance noted). See exploration log for hammer weight and drop.**

**A "P" indicates sampler pushed using the weight of the drill rig.**

### Laboratory / Field Tests

| | |
|---|---|
| %F | Percent fines |
| AL | Atterberg limits |
| CA | Chemical analysis |
| CP | Laboratory compaction test |
| CS | Consolidation test |
| DS | Direct shear |
| HA | Hydrometer analysis |
| MC | Moisture content |
| MD | Moisture content and dry density |
| OC | Organic content |
| PM | Permeability or hydraulic conductivity |
| PI | Plasticity index |
| PP | Pocket penetrometer |
| PPM | Parts per million |
| SA | Sieve analysis |
| TX | Triaxial compression |
| UC | Unconfined compression |
| VS | Vane shear |

### Sheen Classification

| | |
|---|---|
| NS | No Visible Sheen |
| SS | Slight Sheen |
| MS | Moderate Sheen |
| HS | Heavy Sheen |
| NT | Not Tested |

NOTE: The reader must refer to the discussion in the report text and the logs of explorations for a proper understanding of subsurface conditions. Descriptions on the logs apply only at the specific exploration locations and at the time the explorations were made; they are not warranted to be representative of subsurface conditions at other locations or times.

## KEY TO EXPLORATION LOGS

GeoEngineers

FIGURE A-1

| Drilled | Start 10/21/2015 | End 10/21/2015 | Total Depth (ft) 6.5 | Logged By TNG Checked By JCV | Driller Western States Drilling | Drilling Method Hollow-stem Auger |
|---|---|---|---|---|---|---|

| | | | | | |
|---|---|---|---|---|---|
| Surface Elevation (ft) Vertical Datum | 285 | Hammer Data | Autohammer 140 (lbs) / 30 (in) Drop | Drilling Equipment | CME-55 Track Rig |
| Latitude Longitude | | System Datum | Geographic | Groundwater: Date Measured / Depth to Water (ft) / Elevation (ft) | |

Notes: Auger Data: 4¼-inch I.D. , 9-inch O.D.

| Elevation (feet) | Depth (feet) | Interval / Recovered (in) | Blows/foot | Collected Sample | Sample Name / Testing | Water Level | Graphic Log | Group Classification | MATERIAL DESCRIPTION | Moisture Content (%) | Fines Content (%) | REMARKS |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 0 | | | | | | | AC | Asphalt concrete pavement (2.5 inches) | | | |
| | | | | | | | | GW | Gray sandy gravel (dense, dry) (fill-base rock 3/4"-0) (8 inches) | | | |
| | | | | | | | | CH | Brown silty clay (stiff, moist) | | | Gravel mixed with soil below base rock |
| | | 18 | 16 | | 1 | | | | | | | PP = 1.25 tsf<br>PP = 2.25 tsf |
| 280 | 5 | 16 | 22 | | 2 | | | | Grades to very stiff with trace sand | | | PP = 3.75 tsf |

Notes: See Figure A-1 for explanation of symbols.

Portland: Date:11/5/15 Path:P:\22\22126001\00\GINT\2212600100.GPJ DBTemplate/LibTemplate:GEOENGINEERS8.GDT/GEI8_GEOTECH_STANDARD

# Log of Boring B-1

GeoEngineers

Project: Open Dental Software - Marietta Street Development
Project Location: Salem, Oregon
Project Number: 22126-001-00

| Drilled | Start 10/21/2015 | End 10/21/2015 | Total Depth (ft) | 6.5 | Logged By TNG / Checked By JCV | Driller | Western States Drilling | Drilling Method | Hollow-stem Auger |
|---|---|---|---|---|---|---|---|---|---|

| | | | | | |
|---|---|---|---|---|---|
| Surface Elevation (ft) / Vertical Datum | 311 | Hammer Data | Autohammer 140 (lbs) / 30 (in) Drop | Drilling Equipment | CME-55 Track Rig |
| Latitude / Longitude | | System / Datum | Geographic | Groundwater | Date Measured / Depth to Water (ft) / Elevation (ft) |

Notes: Auger Data: 4¼-inch I.D. , 9-inch O.D.

| Elevation (feet) | Depth (feet) | Interval | Recovered (in) | Blows/foot | Collected Sample | Sample Name / Testing | Water Level | Graphic Log | Group Classification | MATERIAL DESCRIPTION | Moisture Content (%) | Fines Content (%) | REMARKS |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 0 | | | | | | | | AC | Asphalt concrete pavement (2.5 inches) | | | |
| | | | | | | | | | GW | Gray sandy gravel (dense, dry) (fill-base rock 3/4"-0) (6 inches) | | | |
| 310 | | | | | | | | | CH | Brown with gray mottling silty clay (very stiff, moist) | | | |
| | | | 14 | 17 | | 1 / MC | | | | | 27 | | |
| | 5 | | 18 | 77 | | 2 | | | | Grades to reddish brown with trace gravel and sand, hard | | | PP = 2.75 tsf |
| 305 | | | | | | | | | | | | | |

Notes: See Figure A-1 for explanation of symbols.

Portland: Date:11/5/15 Path:P:\22\22126001\00\GINT\2212600100.GPJ DBTemplate/LibTemplate:GEOENGINEERS8.GDT/GEI8_GEOTECH_STANDARD

## Log of Boring B-2

GEOENGINEERS

Project: Open Dental Software - Marietta Street Development
Project Location: Salem, Oregon
Project Number: 22126-001-00

Figure A-3
Sheet 1 of 1

| Drilled | Start 10/21/2015 | End 10/21/2015 | Total Depth (ft) | 6.5 | Logged By TNG / Checked By JCV | Driller | Western States Drilling | Drilling Method | Hollow-stem Auger |
|---|---|---|---|---|---|---|---|---|---|

| | | | |
|---|---|---|---|
| Surface Elevation (ft) / Vertical Datum | 305 | Hammer Data | Autohammer 140 (lbs) / 30 (in) Drop |
| Drilling Equipment | CME-55 Track Rig | | |
| Latitude / Longitude | | System / Datum | Geographic |

| Groundwater Date Measured | Depth to Water (ft) | Elevation (ft) |
|---|---|---|
| | | |

Notes: Auger Data: 4¼-inch I.D. , 9-inch O.D.

| Elevation (feet) | Depth (feet) | Interval | Recovered (in) | Blows/foot | Collected Sample | Sample Name / Testing | Water Level | Graphic Log | Group Classification | MATERIAL DESCRIPTION | Moisture Content (%) | Fines Content (%) | REMARKS |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 0 | | | | | | | | AC | Asphalt concrete pavement (2.5 inches) | | | |
| | | | | | | | | | GW | Gray sandy gravel (dense, dry) (fill-base rock 3/4"-0) (11 inches) | | | |
| | | | | | | | | | CH | Brown silty clay (very stiff, dry) | | | |
| | | | 10 | 20 | | 1 | | | | | | | |
| 300 | 5 | | 16 | 58 | | 2 / MC | | | | Grades to reddish brown with trace gravel and sand, hard, moist | 22 | | PP = 4.5+ tsf |

Notes: See Figure A-1 for explanation of symbols.



## Log of Boring B-3

GeoEngineers

Project: Open Dental Software - Marietta Street Development
Project Location: Salem, Oregon
Project Number: 22126-001-00

Figure A-4
Sheet 1 of 1

| Drilled | Start 10/21/2015 | End 10/21/2015 | Total Depth (ft) 16.5 | Logged By TNG<br>Checked By JCV | Driller Western States Drilling | Drilling Method Hollow-stem Auger |
|---|---|---|---|---|---|---|

| | | |
|---|---|---|
| Surface Elevation (ft) / Vertical Datum: 297 | Hammer Data: Autohammer 140 (lbs) / 30 (in) Drop | Drilling Equipment: CME-55 Track Rig |
| Latitude / Longitude | System / Datum: Geographic | Groundwater: Date Measured / Depth to Water (ft) / Elevation (ft) |
| Notes: Auger Data: 4¼-inch I.D. , 9-inch O.D. | | |

| Elevation (feet) | Depth (feet) | Interval | Recovered (in) | Blows/foot | Collected Sample | Sample Name / Testing | Water Level | Graphic Log | Group Classification | MATERIAL DESCRIPTION | Moisture Content (%) | Fines Content (%) | REMARKS |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 0 | | | | | | | | ML | Brown silt with sand and trace gravel (hard, moist) | | | |
| 295 | | | | | | | | | | | | | |
| | | | 18 | 44 | | 1 / MC | | | | | 22 | | PP = 1.5 tsf |
| | 5 | | 18 | 35 | | 2 | | | | | | | |
| 290 | | | | | | | | | | | | | |
| | 10 | | 17 | 38 | | 3 | | | | Grades with more sand | | | PP = 1.75 tsf |
| 285 | | | | | | | | | | | | | |
| | 15 | | 18 | 21 | | 4 | | | | Grades with lenses of purple, very stiff | | | PP = 2.0 tsf |

Notes: See Figure A-1 for explanation of symbols.

# Log of Boring B-4

GeoEngineers

Project: Open Dental Software - Marietta Street Development
Project Location: Salem, Oregon
Project Number: 22126-001-00

Figure A-5
Sheet 1 of 1

| Drilled | Start 10/21/2015 | End 10/21/2015 | Total Depth (ft) 11 | Logged By TNG / Checked By JCV | Driller Western States Drilling | Drilling Method Hollow-stem Auger |
|---|---|---|---|---|---|---|

| | | |
|---|---|---|
| Surface Elevation (ft) / Vertical Datum: 298 | Hammer Data: Autohammer 140 (lbs) / 30 (in) Drop | Drilling Equipment: CME-55 Track Rig |
| Latitude / Longitude | System / Datum: Geographic | Groundwater: Date Measured / Depth to Water (ft) / Elevation (ft) |

Notes: Auger Data: 4¼-inch I.D. , 9-inch O.D.

| Elevation (feet) | Depth (feet) | Interval | Recovered (in) | Blows/foot | Collected Sample | Sample Name / Testing | Water Level | Graphic Log | Group Classification | MATERIAL DESCRIPTION | Moisture Content (%) | Fines Content (%) | REMARKS |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 0 | | | | | | | | ML | Brown silt with trace sand (stiff, moist) | | | |
| 295 | | | 18 | 9 | | 1 | | | | | | | PP = 1.5 tsf |
| | 5 | | 18 | 31 | | 2 | | | | Grades with more sand | | | |
| 290 | | | | | | | | | | | | | |
| | 10 | | 18 | 29 | | 3 / MC | | | | Brown sandy silt with gravel (very stiff to hard, moist) | 27 | | Driller indicates refusal at 9.5 feet, hitting rock/boulder with auger. Collected sample below 9.5 feet due to no rock in center of borehole |

Refusal at 9.5 feet on rock/boulder

Notes: See Figure A-1 for explanation of symbols.



