GeoPacific Engineering, Inc.
Real-World Geotechnical Solutions
Investigation • Design • Construction Support

Geotechnical Engineering Study

Marquis Oregon City Parking Lot Expansion
1680 Molalla Avenue
Oregon City, Oregon 97045

GeoPacific Engineering, Inc. Job No. 18-4984
August 29, 2018

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Real-World Geotechnical Solutions
Investigation • Design • Construction Support

August 28, 2018
Project No. 18-4984

Marquis Companies
Mr. Scott Miller
4560 SE International Way, Suite 100
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Phone: (971) 206-5200

SUBJECT: GEOTECHNICAL ENGINEERING STUDY
MARQUIS OREGON CITY PARKING LOT EXPANSION
1680 MOLALLA AVENUE
OREGON CITY, OREGON 97045

PROJECT INFORMATION

This report presents the results of a geotechnical engineering study conducted by GeoPacific Engineering, Inc. (GeoPacific) for the above-referenced project. The purpose of our study was to investigate subsurface conditions at the site and provide recommendations for stormwater management and the construction of new pavement sections. This geotechnical study was performed in accordance with GeoPacific Proposal No. P-6631, dated June 13, 2018, and your subsequent authorization of our proposal and *General Conditions for Geotechnical Services*.

Site Location:	1680 Molalla Avenue Oregon City, Oregon 97045 (see Figures 1 through 3)
Civil Designer:	Emerio Design Kyung Han 6445 SW Fallbrook Place, Suite 100 Beaverton, Oregon 97008 Phone: (503) 746-8812 Email: kyung@emeriodesign.com
Jurisdictional Agency:	City of Oregon City, Oregon
Prepared By:	GeoPacific Engineering, Inc 14835 SW 72nd Avenue Portland, Oregon 97224 Tel (503) 598-8445 Fax (503) 941-9281

SITE AND PROJECT DESCRIPTION

The site is approximately 1.84 acres in size, and is located at 1680 Molalla Avenue in Oregon City, Oregon (Figure 1). The site is bordered by Beavercreek Road to the north, Molalla Avenue to the east, and commercial businesses to the south, west, and northwest. Vegetation onsite consists primarily of short grasses, shrubs, and medium to large trees. The central portion of the site is occupied by the Marquis Oregon City Post Acute Rehab Center with parking and drive areas surrounding the existing building. The southeastern portion of the site is undeveloped and forested with conifer trees. Topography at the site is relatively flat with site elevations ranging from 429 to 432 feet amsl.

Based upon communication with the client and review of preliminary project plans, GeoPacific understands that the proposed development at the site will consist of the expansion of the existing parking lot primarily onto the southeastern portion of the site, and associated stormwater facilities.

REGIONAL GEOLOGIC SETTING

Regionally, the subject site lies within the Willamette Valley/Puget Sound lowland, a broad structural depression situated between the Coast Range on the west and the Cascade Range on the east. A series of discontinuous faults subdivide the Willamette Valley into a mosaic of fault-bounded, structural blocks (Yeats et al., 1996). Uplifted structural blocks form bedrock highlands, while down-warped structural blocks form sedimentary basins. Valley-fill sediment in the adjacent basin achieves a maximum thickness of 1,500 feet and overlies Miocene Columbia River Basalt at depth (Madin, 1990; Yeats et al., 1996).

The subject site lies on a broad volcanic plateau underlain by the Boring Lava which formed during a period of Plio-Pleistocene (5 to 0.2 million years ago) volcanism and faulting (Schlicker and Finlayson, 1979). The Boring Lava consists mainly of basaltic lava flows, but locally contains tuff breccia, ash, tuff, cinders, and scoriaceous volcanic debris flows deposited on the flanks of volcanic cones. The flows are commonly light gray to nearly black, with lighter tones predominating, and are characterized by columnar jointing and flow structures. The upper surface of the Boring Lava is typically weathered to depths of 25 feet or more with the upper 5 to 15 feet consisting of red-brown, clayey silt to silty clay soil.

FIELD EXPLORATION AND SUBSURFACE CONDITIONS

Our site-specific explorations for this report was conducted on July 27, 2018 and consisted of 3 hand auger borings extending to a maximum depth of 5.5 feet below ground surface (bgs), and 2 portable dynamic cone penetration tests (PDCPs). Hand augers HA-1 and HA-3 were performed to observe soil and groundwater conditions. Infiltration testing was conducted within hand auger HA-2 at 4.6 feet below the ground surface. The approximate locations of the explorations are shown on the attached site plans (Figures 2 and 3). It should be noted that the exploration locations were located in the field by pacing or taping distances from apparent property corners and other site features shown on the plans provided. As such, the locations of the explorations should be considered approximate. During the exploration, GeoPacific observed and recorded pertinent soil information such as color, stratigraphy, strength, and soil moisture content. Soils were classified in general accordance with the Unified Soil Classification System (USCS). At the completion of the explorations, the hand augers were backfilled loosely with onsite soils. Exploration logs corresponding to hand augers HA-1 through HA-3, PDCP-1, and PDCP-2 are attached to the appendix of this report. Soil and groundwater conditions encountered in our explorations are summarized below.

Soil Descriptions

Undocumented Fill: Underlying the ground surface at the location of hand auger HA-2, we observed undocumented fill material consisting of medium stiff, brown, moderately organic, damp, SILT (ML-OL). The fill material contained subrounded to angular gravel and fine to medium roots. The undocumented fill was surfaced with grass and developed approximately 6 inches of topsoil on the ground surface. The undocumented fill extended to an approximate depth of 18 inches below the ground surface at the location of hand auger HA-2.

Topsoil Horizon: Underlying the ground surface at the location of hand augers HA-1 and HA-3, we observed a topsoil horizon consisting of medium stiff, dark brown, moderately organic, damp, SILT (ML-OL). The topsoil layer contained fine to medium roots. The topsoil horizon extended to an approximate depth of 10 inches below the ground surface in hand auger borings HA-1 and HA-3.

Residual Soil: Underlying the topsoil in hand augers HA-1 and HA-3, and the undocumented fill in hand auger HA-2, we observed residual soil consisting of stiff to very stiff, damp to moist, low plasticity, reddish brown, Lean CLAY (CL). The residual soil gradually graded to weathered rock at an approximate depth of 4-5 feet in our hand auger explorations. Based upon our observations of the soil type and review of geologic mapping, residual soil encountered in our explorations was derived from weathering of the underlying Boring Lava Formation.

***Boring Lava* –** Beneath the residual soil, we encountered weathered rock belonging to the Boring Lava Formation in all explorations. The upper foot of the weathered rock was generally extremely soft to very soft (R0-R1). We experienced practical refusal on very soft (R1) basalt at a depth of 4.6 feet in hand auger HA-1, 5.5 feet in hand auger HA-2, and 4.2 feet in hand auger HA-3.

