EXHIBIT E

APPROVED
City of Florence
Community Development
Department
E
Exhibit
PC 21 22 DR 01
File Number

Branch ENGINEERING inc
Since 1977
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structural · geotechnical
SURVEYING

February 10, 2021

Northwest Housing Alternatives
13819 SE McLoughlin Blvd.
Milwaukie, Oregon 97402

RE: GEOTECHNICAL ENGINEERING INVESTIGATION
SHORE PINES HOUSING PROJECT
TAX MAP & LOT NO. 18-12-14-33 00500
FLORENCE, OREGON
BRANCH ENGINEERING INC. PROJECT NO. 19-132

Pursuant to your authorization Branch Engineering Inc. (BEI) performed a geotechnical engineering investigation at the subject site for the proposed development of apartment buildings on the subject site.

On January 7, 2021 six (6) exploratory test pits were advanced, using a rubber-tracked mini-excavator. Test pits were advanced to a maximum depth of 6-feet below ground surface (BGS). The subsurface soil conditions in the test pits were logged in accordance the USCS (Unified Soil Classification System) ASTM D2488. One (1) falling head infiltration test and two (2) portable dynamic cone penetrometer (DCP) field tests were performed. The accompanying report presents the results of our site research, field exploration and testing, data analysis, our conclusions and geotechnical engineering recommendations for the project. The site is suitable for the planned development, provided the recommendations of this report are implemented in the design and construction of the project.

Sincerely,
Branch Engineering Inc.

REGISTERED PROFESSIONAL ENGINEER
#16,170
Ronald J. Derrick
OREGON
SEPT 15, 1992
RONALD J. DERRICK
EXPIRES: 12/31/2021

Digitally signed by Ronald J. Derrick
Date: 2021.02.10 14:43:11 -08'00'

Ronald J. Derrick, P.E., G.E.
Principal Geotechnical Engineer

Matt Renner P.E.
Construction Engineer

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1.0 INTRODUCTION

The subject site is located in Florence, Oregon at the project coordinates of 44.000523° north and 124.100174° west. The site consists of one parcel of vacant land, tax lot 500, approximately 2.5-acres in size.

This report presents the results and findings of Branch Engineering, Inc. (BEI) field observations, testing, and research for the subject site. Our investigation included the evaluation of the subsurface conditions at the site to provide geotechnical recommendations for the design and construction of proposed multi-unit residential buildings and site improvements for access and parking.

1.1 Project and Site Description

Our understanding of the project is that two apartment buildings with associated site improvements for parking and utilities is proposed. Based on preliminary site drawings and documents provided to BEI we anticipate that there will approximately 68 total units comprised of 1-bedroom and 3-bedroom apartments. Access to the site will be taken from a driveway connection to Highway 101 on the western site perimeter.

At the time of our visit the site was covered in dense vegetation. Shore pines and spruce trees approximately 10-inches to 24-inches in diameter are present across the site with wax myrtle up to 8-inches in diameter in the understory. Dense understory vegetation includes rhododendrons, salal, and Oregon grape. To access the test pit locations rough pathways were cleared with the mini-excavator, shrubs and small understory trees were removed however an effort was made to insure no significantly sized trees were disturbed. The site topography is relatively flat, and difficult to assess in the field due to the existing vegetation however, an existing conditions survey by BEI indicates the site surface has approximately five-feet of elevation change, sloping slightly downhill from west to east.

The site is bordered by a Presbyterian Church to the north which allowed BEI to stage equipment and access the site for our investigation. Single-family residential development is present to the east of the site and separated from the site by an alley easement running north-south. A 42-inch diameter storm line flowing north to south is present in the easement. The southern property line is bordered by a residential property and Highway 101 runs north-south along the western boundary of the site. Areas of active sand dunes are present approximately 0.5-miles from the site to the northwest and the east southeast with infill development bordering Highway 101 which runs north-south in between the dune areas. The nearest water bodies are ponds within the Sandpines Golf Course approximately 0.4-miles west of the site and ponds adjacent to dunes approximately 0.4 miles southeast of the site. Munsel Lake is located approximately 0.5-miles to the northeast of the site.