## Log of Boring B-5

GeoEngineers

Project: Open Dental Software - Marietta Street Development
Project Location: Salem, Oregon
Project Number: 22126-001-00

| Drilled | Start 10/21/2015 | End 10/21/2015 | Total Depth (ft) | 16.5 | Logged By TNG / Checked By JCV | Driller | Western States Drilling | Drilling Method | Hollow-stem Auger |
|---|---|---|---|---|---|---|---|---|---|

| | | | | | |
|---|---|---|---|---|---|
| Surface Elevation (ft) Vertical Datum | 301 | Hammer Data | Autohammer 140 (lbs) / 30 (in) Drop | Drilling Equipment | CME-55 Track Rig |
| Latitude Longitude | | System Datum | Geographic | Groundwater | Date Measured / Depth to Water (ft) / Elevation (ft) |

Notes: Auger Data: 4¼-inch I.D. , 9-inch O.D.

| Elevation (feet) | Depth (feet) | FIELD DATA: Interval | Recovered (in) | Blows/foot | Collected Sample | Sample Name / Testing | Water Level | Graphic Log | Group Classification | MATERIAL DESCRIPTION | Moisture Content (%) | Fines Content (%) | REMARKS |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 301 | 0 | | | | | | | | ML | Brown to tan sandy silt (stiff, moist) | | | |
| 300 | | | | | | | | | | | | | |
| | 2.5 | | 18 | 11 | | 1 | | | | | | | PP = 1.5-2.0 tsf |
| | 5 | | 18 | 15 | | 2 / %F | | | | | 42 | 60 | PP = 0.5 tsf |
| 295 | | | | | | | | | | | | | PP = 1.5 tsf |
| | 10 | | 18 | 69 | | 3 | | | | Brown and tan sandy silt with gravel (hard, moist) | | | PP = 2.0 tsf |
| 290 | | | | | | | | | | | | | |
| | 15 | | 18 | 17 | | 4 / MC | | | | Grades to stiff | 31 | | PP = 1.5-2.0 tsf |
| 285 | | | | | | | | | | | | | |

Notes: See Figure A-1 for explanation of symbols.

Portland: Date:11/5/15 Path:P:\22\22126001\00\GINT\2212600100.GPJ DBTemplate/LibTemplate:GEOENGINEERS8.GDT/GEI8_GEOTECH_STANDARD

## Log of Boring B-6

GeoEngineers

Project: Open Dental Software - Marietta Street Development
Project Location: Salem, Oregon
Project Number: 22126-001-00

| Drilled | Start | End | Total Depth (ft) | Logged By | Checked By | Driller | Drilling Method |
|---|---|---|---|---|---|---|---|
| | 10/21/2015 | 10/21/2015 | 16.5 | TNG | JCV | Western States Drilling | Hollow-stem Auger |

| | | | | | |
|---|---|---|---|---|---|
| Surface Elevation (ft) Vertical Datum | 300 | Hammer Data | Autohammer 140 (lbs) / 30 (in) Drop | Drilling Equipment | CME-55 Track Rig |
| Latitude Longitude | | System Datum | Geographic | Groundwater | |

| Groundwater Date Measured | Depth to Water (ft) | Elevation (ft) |
|---|---|---|

Notes: Auger Data: 4¼-inch I.D. , 9-inch O.D.

| Elevation (feet) | Depth (feet) | Interval | Recovered (in) | Blows/foot | Collected Sample | Sample Name / Testing | Water Level | Graphic Log | Group Classification | MATERIAL DESCRIPTION | Moisture Content (%) | Fines Content (%) | REMARKS |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 0 | | | | | | | | SM | Tan to brown with black mottling silty sand (medium dense, moist) | | | |
| | | | 18 | 14 | | 1 / %F | | | | | 38 | 45 | PP = 1.25-1.75 tsf |
| 295 | 5 | | 18 | 21 | | 2 | | | | Grades with gravel | | | PP = 2.0 tsf |
| | | | | | | | | | ML/SM | Tan to brown with black mottling silty sand to sandy silt (stiff/medium dense, moist) | | | |
| 290 | 10 | | 18 | 25 | | 3 / MC | | | | | 40 | | PP = 1.5 tsf |
| 285 | 15 | | 8 | 25 | | 4 | | | | | | | |

Notes: See Figure A-1 for explanation of symbols.

Portland: Date:11/5/15 Path:P:\22\22126001\00\GINT\2212600100.GPJ DBTemplate/LibTemplate:GEOENGINEERS8.GDT/GEI8_GEOTECH_STANDARD

## Log of Boring B-7

GEOENGINEERS

Project: Open Dental Software - Marietta Street Development
Project Location: Salem, Oregon
Project Number: 22126-001-00

| Drilled | Start 10/21/2015 | End 10/21/2015 | Total Depth (ft) | 16.5 | Logged By TNG Checked By JCV | Driller | Western States Drilling | Drilling Method | Hollow-stem Auger |
|---|---|---|---|---|---|---|---|---|---|

| | | | |
|---|---|---|---|
| Surface Elevation (ft) Vertical Datum | 295 | Hammer Data | Autohammer 140 (lbs) / 30 (in) Drop |
| Drilling Equipment | CME-55 Track Rig | | |
| Latitude Longitude | | System Datum | Geographic |

Groundwater

| Date Measured | Depth to Water (ft) | Elevation (ft) |
|---|---|---|

Notes: Auger Data: 4¼-inch I.D. , 9-inch O.D.

| Elevation (feet) | Depth (feet) | FIELD DATA: Interval | Recovered (in) | Blows/foot | Collected Sample | Sample Name Testing | Water Level | Graphic Log | Group Classification | MATERIAL DESCRIPTION | Moisture Content (%) | Fines Content (%) | REMARKS |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 0 | | | | | | | | ML | Brown silt with sand (very stiff, moist) | | | |
| | | | 18 | 21 | | 1 | | | | | | | PP = 1.5 tsf |
| 290 | 5 | | 17 | 37 | | 2 MC | | | | Grades with light tan clay, hard | 26 | | PP = 1.25 tsf |
| 285 | 10 | | 18 | 13 | | 3 | | | | Grades to stiff | | | PP = 1.0 tsf |
| 280 | 15 | | 18 | 11 | | 4 | | | | Brown to tan with black mottling silt with clay and sand (stiff, moist) | | | PP = 1.25 tsf |

Notes: See Figure A-1 for explanation of symbols.



## Log of Boring B-8

GeoEngineers

Project: Open Dental Software - Marietta Street Development
Project Location: Salem, Oregon
Project Number: 22126-001-00

Figure A-9
Sheet 1 of 1

| Drilled | Start 10/21/2015 | End 10/21/2015 | Total Depth (ft) | 12 | Logged By TNG / Checked By JCV | Driller | Western States Drilling | Drilling Method | Hollow-stem Auger |
|---|---|---|---|---|---|---|---|---|---|

| | | | |
|---|---|---|---|
| Surface Elevation (ft) / Vertical Datum: 295 | Hammer Data: Autohammer 140 (lbs) / 30 (in) Drop | Drilling Equipment: CME-55 Track Rig | |
| Latitude / Longitude | System / Datum: Geographic | Groundwater — Date Measured | Depth to Water (ft) — Elevation (ft) |

Notes: Auger Data: 4¼-inch I.D. , 9-inch O.D.

| Elevation (feet) | Depth (feet) | Interval | Recovered (in) | Blows/foot | Collected Sample | Sample Name / Testing | Water Level | Graphic Log | Group Classification | MATERIAL DESCRIPTION | Moisture Content (%) | Fines Content (%) | REMARKS |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 0 | | | | | | | | ML | Brown to red-brown sandy silt with black nodules and trace gravel (hard, moist) | | | |
| | | | 18 | 65 | | 1 | | | | | | | PP = 1.75-2.0 tsf |
| 290 | 5 | | 18 | 36 | | 2 | | | | | | | PP = 1.25-1.75 tsf |
| 285 | 10 | | 18 | 23 | | 3 / MC | | | | Brown silt with fine sand (very stiff, moist) | 34 | | PP = 1.0-1.5 tsf |

Refusal at 12 feet on boulder

Notes: See Figure A-1 for explanation of symbols.



## Log of Boring B-9

GeoEngineers

Project: Open Dental Software - Marietta Street Development
Project Location: Salem, Oregon
Project Number: 22126-001-00

| Drilled | Start 10/21/2015 | End 10/21/2015 | Total Depth (ft) 16.5 | Logged By TNG / Checked By JCV | Driller Western States Drilling | Drilling Method Hollow-stem Auger |
|---|---|---|---|---|---|---|

| | | | |
|---|---|---|---|
| Surface Elevation (ft) / Vertical Datum: 296 | Hammer Data: Autohammer 140 (lbs) / 30 (in) Drop | Drilling Equipment: CME-55 Track Rig | |
| Latitude / Longitude | System / Datum: Geographic | Groundwater | Date Measured / Depth to Water (ft) / Elevation (ft) |

Notes: Auger Data: 4¼-inch I.D. , 9-inch O.D.

| Elevation (feet) | Depth (feet) | FIELD DATA: Interval | Recovered (in) | Blows/foot | Collected Sample | Sample Name / Testing | Water Level | Graphic Log | Group Classification | MATERIAL DESCRIPTION | Moisture Content (%) | Fines Content (%) | REMARKS |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 0 | | | | | | | | ML | Brown sandy silt (very stiff, moist) (fill) | | | |
| 295 | | | | | | | | | | | | | |
| | | | 18 | 26 | | 1 | | | | | | | PP = 1.5 tsf |
| | 5 | | 18 | 23 | | 2 / MC | | | | | 29 | | PP = 1.5 tsf |
| 290 | | | | | | | | | | | | | |
| | 10 | | 18 | 21 | | 3 | | | | | | | PP = 1.75-2.0 tsf |
| 285 | | | | | | | | | | | | | |
| | 15 | | 18 | 11 | | 4 | | | CL | Brown silty clay with black nodules, sand, and trace gravel (stiff, moist) | | | PP = 1.0 tsf |
| 280 | | | | | | | | | | Grades to wet at 16 feet | | | |

Notes: See Figure A-1 for explanation of symbols.