Table 1 - Rock Hardness Classification Chart

ODOT Rock Hardness Rating	Field Criteria	Unconfined Compressive Strength	Typical Equipment Needed For Excavation
Extremely Soft (R0)	Indented by thumbnail	<100 psi	Small excavator
Very Soft (R1)	Scratched by thumbnail, crumbled by rock hammer	100-1,000 psi	Small excavator
Soft (R2)	Not scratched by thumbnail, indented by rock hammer	1,000-4,000 psi	Medium excavator (slow digging with small excavator)
Medium Hard (R3)	Scratched or fractured by rock hammer	4,000-8,000 psi	Medium to large excavator (slow to very slow digging), typically requires chipping with hydraulic hammer or mass excavation)
Hard (R4)	Scratched or fractured w/ difficulty	8,000-16,000 psi	Slow chipping with hydraulic hammer and/or blasting
Very Hard (R5)	Not scratched or fractured after many blows, hammer rebounds	>16,000 psi	Blasting

Groundwater and Soil Moisture

On July 27, 2018, observed soil moisture conditions within our test pit explorations were generally damp, grading moist at approximately 2-3 feet below the ground surface. Perched groundwater or seepage was not encountered during our site exploration. According to the *Estimated Depth to Groundwater in the Portland, Oregon Area, (United States Geological Survey, Snyder, 2018 website)*, groundwater may be present at an approximate depth of 15 to 25 feet below the ground surface. It is anticipated that groundwater conditions will vary depending on the season, local subsurface conditions, changes in site utilization, and other factors.

Infiltration Testing

Soil infiltration testing was performed using the open-hole method in hand auger HA-2. The approximate locations of the subsurface explorations are indicated on Figures 2 and 3. The test location was pre-saturated prior to testing. During testing the water level was measured to the nearest 0.01 foot (1/8 inch) from a fixed point, and the change in water level was recorded at regular intervals until three successive measurements showing a consistent infiltration rate were achieved.

Table 2 summarizes the results of our infiltration testing. Soils at the test location were observed and sampled in order to characterize the subsurface profile. Tested native soils classified as Lean CLAY (CL). The result of the infiltration testing indicates an infiltration rate that was not measurable in the field (0.0 inches per hour) from 0 to 5.5 feet below the ground surface. The measured rate for this test reflects both vertical and horizontal flow pathways. The infiltration results presented in Table 2 do not incorporate factors of safety.

Table 2 - Summary of Infiltration Test Results

Test Location	Depth (feet)	Soil Type	Infiltration Rate (inches/hr)	Hydraulic Head Range (inches)
HA-2	4.6	CL	0.0	12

Portable Dynamic Cone Penetrometer Testing

On July 27, 2018, GeoPacific Engineering conducted in place strength testing of native soils in two locations, indicated on Figure 2. A portable dynamic cone penetrometer was used to collect data for design of the pavement sections. Table 3 summarizes the results of our PDCP testing. PDCP testing data is attached to this report.

Table 3 - PDCP Field Test Results and Representative CBR Values

Field Test Designation	Material Tested	Depth Interval of Test (inches)	Average Penetration Per Blow (mm)	Correlated CBR Value
PDCP-1	Native SILT	5-45	7	34
PDCP-2	Native SILT	17-47	7	34

CONCLUSIONS AND RECOMMENDATIONS

Our site investigation indicated that the proposed construction is geotechnically feasible, provided that the recommendations of this report are incorporated into the design and construction phases of the project.

In our opinion, the primary geotechnical concern associated with construction at the site is the presence of residual soil and weathered rock. The residual soil exhibits negligible hydraulic conductivity. Based on results of our soil infiltration testing, soils at the subject site exhibited infiltration rates that were not measurable in the field. In our explorations, weathered rock was encountered between 3.8 to 4.2 feet below the ground surface. Generally, at least 5 feet of separation is recommended between infiltration facilities and rock. Based on the subsurface conditions encountered, subsurface infiltration of stormwater is not recommended for this site.

The second geotechnical concern associated with construction at the site is the potential for bedrock at shallow depths across the site. The Boring Lava Formation, which underlies the site, is known for rounded residual boulders, which could hamper excavations, such as for stormwater management facilities and utility trenching. The potential for encountering boulders should be anticipated. The following report sections provide recommendations for site development and construction in accordance with the current applicable codes and local standards of practice.

Site Preparation Recommendations

Areas of proposed construction should be cleared of vegetation, stockpiled soils, and any organic and inorganic debris. Inorganic debris and organic materials from clearing should be removed from the site. Organic-rich soils and root zones should then be stripped from construction areas of the site or where engineered fill is to be placed. Based upon our observations, the residual soil appears to be adequate for reuse as engineered fill provided the soil is adequately aerated to within 2 percent of optimum moisture during site grading.

The depth of stripping of organic soils and topsoil is estimated to be approximately 6 inches across the undeveloped portion of the site. However, depth of organic soil layers may increase in areas not explored. The southeast portion of the site, where the majority of new parking lot expansion is proposed, is densely forested, and deep stripping will likely be required in that area to remove organic material and topsoil. The final depth of soil removal will be determined on the basis of a site inspection after the stripping/excavation has been performed. Stripped topsoil should be removed from the site. Any remaining topsoil should be stockpiled only in designated areas and stripping operations should be observed and documented by the geotechnical engineer or his representative. Deeper stripping to remove large tree roots or other organics may be necessary in portions of the site. It is possible that portions of the soil containing medium to large roots, but not much other organic content, may be remediated by ripping/tilling, root-picking, and recompacting. Prior to placement of engineered fill, subgrade soils should be aerated and recompacted. If unstable soil is encountered in low-lying, high seasonal groundwater areas, crushed aggregate or cement amended stabilization may be necessary.

If encountered, undocumented fills and any subsurface structures (dry wells, basements, driveway and landscaping fill, old utility lines, septic leach fields, etc.) should be completely removed and the excavations backfilled with engineered fill.

Engineered Fill

We anticipate that onsite soils, consisting of SILT and Lean CLAY will largely be suitable for use as engineered fill. All grading for the proposed construction should be performed as engineered grading in accordance with the applicable building code at the time of construction with the exceptions and additions noted herein. Areas proposed for fill placement should be prepared as described in the site preparation section. Surface soils should then be scarified and recompacted prior to placement of structural fill. Proper test frequency and earthwork documentation usually requires daily observation and testing during stripping, rough grading, and placement of engineered fill. Imported fill material must be approved by the geotechnical engineer prior to being imported to the site. Oversize material greater than 12 inches in diameter should not be used in engineered fill.

