PINNACLE
ENGINEERING, INC.
GEOTECHNICAL · STRUCTURAL · CIVIL
QUALITY DESIGNS - ENGINEERED TO LAST

February 18, 2025

Vine Maple Design
1130 Baltimore Ave, Ste A 86
Bandon, OR 97411

Re: Residential Geotechnical Study Report
1190 Beach Loop Rd, Bandon, OR 97411
Project # 30725

Dear Mr. Reilly,

A. Scope

At your request, representatives of Pinnacle Engineering, Inc. (PEI) and Western Testing, LLC (WTL) conducted a record search, and then visited the above referenced lot owned by Julia A Johnson Revocable Trust, on January 17, 2025. The purpose of our site visit was to sample the soil beneath the foundation of the proposed structure to 10 feet below the ground surface or practical refusal. Site exploration and laboratory testing were conducted to provide a basis for geotechnical recommendations for site development and bearing capacity.

This report was prepared and is consistent with standard geologic practices and contains applicable provisions of "Guideline for Preparing Engineering Geologic Reports", 5/30/2014. The report is valid for a period of five years from the date of preparation. No extensions to this timeline shall be granted. All of the applicable content requirements of subsection 17.78.040 of the City of Bandon Municipal Code have been addressed in the report or are not applicable to the review.

B. Prior Geotechnical Report

No prior geotechnical reports have been prepared for the subject site. PEI has completed several geotechnical reports in the area.

C. Site Geology and Geotechnical Characterization

C.1. Project Area Geology

The site is located within the Oregon Coast Range Geological province. The surface geology has been mapped as upland coastal dune deposits of Holocene and Upper Pleistocene over coastal marine terraces.[1]

Qds	**Upland coastal dune deposits (Holocene and upper Pleistocene):** *Unconsolidated, well-sorted, fine- to medium-grained sand deposited by the wind in upland coastal dune fields. Dune fields are situated on top of coastal marine terraces and cover a large portion of the coast between Port Orford and Bandon.*

Open-File Report O-14-01, Geologic map of the southern Oregon coast between Port Orford and Bandon, Curry and Coos Counties, Oregon, 2014, Thomas J. Wiley, Jason D. McClaughry, Lina Ma, Katherine A. Mickelson, Clark A. Niewendorp, Laura L. Stimely, Heather H. Herinckx, and Jonathan Rivas

C.2. Seismicity and Seismotectonic Considerations

C.2.a. Area and Site Seismicity

Extensive seismotectonic studies continuing since 1990 have concluded that western Oregon is subject to a much greater probability of both random and plate-subduction seismic events of far greater magnitude and far more frequently than was historically believed.

- Regionally, the Cascadia Subduction Zone is considered as a feasible source of Magnitude 7.75, or greater, earthquakes.
- Intraplate earthquakes, focused at a relatively great depth within the Juan de Fuca plate subduction beneath western Oregon and Washington, are capable of producing magnitude 7.0 earthquakes. Deep focus intraplate earthquakes are theoretically possible, but considered rare in Oregon.
- Relatively shallow crustal earthquakes are more likely, with an upper bound considered to be on the order of Magnitude 6.0.
- The design spectral response acceleration for the project area are as follows:

$S_S = 2.041$ g	$S_{MS} = 2.041$ g	$S_{DS} = 1.361$ g
$S_1 = 0.972$ g	$S_{M1} = 1.652$ g	$S_{D1} = 1.102$ g

C.2.b. Site Stability

Beneath a thin root zone, the site is generally underlain by brown to orange poorly graded sand fill material that overlies native light brown to orange silty sand at depths ranging from three to four feet. Native surface soil depths are typically greater than ten feet.

The project area is considered susceptible to an Extra Extra Large *tsunam*i caused by a Cascadia Subduction Zone Megathrust Earthquake.

The project area is not considered susceptible to *seiche*.

1 Geologic Map of the Southern Oregon Coast Between Port Orford and Bandon, Curry and Coos Counties, Oregon, 2014, Thomas J. Wiley, Jason D. McClaughry, Lina Ma, Katherine A. Mickelson, Clark A. Niewendorp, Laura L. Stimely, Heather H. Herinckx, and Jonathan Rivas, Oregon department of Geology and Mineral Industries, Open-File Report O-14-01.

Liquefaction occurs when saturated deposits of loose, cohesionless, fine grained soils – generally sands and sand-silt mixtures – are subjected to strong seismic shaking. If these deposits are saturated and cannot drain rapidly, the pore water pressure will increase. With increased oscillation, the pore water pressure can increase to equal the value of the overburden pressure. The shear strength of a cohesionless soil is directly proportional to the effective stress, which is equal to the difference between the overburden pressure and the pore water pressure. Therefore, when the pore water pressure increases to approach the value of the overburden pressure, the shear strength of the soil decreases to zero, and the soil deposits morph into a liquid state.