Portland: Date:11/5/15 Path:P:\22\22126001\00\GINT\2212600100.GPJ DBTemplate/LibTemplate:GEOENGINEERS8.GDT/GEI8_GEOTECH_STANDARD

## Log of Boring B-10

GEOENGINEERS

Project: Open Dental Software - Marietta Street Development
Project Location: Salem, Oregon
Project Number: 22126-001-00

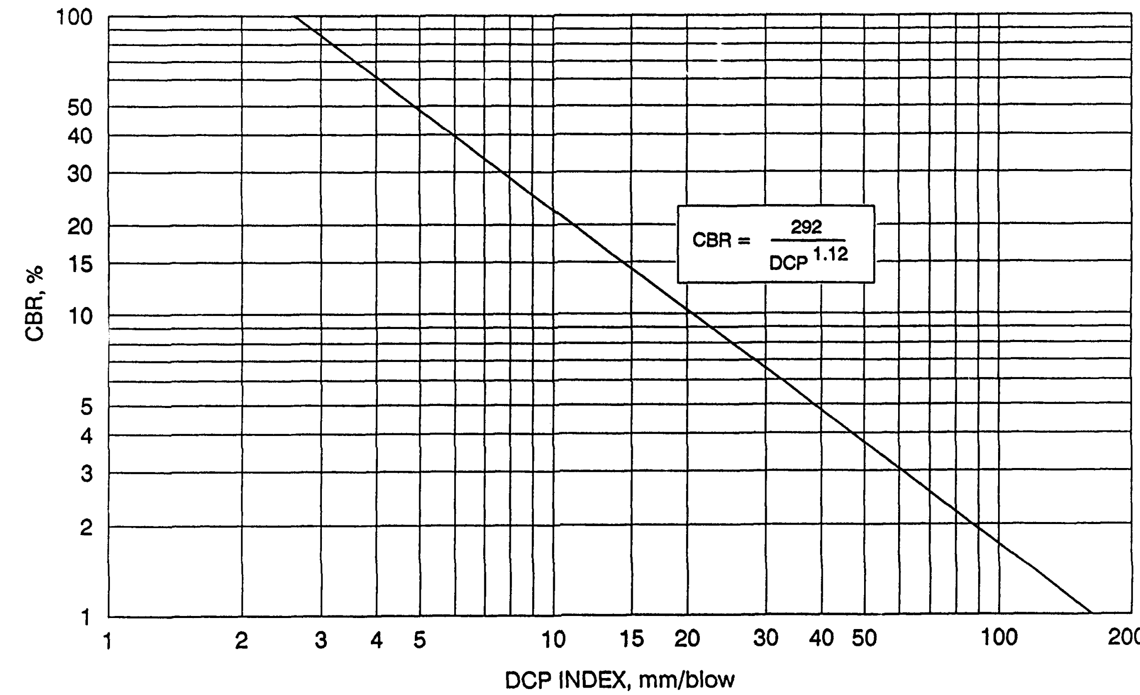
Location: Maritta St. Devlopment
Depth to bottom: 5'
Tester's Name: Tygh Gianella
Tester's Company: GeoEngineers, Inc.

Date: 10/20/2015
Tester's Contact No: 503-951-1810

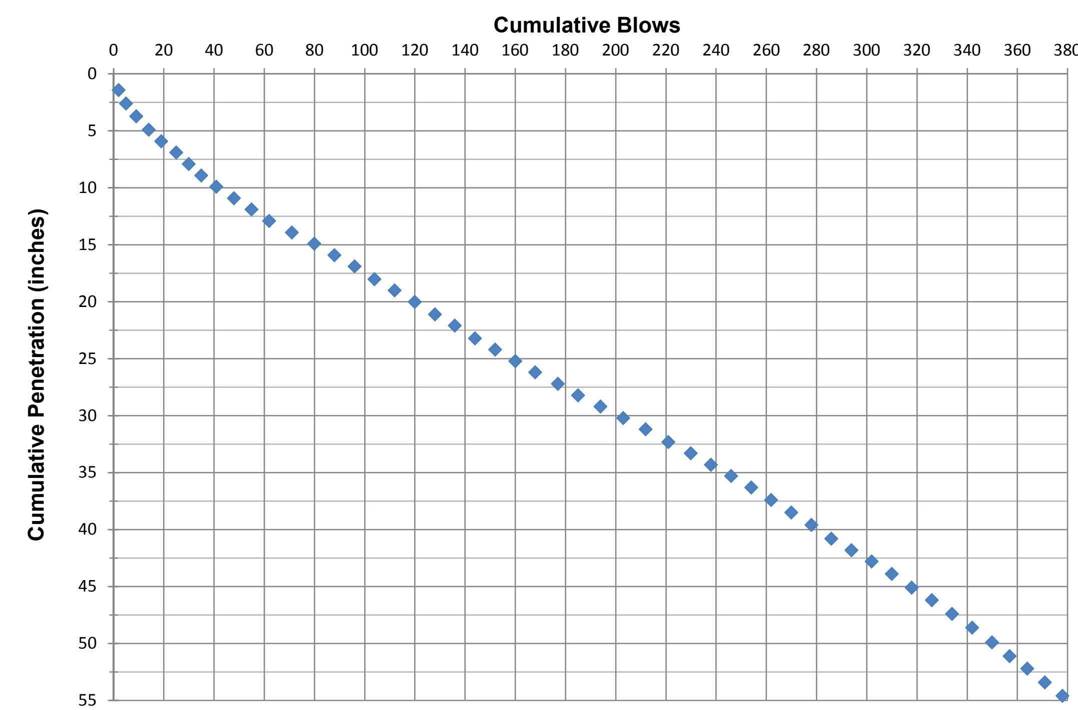
Test Hole Number: DCP-1
Test Method: Dynamic Cone Penetration
GeoEngineers Job: 022126-001-00

| Depth, feet | Soil Texture |
|---|---|
| 0-4.5' | Brown clay (stiff, moist) |
| 4.5'-5' | Brown clay with trace sand (very stiff, moist) |
| | |
| | |

| Test increment | Number of blows | Cumulative blows | Depth roadway surface | Cummulative Penetration | Penetration per blow set | Penetration per blow | Hammer blow factor | DCP Index | DCP Index | CBR | $M_R$ |
|---|---|---|---|---|---|---|---|---|---|---|---|
| # | # | # | (in) | (in) | (in) | (in) | 1 for 8-kg 2 for 4.6-kg hammer | in/blow | mm/blow | % | (psi) |
| 1 | 2 | 2 | 7.4 | 1.4 | 1.4 | 0.7 | 2 | 1.4 | 35.6 | 5.3 | 4262 |
| 2 | 3 | 5 | 8.6 | 2.6 | 1.2 | 0.4 | 2 | 0.8 | 20.3 | 10.0 | 5301 |
| 3 | 4 | 9 | 9.7 | 3.7 | 1.1 | 0.3 | 2 | 0.6 | 14.0 | 15.2 | 6135 |
| 4 | 5 | 14 | 10.9 | 4.9 | 1.2 | 0.2 | 2 | 0.5 | 12.2 | 17.7 | 6470 |
| 5 | 5 | 19 | 11.9 | 5.9 | 1.0 | 0.2 | 2 | 0.4 | 10.2 | 21.8 | 6947 |
| 6 | 6 | 25 | 12.9 | 6.9 | 1.0 | 0.2 | 2 | 0.3 | 8.5 | 26.7 | 7459 |
| 7 | 5 | 30 | 13.9 | 7.9 | 1.0 | 0.2 | 2 | 0.4 | 10.2 | 21.8 | 6947 |
| 8 | 5 | 35 | 14.9 | 8.9 | 1.0 | 0.2 | 2 | 0.4 | 10.2 | 21.8 | 6947 |
| 9 | 6 | 41 | 15.9 | 9.9 | 1.0 | 0.2 | 2 | 0.3 | 8.5 | 26.7 | 7459 |
| 10 | 7 | 48 | 16.9 | 10.9 | 1.0 | 0.1 | 2 | 0.3 | 7.3 | 31.7 | 7921 |
| 11 | 7 | 55 | 17.9 | 11.9 | 1.0 | 0.1 | 2 | 0.3 | 7.3 | 31.7 | 7921 |
| 12 | 7 | 62 | 18.9 | 12.9 | 1.0 | 0.1 | 2 | 0.3 | 7.3 | 31.7 | 7921 |
| 13 | 9 | 71 | 19.9 | 13.9 | 1.0 | 0.1 | 2 | 0.2 | 5.6 | 42.0 | 8736 |
| 14 | 9 | 80 | 20.9 | 14.9 | 1.0 | 0.1 | 2 | 0.2 | 5.6 | 42.0 | 8736 |
| 15 | 8 | 88 | 21.9 | 15.9 | 1.0 | 0.1 | 2 | 0.3 | 6.4 | 36.8 | 8344 |
| 16 | 8 | 96 | 22.9 | 16.9 | 1.0 | 0.1 | 2 | 0.3 | 6.3 | 36.8 | 8344 |
| 17 | 8 | 104 | 24.0 | 18 | 1.1 | 0.1 | 2 | 0.3 | 7.0 | 33.1 | 8040 |
| 18 | 8 | 112 | 25.0 | 19 | 1.0 | 0.1 | 2 | 0.3 | 6.4 | 36.8 | 8344 |
| 19 | 8 | 120 | 26.0 | 20 | 1.0 | 0.1 | 2 | 0.3 | 6.4 | 36.8 | 8344 |
| 20 | 8 | 128 | 27.1 | 21.1 | 1.1 | 0.1 | 2 | 0.3 | 7.0 | 33.1 | 8040 |
| 21 | 8 | 136 | 28.1 | 22.1 | 1.0 | 0.1 | 2 | 0.3 | 6.4 | 36.8 | 8344 |
| 22 | 8 | 144 | 29.2 | 23.2 | 1.1 | 0.1 | 2 | 0.3 | 7.0 | 33.1 | 8040 |
| 23 | 8 | 152 | 30.2 | 24.2 | 1.0 | 0.1 | 2 | 0.3 | 6.4 | 36.8 | 8344 |
| 24 | 8 | 160 | 31.2 | 25.2 | 1.0 | 0.1 | 2 | 0.3 | 6.4 | 36.8 | 8344 |
| 25 | 8 | 168 | 32.2 | 26.2 | 1.0 | 0.1 | 2 | 0.3 | 6.4 | 36.8 | 8344 |
| 26 | 9 | 177 | 33.2 | 27.2 | 1.0 | 0.1 | 2 | 0.2 | 5.6 | 42.0 | 8736 |
| 27 | 8 | 185 | 34.2 | 28.2 | 1.0 | 0.1 | 2 | 0.3 | 6.4 | 36.8 | 8344 |
| 28 | 9 | 194 | 35.2 | 29.2 | 1.0 | 0.1 | 2 | 0.2 | 5.6 | 42.0 | 8736 |
| 29 | 9 | 203 | 36.2 | 30.2 | 1.0 | 0.1 | 2 | 0.2 | 5.6 | 42.0 | 8736 |
| 30 | 9 | 212 | 37.2 | 31.2 | 1.0 | 0.1 | 2 | 0.2 | 5.6 | 42.0 | 8736 |
| 31 | 9 | 221 | 38.3 | 32.3 | 1.1 | 0.1 | 2 | 0.2 | 6.2 | 37.8 | 8418 |
| 32 | 9 | 230 | 39.3 | 33.3 | 1.0 | 0.1 | 2 | 0.2 | 5.6 | 42.0 | 8736 |
| 33 | 8 | 238 | 40.3 | 34.3 | 1.0 | 0.1 | 2 | 0.3 | 6.4 | 36.8 | 8344 |
| 34 | 8 | 246 | 41.3 | 35.3 | 1.0 | 0.1 | 2 | 0.3 | 6.4 | 36.8 | 8344 |
| 35 | 8 | 254 | 42.3 | 36.3 | 1.0 | 0.1 | 2 | 0.3 | 6.4 | 36.8 | 8344 |
| 36 | 8 | 262 | 43.4 | 37.4 | 1.1 | 0.1 | 2 | 0.3 | 7.0 | 33.1 | 8040 |
| 37 | 8 | 270 | 44.5 | 38.5 | 1.1 | 0.1 | 2 | 0.3 | 7.0 | 33.1 | 8040 |
| 38 | 8 | 278 | 45.6 | 39.6 | 1.1 | 0.1 | 2 | 0.3 | 7.0 | 33.1 | 8040 |
| 39 | 8 | 286 | 46.8 | 40.8 | 1.2 | 0.1 | 2 | 0.3 | 7.6 | 30.0 | 7772 |
| 40 | 8 | 294 | 47.8 | 41.8 | 1.0 | 0.1 | 2 | 0.3 | 6.4 | 36.8 | 8344 |
| 41 | 8 | 302 | 48.8 | 42.8 | 1.0 | 0.1 | 2 | 0.3 | 6.4 | 36.8 | 8344 |
| 42 | 8 | 310 | 49.9 | 43.9 | 1.1 | 0.1 | 2 | 0.3 | 7.0 | 33.1 | 8040 |
| 43 | 8 | 318 | 51.1 | 45.1 | 1.2 | 0.2 | 2 | 0.3 | 7.6 | 30.0 | 7772 |
| 44 | 8 | 326 | 52.2 | 46.2 | 1.1 | 0.1 | 2 | 0.3 | 7.0 | 33.1 | 8040 |
| 45 | 8 | 334 | 53.4 | 47.4 | 1.2 | 0.1 | 2 | 0.3 | 7.6 | 30.0 | 7772 |
| 46 | 8 | 342 | 54.6 | 48.6 | 1.2 | 0.2 | 2 | 0.3 | 7.6 | 30.0 | 7772 |
| 47 | 8 | 350 | 55.9 | 49.9 | 1.3 | 0.2 | 2 | 0.3 | 8.3 | 27.5 | 7533 |
| 48 | 7 | 357 | 57.1 | 51.1 | 1.2 | 0.2 | 2 | 0.3 | 8.7 | 25.9 | 7377 |
| 49 | 7 | 364 | 58.2 | 52.2 | 1.1 | 0.2 | 2 | 0.3 | 8.0 | 28.5 | 7632 |
| 50 | 7 | 371 | 59.4 | 53.4 | 1.2 | 0.2 | 2 | 0.3 | 8.7 | 25.9 | 7377 |
| 51 | 7 | 378 | 60.6 | 54.6 | 1.2 | 0.2 | 2 | 0.3 | 8.7 | 25.9 | 7377 |