Engineered fill should be compacted in horizontal lifts not exceeding 8 inches using standard compaction equipment. We recommend that engineered fill be compacted to at least 95 percent of the maximum dry density determined by ASTM D698 (Standard Proctor) or equivalent. Field density testing should conform to ASTM D2922 and D3017, or D1556. All engineered fill should be observed and tested by the project geotechnical engineer or his representative. Typically, one density test is performed for at least every 2 vertical feet of fill placed or every 500 yd^3, whichever requires more testing. Because testing is performed on an on-call basis, we recommend that the earthwork contractor be held contractually responsible for test scheduling and frequency. Site earthwork will be impacted by soil moisture and shallow groundwater conditions.

Excavating Conditions and Utility Trench Backfill

We anticipate that on-site soils can be excavated using conventional heavy equipment. Maintenance of safe working conditions, including temporary excavation stability, is the responsibility of the contractor. Actual slope inclinations at the time of construction should be determined based on safety requirements and actual soil and groundwater conditions. All temporary cuts in excess of 4 feet in height should be sloped in accordance with U.S. Occupational Safety and Health Administration (OSHA) regulations (29 CFR Part 1926), or be shored. The existing native soils classify as Type B Soil and temporary excavation side slope inclinations as steep as 1H:1V may be assumed for planning purposes. This cut slope inclination is applicable to excavations above the water table only.

Shallow, perched groundwater may be encountered during the wet weather season and should be anticipated in excavations and utility trenches. Vibrations created by traffic and construction equipment may cause some caving and raveling of excavation walls. In such an event, lateral support for the excavation walls should be provided by the contractor to prevent loss of ground support and possible distress to existing or previously constructed structural improvements.
PVC pipe should be installed in accordance with the procedures specified in ASTM D2321 and Oregon City standards. We recommend that structural trench backfill be compacted to at least 95 percent of the maximum dry density obtained by the Standard Proctor (ASTM D698) or equivalent. Initial backfill lift thicknesses for a ¾"-0 crushed aggregate base may need to be as great as 4 feet to reduce the risk of flattening underlying flexible pipe. Subsequent lift thickness should not exceed 1 foot. If imported granular fill material is used, then the lifts for large vibrating plate-compaction equipment (e.g. hoe compactor attachments) may be up to 2 feet, provided that proper compaction is being achieved and each lift is tested. Use of large vibrating compaction equipment should be carefully monitored near existing structures and improvements due to the potential for vibration-induced damage.

Adequate density testing should be performed during construction to verify that the recommended relative compaction is achieved. Typically, at least one density test is taken for every 4 vertical feet of backfill on each 200-lineal-foot section of trench.

Erosion Control Considerations

During our field exploration program, we did not observe soil conditions that would be considered highly susceptible to erosion. In our opinion, the primary concern regarding erosion potential will occur during construction in areas that have been stripped of vegetation. Erosion at the site during construction can be minimized by implementing the project erosion control plan, which should include judicious use of straw wattles, fiber rolls, and silt fences. If used, these erosion control devices should remain in place throughout site preparation and construction.

Erosion and sedimentation of exposed soils can also be minimized by quickly re-vegetating exposed areas of soil, and by staging construction such that large areas of the project site are not denuded and exposed at the same time. Areas of exposed soil requiring immediate and/or temporary protection against exposure should be covered with either mulch or erosion control netting/blankets. Areas of exposed soil requiring permanent stabilization should be seeded with an approved grass seed mixture, or hydroseeded with an approved seed-mulch-fertilizer mixture.

Wet Weather Earthwork

Soils underlying the site are likely to be moisture sensitive and may be difficult to handle or traverse with construction equipment during periods of wet weather. Earthwork is typically most economical when performed under dry weather conditions. Earthwork performed during the wet-weather season will probably require expensive measures such as cement treatment or imported granular material to compact areas where fill may be proposed to the recommended engineering specifications. If earthwork is to be performed or fill is to be placed in wet weather or under wet conditions when soil moisture content is difficult to control, the following recommendations should be incorporated into the contract specifications.

- Earthwork should be performed in small areas to minimize exposure to wet weather. Excavation or the removal of unsuitable soils should be followed promptly by the placement and compaction of clean engineered fill. The size and type of construction equipment used may have to be limited to prevent soil disturbance. Under some circumstances, it may be necessary to excavate soils with a backhoe to minimize subgrade disturbance caused by equipment traffic;
- The ground surface within the construction area should be graded to promote run-off of surface water and to prevent the ponding of water;
- Material used as engineered fill should consist of clean, granular soil containing less than 5 percent passing the No. 200 sieve. The fines should be non-plastic. Alternatively, cement treatment of on-site soils may be performed to facilitate wet weather placement;
- The ground surface within the construction area should be sealed by a smooth drum vibratory roller, or equivalent, and under no circumstances should be left uncompacted and exposed to moisture. Soils which become too wet for compaction should be removed and replaced with clean granular materials;
- Excavation and placement of fill should be observed by the geotechnical engineer to verify that all unsuitable materials are removed and suitable compaction and site drainage is achieved; and
- Geotextile silt fences, straw wattles, and fiber rolls should be strategically located to control erosion.

If cement or lime treatment is used to facilitate wet weather construction, GeoPacific should be contacted to provide additional recommendations and field monitoring.

Flexible Pavement Design – Private Parking and Drive Areas – 20 Year Design Life

We understand that development at the site will include construction of private parking and drive areas inside the project. Based on the results of PDCP testing, the subgrade exhibits an average CBR value of 31 in dry weather conditions. For the new private pavement sections we conservatively assume that the subgrade will exhibit a resilient modulus of at least 9,000, which correlates to a CBR value of 6. We assumed an anticipated 18-kip ESAL count of approximately 75,000 over 20 years, accounting for projected population growth. Our design considers 550 trips per day with 3 percent heavy trucks. If higher amounts of truck traffic are expected for the site, GeoPacific should be consulted to provided revised pavement design recommendations. Table 4 presents our flexible pavement design input parameters. Table 5 presents our recommended minimum dry-weather pavement section for the proposed roadway, supporting 20 years of vehicle traffic per Oregon City standards. Pavement design calculations are attached to this report.

Table 4 – Flexible Pavement Section Design Input Parameters for New Private Pavement Sections

Input Parameter	Design Value
18-kip ESAL Initial Performance Period (20 Years)	75,000
Initial Serviceability	4.2
Terminal Serviceability	2.5
Reliability Level	85 Percent
Overall Standard Deviation	0.5
Roadbed Soil Resilient Modulus (PSI)	9,000
Structural Number	**2.06**

Table 5 - Recommended Minimum Dry-Weather Pavement Section for New Private Pavement Sections

Material Layer	Private Pavement (inches)	Structural Coefficient	Compaction Standard
Asphaltic Concrete (AC)	3	.42	91%/ 92% of Rice Density AASHTO T-209
Crushed Aggregate Base ¾"-0 (leveling course)	2	.10	95% of Modified Proctor AASHTO T-180
Crushed Aggregate Base 1½"-0	8	.10	95% of Modified Proctor AASHTO T-180
Subgrade	12	9,000 PSI	95% of Standard Proctor AASHTO T-99 or equivalent
Total Calculated Structural Number		**2.26**	

The subgrade should be ripped or tilled to a depth of 12 inches, moisture conditioned, root-picked, and compacted in-place prior to the placement of crushed aggregate base for pavement. Any pockets of organic debris or loose fill encountered during ripping or tilling should be removed and replaced with engineered fill (see *Site Preparation Recommendations* section). In order to verify subgrade strength, we recommend proof-rolling directly on subgrade with a loaded dump truck during dry weather and on top of base course in wet weather. Soft areas that pump, rut, or weave should be stabilized prior to paving.