As a result of the shallow depth to groundwater, grain size and consistency of the soil encountered in our sub surface explorations, the site material has a high susceptibility to liquefaction during a major seismic event. We recommend that there is a moderate hazard of earthquake-induced liquefaction subsidence at the project site. Recommended Mitigation Includes: building foundation systems consisting of a reinforced structural fill section.

The DOGAMI Hazard Viewer shows Moderate landslide hazard in the immediate area. However, there are no mapped landslides near the site that could affect it. Slope gradients of the western facing slope are estimated to range between 1.3H:1V – 1.45H:1V and are heavily vegetated with shrubs and grasses. Therefore, it is our opinion that the site is not at risk for slope instability.

C.2.c. Site Classification

Soils underlying the site are consistent with Site Class D, as defined by the current edition of the Oregon Structural Specialty Code (OSSC).

C.2.d. Seismic Refraction Survey

A seismic refraction survey was neither requested by our client nor conducted for this investigation. Qualitatively;

- The surface layer of poorly graded SAND can be expected to transmit lateral accelerations typical of lower velocity range of 600 to 800 ft/sec.
- Underlying the surface soils the silty sand can be expected to transmit lateral accelerations typical of lower velocity range of 600 to 800 ft/sec.

D. FIELD STUDIES

D.1. Surface Reconnaissance

Contemporaneous with the geotechnical site characterization, a surface reconnaissance was conducted. The surface reconnaissance concluded that there were no observable site defects that would compromise viability of the site for the planned use.

D.2. Surface Hydrology

The subject site is located on a westerly trending hillside above Tupper Creek in Bandon. A flat bench made level with loose, sand FILL that extends approximately 135 feet from Beach Loop Drive SW terminating at the top of a west-facing slope that descends approximately 80 feet to the beach. The moderate slopes can facilitate the relatively rapid runoff of surface waters during rainfall events.

The shallow natural SAND layers are relatively free draining and allow for percolation. Sub-surface rock layers beneath the surface soils are estimated to transmit a moderate amount of water during wet months.

Post development, the surface water runoff will be conveyed via gutters, ditches and storm drains then, ultimately, Tupper Creek.

D.3. Field Observations

Field observations included soil description, classification, qualitative density measurement, measurements of thicknesses of the various soil horizons, and depth to or presence of groundwater.

D.4. Site Exploration and Field Testing

Field investigations conducted on January 17, 2025 included geologic reconnaissance of the site and immediate surrounding area, and observation, sampling and testing in conformance to ASTM D-2488 of the underlying soils encountered in three test pits.

Test pits were excavated with a Takeuchi 235 excavator with 18" bucket at the locations depicted on Figure 2. The test pits were observed, logged and samples retrieved by a certified technician. The summary logs of test pits are contained in Appendix A

Samples were retrieved at visible soil horizon changes. Most of the samples were obtained using a Modified California Barrel advanced by hand driving, which produces a measure of soil density while recovering moderately disturbed samples for strength and performance testing. Bulk samples were also retrieved at the depths and locations indicated on the test pit logs

In addition to basic field soil classification tests, *in situ* field density tests were conducted on natural site soils.

The test pits were left unfilled for a brief time to allow groundwater levels to stabilize if present. Groundwater was encountered at TP1 at a depth of 8.5 feet and TP3 at a depth of 10.5 feet during the field exploration. Groundwater was not encountered at TP2.

Please note that shear strengths and estimated bearing capacities, if noted on the field logs are field estimates of ultimate values, recorded for correlation of laboratory results and are only provided for comparative purposes. They should not be used for design. <u>We should be contacted before utilization of values other than those recommended in Section F to confirm applicability and that the designer's interpretation is consistent with our understanding of design properties.</u>

D.5. Geotechnical Characterization

Soil descriptions and layer interfaces are interpreted from observations on site. While the layers are shown as having distinct boundaries in field logs, in reality, the change is gradual.

The native surface soil is typically a coarse-grained material to a depth greater than ten feet. The surface soils are characterized as being loose to dense, silty sand, light brown to orange, and low to non-plastic.

A portion of the site in the area of TP 2 and TP 3 includes an additional three to four feet thick section of fill material placed over the native surface. This fill material is characterized as being loose, brown to orange, poorly graded sand. The soils at TP1 indicate that the site is relatively undisturbed at that location. It appears that portions of the site were previously cut with the fill material placed on the southern half of the lot.

Hard rock was not directly encountered during site exploration; however, based on nearby well and geotechnical logs it is estimated that hard rock will likely be encountered within 25 feet of the surface in the project area.

The site soils are compactible after the removal of the vegetative component and may be used as site fills if construction occurs during dry weather. Site soils should not be used as fills beneath foundations. The vegetative component is suitable for use as landscaping material or for sculpting wetlands mitigation areas.