(after Webster et al., 1992)
Webster, S. L., Grau, R. H., and Williams, T. P. (1992). Description and application of dual mass dynamic cone penetrometer. Department of the Army Waterways Equipment Station, No. GL-92-3.




ODOT Pavement Design Guide. (2011). Pavement Sevices Unit, Oregon Department of Transportation.
$M_R = C_f \times 49023 \times S^{-0.39}$
$M_R$ = resilient modulus (psi)
$C_f$ = conversion coefficient
S = DCP Index (mm/blow)

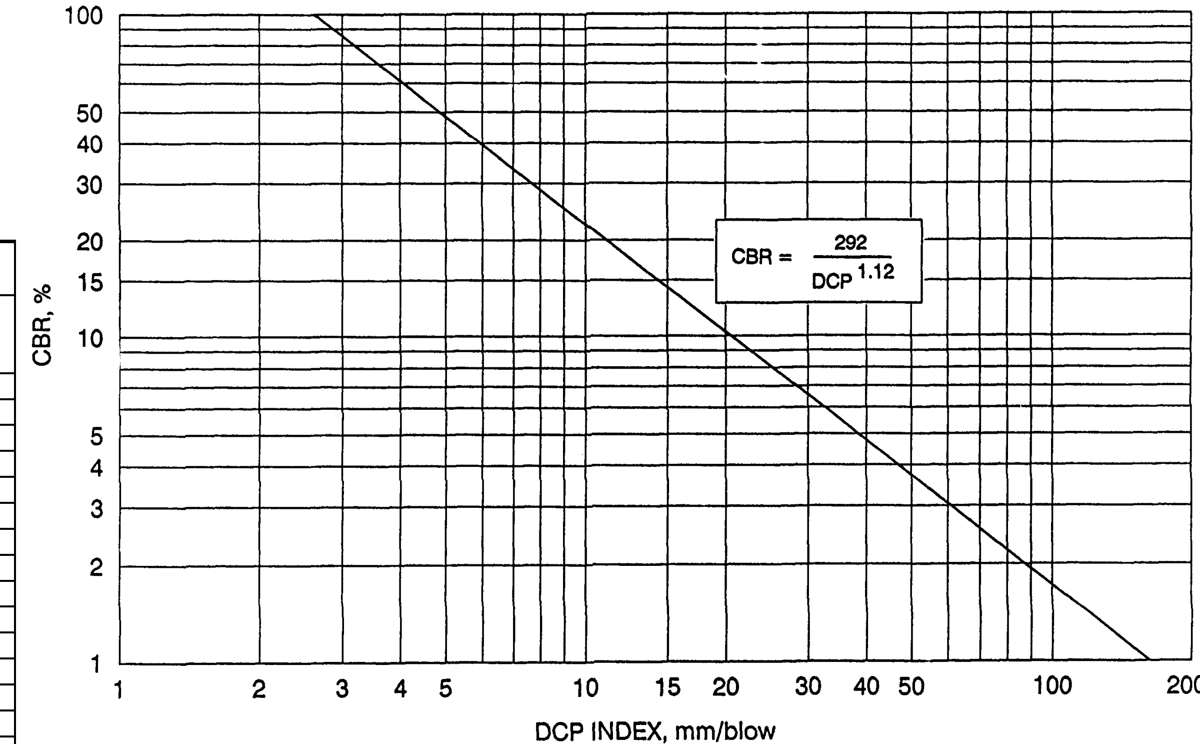
Location: Maritta St. Devlopment
Depth to bottom: 5'
Tester's Name: Tygh Gianella
Tester's Company: GeoEngineers, Inc.

Date: 10/20/2015
Tester's Contact No: 503-951-1810

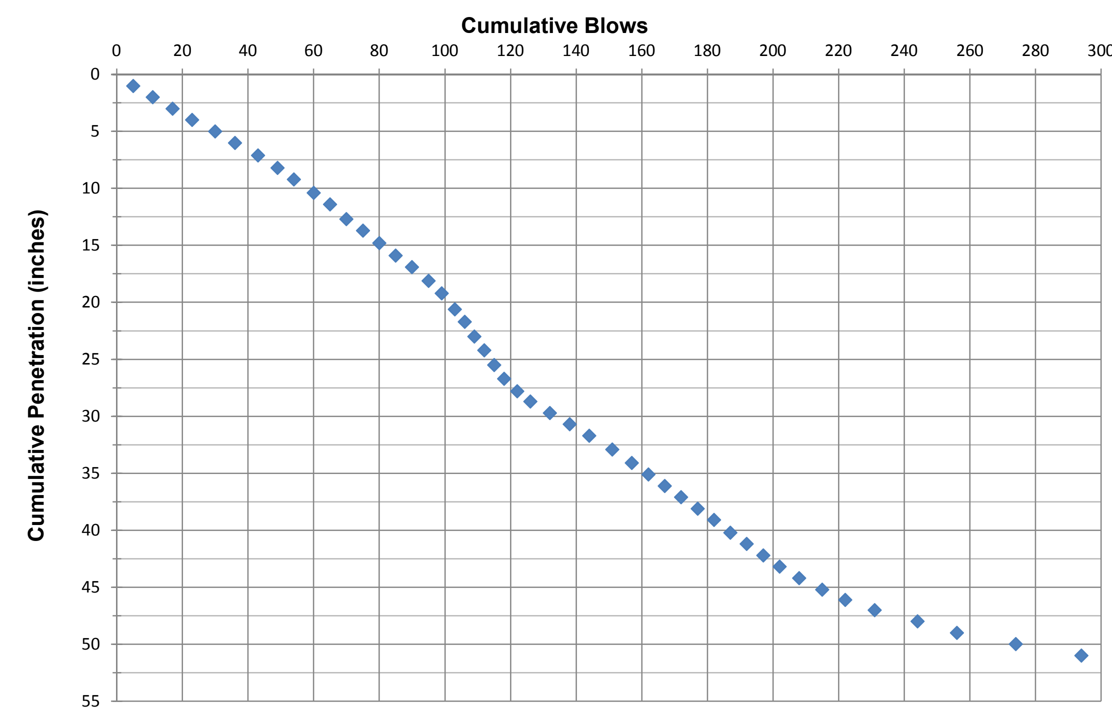
Test Hole Number: DCP-2
Test Method: Dynamic Cone Penetration
GeoEngineers Job: 022126-001-00

| Depth, feet | Soil Texture |
|---|---|
| 0-4.5' | Brown clay with black mottle (stiff, moist) |
| 4.5'-5' | Reddish brown clay with trace gravel and sand (hard, moist) |
| | |
| | |