If pavement areas are to be constructed during wet weather, the subgrade and construction plan should be reviewed by the project geotechnical engineer at the time of construction so that condition specific recommendations can be provided. The moisture sensitive subgrade soils make the site a difficult wet weather construction project. General recommendations for wet weather pavement sections are provided below.

During placement of pavement section materials, density testing should be performed to verify compliance with project specifications. Generally, one subgrade, one base course, and one asphalt compaction test is performed for every 100 to 200 linear feet of paving.

<u>Wet Weather Construction Pavement Section</u>

This section presents our recommendations for wet weather pavement section and construction for new pavement sections at the project. These wet weather pavement section recommendations are intended for use in situations where it is not feasible to compact the subgrade soils to Oregon Cities requirements, due to wet subgrade soil conditions, and/or construction during wet weather.

Based on our site review, we recommend a wet weather section with a minimum subgrade deepening of 6 to 12 inches to accommodate a working subbase of additional 1½”-0 crushed rock. Geotextile fabric, Mirafi 500x or equivalent, should be placed on subgrade soils prior to placement of base rock.

In some instances it may be preferable to use a subbase material in combination with overexcavation and increasing the thickness of the rock section. GeoPacific should be consulted for additional recommendations regarding use of additional subbase in wet weather pavement sections if it is desired to pursue this alternative. Cement treatment of the subgrade may also be considered instead of overexcavation. For planning purposes, we anticipate that treatment of the onsite soils would involve mixing cement powder to approximately 6 percent cement content and a mixing depth on the order of 12 to 18 inches.

With implementation of the above recommendations, it is our opinion that the resulting pavement section will provide equivalent or greater structural strength than the dry weather pavement section currently planned. However, it should be noted that construction in wet weather is risky and the performance of pavement subgrades depend on a number of factors including the weather conditions, the contractor’s methods, and the amount of traffic the road is subjected to. There is a potential that soft spots may develop even with implementation of the wet weather provisions recommended in this letter. If soft spots in the subgrade are identified during roadway excavation, or develop prior to paving, the soft spots should be overexcavated and backfilled with additional crushed rock.

During subgrade excavation, care should be taken to avoid disturbing the subgrade soils. Removals should be performed using an excavator with a smooth-bladed bucket. Truck traffic should be limited until an adequate working surface has been established. We suggest that the crushed rock be spread using bulldozer equipment rather than dump trucks, to reduce the amount of traffic and potential disturbance of subgrade soils.

Care should be taken to avoid overcompaction of the base course materials, which could create pumping, unstable subgrade soil conditions. Heavy and/or vibratory compaction efforts should be applied with caution. Following placement and compaction of the crushed rock to project specifications (95 percent of Modified Proctor), a finish proof-roll should be performed before paving.

The above recommendations are subject to field verification. GeoPacific should be on-site during construction to verify subgrade strength and to take density tests on the engineered fill, base rock and asphaltic pavement materials.

Stormwater Management

We understand that it is desired to incorporate subsurface infiltration of stormwater into the design of stormwater management facilities. However, during our geotechnical investigation of the site, we observed infiltration rates that were negligible (0.0 inches per hour), and encountered weathered rock at relatively shallow depths across the site. In our explorations, weathered rock was encountered between 3.8 to 4.2 feet below the ground surface. Generally, at least 5 feet of separation is recommended between infiltration facilities and rock.

Based on the subsurface conditions encountered, subsurface infiltration of stormwater is not recommended for this site. Our opinion is based on low measured infiltration rates and the fact that the native soil layer overlying weathered rock is generally less than 5 feet thick.

Stormwater management systems should be constructed as specified by the designer and/or in accordance with jurisdictional design manuals. Stormwater exceeding storage capacities will need to be directed to a suitable surface discharge location, away from structures. Stormwater management systems may need to include overflow outlets, surface water control measures and/or be connected to the street storm drain system, if available. In no case should uncontrolled stormwater be allowed to flow over slopes.

Subsurface stormwater disposal systems have the potential to affect groundwater quality since they provide a more direct pathway to groundwater aquifers. Consequently, disposal systems should be constructed and maintained in accordance with Oregon Department of Environmental Quality (DEQ) requirements for groundwater protection. Systems receiving runoff from pavement areas should include water quality elements; such as oil traps, filters, or similar measures.

UNCERTAINTIES AND LIMITATIONS

Infiltration test methods and procedures attempt to simulate the as-built conditions of a planned subsurface disposal system. However, due to natural variations in soil properties, actual infiltration rates may vary from the measured and/or recommended design rates. Storm events in excess of the design event are possible, and systems should be constructed such that potential overflow is discharged in a controlled manner away from structures.

We have prepared this report for the owner and their consultants for use of this project only. This report should be provided in its entirety to prospective contractors for bidding and estimating purposes; however, the conclusions and interpretations presented in this report should not be construed as a warranty of the subsurface conditions. Experience has shown that soil and groundwater conditions can vary significantly over small distances. Inconsistent conditions can occur between explorations that may not be detected by a geotechnical study. If, during future site operations, subsurface conditions are encountered which vary appreciably from those described herein, GeoPacific should be notified for review of the recommendations of this report, and revision of such if necessary.

Sufficient geotechnical monitoring, testing and consultation should be provided during construction to confirm that the conditions encountered are consistent with those indicated by explorations. The checklist attached to this report outlines recommended geotechnical observations and testing for the project. Recommendations for design changes will be provided should conditions revealed during construction differ from those anticipated, and to verify that the geotechnical aspects of construction comply with the contract plans and specifications.

Within the limitations of scope, schedule and budget, GeoPacific attempted to execute these services in accordance with generally accepted professional principles and practices in the fields of geotechnical engineering and engineering geology at the time the report was prepared. No warranty, expressed or implied, is made. The scope of our work did not include environmental assessments or evaluations regarding the presence or absence of wetlands or hazardous or toxic substances in the soil, surface water, or groundwater at this site.

We appreciate this opportunity to be of service.