The site soils can be excavated with light to moderate effort by moderate energy excavation equipment. Bedrock is not likely to be encountered during foundation excavation. Where rock is encountered, it is likely to be weathered for the first three to five feet and can be ripped with a high energy excavator. Competent bedrock will require blasting or intensive air hammering for removal.

D.6. Groundwater

Groundwater (the phreatic surface) in the form of seepage was encountered at a depth of 8.5 feet at TP1 and 10.5 feet at TP3 during this exploration. It is likely that the phreatic surface will fluctuate both seasonally and during the typical five year hydrologic cycle. Considering annual precipitation records during the past several years, the absence of measurable changes in the ground water surface should not be regarded as evidence that higher groundwater conditions will not occur in the future. Experience indicates that the phreatic surface will vary seasonally by approximately five feet and will vary by approximately ten feet between hydrologic extremes, an average ten year period. We project that the average high groundwater elevation will be greater than 4 feet below the finished surface. Seepage, occasionally in considerable amounts, should be expected at the transitional zone between the residual soils and the underlying transitional bedrock.

D.7. Soil Permeability

Permeability tests were not performed for this study. Qualitatively, flow velocities within the proposed structural fill soil can be expected to range between 10^{-4} and 10^{-5} cm/sec and as high as 10^{-2} cm/sec at the bedrock interface where fine grained soils transition to weathered formational material. Where sandy or fractured layers exist, their permeability will be on the order of 10^{-3} cm/sec.

E. LABORATORY TESTING

All of the samples recovered during the site exploration were visually reexamined at our Roseburg laboratory to verify the field descriptions. To assist in soil classification and assessing long term stability of the site soils and physical characteristics, including bearing capacity, natural moisture/density relationship, and plasticity indices. Samples were then classified in conformance with the Unified Soil Classification System (USCS) per ASTM D-2487.

E.1. Soil Classification

The USCS identifies soil type by single letter prefix and subgroup by single letter suffix as follows;

Table E 1
USCS Classification

Soil Type	Prefix	Subgroup	Suffix
Gravel	G	Well Graded	W
Sand	S	Poorly Graded	P
Silt	M	Silty	M
Clay	C	Clayey	C
Organic	O	w_L < 50%	L
Peat	Pt	w_H > 50%	H

E.2. Electro-Chemical Parameters

Electro-Chemical analysis was neither requested nor conducted during this investigative effort.

E.3. Strength Parameters

For strength calculations, we recommend the following values for angles of internal friction and residual cohesion at 4% strain;

Table E 2 Strength Parameters			
Normal Load	**Soil Type**	**Phi**	**Cohesion**
500 #/ft^2	Poorly graded SAND	32 degrees	0 #/ft^2
	Silty SAND	30 degrees	0 #/ft^2
	Imported ABC FILL @ 90% density per D 1557	33 degrees	0#/ft^2
3,000 #/ft^2	Poorly graded SAND	30 degrees	0 #/ft^2
	Silty SAND	28 degrees	0 #/ft^2
	Imported ABC FILL @ 90% density per D 1557	37 degrees	0#/ft^2

Note that the above values are based on historic typical minimum values determined in other tests of similar soils. For imported fill, we should be contacted to verify values after an actual fill source has been selected.

E.4. Performance Parameters

In addition to the strength parameters described above, swell and consolidation characteristics of the natural soil were carefully considered, both in terms of primary and secondary (long term) volume change. Testing was conducted per ASTM D 2435 (modified), with saturation at a load of 225 psf to simulate the soil load resulting from a concrete slab and fill beneath the slab. The following volume changes were noted;

Table E 3 Performance Parameters				
Pressure	**Consolidation (Collapse)**	**Swell pressure**	**Location**	**Remarks**
225 psf	(0.0%)	--	TP2 @ 4 ft	Silty Sand
1,500 psf	1.4%	--	TP2 @ 4 ft	Silty Sand

Note that swell pressures listed in Table E 3 are recommended design values.

Recommended bearing pressures are presented in Section F of this report.

F. ENGINEERING STUDIES AND RECOMMENDATIONS

F.1. General

The engineering studies and recommendations summarized in this section provide foundation design parameters for the proposed residential structure and for other appurtenant construction. Unless specifically noted otherwise herein, all density tests and recommended densities refer to ASTM D 1557 (Modified Proctor) at optimum to 2% above optimum moisture, unless specifically noted otherwise.

For the purposes of this analysis, maximum column loads were assumed to be on the order of five kips. Wall loads were assumed to be on the order of one kip/lf. Construction methodology was assumed to consist of conventional light wood framing.