| Test increment | Number of blows | Cumulative blows | Depth roadway surface | Cummulative Penetration | Penetration per blow set | Penetration per blow | Hammer blow factor | DCP Index | DCP Index | CBR | $M_R$ |
|---|---|---|---|---|---|---|---|---|---|---|---|
| # | # | # | (in) | (in) | (in) | (in) | 1 for 8-kg 2 for 4.6-kg hammer | in/blow | mm/blow | % | (psi) |
| 1 | 5 | 5 | 10.0 | 1.0 | 1.0 | 0.2 | 2 | 0.4 | 10.2 | 21.8 | 6947 |
| 2 | 6 | 11 | 11.0 | 2.0 | 1.0 | 0.2 | 2 | 0.3 | 8.5 | 26.7 | 7459 |
| 3 | 6 | 17 | 12.0 | 3.0 | 1.0 | 0.2 | 2 | 0.3 | 8.5 | 26.7 | 7459 |
| 4 | 6 | 23 | 13.0 | 4.0 | 1.0 | 0.2 | 2 | 0.3 | 8.5 | 26.7 | 7459 |
| 5 | 7 | 30 | 14.0 | 5.0 | 1.0 | 0.1 | 2 | 0.3 | 7.3 | 31.7 | 7921 |
| 6 | 6 | 36 | 15.0 | 6.0 | 1.0 | 0.2 | 2 | 0.3 | 8.5 | 26.7 | 7459 |
| 7 | 7 | 43 | 16.1 | 7.1 | 1.1 | 0.2 | 2 | 0.3 | 8.0 | 28.5 | 7632 |
| 8 | 6 | 49 | 17.2 | 8.2 | 1.1 | 0.2 | 2 | 0.4 | 9.3 | 24.0 | 7186 |
| 9 | 5 | 54 | 18.2 | 9.2 | 1.0 | 0.2 | 2 | 0.4 | 10.2 | 21.8 | 6947 |
| 10 | 6 | 60 | 19.4 | 10.4 | 1.2 | 0.2 | 2 | 0.4 | 10.2 | 21.8 | 6947 |
| 11 | 5 | 65 | 20.4 | 11.4 | 1.0 | 0.2 | 2 | 0.4 | 10.2 | 21.8 | 6947 |
| 12 | 5 | 70 | 21.7 | 12.7 | 1.3 | 0.3 | 2 | 0.5 | 13.2 | 16.2 | 6271 |
| 13 | 5 | 75 | 22.7 | 13.7 | 1.0 | 0.2 | 2 | 0.4 | 10.2 | 21.8 | 6947 |
| 14 | 5 | 80 | 23.8 | 14.8 | 1.1 | 0.2 | 2 | 0.4 | 11.2 | 19.6 | 6693 |
| 15 | 5 | 85 | 24.9 | 15.9 | 1.1 | 0.2 | 2 | 0.4 | 11.2 | 19.6 | 6693 |
| 16 | 5 | 90 | 25.9 | 16.9 | 1.0 | 0.2 | 2 | 0.4 | 10.2 | 21.8 | 6947 |
| 17 | 5 | 95 | 27.1 | 18.1 | 1.2 | 0.2 | 2 | 0.5 | 12.2 | 17.7 | 6470 |
| 18 | 4 | 99 | 28.2 | 19.2 | 1.1 | 0.3 | 2 | 0.5 | 14.0 | 15.2 | 6135 |
| 19 | 4 | 103 | 29.6 | 20.6 | 1.4 | 0.4 | 2 | 0.7 | 17.8 | 11.6 | 5585 |
| 20 | 3 | 106 | 30.7 | 21.7 | 1.1 | 0.4 | 2 | 0.7 | 18.6 | 11.0 | 5484 |
| 21 | 3 | 109 | 32.0 | 23.0 | 1.3 | 0.4 | 2 | 0.9 | 22.0 | 9.2 | 5138 |
| 22 | 3 | 112 | 33.2 | 24.2 | 1.2 | 0.4 | 2 | 0.8 | 20.3 | 10.0 | 5301 |
| 23 | 3 | 115 | 34.5 | 25.5 | 1.3 | 0.4 | 2 | 0.9 | 22.0 | 9.2 | 5138 |
| 24 | 3 | 118 | 35.7 | 26.7 | 1.2 | 0.4 | 2 | 0.8 | 20.3 | 10.0 | 5301 |
| 25 | 4 | 122 | 36.8 | 27.8 | 1.1 | 0.3 | 2 | 0.6 | 14.0 | 15.2 | 6135 |
| 26 | 4 | 126 | 37.7 | 28.7 | 0.9 | 0.2 | 2 | 0.4 | 11.4 | 19.1 | 6635 |
| 27 | 6 | 132 | 38.7 | 29.7 | 1.0 | 0.2 | 2 | 0.3 | 8.5 | 26.7 | 7459 |
| 28 | 6 | 138 | 39.7 | 30.7 | 1.0 | 0.2 | 2 | 0.3 | 8.5 | 26.7 | 7459 |
| 29 | 6 | 144 | 40.7 | 31.7 | 1.0 | 0.2 | 2 | 0.3 | 8.5 | 26.7 | 7459 |
| 30 | 7 | 151 | 41.9 | 32.9 | 1.2 | 0.2 | 2 | 0.3 | 8.7 | 25.9 | 7377 |
| 31 | 6 | 157 | 43.1 | 34.1 | 1.2 | 0.2 | 2 | 0.4 | 10.2 | 21.8 | 6947 |
| 32 | 5 | 162 | 44.1 | 35.1 | 1.0 | 0.2 | 2 | 0.4 | 10.2 | 21.8 | 6947 |
| 33 | 5 | 167 | 45.1 | 36.1 | 1.0 | 0.2 | 2 | 0.4 | 10.2 | 21.8 | 6947 |
| 34 | 5 | 172 | 46.1 | 37.1 | 1.0 | 0.2 | 2 | 0.4 | 10.2 | 21.8 | 6947 |
| 35 | 5 | 177 | 47.1 | 38.1 | 1.0 | 0.2 | 2 | 0.4 | 10.2 | 21.8 | 6947 |
| 36 | 5 | 182 | 48.1 | 39.1 | 1.0 | 0.2 | 2 | 0.4 | 10.2 | 21.8 | 6947 |
| 37 | 5 | 187 | 49.2 | 40.2 | 1.1 | 0.2 | 2 | 0.4 | 11.2 | 19.6 | 6693 |
| 38 | 5 | 192 | 50.2 | 41.2 | 1.0 | 0.2 | 2 | 0.4 | 10.2 | 21.8 | 6947 |
| 39 | 5 | 197 | 51.2 | 42.2 | 1.0 | 0.2 | 2 | 0.4 | 10.2 | 21.8 | 6947 |
| 40 | 5 | 202 | 52.2 | 43.2 | 1.0 | 0.2 | 2 | 0.4 | 10.2 | 21.8 | 6947 |
| 41 | 6 | 208 | 53.2 | 44.2 | 1.0 | 0.2 | 2 | 0.3 | 8.5 | 26.7 | 7459 |
| 42 | 7 | 215 | 54.2 | 45.2 | 1.0 | 0.1 | 2 | 0.3 | 7.3 | 31.7 | 7921 |
| 43 | 7 | 222 | 55.1 | 46.1 | 0.9 | 0.1 | 2 | 0.3 | 6.5 | 35.7 | 8253 |
| 44 | 9 | 231 | 56.0 | 47.0 | 0.9 | 0.1 | 2 | 0.2 | 5.1 | 47.3 | 9103 |
| 45 | 13 | 244 | 57.0 | 48.0 | 1.0 | 0.1 | 2 | 0.2 | 3.9 | 63.5 | 10084 |
| 46 | 12 | 256 | 58.0 | 49.0 | 1.0 | 0.1 | 2 | 0.2 | 4.2 | 58.0 | 9774 |
| 47 | 18 | 274 | 59.0 | 50.0 | 1.0 | 0.1 | 2 | 0.1 | 2.8 | 91.4 | 11448 |
| 48 | 20 | 294 | 60.0 | 51.0 | 1.0 | 0.1 | 2 | 0.1 | 2.5 | 102.8 | 11928 |




(after Webster et al., 1992)
Webster, S. L., Grau, R. H., and Williams, T. P. (1992). Description and application of dual mass dynamic cone penetrometer. Department of the Army Waterways Equipment Station, No. GL-92-3.



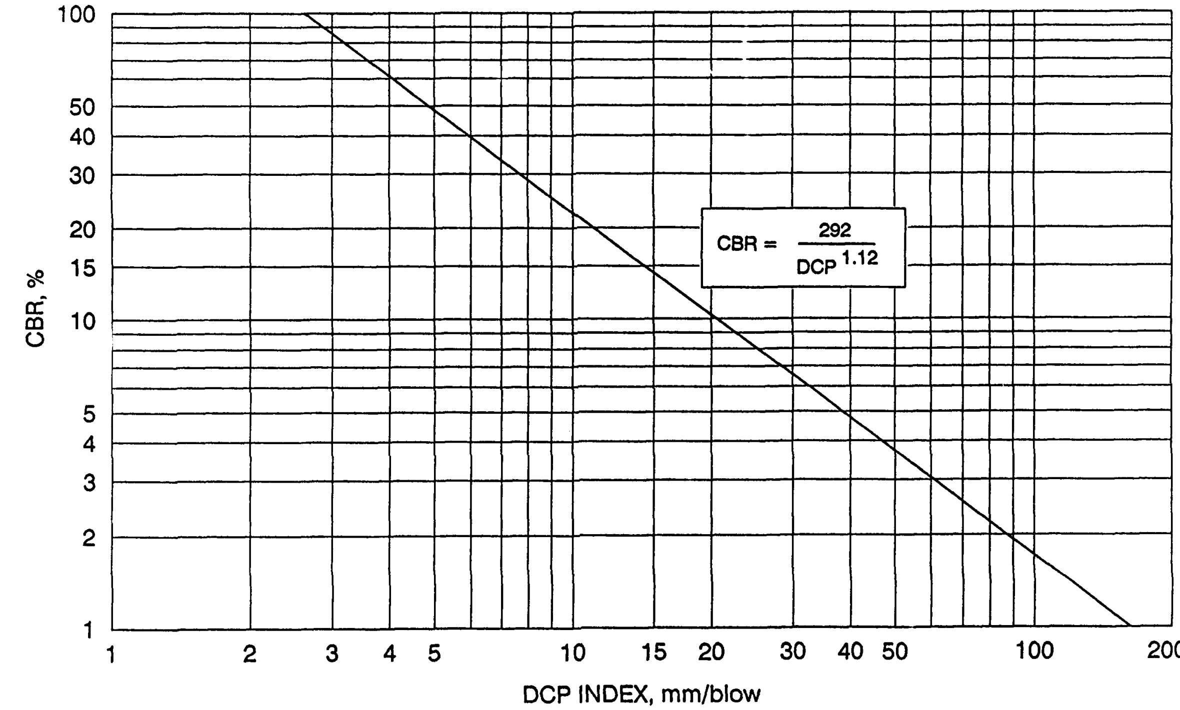

ODOT Pavement Design Guide. (2011). Pavement Sevices Unit, Oregon Department of Transportation.
$M_R = C_f \times 49023 \times S^{-0.39}$
$M_R$ = resilient modulus (psi)
$C_f$ = conversion coefficient
S = DCP Index (mm/blow)

| | |
|---|---|
| Location: Maritta St. Devlopment | Date: 10/20/2015 |
| Depth to bottom: 5' | Tester's Contact No: 503-951-1810 |
| Tester's Name: Tygh Gianella | |
| Tester's Company: GeoEngineers, Inc. | |