1.2 Scope of Work

Our scope of work included a site reconnaissance and subsurface investigation on January 7, 2021. Six (6) exploratory test pits were advanced at the locations shown on the attached Figure-1 Site Exploration Map with the observed soil stratigraphy classified in accordance with the American Society of Testing and Materials (ASTM) Method D-2488.

A portable dynamic cone penetrometer (DCP) which consists of graduated steel rods driven into the soil by dropping a 35-lb slide hammer a vertical distance of 18-inches was used to assess the consistency of the site soil at select locations. Infiltration testing was performed at one location shown on the attached Figure-1. Shallow groundwater presence impacted planned additional tests.

Field log summaries of the site exploratory test pits, including field test results, are presented in Appendix A. Also included in Appendix A are copies of nearby well logs from the Oregon Department of Water Resources on-line database, and the soil survey mapping of the site.

1.3 Site Information Resources

The following site investigation activities were performed and literature resources were reviewed for pertinent site information:

- Review of the United States Department of the Interior Geological Survey (USGS) 2017 Mercer Lake, Oregon Quadrangle Map and the 2017 Florence, OR Quadrangle Map.
- Six (6) exploratory test pits were advanced to a maximum depth of 6-feet below ground surface (BGS) were performed on the site at the approximate locations shown on Figure-1. A single falling head infiltration test was performed at the location shown on Figure-1.
- Review of the Lane County area Web Soil Survey, United States Department of Agricultural (USDA) Natural Resources Conservation Service (NRCS), see Appendix A.
- Review of the USGS Geologic Map of Oregon, (USGS 1991, Walker & MacLeod).
- Review of Oregon Department of Water Resources Well Logs from nearby locations, see Appendix A.
- State of Oregon, Department of Geology and Mining Industries (DOGAMI) website, Statewide Geohazards Viewer (HazVu), http://www.oregongeology.org/hazvu/

2.0 SITE SUBSURFACE CONDITIONS

The analyses, conclusions and recommendations contained in this report are based on site conditions as they presently exist and assume the exploratory test pit excavations, presented in Appendix A, are representative of the subsurface conditions throughout the site. If, during construction, subsurface conditions differ from those encountered in the exploratory test pits; BEI requests that we be informed to review the site conditions and adjust our recommendations, if necessary.

2.1 Site Soils

The NRCS Web Soil Survey maps one soil unit across the majority of the site area, Yaquina loamy fine sand with Waldport fine sand, 0 to 12 percent slopes mapped along the western site boundary.

Both soil units are described as eolian sands of mixed origins, the Yaquina loamy sand is described as somewhat poorly drained fine sand found in dune slacks with the Waldport fine sand described as excessively drained.

In the exploratory test pits, we observed a relatively consistent soil profile across the site. Beneath a topsoil zone consisting of roots, organic litter, dark brown silt and sand, light brown, moist, medium dense, fine grain sand was observed. The depth of the topsoil zone ranged from approximately 8-inches to 12-inches BGS. The consistency of the sand changes from medium dense to loose at approximately 2-feet to 3.5-feet, or as groundwater was encountered. Blow counts recorded during DCP indicated a medium dense consistency of the soil in the upper 2.5-feet to 3.5-feet of the soil profile. As the excavation depths increased below approximately 4-feet BGS sidewall caving was frequently observed. The presence of ground water seepage likely exacerbated the caving once the depths of the excavations encountered groundwater seepage.

2.2 Ground Water

Ground water was observed in all of the exploratory test pits. The water was initially observed as sidewall seepage at depths ranging from 24-inches to 40-inches BGS. As excavations were advanced the seepage volume steadily increased and static water filled the excavations.