F.2. Site Preparation and Grading

F.2.a. Clearing, Grubbing and Stripping

All areas proposed for roadways, structures, driveways, parking, walkways or structural fill should be cleared and grubbed of all trees, stumps, brush and other debris and/or deleterious materials. The site should then be stripped and cleared of all vegetation, sod and organic topsoil. The depth for stripping is likely to vary between 8 and 12 inches of existing vegetation over the entire site.

PEI should be contacted to verify suitable subgrade after the areas for the proposed foundation have been stripped.

F.2.b. Removal of Unsuitable Soil

All areas beneath the building pad should be overexcavated for the placement of the reinforced structural fill assembly below the bottom of footings per section F.3.

Approximately 2 – 3 feet of undocumented fill was observed in the TP2 and TP3 location. This soil, if present within building pad locations, should be removed to its full extent. We anticipate that the amount of fill will decrease towards the north. We anticipate that the medium dense to dense light brown to orange silty sand found 2-3 feet BGS would be suitable in support of new foundations. Over-excavations should be lined with Type 2 drainage geotextile and backfilled with compacted structural fill to bottom of footing elevation per section F.3. Alternative geogrids may be used with approval from Engineer.

Where areas of unsuitable soil, wood waste, building debris or other deleterious materials are encountered during excavation, they should be removed and replaced with compacted structural fill with the over-excavation lined with Type 2 drainage geotextile as recommended or specified by the Engineer.

F.2.c. Density Testing and Subgrade Re-compaction

After building pad is constructed, the exposed subgrade should be tested per Oregon Department of Transportation Test Method 158 (ODOT TM 158) and observed by the geotechnical engineer's representative. Such testing should not be attempted in wet weather and should be discontinued if the subgrade pumps, deflects under load, or otherwise deforms.

Where soils are disturbed or if they pump when tested, they should be excavated, moisture conditioned, and re-compacted or replaced with imported structural fill. Effective recompaction of the fine grained soil will require moisture conditioning and will require less effort if compacted with a sheepsfoot roller. Moisture conditioning and recompaction beneath pavement or slabs should extend to a depth of between 10 and 12 inches. The recompaction should achieve 90% of maximum density, as determined by ASTM D 1557.

In locations where the subgrade consists of soils that are firm and generally unyielding, moisture conditioning and recompaction is not required. PEI should be contacted to perform *in situ* strength tests of subgrade soils and to advise regarding moisture conditioning and compaction.

F.3. Structural Fill Placement and Compaction

Structural fill is defined as any fill placed and compacted to specified densities and located under roadways, structures, driveways, sidewalks and other load-bearing areas.

F.3.a. Structural Fill Materials

Structural fill should consist of a free-draining granular material with a maximum particle size of 3 inches or 2/3 of the un-compacted lift thickness, whichever is lesser. The material should be well graded with less than 5% non-plastic fines. During dry weather, any organic-free, non-expansive, compactable granular material meeting the maximum size criteria is typically acceptable for this use. Locally available crushed rock and jaw run crushed shale have performed adequately for most applications of structural fill. The site fill described herein is considered suitable.

F.3.b. Structural Fill Placement

We recommend that structural fill for the building pad be placed as a layered assembly. To mitigate liquefaction, the accepted excavation should be lined with geogrid Tensar NXST or approved equal. Stage 1 fill consisting of 12 inches of compacted thickness of 3 inch minus material as described above should be placed and compacted on the geogrid.

The top surface of the Stage 1 fill should be covered with Tensar NXST geogrid, then the Stage 2 fill consisting of 9 inches of 1 inch minus base rock, placed and compacted.

In order to accomplish effective compaction for the full fill footprint, we recommend that fill limits extend to ten feet outside of the foundation limits.

Structural fill should be placed in horizontal lifts not exceeding 10 inches loose thickness. Thinner lifts may be necessary to obtain specified density. Each lift should be compacted to 90% of the maximum density per ASTM D 1557. The lift thickness may be increased if specified density is consistently achieved with prior approval by the Engineer.

F.3.c. Compaction

To facilitate the earthwork and compaction process, the earthwork contractor should place and compact fill materials at 1% to 2% above their optimum moisture content. If fill source soils are too wet to compact, they may be dried by continuous windrowing and aeration to achieve optimum moisture. If soils become dry, moisture should be added to maintain the moisture content at or near optimum during compaction operations.

If soil having swell potential is used for fills beneath structures, it should be moisture conditioned to 2% to 4% over optimum and compacted to 88% of maximum density per ASTM D 1557. Swell properties should be determined by laboratory testing prior to use as structural fill.

F.3.c.1. Fill Observation and Testing Methods

Field density testing by nuclear methods is appropriate for compaction of 2-1/2 inch to 3/4 inch minus crushed base rock, fine grained soils, decomposed granite, weathered SANDSTONE and other materials 2-1/2 inches or smaller in size. Due to the effect of particle size on test methods, other methods of compaction testing may be favored. Testing of only the upper lifts is not adequate to verify compaction. Each lift of fill must be compaction tested to verify proper compaction has been achieved.