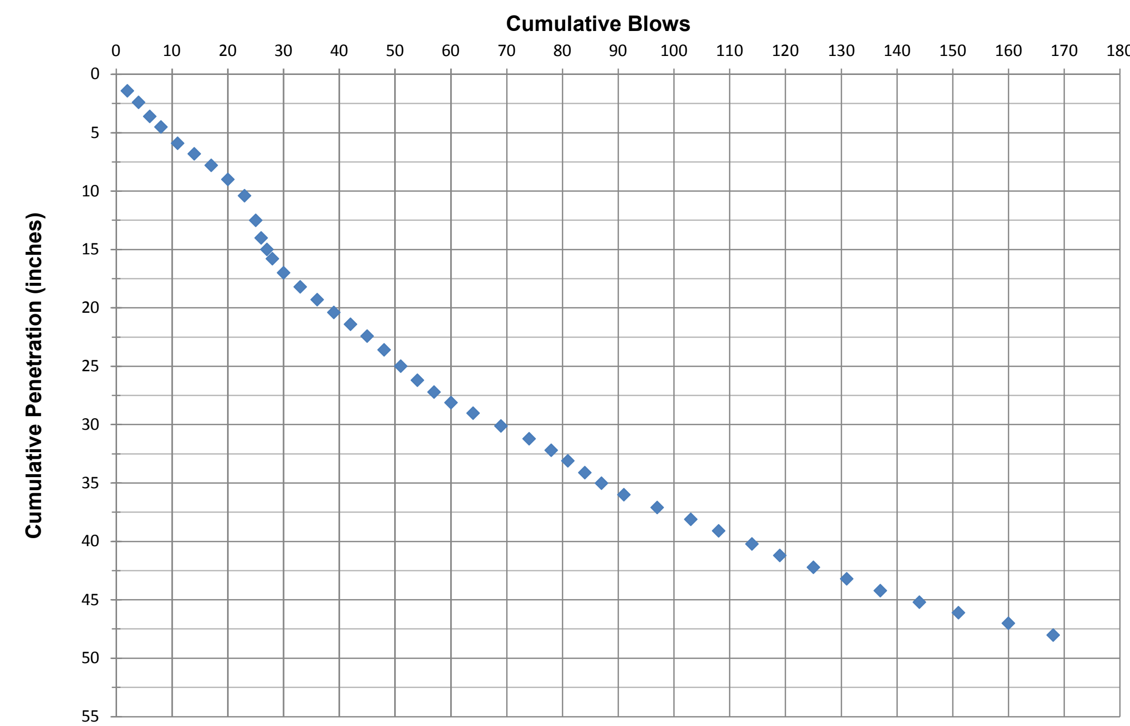
Test Hole Number: DCP-3
Test Method: Dynamic Cone Penetration
GeoEngineers Job: 022126-001-00

| Depth, feet | Soil Texture |
|---|---|
| 0-4.5' | Brown clay (stiff, dry) |
| 4.5'-5' | Reddish brown clay with trace gravel and sand (hard, moist) |
| | |
| | |

| Test increment | Number of blows | Cumulative blows | Depth roadway surface | Cummulative Penetration | Penetration per blow set | Penetration per blow | Hammer blow factor | DCP Index | DCP Index | CBR | $M_R$ |
|---|---|---|---|---|---|---|---|---|---|---|---|
| # | # | # | (in) | (in) | (in) | (in) | 1 for 8-kg 2 for 4.6-kg hammer | in/blow | mm/blow | % | (psi) |
| 1 | 2 | 2 | 13.4 | 1.4 | 1.4 | 0.7 | 2 | 1.4 | 35.6 | 5.3 | 4262 |
| 2 | 2 | 4 | 14.4 | 2.4 | 1.0 | 0.5 | 2 | 1.0 | 25.4 | 7.8 | 4859 |
| 3 | 2 | 6 | 15.6 | 3.6 | 1.2 | 0.6 | 2 | 1.2 | 30.5 | 6.4 | 4526 |
| 4 | 2 | 8 | 16.5 | 4.5 | 0.9 | 0.5 | 2 | 0.9 | 22.9 | 8.8 | 5063 |
| 5 | 3 | 11 | 17.9 | 5.9 | 1.4 | 0.5 | 2 | 0.9 | 23.7 | 8.4 | 4992 |
| 6 | 3 | 14 | 18.8 | 6.8 | 0.9 | 0.3 | 2 | 0.6 | 15.2 | 13.8 | 5931 |
| 7 | 3 | 17 | 19.8 | 7.8 | 1.0 | 0.3 | 2 | 0.7 | 16.9 | 12.3 | 5692 |
| 8 | 3 | 20 | 21.0 | 9.0 | 1.2 | 0.4 | 2 | 0.8 | 20.3 | 10.0 | 5301 |
| 9 | 3 | 23 | 22.4 | 10.4 | 1.4 | 0.5 | 2 | 0.9 | 23.7 | 8.4 | 4992 |
| 10 | 2 | 25 | 24.5 | 12.5 | 2.1 | 1.1 | 2 | 2.1 | 53.3 | 3.4 | 3638 |
| 11 | 1 | 26 | 26.0 | 14.0 | 1.5 | 1.5 | 2 | 3.0 | 76.2 | 2.3 | 3166 |
| 12 | 1 | 27 | 27.0 | 15.0 | 1.0 | 1.0 | 2 | 2.0 | 50.8 | 3.6 | 3708 |
| 13 | 1 | 28 | 27.8 | 15.8 | 0.8 | 0.8 | 2 | 1.6 | 40.6 | 4.6 | 4046 |
| 14 | 2 | 30 | 29.0 | 17.0 | 1.2 | 0.6 | 2 | 1.2 | 30.5 | 6.4 | 4526 |
| 15 | 3 | 33 | 30.2 | 18.2 | 1.2 | 0.4 | 2 | 0.8 | 20.3 | 10.0 | 5301 |
| 16 | 3 | 36 | 31.3 | 19.3 | 1.1 | 0.4 | 2 | 0.7 | 18.6 | 11.0 | 5484 |
| 17 | 3 | 39 | 32.4 | 20.4 | 1.1 | 0.4 | 2 | 0.7 | 18.6 | 11.0 | 5484 |
| 18 | 3 | 42 | 33.4 | 21.4 | 1.0 | 0.3 | 2 | 0.7 | 16.9 | 12.3 | 5692 |
| 19 | 3 | 45 | 34.4 | 22.4 | 1.0 | 0.3 | 2 | 0.7 | 16.9 | 12.3 | 5692 |
| 20 | 3 | 48 | 35.6 | 23.6 | 1.2 | 0.4 | 2 | 0.8 | 20.3 | 10.0 | 5301 |
| 21 | 3 | 51 | 37.0 | 25.0 | 1.4 | 0.5 | 2 | 0.9 | 23.7 | 8.4 | 4992 |
| 22 | 3 | 54 | 38.2 | 26.2 | 1.2 | 0.4 | 2 | 0.8 | 20.3 | 10.0 | 5301 |
| 23 | 3 | 57 | 39.2 | 27.2 | 1.0 | 0.3 | 2 | 0.7 | 16.9 | 12.3 | 5692 |
| 24 | 3 | 60 | 40.1 | 28.1 | 0.9 | 0.3 | 2 | 0.6 | 15.2 | 13.8 | 5931 |
| 25 | 4 | 64 | 41.0 | 29.0 | 0.9 | 0.2 | 2 | 0.4 | 11.4 | 19.1 | 6635 |
| 26 | 5 | 69 | 42.1 | 30.1 | 1.1 | 0.2 | 2 | 0.4 | 11.2 | 19.6 | 6693 |
| 27 | 5 | 74 | 43.2 | 31.2 | 1.1 | 0.2 | 2 | 0.4 | 11.2 | 19.6 | 6693 |
| 28 | 4 | 78 | 44.2 | 32.2 | 1.0 | 0.3 | 2 | 0.5 | 12.7 | 16.9 | 6368 |
| 29 | 3 | 81 | 45.1 | 33.1 | 0.9 | 0.3 | 2 | 0.6 | 15.2 | 13.8 | 5931 |
| 30 | 3 | 84 | 46.1 | 34.1 | 1.0 | 0.3 | 2 | 0.7 | 16.9 | 12.3 | 5692 |
| 31 | 3 | 87 | 47.0 | 35.0 | 0.9 | 0.3 | 2 | 0.6 | 15.2 | 13.8 | 5931 |
| 32 | 4 | 91 | 48.0 | 36.0 | 1.0 | 0.3 | 2 | 0.5 | 12.7 | 16.9 | 6368 |
| 33 | 6 | 97 | 49.1 | 37.1 | 1.1 | 0.2 | 2 | 0.4 | 9.3 | 24.0 | 7186 |
| 34 | 6 | 103 | 50.1 | 38.1 | 1.0 | 0.2 | 2 | 0.3 | 8.5 | 26.7 | 7459 |
| 35 | 5 | 108 | 51.1 | 39.1 | 1.0 | 0.2 | 2 | 0.4 | 10.2 | 21.8 | 6947 |
| 36 | 6 | 114 | 52.2 | 40.2 | 1.1 | 0.2 | 2 | 0.4 | 9.3 | 24.0 | 7186 |
| 37 | 5 | 119 | 53.2 | 41.2 | 1.0 | 0.2 | 2 | 0.4 | 10.2 | 21.8 | 6947 |
| 38 | 6 | 125 | 54.2 | 42.2 | 1.0 | 0.2 | 2 | 0.3 | 8.5 | 26.7 | 7459 |
| 39 | 6 | 131 | 55.2 | 43.2 | 1.0 | 0.2 | 2 | 0.3 | 8.5 | 26.7 | 7459 |
| 40 | 6 | 137 | 56.2 | 44.2 | 1.0 | 0.2 | 2 | 0.3 | 8.5 | 26.7 | 7459 |
| 41 | 7 | 144 | 57.2 | 45.2 | 1.0 | 0.1 | 2 | 0.3 | 7.3 | 31.7 | 7921 |
| 42 | 7 | 151 | 58.1 | 46.1 | 0.9 | 0.1 | 2 | 0.3 | 6.5 | 35.7 | 8253 |
| 43 | 9 | 160 | 59.0 | 47.0 | 0.9 | 0.1 | 2 | 0.2 | 5.1 | 47.3 | 9103 |
| 44 | 8 | 168 | 60.0 | 48.0 | 1.0 | 0.1 | 2 | 0.3 | 6.4 | 36.8 | 8344 |




(after Webster et al., 1992)
Webster, S. L., Grau, R. H., and Williams, T. P. (1992). Description and application of dual mass dynamic cone penetrometer. Department of the Army Waterways Equipment Station, No. GL-92-3.