We expect that ground water levels, from the regional water table, will fluctuate with the seasons and should be expected to be highest during the late winter and spring months when rainstorms are more intense and frequent, and soils are near saturation. The database of well logs from the Oregon Water Resources Department was researched in the site vicinity. No well logs were found at, or adjacent to the site location however groundwater monitoring well log information from USGS work in 1959-1961 was found recorded in the adjacent 1/4 section of land, approximately 0.25 miles east of the site. The well logs indicated static water levels were recorded at approximately 8-feet BGS. Records attached to the well logs indicate the ground water levels were recorded periodically over a two-year period and seasonally fluctuated approximately 5-feet in depth.

Our site investigation followed periods of significant precipitation and the groundwater levels recorded are interpreted to be near the peak seasonal levels. Recommendations to prevent groundwater from being encountered in building pad excavations are presented in Section 5 below, however groundwater seepage should be expected in excavations exceeding 24-inches below the existing ground surface during the wet season. Dewatering measures for utility installation should be expected if construction during the wet season occurs and trench depths exceed 24-inches to 36-inches BGS and may be required for building pad excavations should the excavations encounter seepage.

The presence of ground water is not expected to adversely impact the proposed development, provided the recommendations of this report are implemented in the design and construction of the project. Dewatering measures may be required depending on the seasonal groundwater depth at the time of construction and the required depth of excavations per the project design.

3.0 GEOLOGIC SETTING

The 1991 Geologic map of Oregon by Walker and MacLeod maps the site geology as lacustrine and fluvial sedimentary rocks which is described as unconsolidated to semi consolidated lacustrine clay, silt, sand, and gravel. Areas of dune sand are mapped to the west, and east of the site. Review of the well logs drilled nearby the site to the east show sand logged to the maximum depth of the well at 81-feet with two 6-inch-thick layers of peat logged at 27-feet and 52.5-feet BGS.

The subject site is located near the northern extent of the longest coastal strip of dunes on the Oregon Coast. The dunes in the area were likely formed post ice-age during the Holocene epoch by eolian processes associated with the activity of wind. The typical pattern seen in the area is transverse dunes (running parallel to the ocean) caused by the varying on, and off shore winds. The subject site is underlain by Holocene-aged sedimentary deposits of unconsolidated to poorly consolidated fine-grained sands.

The site is located on the Oregon Coast, the entire Oregon Coast is located near the Cascadia Subduction Zone (CSZ), which is a zone of converging tectonic plates that historically produces major earthquake events. The DOGAMI HazVu website shows the subject site is expected to experience severe shaking in the event of a Cascadia Subduction Zone earthquake, and very strong shaking for lesser earthquakes, with a high hazard for earthquake-initiated soil liquefaction. A depiction of the historical Subduction Zone earthquake events is shown below.

Occurrence and Relative Size of Cascadia Subduction Zone Megathrust Earthquakes

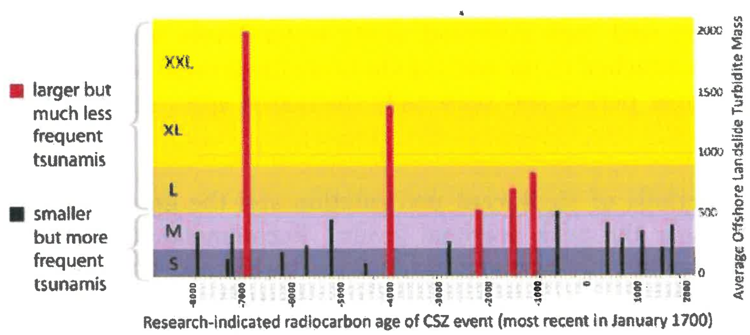

Figure 3: This chart depicts the timing, frequency, and magnitude of the last 19 great Cascadia Subduction Zone events over the past 10,000 years. The most recent event occurred on January 26, 1700. The 1700 event is considered to be a "medium sized" event. The data used to create this chart came from research that examined the many submarine landslides, known as "turbidites," that are triggered only by these great earthquakes (Witter and others, 2011). The loose correlation is "the bigger the turbidite, the bigger the earthquake."