F.3.d. Non-Structural Fill

All waste soil, organic stripping or other deleterious soil is considered suitable only for non-structural fills. These materials may provide excellent landscape soils and lawn topsoil material if placed in landscape areas and waste soil areas, but should not be placed under permanent structures or within structural fill. It is recommended that these soils be compacted to 88% relative compaction to help seal them from surface water. They should only be utilized in berms less than 10 feet in height having slopes no steeper than 3 1/2 H to 1 V.

F.4. Slopes

Permanent cut and small permanent fill slopes may be required for construction of the site fill and structure building pads.

F.4.a. Cut Slopes

Permanent cut slopes may result from site excavation, overlot grading, and placement of fills. Temporary cut slopes will be required for construction of retaining structures and other portions of the project. For brief periods, these soils may be excavated at steeper angles than listed below. The silty sand soils may stand vertical to a depth of 4 feet for brief periods, except where saturated. In deeper trenches, side walls are likely to slough. We recommend cut slope angles no steeper than;

Table G 1 - Cut Slopes

Soil Classification	Type of Cut	Inclination
SILT Soils	Temporary Cuts	1½ H to 1V
SILT Soils	Permanent Cuts	2½ H to 1V
SAND Soils	Temporary Cuts	1 H to 1V
SAND Soils	Permanent Cuts	1 3/4 H to 1V

F.4.b. Fill Slopes

If periodic CoMET services are provided, we recommend the following maximum permanent fill slope inclinations.

Table G 3 - Fill Slopes		
Soil Classification	**Type of Fill**	**Inclination**
CLAY and SILT soils	All	2 H to 1 V
SAND soils	All	1 3/4 H to 1 V
Compacted, crushed base course	All	1 1/2 H to 1V

All fill slope materials should be considered and constructed as Structural Fill, compacted as described above. In order to accomplish effective compaction for the full fill footprint, we recommend that fills deeper than six feet be over built by five feet width, then the face cut back to achieve the design fill face.

The underlying subgrade must be prepared and compacted prior to fill placement. Keys and benches are critical and must be excavated prior to placement of fill on sloping subgrade. Effective compaction is necessary. Use of sheepsfoot rollers with the fine-grained soil is recommended to integrate each lift with the one below. Rubber-tired rollers can also achieve this result, but smooth-drum rollers should be used with coarse grained material only. Care should be exercised when placing dried hard clay to avoid leaving voids within the fill mass, such voids may allow the soil to lose strength when wetted.

F.4.c. Slope Creep

It is likely that surface creep will occur at locations where the organic SILT soils are utilized to construct non-structural fill slopes and in the organic layer of natural slopes. Creep will occur in response to seasonal volume changes resulting from variations in the moisture content. After repeated cycles a slight shift of the soil in the downslope direction will result and may become apparent.

F.4.d. Recommended Clearances

Recognizing the difficulty achieving specified density for unconfined soils, i.e., the edge of slopes, the minimum recommended separation between the crest or face of the descending slopes and edge of footing should be 8 feet.

Note that these slope setbacks apply to slopes constructed in conformance with this report. Slopes that have not been constructed in conformance with this report may require a greater set-back distance from toe or crest of slopes.

Note that where minimum clearances recommended in this report from crests of slopes are not achievable, the footing bearing elevation may be deepened or it may bear on a deep foundation (drilled shafts or helical piers) to achieve the recommended clearance. Drilled shafts are favored over helical piers due to the greater bending strength under lateral loads. PEI can provide a required depth for deepened footings upon request.

F.4.e. Fill Placed on Slopes

Fill placed on slopes steeper than 6 H to 1 V, including facing of slopes with rock ballast or a buttress, require additional stabilization features;

F.4.e.1. Key Trench - A key trench is required at the toe of all fills placed on slopes between 6H to 1V and 1H to 1V. The minimum depth of key trench excavated below undisturbed surface grade is;

- 3 feet into medium stiff native soils for fill slopes up to 8 feet high,
- 5 feet for fill slopes up to 15 feet high.
- Slopes exceeding 15 feet in height above the lowest point along the toe and those steeper than 1H to 1V should be individually analyzed to verify slope stability and to design foundation key trench requirements.

The key trench should be constructed parallel to contour lines, be wide enough to accommodate excavation and compaction equipment, and its base should be level or back sloped into the hillside to facilitate collection of seepage. It should be drained as depicted in Figure 5.

F.4.e.2. Benching - The underlying natural slope should be prepared by cutting flat benches above the key trench. Fill placed on a sloping subgrade should be blended with natural soil and compacted.