ODOT Pavement Design Guide. (2011). Pavement Sevices Unit, Oregon Department of Transportation.
$M_R = C_f \times 49023 \times S^{-0.39}$
$M_R$ = resilient modulus (psi)
$C_f$ = conversion coefficient
S = DCP Index (mm/blow)

Location: Marietta Street Salem
Date: 10/20/2015 - 10/21/2015
Test Hole Number: I-1
Depth to bottom: 5'
Dimension: Pipe ID = 4.0"
Test Method: Encased Falling Head
Tester's Name: Tygh Gianella
GeoEngineers Job: 022126-001-00
Tester's Company: GeoEngineers, Inc.
Tester's Contact No: 503-951-1810

| Depth, feet | Soil Texture |
|---|---|
| 0-5' | Brown Silt (stiff, dry) |
| | |
| | |
| | |

| Date/Time of Day | Time Interval (min) | Total Time (min) | Depth to Water from Top of Pipe (inches) | Dist. Interval (inches) | Infiltration (inches/hour) |
|---|---|---|---|---|---|
| 10/20/2015 10:04 | 0 | 0 | 97.86 | | |
| 10/20/2015 14:50 | 286 | 286 | 97.92 | 0.06 | 0.01 |
| 10/21/2015 7:45 | 1015 | 1301 | 98.64 | 0.72 | 0.04 |
| 10/21/2015 15:11 | 446 | 1747 | 98.70 | 0.06 | 0.01 |
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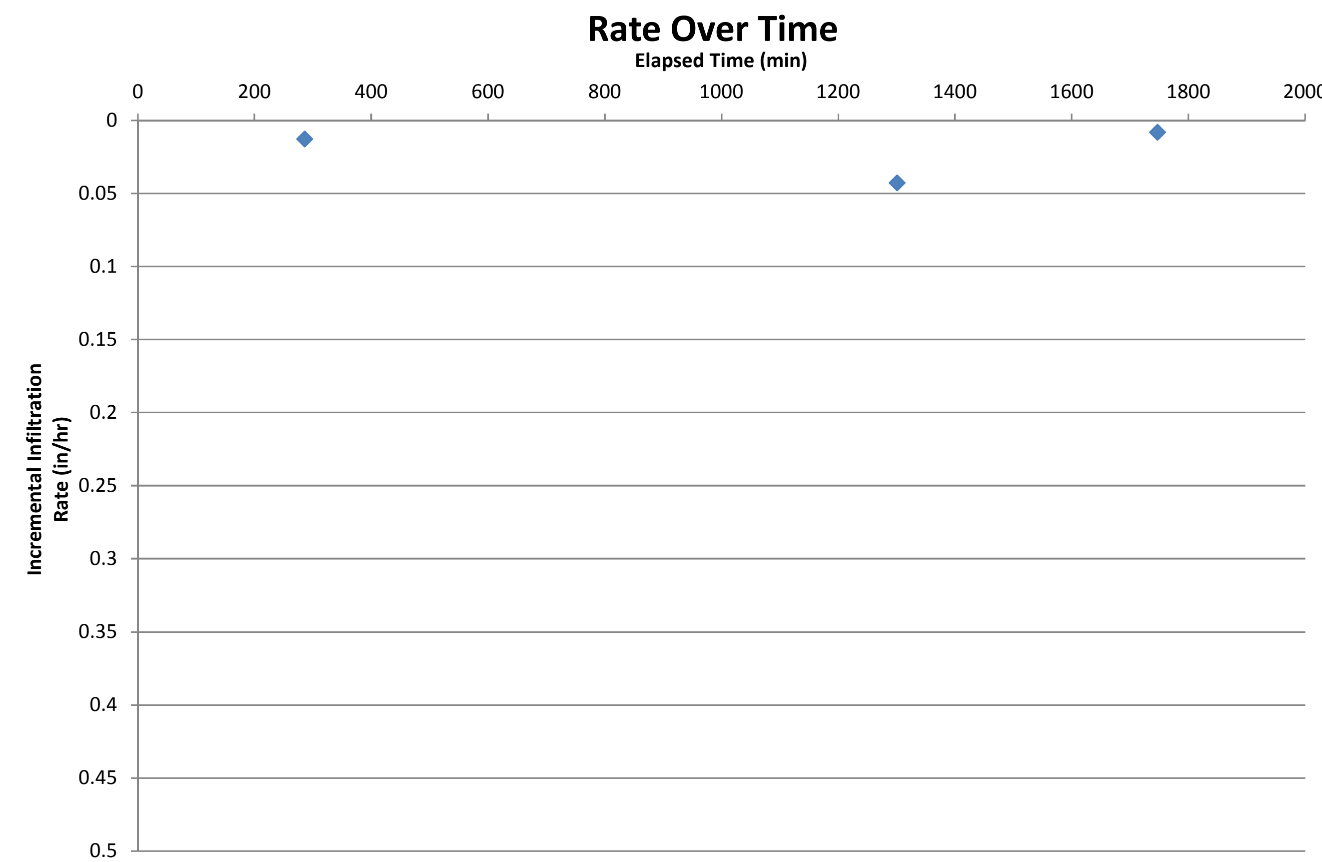

Location: Marietta Street Salem
Depth to bottom: 3'-2.5"
Tester's Name: Tygh Gianella
Tester's Company: GeoEngineers, Inc.

Date: 10/20/2015 - 10/21/2015
Dimension: Pipe ID = 4.0"
Tester's Contact No: 503-951-1810

Test Hole Number: I-2
Test Method: Encased Falling Head
GeoEngineers Job: 022126-001-00

| Depth, feet | Soil Texture |
|---|---|
| 0-5' | Brown silt (stiff, dry) |
| | |
| | |
| | |

| Date/Time of Day | Time Interval (min) | Total Time (min) | Depth to Water from Top of Pipe (inches) | Dist. Interval (inches) | Infiltration (inches/hour) |
|---|---|---|---|---|---|
| 10/20/2015 10:50 | 0 | 0 | 35.64 | | |
| 10/20/2015 14:55 | 245 | 245 | 35.76 | 0.12 | 0.03 |
| 10/21/2015 7:44 | 1009 | 1254 | 36.12 | 0.36 | 0.02 |
| 10/21/2015 15:11 | 446 | 1700 | 36.24 | 0.12 | 0.02 |
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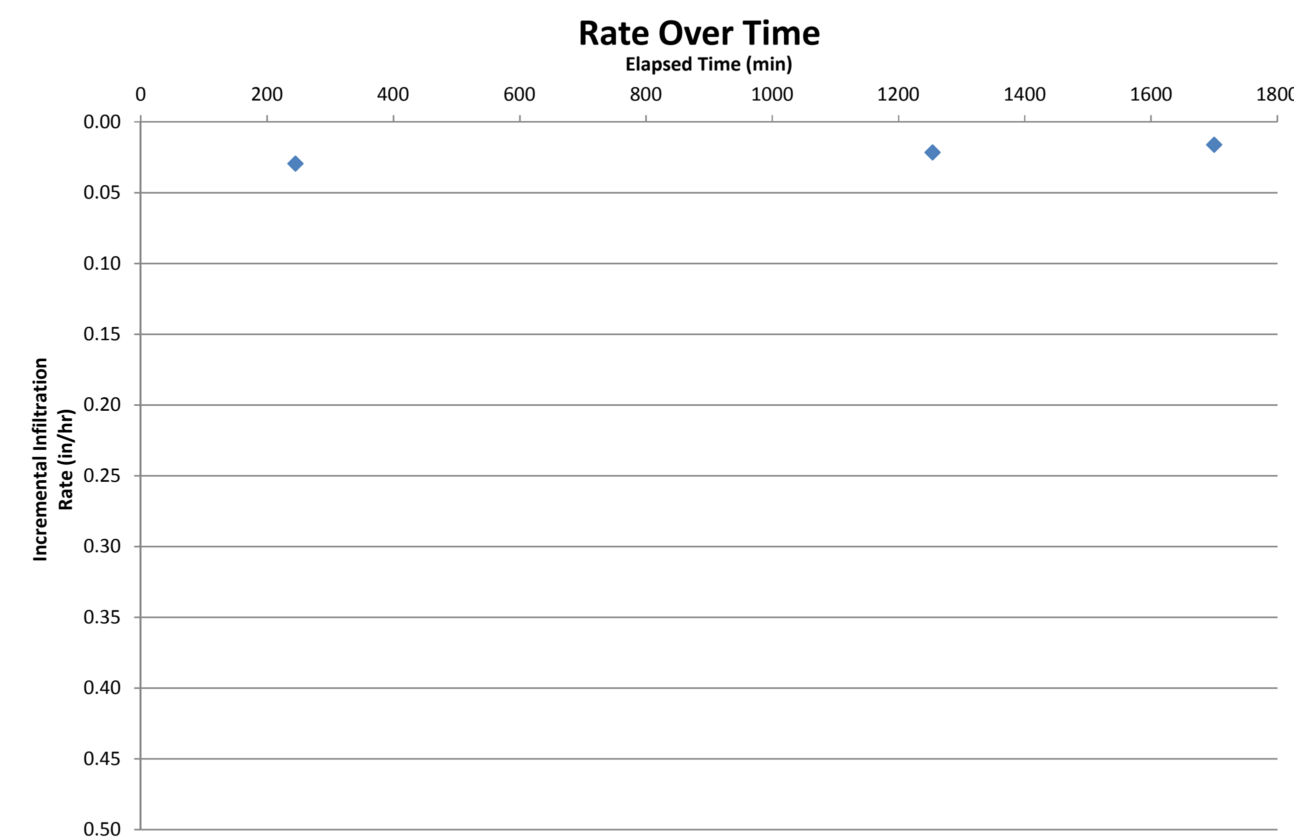