3.1 Seismic Site Classification

Based on the soil properties encountered in our site pits and near-site well log information, Site Class D (Table 20.1-1 ASCE 7) is recommended for foundation bearing on the medium dense sand encountered in the test pits in the upper 18-inches of the soil profile, or engineered fill placed on the medium dense sand. If foundations are to be located below 18-inches of the existing ground surface Site Class E should be used for design based on the loose sand and groundwater underlying the upper soils. We do not recommend that foundations be placed below 24-inches BGS where the shallowest groundwater seepage was observed. Pursuant to the 2019 Oregon Structural Specialty Code the following potential geologic and seismic hazards are addressed.

- Slope Instability: The site topography is relatively flat, with no observable or mapped slopes present on the site or in a location that may impact the site. Our review of the online Department of Geologic and Mining Industries (DOGAMI) hazard viewer maps the risk of landslide hazard as low, the nearest mapped landslide is approximately 0.9-miles to the east of the site. The risk of landslides impacting the site is low.

- Liquefaction: The site soil consists of sand and the susceptibility to liquefaction and settlement exists if saturated at the time of a seismic event. The DOGAMI hazard viewer categorizes the risk of liquefaction on the site during seismic events as high. This is consistent with the high groundwater conditions we observed during our investigation however, we recognize that the mapped high risk of liquefaction extends beyond the site and surrounding area to include the majority of the City of Florence's Urban Growth Boundary (UGB). The existing risk of liquefaction is present for residential structures located in the greater area of the site and our recommendations in Section 5 of this report are intended to mitigate those risks to maximum extent practical by densification of the existing soil and raising site grades with engineered fill.

- Fault Rupture: There are no known active faults on the site, with mapped faults approximately 7.5-miles southwest and 8-miles east of the site and numerous fault zones mapped off the coastline. Only the off-shore faults are considered active based on a 1995 report by Geomatrix Consultants report. The risk of surface rupture is low.

- Lateral Spreading: Lateral spreading is a condition where underlying soils liquefy, thereby losing shear strength, and upper deposits migrate laterally, generally in a down slope direction. One of the conditions for lateral spread to occur is an abrupt change in topography that presents an unconfined surface for material to move in that direction. The existing site topography is flat and presently the risk of lateral spreading is low. If changes to the site surface elevations are made during site grading work the recommendations in this report are intended to mitigate the risk of lateral spreading.

- Tsunami: The risk of a large tsunami impacting the Oregon coast in the event of CSZ earthquake is high. The site is mapped as being outside the tsunami inundation line; however, this level is only an estimate and erosion of lower ground may cause instability of ground at higher elevations. The risk of tsunami impact to the site is estimated to be moderate.

4.0 CONCLUSIONS

Based on our field observations, subsurface explorations, and data analyses, we conclude that the site is geologic and geotechnically suitable for the proposed development provided that the recommendations of this report are incorporated into the design and construction of the project. Our investigation did reveal the presence of groundwater at relatively shallow depth and recommendations to mitigate the potential impact to the proposed development from the groundwater are presented in the following sections of this report. We conclude that no further geotechnical analysis is required on the subject site for the proposed site improvements.

5.0 RECOMMENDATIONS

The following sections present site-specific recommendations and design parameters for site preparation, drainage, foundations, utility excavations, and slab/pavement design. General material and construction specifications for the items discussed herein are provided in Appendix B.

The subsurface conditions observed in our site investigation are consistent; however, our field explorations only represent a very small portion of the site. Should loose or unsuitable soils extend to a depth greater than that described herein, or areas of distinct soil variation be discovered, this office shall be notified to perform site observation and additional excavation may be required.

5.1 Site Preparation, Grading, and Foundation Subgrade Requirements

The following recommendations are for earthwork in the building foundation areas, roadways, and parking areas. Earthwork shall be performed in general accordance with the standard of practice as described in Appendix J of the 2019 Oregon Structural Specialty Code and as specified in this report.