Sincerely,

GEOPACIFIC ENGINEERING, IN

REGISTERED PROFESSIONAL ENGINEER
90013PE
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MAY 22, 2015
BENJAMIN G. ANDERSON
Expires: 12/31/19

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REFERENCES

Gannet, Marshall W., and Caldwell, Rodney R., Generalized Geologic Map of the Willamette Lowland, U.S. Department of the interior, U.S. Geological Survey, 1998.

Oregon Department of Geology and Mineral Industries, Statewide Geohazards Viewer, www.oregongeology.org/hazvu.

Oregon Department of Geology and Mineral Industries, Madin, Ian P., Ma, Lina, and Niewendorp, Clark A., *Open-File Report 0-08-06, Preliminary Geologic Map of the Linnton 7.5' Quadrangle, Multnomah and Washington Counties*, Oregon, 2008.

United States Geological Survey, USGS Earthquake Hazards Program Website (earthquake.usgs.gov).

Snyder, D.T., 2008, Estimated Depth to Ground Water and Configuration of the Water Table in the Portland, Oregon Area: U.S. Geological Survey Scientific Investigations Report 2008–5059, 41 p., 3 plates.

Yeats, R.S., Graven, E.P., Werner, K.S., Goldfinger, C., and Popowski, T., 1996, Tectonics of the Willamette Valley, Oregon: in Assessing earthquake hazards and reducing risk in the Pacific Northwest, v. 1: U.S. Geological Survey Professional Paper 1560, P. 183-222, 5 plates, scale 1:100,000.

Web Soil Survey*,* Natural Resources Conservation Service, United States Department of Agriculture *2015 website.*

(http://websoilsurvey.nrcs.usda.gov/app/HomePage.htm.).

FIGURES

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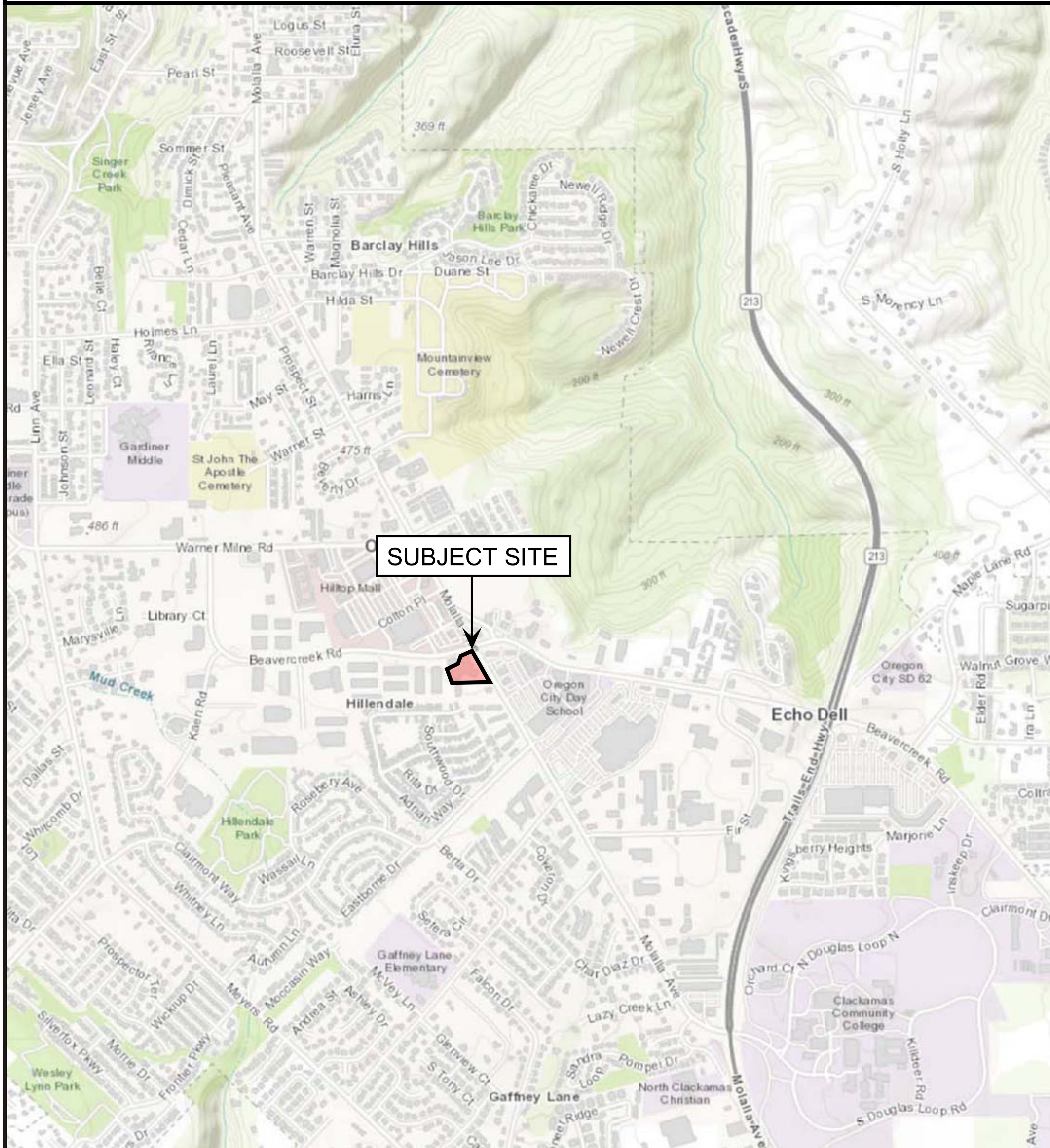
SITE VICINITY MAP



Base map: DOGAMI HAZVU Maps 2018
Date: 8/09/2018
Drawn by: TJT

1:24,000
0 0.13 0.25 0.5 mi
0 0.23 0.45 0.9 km

North

Project: Marquis Oregon City Parking Lot, Oregon City, Oregon 97045	Project No. 18-4984	FIGURE 1

GeoPacific Engineering, Inc.
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SITE AERIAL AND EXPLORATION LOCATIONS

Legend: Base Map Obtained From Google Earth 2018

HA-1 Hand Auger Boring Designation and Approximate Location

PDCP-1 PDCP Testing Designation and Approximate Location

APPROXIMATE SCALE (FEET)

40 0 40 80

Drawn by: TJT

Date: 8/09/2018

North

Project: Marquis Oregon City Parking Lot
1680 Molalla Avenue
Oregon City, Oregon 97045

Project No. 18-4984

FIGURE 2

GeoPacific Engineering, Inc.