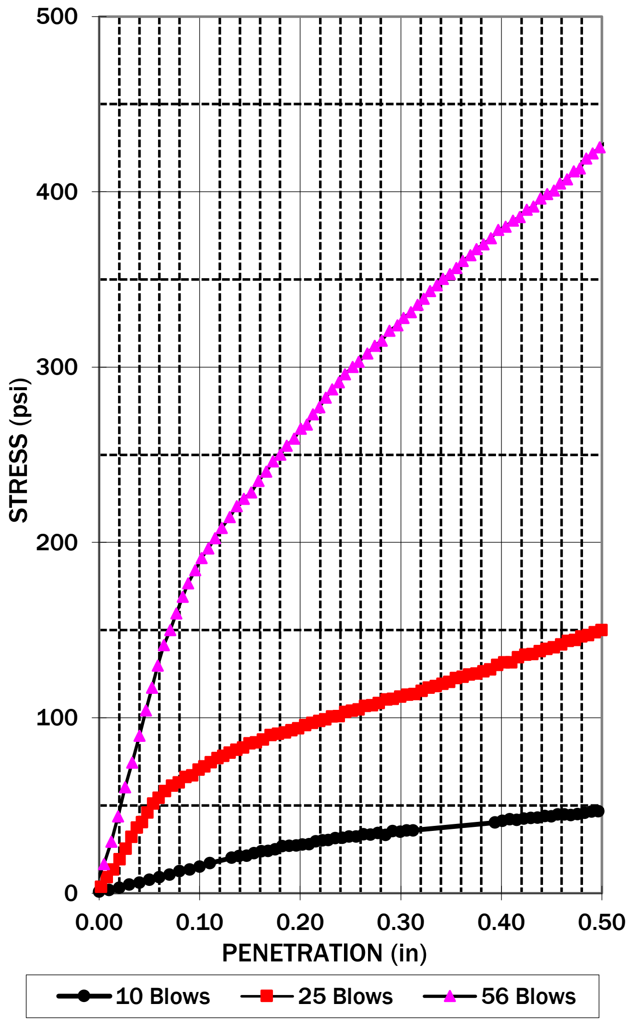


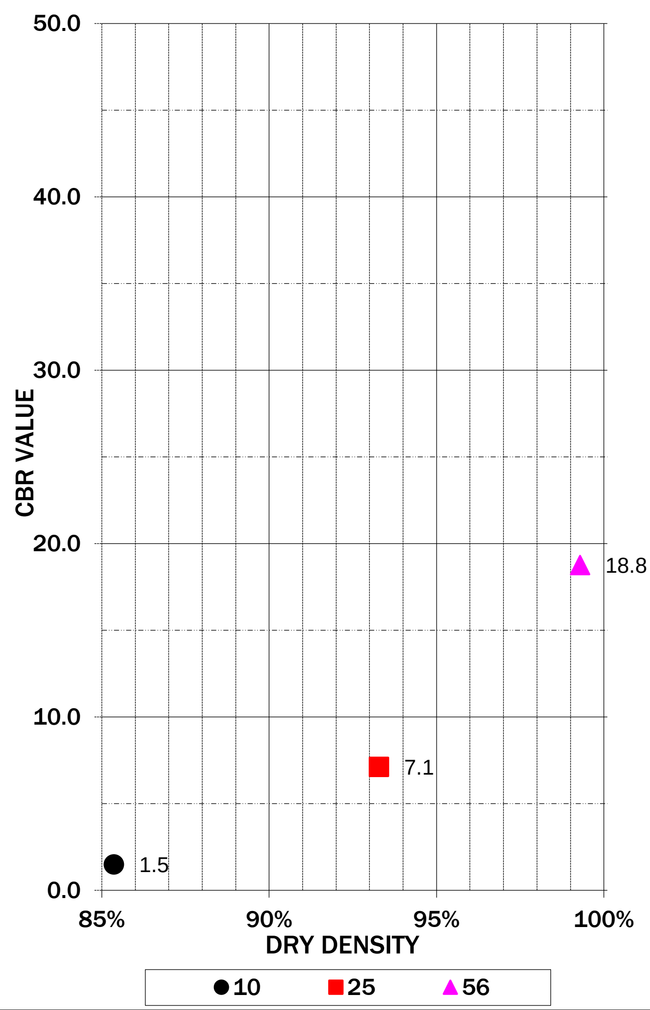


| CBR Information | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | Molded | | | Soaked | | | Surcharge Wt. | Volume Change | CBR (%) | |
| No. of Blows | Dry Density (pcf) | Percent of Max. Dry Density | Moisture Content % | Dry Density (pcf) | Percent of Max. Dry Density | Moisture Content % | (lb) | % | 0.1" | 0.2" |
| 10 | 102 | 85% | 15.1 | 101 | 84% | 23.3 | 10 | 2.2 | 1.5 | 1.8 |
| 25 | 112 | 93% | 15.4 | 110 | 92% | 19.7 | 10 | 2.0 | 7.1 | 6.3 |
| 56 | 119 | 99% | 15.4 | 116 | 97% | 18.0 | 10 | 1.9 | 18.8 | 17.7 |

| Proctor Information | | | |
|---|---|---|---|
| Maximum Dry Density: | 119.8 | Rock Corrected Maximum: | NA |
| Optimum % Moisture: | 14.6 | Rock Corrected Optimum % Moisture: | NA |
| Test Standard: | 1557 | Percent +3/4": | <5% |
| Method: | C | Correction Method: | NA |



The California Bearing Ratio was obtained in general accordance with ASTM D 1883. The CBR Value for the 10 blow sample is approximated base on data acquired. The first 0.1 inch of penetration data was not recorded due to a computer error while running the test.

**California Bearing Ratio Test**

Open Dental Software – Marietta Street Development
Salem, Oregon

GeoEngineers

**Figure A-17**

22126-001-00 Date Exported 11/5/2015 by kjanci

# APPENDIX B
## Report Limitations and Guidelines for Use

## APPENDIX B
## REPORT LIMITATIONS AND GUIDELINES FOR USE[1]

This appendix provides information to help you manage your risks with respect to the use of this report.

### Read These Provisions Closely

It is important to recognize that the geoscience practices (geotechnical engineering, geology and environmental science) rely on professional judgment and opinion to a greater extent than other engineering and natural science disciplines, where more precise and/or readily observable data may exist. To help clients better understand how this difference pertains to our services, GeoEngineers includes the following explanatory "limitations" provisions in its reports. Please confer with GeoEngineers if you need to know more how these "Report Limitations and Guidelines for Use" apply to your project or site.

### Geotechnical Services Are Performed for Specific Purposes, Persons and Projects

This report has been prepared for Westech Engineering, Inc. and for the proposed Open Dental Software development project specifically identified in the report. The information contained herein is not applicable to other sites or projects.

GeoEngineers structures its services to meet the specific needs of its clients. No party other than the party to whom this report is addressed may rely on the product of our services unless we agree to such reliance in advance and in writing. Within the limitations of the agreed scope of services for the Project, and its schedule and budget, our services have been executed in accordance with our Agreement with Westech Engineering, Inc. dated October 15, 2015 and generally accepted geotechnical practices in this area at the time this report was prepared. We do not authorize, and will not be responsible for, the use of this report for any purposes or projects other than those identified in the report.

### A Geotechnical Engineering or Geologic Report is Based on a Unique Set of Project-Specific Factors

This report has been prepared for the proposed Open Dental development northeast of the intersection of Marietta Street SE and 32nd Avenue SE in Salem, Oregon. GeoEngineers considered a number of unique, project-specific factors when establishing the scope of services for this project and report. Unless GeoEngineers specifically indicates otherwise, it is important not to rely on this report if it was:

- not prepared for you,
- not prepared for your project,
- not prepared for the specific site explored, or
- completed before important project changes were made.

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[1] Developed based on material provided by ASFE, Professional Firms Practicing in the Geosciences; www.asfe.org.

For example, changes that can affect the applicability of this report include those that affect:

- the function of the proposed structure;
- elevation, configuration, location, orientation or weight of the proposed structure;

If changes occur after the date of this report, GeoEngineers cannot be responsible for any consequences of such changes in relation to this report unless we have been given the opportunity to review our interpretations and recommendations. Based on that review, we can provide written modifications or confirmation, as appropriate.

### Environmental Concerns Are Not Covered

Unless environmental services were specifically included in our scope of services, this report does not provide any environmental findings, conclusions, or recommendations, including but not limited to, the likelihood of encountering underground storage tanks or regulated contaminants.

### Subsurface Conditions Can Change

This geotechnical or geologic report is based on conditions that existed at the time the study was performed. The findings and conclusions of this report may be affected by the passage of time, by man-made events such as construction on or adjacent to the site, new information or technology that becomes available subsequent to the report date, or by natural events such as floods, earthquakes, slope instability or groundwater fluctuations. If more than a few months have passed since issuance of our report or work product, or if any of the described events may have occurred, please contact GeoEngineers before applying this report for its intended purpose so that we may evaluate whether changed conditions affect the continued reliability or applicability of our conclusions and recommendations.

### Geotechnical and Geologic Findings Are Professional Opinions

Our interpretations of subsurface conditions are based on field observations from widely spaced sampling locations at the site. Site exploration identifies the specific subsurface conditions only at those points where subsurface tests are conducted or samples are taken. GeoEngineers reviewed field and laboratory data and then applied its professional judgment to render an informed opinion about subsurface conditions at other locations. Actual subsurface conditions may differ, sometimes significantly, from the opinions presented in this report. Our report, conclusions and interpretations are not a warranty of the actual subsurface conditions.

### Geotechnical Engineering Report Recommendations Are Not Final

We have developed the following recommendations based on data gathered from subsurface investigation(s). These investigations sample just a small percentage of a site to create a snapshot of the subsurface conditions elsewhere on the site. Such sampling on its own cannot provide a complete and accurate view of subsurface conditions for the entire site. Therefore, the recommendations included in this report are preliminary and should not be considered final. GeoEngineers' recommendations can be finalized only by observing actual subsurface conditions revealed during construction. GeoEngineers cannot assume responsibility or liability for the recommendations in this report if we do not perform construction observation.

We recommend that you allow sufficient monitoring, testing and consultation during construction by GeoEngineers to confirm that the conditions encountered are consistent with those indicated by the explorations, to provide recommendations for design changes if the conditions revealed during the work differ from those anticipated, and to evaluate whether earthwork activities are completed in accordance with our recommendations. Retaining GeoEngineers for construction observation for this project is the most effective means of managing the risks associated with unanticipated conditions. If another party performs field observation and confirms our expectations, the other party must take full responsibility for both the observations and recommendations. Please note, however, that another party would lack our project-specific knowledge and resources.

### A Geotechnical Engineering or Geologic Report Could Be Subject to Misinterpretation

Misinterpretation of this report by members of the design team or by contractors can result in costly problems. GeoEngineers can help reduce the risks of misinterpretation by conferring with appropriate members of the design team after submitting the report, reviewing pertinent elements of the design team's plans and specifications, participating in pre-bid and preconstruction conferences, and providing construction observation.

### Do Not Redraw the Exploration Logs

Geotechnical engineers and geologists prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. The logs included in a geotechnical engineering or geologic report should never be redrawn for inclusion in architectural or other design drawings. Photographic or electronic reproduction is acceptable, but separating logs from the report can create a risk of misinterpretation.

### Give Contractors a Complete Report and Guidance

To help reduce the risk of problems associated with unanticipated subsurface conditions, GeoEngineers recommends giving contractors the complete geotechnical engineering or geologic report, including these "Report Limitations and Guidelines for Use." When providing the report, you should preface it with a clearly written letter of transmittal that:

- advises contractors that the report was not prepared for purposes of bid development and that its accuracy is limited; and
- encourages contractors to confer with GeoEngineers and/or to conduct additional study to obtain the specific types of information they need or prefer.

### Contractors Are Responsible for Site Safety on Their Own Construction Projects

Our geotechnical recommendations are not intended to direct the contractor's procedures, methods, schedule or management of the work site. The contractor is solely responsible for job site safety and for managing construction operations to minimize risks to on-site personnel and adjacent properties.

### Biological Pollutants

GeoEngineers' Scope of Work specifically excludes the investigation, detection, prevention or assessment of the presence of Biological Pollutants. Accordingly, this report does not include any interpretations, recommendations, findings or conclusions regarding the detecting, assessing, preventing or abating of Biological Pollutants, and no conclusions or inferences should be drawn regarding Biological Pollutants as they may relate to this project. The term "Biological Pollutants" includes, but is not limited to, molds, fungi, spores, bacteria and viruses, and/or any of their byproducts.

A Client that desires these specialized services is advised to obtain them from a consultant who offers services in this specialized field.

Have we delivered World Class Client Service?

Please let us know by visiting **www.geoengineers.com/feedback**.

GeoEngineers