All areas intended to directly or laterally support structures or pavement areas shall be stripped of vegetation, organic soil, unsuitable fill, and/or other deleterious material. These stripping's shall be removed from the site or reserved for use in landscaping or non-structural areas. The depth to subgrade is expected to be approximately 12-inches BGS unless deeper due to tree removal. Once subgrade is exposed, expected to be medium dense sand, the recommended subgrade preparation is as follows:

Foundation Subgrade Preparation
In areas of foundation footings, organic topsoil and loose sand shall be removed to consistently medium dense sand either for the placement of foundation forms or structural fill. Upon excavation to suitable subgrade, expected at approximately 12-inches BGS, the subgrade shall be wetted and rolled with a vibratory smooth drum roller until no visual settlement of the subgrade is detected. Conventional strip and spread footings may be used for the foundation system of the proposed structure.

We recommend that the subgrade be covered with a minimum of 4-inches of compacted aggregate to mitigate wind and water erosion. After construction of footings, the perimeter of the footings shall be protected from erosion to mitigate undermining of footings. If structural fill is used to raise subgrade elevations, the fill shall conform to the recommendations in Sections 5.2 below.

Pavement Subgrade Preparation
In areas of pavement for vehicle access or parking, we recommend that the existing vegetation, topsoil, and areas of loose soil be removed to consistent subgrade material as described above. The expected depth of excavation to the subgrade material described above is approximately 12-inches. Upon excavation to suitable subgrade, the subgrade shall be wetted and rolled with a vibratory smooth drum roller until no additional visual settlement of the subgrade is detected. Fill placed to raise pavement subgrade elevations shall be placed on suitable subgrade, and conform to the recommendations below.

We recommend that a minimum of 6-inches of compacted aggregate be placed on the subgrade in light vehicle pavement areas. Heavy construction traffic will require additional aggregate thickness, a minimum of 12-inches, to mitigate rutting of the subgrade.

During subgrade excavation in foundation and pavement areas we recommend the Geotechnical Engineer of Record, or designated representative visit the site to observe the subgrade material prior to placement of structural fill or aggregate.

Site Grading

During our site investigation on January 7, 2021, we observed a seasonally high ground water table. During the site development design, it may be advantageous to raise the site surface elevations for ensuring the foundation footings are kept above the levels of the observed groundwater seepage. Raising site elevations may also be useful for stormwater drainage design and treatment. The medium dense sand underlying the existing topsoil zone is a suitable base for the placement of fill provided any areas of loose sand are compacted prior to fill placement. Locally available, clean fine grain sand is suitable for use as engineered fill provided the recommendations in Section 5.2 below are incorporated into the fill placement.

Fill slopes constructed from sand shall have a maximum slope angle of 3:1 (H:V) and shall extend horizontally outward from foundations per the setbacks described below in Section 5.5. If fill slopes are constructed from compacted aggregate, a maximum slope of 2:1 is acceptable. Sand material placed as fill shall be protected from erosion by covering the sand with compacted aggregate 4-inches in thickness, establishing vegetation on the slopes, or other means.

5.2 Engineered Fill Recommendations

All engineered fill placed on the site shall consist of homogenous material and shall meet the following recommendations. Clean, native sand is suitable for use as structural fill material.

- Areas of structural fill placement shall be stripped of organic material, loose soil, and subgrade approved by the Geotechnical Engineer prior to the placement of fill materials. Sloped areas in excess of 20% shall be properly keyed and benched horizontally into competent material as the fill height progresses. Proof-rolling or hand-probing of the subgrade may be required to assess competence.

- Prior to placement, fill material shall be approved by the Geotechnical Engineer. Acceptable fill shall be free of organics or other deleterious materials. The sand present on the site and site vicinity is acceptable for use as engineered fill upon removal of any organic material.

- The fill shall be moisture conditioned within +/-3% of optimum moisture content and compacted in lifts with loose lift thickness not exceeding 8- inches with appropriate equipment for the fill material.

- Periodic visits to the site to verify lift thickness, source material, and compaction efforts shall be conducted by the Geotechnical Engineer or designated representative and documented.