F.4.e.3. Internal Drainage - All seepage or wet zones encountered during construction of keys or benches should be intercepted with internal drains. A minimum of 2 sub-drains are required on all slopes more than 10 feet in height measured from the lowest point along the toe, one at the keyway trench and one ten feet vertically above the key trench. Additional drains may be required as directed by the Engineer. See Figure 5 for drain detail.

The keyway drain should be placed at the rear (uphill) edge of keyway. Engineer should be contacted to observe the drainage components upon excavation of the toe trench. At engineer's discretion, additional horizontal drains may be required upslope. ***Additional slope drains may be required by engineer during site observation, where either soil characteristics or moisture conditions, in his sole opinion, warrant.***

F.5. Pavement Analysis and Design

F.5.a. Asphaltic Concrete Pavements

Site specific paving design was beyond the scope of this investigation, however, it should generally consist of compacted bituminous surface mix placed over a layer of 1 1/2 inch minus aggregate base and compacted sub-base. Geotextile should be used as a separation medium to isolate localized sub grade failures. For design purposes, CBR's can be expected to vary between one for soaked subgrade in fill areas to in excess of twenty in areas of competent weathered rock. If assistance is desired with site specific pavement design, please contact us.

Material quality and placement of the surface assembly should conform to the 2024 edition of the Oregon Standard Specifications for Construction.

F.5.b. Non-Structural Slabs on Grade

Exterior concrete slabs on grade will be subjected to moisture induced movement which is likely to result in cracking and vertical offsets at joints and connections with other structures. More uniform support can be achieved by placing a minimum thickness of 8 inches of crushed rock, crushed shale or decomposed granite fill beneath the slabs in these areas and conforming to the concrete pavement recommendations per the Portland Cement Association. Slabs and walkways reinforced with #3 or #4 deformed steel reinforcing bars both ways will also withstand moisture induced movement better than unreinforced flatwork. The reinforcing should extend across joints (or use dowels, Diamond Dowels, etc.) to decrease differential vertical movement. Jointing patterns to provide predetermined crack locations will also generally improve the appearance of the finished flatwork. Concrete work should conform to American Concrete Institute (ACI) Specification 306 and 318.

F.6. Site Drainage and Erosion Control

F.6.a. Buildings

Final grading should accomplish rapid positive drainage away from the structure for a horizontal distance of at least 10 feet at a minimum grade of 5%. This water should be channeled to surface drains or swales for proper disposal. The landscaping around the structure should be graded such that drainage discharges clear of the foundation influence area. Downspouts should be connected to a sealed system which discharges to a location clear of the foundation influence area.

F.6.b. Crawlspace Drainage

Crawl spaces should be sloped to drain to one or more low point drains. There should be no low areas that allow ponding. These low point drains should discharge through or under the foundations to the surface water disposal system.

F.6.c. Upslope of Structures

The area immediately upslope of most structures and components is likely to pond surface moisture. We recommend that the upslope area be graded to collect and dispose of surface moisture.

F.6.d. Surface Areas

Surface and subsurface water flows should be intercepted by swales and/or catch basins and conveyed through tight lines to acceptable discharge locations. We recommend that hard surfaces be provided, sloped and shaped to channel water away from the structure.

F.6.e. Erosion Control

Site soils are susceptible to erosion if unprotected. The site grades are such that erosion and sediment transport during construction are expected to be significant. The site cuts and fills, building pad, etc. should be graded such that surface water is collected and disposed without causing erosion or siltation. Sediment laden water should be addressed using state guided erosion control best management practices.

The following erosion control measures shall be employed:

- Stripping of vegetation, grading, or other soil disturbance shall be done in a manner which will minimize soil erosion, stabilize the soil as quickly as practicable, and expose the smallest practical area at any one-time during construction.
- Development plans shall minimize cut or fill operations so as to prevent off-site impacts.
- Temporary vegetation and/or mulching shall be used to protect exposed critical areas during development.
- Permanent plantings and any required structural erosion control and drainage measures shall be installed as soon as practical.
- Provisions shall be made to effectively accommodate increased runoff caused by altered soil and surface conditions during and after development. The rate of surface water runoff shall be structurally retarded where necessary.
- Provisions shall be made to prevent surface water from damaging the cut face of excavations or the sloping surface of fills by installation of temporary or permanent drainage across or above such areas, or by other suitable stabilization measures such as mulching, seeding, planting, or armoring with rolled erosion control products, stone, or other similar methods.
- All drainage provisions shall be designed to adequately carry existing and potential surface runoff from the twenty-year frequency storm to suitable drainageways such as storm drains, natural watercourses, or drainage swales. In no case shall runoff be directed in such a way that it significantly decreases the stability of known landslides or areas identified as unstable slopes prone to earth movement, either by erosion or increase of groundwater pressure.
- Where drainage swales are used to divert surface waters, they shall be vegetated or protected as necessary to prevent offsite erosion and sediment transport.
- Erosion and sediment control devices shall be required where necessary to prevent polluting discharges from occurring. Control devices and measures which may be required include, but are not limited to:

  a) Energy absorbing devices to reduce runoff water velocity. Straw waddles and check dams are effective at reducing runoff velocity.

b) Sedimentation controls such as sediment or debris basins. Any trapped materials shall be removed to an approved disposal site on an approved schedule.

c) Dispersal of water runoff from developed areas over large undisturbed areas.