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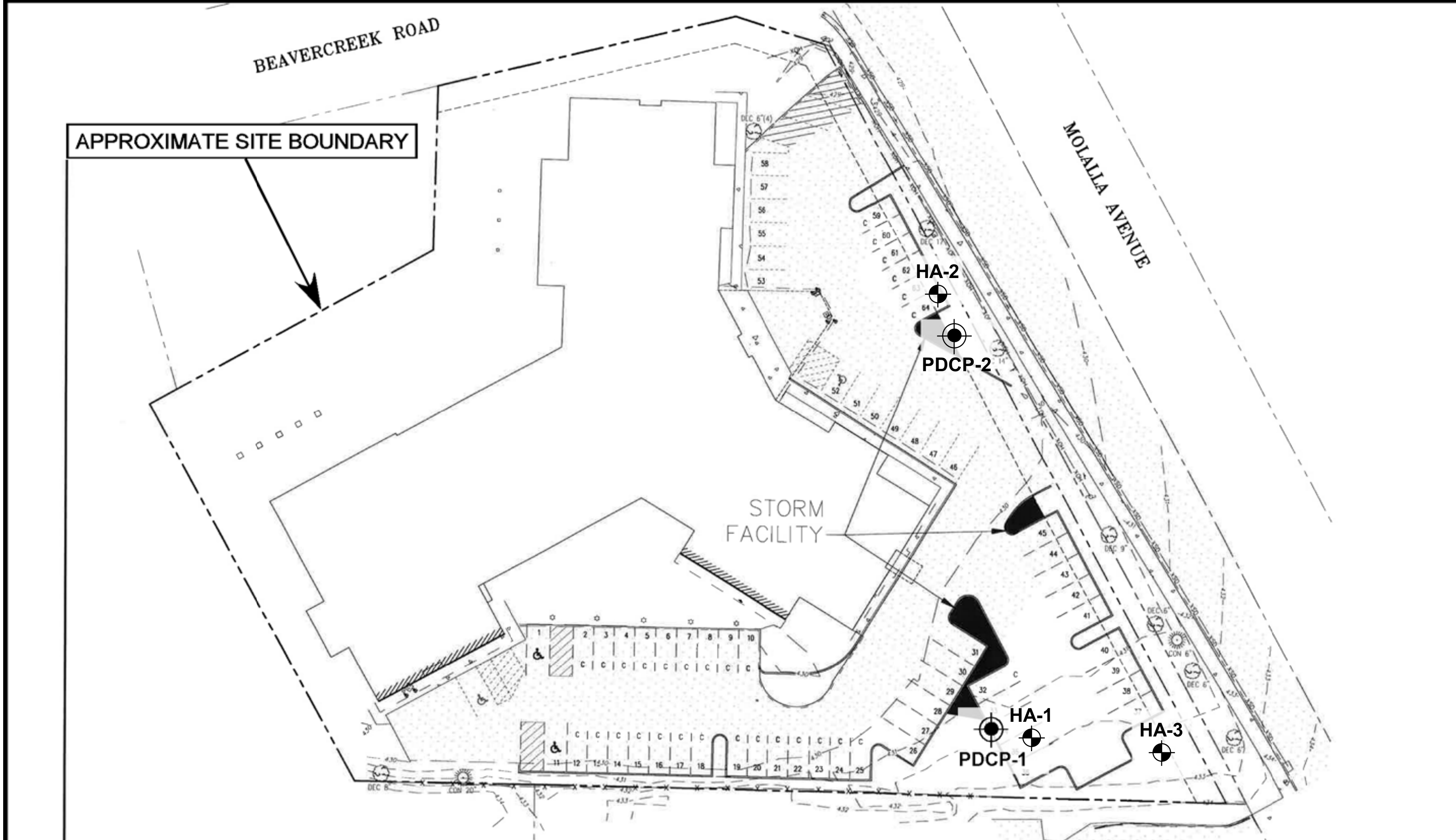
SITE PLAN AND EXPLORATION LOCATIONS



Legend: Base Map Provided By Emerio Designs

HA-1 Hand Auger Boring Designation and Approximate Location

PDCP-1 PDCP Designation and Approximate Location

APPROXIMATE SCALE (FEET)

30 0 30 60

Drawn by: TJT
Date: 8/09/2018

North

Project: Marquis Oregon City Parking Lot
1680 Molalla Avenue
Oregon City, Oregon 97045

Project No. 18-4984

FIGURE 3

EXPLORATION LOGS

GeoPacific Engineering, Inc.
14835 SW 72nd Avenue
Portland, Oregon 97224
Tel: (503) 598-8445 Fax: (503) 941-9281

HAND AUGER LOG

Project: Marquis Oregon City Parking Lot 1680 Molalla Avenue Oregon City, Oregon	Project No. 18-4984	Hand Auger No. **HA-1**

Depth (ft)	Pocket Penetrometer (tons/ft^2)	Sample Type	In-Situ Dry Density (lb/ft^3)	Moisture Content (%)	Water Bearing Zone	Material Description
0–0.8						SILT (ML), brown, medium stiff, moderately organic, trace fine and medium roots damp, surfaced with pine needles and leaves. (Topsoil).
0.8–4.0						Lean CLAY (CL), reddish brown, stiff to very stiff, with trace fragments of weathered rock, trace black and orange staining, damp. (Residual Soil). 1 inch thick layer of gray ash 1.5 feet bgs. Grades to moist and with more pronounced angular basalt at 2.2 feet bgs.
4.0–4.6						Highly weathered BASALT, light to dark gray, extremely soft (R0), reddish-brown matrix of silty clay, black staining, moist. (Boring Lava)
						Hand auger terminated at 4.6 feet bgs due to refusal on weathered rock. No seepage or groundwater encountered No caving observed

Depth scale: 1, 2, 3, 4, 5, 6, 7, 8

LEGEND

100 to 1,000 g — Bag Sample
5 Gal. Bucket — Bucket Sample
Shelby Tube Sample
Seepage
Water Bearing Zone
Water Level at Abandonment

Date Excavated: 07/27/2018

Logged By: TJT

Surface Elevation:

GeoPacific Engineering, Inc.
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Portland, Oregon 97224
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HAND AUGER LOG

Project: Marquis Oregon City Parking Lot, 1680 Molalla Avenue, Oregon City, Oregon	Project No. 18-4984	Hand Auger No. **HA-2**

Depth (ft)	Pocket Penetrometer (tons/ft²)	Sample Type	In-Situ Dry Density (lb/ft³)	Moisture Content (%)	Water Bearing Zone	Material Description
0–1.5						SILT (ML), brown, medium stiff, moderately organic, contains subrounded to angular gravel, with trace fine and medium roots, damp, 6 inches of topsoil developed on the surface, surfaced with grass. (Undocumented Fill).
1.5–4.3						Lean CLAY (CL), reddish brown, stiff to very stiff, with trace fragments of weathered rock, trace black and orange staining, damp. (Residual Soil). Grades to with more pronounced angular basalt at 2.0 feet bgs. Grades to moist at 3.5 feet bgs.
4.3–5.5						Highly weathered BASALT, light to dark gray, extremely soft (R0), reddish-brown matrix of silty clay, black staining, moist. (Boring Lava). Infiltration testing conducted at 4.6 feet bgs. Measured rate = 0.0 inches/hour