- The recommended compaction level for engineered fill is 90% of ASHTO T-180/ASTM 1557-D (modified Proctor) unless otherwise specified. Compaction shall be measured by testing with nuclear densometer ASTM D-6938, or D-1556 sand cone method. If compaction testing by nuclear densometer is not possible due to the nature of the approved fill material, proof rolling with a fully loaded 10 CY dump truck observed by the Geotechnical Engineer or designated representative shall be conducted.

5.3 Lateral Earth Pressures and Friction Coefficient

We are not aware of any retaining wall structures proposed are part of the site development, the following equivalent fluid pressure parameters may be used for design of site retaining structures that are free draining with no hydrostatic pressures.

Table-1 Lateral Earth Pressures

Material	Passive Earth Pressure (Kp)[1]	Active Earth Pressure (Ka)	At-Rest Earth Pressure (Ko)[2]
Sand (Level Backfill)	325 pcf	30 pcf	45 pcf
Sand (2:1 Backfill Slope)	325 pcf	40 pcf	55 pcf

1 – Neglect upper foot of material unless covered by foundation footing or pavement.
2 – For walls restrained at the top from movement
For seismic design increase earth pressure by 0.7 of the peak ground acceleration (PGA) and apply at 0.4H above the base of the wall, where H is the wall height.

The coefficient of friction for concrete poured neat against undisturbed or compacted sand subgrade is 0.4, a coefficient of 0.5 may be used for concrete poured on a minimum of 12-inches of compacted aggregate.

5.4 Drainage and Infiltration Testing

An on-site storm drainage system is expected to be engineered for this project. Alteration of existing grades for this project will likely change drainage patterns but should not adversely affect adjacent properties. We recommend that areas of structural fill be evaluated to ensure proper drainage away from structures is maintained. Accumulation of drainage near structural fills may result in saturation and softening of material. Final perimeter landscape grades shall slope away from the foundation and surface water shall not be allowed to pond adjacent to foundations.

As part of our site investigation a falling head infiltration test was performed at the location shown on the attached Figure-1. Additional infiltration tests were planned at depths of 3-feet to 4-feet BGS however groundwater seepage which led to static water in the excavations was encountered. The infiltration test that was performed was at a depth of approximately 28-inches BGS, above the depth where groundwater seepage was encountered in the adjacent test pit excavation. The infiltration test results are attached in Appendix A with the average infiltration rates exceeding 60-inches per hour.

5.5 Soil Bearing Capacity

Based on our site observations and review of proposed building plans, conventional spread footings are suitable for the proposed site development provided the building pad area preparation is in conformance with the recommendations described above in Section 5.1.

The allowable bearing capacity for foundation elements placed on compacted sand subgrade or prepared structural fill is 2,000 psf. The allowable bearing capacity may be increased by 1/3 for short-term loading such as wind and seismic. Additionally, structural fill should extend laterally, from all foundation edges, a minimum distance or 2-feet or within a 1:1 plane from at least 1-foot outside the edge of footing. Perimeter landscape grades shall be sloped away from all foundations and water should not be allowed to pond within 10-feet of footings.

The following recommendations shall be implemented in the design and construction of the project. Periodic site observations by a geotechnical representative of Branch Engineering, Inc. are recommended during the construction of the project. The specific phases of construction that should be observed are:

Table 2:

Recommended Construction Phases to be Observed by the Geotechnical Engineer	
Phase	*Observation*
At completion of street excavation	Subgrade observation by the geotechnical engineer before fabric and aggregate placement.
Imported fill material	Observation of material or information on material type and source.
Placement or compaction of fill material	Observation by geotechnical engineer or test results by qualified testing agency.

5.6 Settlement

The maximum building foundation loads are estimated to be 1.5 kip/linear foot for wall loads and/or 3 kips for column loads. Site-specific consolidation testing was not performed; however, based on soil observations and test results in similar soil conditions, the estimated total settlement at the site is not expected to exceed 0.75-inches with a differential settlement up to 0.5-inches over a span of 20 feet. The settlement estimates are based on the building load effects and area expected to occur over a short-term, generally by the time construction is completed.