- Disposed spoil material or stockpiled topsoil shall be prevented from eroding into streams or drainageways by applying mulch or other protective covering; or by location at a sufficient distance from streams or drainageways; or by other sediment reduction measures.
- Such non-erosion pollution associated with construction such as pesticides, fertilizers, petrochemicals, solid wastes, construction chemicals, or wastewaters shall be prevented from leaving the construction site through proper handling, disposal, site monitoring and clean-up activities.

Typical project landscaping should be adequate for long-term erosion control. ***In no case should concentrated surface water runoff be allowed to flow from swales and over the top edge and/or down the face of any slopes.***

F.7. Building Foundations

F.7.a. General

A combination of spread and continuous footings is recommended for residential structures. Footings should bear on non-swelling imported structural fill.

F.7.b. Spread and Continuous Footings

F.7.b.1. Subgrade Preparation

The foundation subgrade should be overexcavated per Section F.2 of this report.

F.7.b.2. Fill

See Section F.3 of this report.

F.7.b.3. Footing or Perimeter Slab Embedment

Bearing components subject to frost induced movement should be embedded a minimum of 12 inches below natural or finish grade to provide lateral support and frost protection. Footing excavations should be backfilled with structural fill.

F.7.b.4. Allowable Bearing Pressure

Building footings placed as recommended above may be designed for the following bearing pressures;

Table F 3 – Allowable Bearing Pressure	
Classification	**Allowable Bearing Pressure**
Compacted Structural Fill	1,500 #/ft^2

F.7.b.4.a. Load Duration and Shape Increases

Allowable bearing pressure may be increased by 1/3 for short term loads. Allowable bearing pressures on square spread footings may be increased by 20%.

F.7.b.5. Minimum Dimensions

The minimum recommended width for continuous footings is 1'- 6" and the minimum recommended dimension for spread footings is 2'-0".

F.7.c. Footing Drains

We recommend that exterior footing drains be provided for below grade components, located at an elevation low enough to intercept groundwater, and limit it from rising above the surface of crawlspaces and the bearing area of interior slabs on grade. Footing drains should discharge clear of the foundation influence area. See Section F.7.f.

F.7.d. Settlement

Building settlement will vary with thickness and swell/consolidation potential of fill, type and thickness of underlying soils and methodology of foundation construction. In addition to settlement, vertical movement due to swelling of the foundation soil is possible for lightly or differentially loaded structural components placed on over-compacted non-natural imported soil having swell potential.

Relying on the loads estimated herein and assuming that the dead load portion will be approximately 1/3 of the total, we project total vertical movement to be less than 1 inch. Differential movement could be as much as 0.75 inches. Post-liquefaction total settlement could be as much as 2 inches.

F.7.e. Interior Floor Slabs

Interior floor slabs should not be rigidly connected to the perimeter footing, i.e., should float within the structure. The following recommendations are provided for slabs constructed on structural fill over properly prepared subgrade soils;

F.7.e.1. Aggregate Base Course (ABC)

A 6 inch thick layer of clean (less than 2% passing the No. 200 sieve) 3/4" minus crushed rock should be placed over the structural fill to provide a positive capillary moisture break and uniform slab support. The capillary break is essential in areas to receive tile and linoleum and other areas with relatively impermeable floor finishes. To decrease drying stress, a 1/4 inch thickness of clean sand should be placed on top of the ABC.

F.7.e.2. Underslab Membrane

A moisture retarder or barrier should be used to decrease seepage or upward migration of moisture through the concrete, but is likely to increase soil moisture and exacerbate expansion if soils having expansion potential are imported. To protect the membrane, a 1/4 inch thickness of clean sand should be placed on top of the membrane.

F.7.e.3. Minimum Slab Thickness
Minimum recommended slab thickness is 4 inches to allow sufficient cover over the reinforcing steel. **Note that all slabs should be designed for the actual use and equipment anticipated.**

F.7.e.4. Isolation
Floor slabs and walls, both bearing and non-bearing, resting on floor slabs should be isolated from other structural components. We would be pleased to provide typical isolation details or to review structural plans prepared by others.

F.7.e.5. Reinforcement
The slabs should be reinforced with deformed reinforcing steel instead of welded wire fabric.

F.7.e.6. Reinforcement Location
Locate reinforcing a dimension of 1/3 slab thickness below the surface. Use “dobies” or bolsters to establish accurate position of reinforcement.