Hand auger terminated at 5,5 feet bgs due to refusal on weathered rock.
Infiltration testing conducted at 4.6 feet below the ground surface.
Measured Rate = 0.0 inches per hour.
No seepage or groundwater encountered
No caving observed

LEGEND: 100 to 1,000 g Bag Sample; 5 Gal. Bucket Bucket Sample; Shelby Tube Sample; Seepage; Water Bearing Zone; Water Level at Abandonment

Date Excavated: 07/27/2018

Logged By: TJT

Surface Elevation:

GeoPacific Engineering, Inc.
14835 SW 72nd Avenue
Portland, Oregon 97224
Tel: (503) 598-8445 Fax: (503) 941-9281

HAND AUGER LOG

Project: Marquis Oregon City Parking Lot 1680 Molalla Avenue Oregon City, Oregon	Project No. 18-4984	Hand Auger No. **HA-3**

Depth (ft)	Pocket Penetrometer (tons/ft^2)	Sample Type	In-Situ Dry Density (lb/ft^3)	Moisture Content (%)	Water Bearing Zone	Material Description
						SILT (ML), brown, medium stiff, moderately organic, trace fine and medium roots damp, surfaced with pine needles and leaves. (Topsoil).
1						Lean CLAY (CL), reddish brown, stiff to very stiff, with trace fragments of weathered rock, trace black and orange staining, damp. (Residual Soil). 1 inch thick layer of gray ash 1.5 feet bgs.
2						Grades to with more pronounced angular basalt at 2.0 feet bgs.
3						Grades to moist at 3.0 feet bgs.
4						Highly weathered BASALT, light to dark gray, extremely soft (R0), reddish-brown matrix of silty clay, black staining, moist. (Boring Lava).
						Hand auger terminated at 4.2 feet bgs due to refusal on residual soil. No seepage or groundwater encountered No caving observed
5						
6						
7						
8						

LEGEND

100 to 1,000 g — Bag Sample
5 Gal. Bucket — Bucket Sample
Shelby Tube Sample
Seepage
Water Bearing Zone
Water Level at Abandonment

Date Excavated: 07/27/2018

Logged By: TJT

Surface Elevation:

PDCP TESTING DATA

GeoPacific Engineering, Inc.
Real-World Geotechnical Solutions
Investigation, Design, Construction Support

14835 SW 72nd Avenue
Portland, Oregon 97224

Tel (503) 598-8445
Fax (503) 941-9281

Portable Dynamic Cone Penetrometer (PDCP) / California Bearing Ratio (CBR) Correlation

Project: Marquis Parking Lot **Date: 07.27.2018** **Existing A/C Thickness: none** **Test:** PDCP-1
Project No. 18-4984 **Engineer: TJT** **Existing Base Aggregate Thickness: none**
Location: South of Existing Parking Lot **Subgrade: Native Lean CLAY** **Notes:** Location on Figure 2

Length of shaft	Height (from ref) at start	Depth below ground at start	Length of shaft	Height (from ref) at start	Depth below ground at start
mm	mm	mm	in	in	in
1320	1320	110	52.0	48.0	4.3

Blows	Height(from ref) in	Height(from ref) mm	Depth (below ground) mm	Depth (inches below ground)	Depth (feet below ground)	mm/blow	CBR
1	4.92	125	125	4.92	0.41	15.00	14.1
5	6.10	155	155	6.10	0.51	6.00	39.3
5	6.89	175	175	6.89	0.57	4.00	61.8
5	8.07	205	205	8.07	0.67	6.00	39.3
5	8.86	225	225	8.86	0.74	4.00	61.8
10	10.83	275	275	10.83	0.90	5.00	48.1
10	12.80	325	325	12.80	1.07	5.00	48.1
10	16.34	415	415	16.34	1.36	9.00	24.9
10	20.87	530	530	20.87	1.74	11.50	18.9
10	24.80	630	630	24.80	2.07	10.00	22.2
10	28.15	715	715	28.15	2.35	8.50	26.6
10	30.71	780	780	30.71	2.56	6.50	35.9
10	32.48	825	825	32.48	2.71	4.50	54.2
10	34.65	880	880	34.65	2.89	5.50	43.3
10	37.01	940	940	37.01	3.08	6.00	39.3
10	39.76	1010	1010	39.76	3.31	7.00	33.0
10	41.93	1065	1065	41.93	3.49	5.50	43.3
10	44.49	1130	1130	44.49	3.71	6.50	35.9
					Average	6.97	38.3

Measurements are after each blow. Mm/blow is difference between previous and current blow

Portable Dynamic Cone Penetrometer (PDCP) / California Bearing Ratio (CBR) Correlation

Project: Marquis Parking Lot **Date: 07.27.2018** **Existing A/C Thickness: none** **Test:** PDCP-2
Project No. 18-4984 **Engineer: TJT** **Existing Base Aggregate Thickness: none**
Location: East of Existing Parking Lot **Subgrade: Native Lean CLAY** **Notes:** Location on Figure 2

Length of shaft	Height (from ref) at start	Depth below ground at start	Length of shaft	Height (from ref) at start	Depth below ground at start
mm	mm	mm	in	in	in
1320	1320	375	52.0	48.0	14.8