These settlement estimates do not account for seismic induced settlement, which may be as much as 2+ inches, but is expected to be relatively uniform across a building footprint. Foundations should be placed a minimum distance from each other to prevent overlapping of stress distributions defined as a 1:1 (H:V) slope projection from all foundation edges to a minimum depth of two (2) times the foundation width of the largest footing.

5.7 Slabs-On-Grade

After site preparation to expose suitable subgrade prepared in accordance with Section 5.1, load bearing concrete slabs shall be underlain by a compacted sand subgrade or leveling course of compacted, crushed aggregate, if necessary. A modulus of subgrade reaction of 150 pci may be used for design of slabs on approved native subgrade material or structural fill. Non-load bearing slabs or pavements do not require geotechnical design criteria; however, BEI recommends a stable subgrade to mitigate un-controlled cracks.

The edges of slabs shall be protected from erosion and undermining of the slab; a vapor barrier system shall be selected by the project architect and may be dependent on slab cover materials.

5.8 Pavement Design Recommendations

Our recommendations for any parking or driveway improvements used a CBR of 8 and the guidance of the 1993 AASHTO Guide for Design of Pavement Structures and 2003 revised Asphalt Pavement Design Guide, published by the Asphalt Pavement Association of Oregon.

For new AC pavement installation in parking areas and light vehicle routes, we recommend a minimum pavement thickness of 3-inches of AC over a minimum of 6-inches of compacted base rock. We recommend that the AC thickness be increased to 4-inches in areas of heavier traffic, such as refuse truck routes or delivery vehicles.

Prior to placement of base rock any soft soil, wet soil, or organic soil shall be removed from the pavement subgrade. The geotechnical engineer of record, or designated representative should visit the site to approve the subgrade soil prior to the placement of the base rock. Proof rolls with a loaded 10 CY haul truck shall be observed on the compacted base rock prior to pavement installation and any areas of deflection under wheel loads shall be corrected by over-excavation replacing subgrade material with additional compacted aggregate.

The base rock shall be compacted to at least 90% relative compaction as determined by ASTM 1557/AASHTO T-180 (modified Proctor). The base rock shall be tested to measure compliance with this compaction standard prior to placement of asphalt concrete.

The above recommended structural pavement sections are designed for the type of vehicle use on the site after construction completion, not for construction vehicle traffic which is generally heavier, occurs over a short time, and impacts the site before full pavement sections are constructed. The construction traffic may cause subgrade failures and the site contractor should consider over-building designated haul routes through the site to mitigate soft areas at the time of final paving.

5.9 Wet Weather/Dry Weather Construction Practices

The site material is sand to depths over 50-feet and is relatively free-draining. Precipitation will not adversely impact site earthwork; however, perched water during the wet season may impact site trenching activities and cause “pumping” of the subgrade with repeated heavy vehicle traffic. Dewatering and/or shoring of excavation sidewalls is expected to be required for excavations exceeding 3-feet BGS of the existing ground surface. Construction traffic routes should have a minimum of 12-inches of aggregate, with preferably 3-inch minus angular aggregate in the lower 8-inches of the temporary road section to mitigate subgrade degradation during wet weather conditions. Final design pavement sections and foundation subgrade recommendations do not account for repeated heavy truck traffic associated with construction.

6.0 REPORT LIMITATIONS

This report has presented BEI's site observations and research, subsurface explorations, geotechnical engineering analyses, and recommendations for the proposed site development. The conclusions in this report are based on the conditions described in this report and are intended for the exclusive use of Northwest Housing Alternatives and their representatives for use in design and construction of the development described herein. The analysis and recommendations may not be suitable for other structures or purposes.

Services performed by the geotechnical engineer for this project have been conducted with the level of care and skill exercised by other current geotechnical professionals in this area. No warranty is herein expressed or implied. The conclusions in this report are based on the site conditions as they currently exist and it is assumed that the limited site locations that were physically investigated generally represent the subsurface conditions at the site. Should site development or site conditions change, or if a substantial amount of time goes by between our site investigation and site development, we reserve the right to review this report for its applicability. If you have any questions regarding the contents of this report please contact our office.