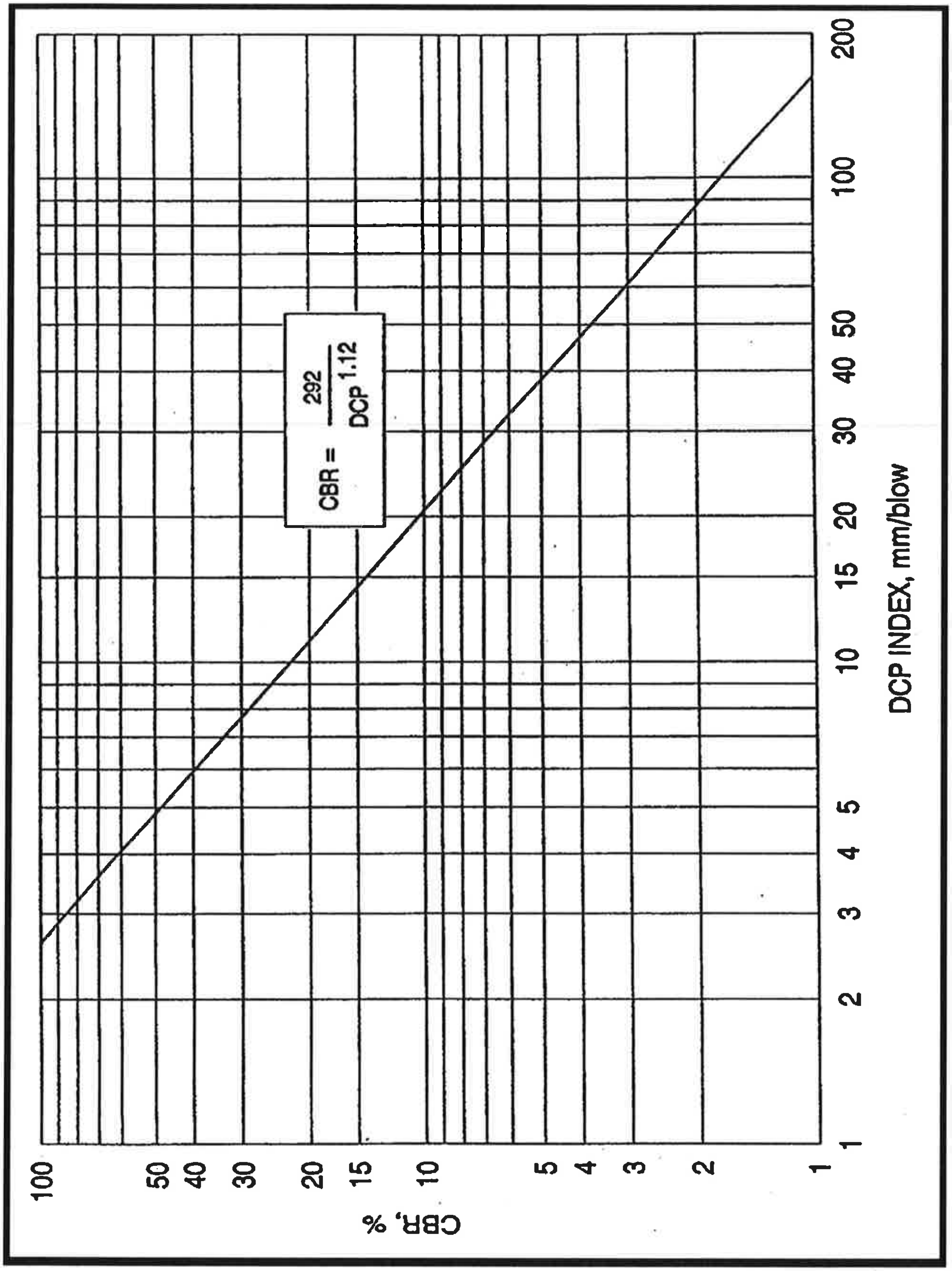
Blows	Height(from ref) in	Height(from ref) mm	Depth (below ground) mm	Depth (inches below ground)	Depth (feet below ground)	mm/blow	CBR
5	17.13	435	435	17.13	1.43	12.00	18.1
5	18.50	470	470	18.50	1.54	7.00	33.0
5	19.88	505	505	19.88	1.66	7.00	33.0
5	21.26	540	540	21.26	1.77	7.00	33.0
5	22.24	565	565	22.24	1.85	5.00	48.1
10	24.80	630	630	24.80	2.07	6.50	35.9
10	27.36	695	695	27.36	2.28	6.50	35.9
10	30.31	770	770	30.31	2.53	7.50	30.6
10	32.68	830	830	32.68	2.72	6.00	39.3
10	34.65	880	880	34.65	2.89	5.00	48.1
10	36.42	925	925	36.42	3.03	4.50	54.2
10	38.78	985	985	38.78	3.23	6.00	39.3
10	41.93	1065	1065	41.93	3.49	8.00	28.4
5	43.90	1115	1115	43.90	3.66	10.00	22.2
5	46.06	1170	1170	46.06	3.84	11.00	19.9
3	47.44	1205	1205	47.44	3.95	11.67	18.6
					Average	6.95	33.6

Measurements are after each blow. Mm/blow is difference between previous and current blow

DCP Index mm/blow	CBR %	DCP Index mm/blow	CBR %
<3	100	51	3.6
3	80	52	3.5
4	60	53-54	3.4
5	50	55	3.3
6	40	56-57	3.2
7	35	58	3.1
8	30	59-60	3.0
9	25	61-62	2.9
10-11	20	63-64	2.8
12	18	65-66	2.7
13	16	67-68	2.6
14	15	69-71	2.5
15	14	72-74	2.4
16	13	75-77	2.3
17	12	78-80	2.2
18-19	11	81-83	2.1
20-21	10	84-87	2.0
22-23	9	88-91	1.9
24-26	8	92-96	1.8
27-29	7	97-101	1.7
30-34	6	102-107	1.6
35-38	5	108-114	1.5
39	4.8	115-121	1.4
40	4.7	122-130	1.3
41	4.6	131-140	1.2
42	4.4	141-152	1.1
43	4.3	153-166	1.0
44	4.2	166-183	0.9
45	4.1	184-205	0.8
46	4.0	206-233	0.7
47	3.9	234-271	0.6
48	3.8	272-324	0.5
49-50	3.7	>324	<0.5

Figure 4: Tabulated Correlation of CBR versus DCP Index

(7)


CBR, %
100
50
40
30
20
15
10
5
4
3
2
1
DCP INDEX, mm/blow
1
2
3
4
5
10
15
20
30
40
50
100
200
CBR = 292 / DCP 1.12

PAVEMENT DESIGN CALCULATIONS

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              DARWin(tm) - Pavement Design

           A Proprietary AASHTOWARE(tm)
             Computer Software Product

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         Flexible Structural Design Module

          GeoPacific Engineering, Inc.
              14835 SW 72nd Avenue
               Portland, OR 97224
              Thomas J. Torkelson

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Project Description
   18-4984, Marquis Oregon City Parking, Private Parking and Drive Areas

Flexible Structural Design Module Data
   18-kip ESALs Over Initial Performance Period: 75,000
                        Initial Serviceability: 4.2
                       Terminal Serviceability: 2.5
                         Reliability Level (%): 85
                   Overall Standard Deviation: .5
        Roadbed Soil Resilient Modulus (PSI): 9,000
                           Stage Construction: 1

                 Calculated Structural Number: 2.06

Specified Layer Design
                                        Layer: 1
                         Material Description: AC
                   Structural Coefficient (Ai): .42
                     Drainage Coefficient (Mi): 1
                     Layer Thickness (Di) (in): 3.00
                           Calculated Layer SN: 1.26

                                        Layer: 2
                         Material Description: 3/4"-0
                   Structural Coefficient (Ai): .1
                     Drainage Coefficient (Mi): 1
                     Layer Thickness (Di) (in): 2.00
                           Calculated Layer SN: .20

                                        Layer: 3
                         Material Description: 1.5"-0
                   Structural Coefficient (Ai): .1
                     Drainage Coefficient (Mi): 1
                     Layer Thickness (Di) (in): 8.00
                           Calculated Layer SN: .80

                          Total Thickness (in): 13.00
                           Total Calculated SN: 2.26
```