F.7.e.7. Fiber
Polypropylene fiber may be added to the concrete mix to help decrease plastic shrinkage cracking; however, it is not a replacement for structural reinforcing.

F.7.e.8. Joints
Contraction and control joints conforming to ACI recommendations should be incorporated in the construction. Saw cut joints or wet scored joints should be accomplished within 12 hours after concrete placement. Construction joints and joints across dissimilar pours should be joined by square dowels to decrease the potential for differential vertical movement or curling.

F.7.f. Footing and Floor Drains

F.7.f.1. Footing Drains
Drains should consist of a rigid, smooth interior perforated drain pipe placed adjacent to the base of the footing. The perforated pipe should be encapsulated in a minimum of 8 inches of clean drain rock or pea gravel wrapped in ODOT drainage geotextile Type 1.

F.7.f.2. Wall Drains
Drains are recommended for below grade walls. These walls should be provided a minimum 12-inch wide zone of drain rock isolated with non-woven drainage geotextile, continuous from the top of footing to one foot below the surface. A preformed, fabric-wrapped, polymer sheet drain, such as Linq Drain, Enkamat, or Amerdrain may be used instead of the vertical drainage zone, provided the excavation is backfilled with clean, free-draining material. Design of such walls should disregard friction between the wall and fill for stability computations, however. Walls demising habitable areas should be provided durable wall sealant coating or other water proofing membrane before installing the sheet drain.

F.7.f.3. Floor Subdrains

Where the drain rock layer below slabs will be lower than the adjacent exterior grades, water will tend to accumulate. In these locations, positive drainage of the under slab layer should be provided.

F.7.f.4. Discharge

Foundation drains and subdrains should be routed to discharge clear of the foundation influence area or slopes. **Interconnection of roof downspouts or surface area drains with foundation, wall, or floor subdrain systems is not allowed.**

F.8. Lateral Earth Pressures and Drainage

F.8.a. Lateral Load Resistance

Lateral loads exerted upon these structures can be resisted by passive pressure acting on buried portions of the foundation and other buried structures and by friction between the bottom of concrete elements of the foundations and slabs and the underlying soil.

Lateral load resistance should be calculated using the values presented in Section E.3 for the recommended depth of embedment as;

P_a or $P_p = \frac{1}{2} k_{(a \text{ or } p)} \gamma H^2$ where;

P_a is active earth pressure

P_p is passive earth pressure

$k_a = \tan^2 (45°-\varphi/2)$

$k_p = 1/ k_a$

γ = soil unit weight

The first one foot below the ground surface should be ignored when computing passive resistance.

- A coefficient of friction of 0.45 is recommended for elements poured neat against structural rock fill or bedrock.
- A coefficient of friction of 0.30 is recommended for elements poured against natural soils.
- The above values should be reduced to 0.2 for areas where bearing is over a non-soil vapor barrier or low permeability membrane.

F.8.b. Lateral Earth Pressures

It is possible that both unrestrained and restrained retaining walls may be constructed for the project. Lateral earth pressures will be imposed on below-ground and backfilled structures or walls, including daylight basements and foundations which do not have uniform heights of fill on both sides. The following recommendations are provided for design and construction of retaining walls:

- Walls which are free to rotate at the top when backfilled should be designed for an equivalent fluid pressure of 45 $\#/ft^3$. This value should be increased to 52 $\#/ft^3$ for a 2 H to 1 V back slope.
- Walls that are fixed at the top should be designed for an equivalent fluid pressure of 60 $\#/ft^3$. This should be increased to 67 $\#/ft^3$ for a 2 H to 1 V back slope.
- A wet soil unit weight of 135 $\#/ft^3$ should be used for design.
- Backfill should consist of non-expansive, free draining, material. The backfill should be placed in lifts at near the optimum moisture content and compacted to between 88 and 90% of the maximum density per ASTM D 1557. Care should be employed to avoid over compacting the backfill. Loosely placed backfill and over-compacted backfill will exert greater pressures on the wall than the pressures considered above.
- To prevent damage, backfill and compaction against walls or embedded structures should be accomplished with hand-operated equipment within a lateral distance of 1/2 to 1/3 the unsupported height of wall. Beyond this zone, normal compaction equipment may be used.
- While proper compaction of wall backfill is critical to long-term performance, care should be taken to avoid over compaction of the backfill materials, which can result in lateral loads greater than the design pressures recommended above.
- For design of retaining walls supporting or bracing structures, a peak horizontal acceleration coefficient of 0.5g is recommended for seismic loads.
- To prevent development of hydrostatic pressures exceeding the lateral earth pressures, a perimeter drainage system is recommended for underground structures, including basements.
- Hydrostatic pressures behind retaining walls should be relieved by installation of free draining backfill behind the walls, with weep holes spaced as necessary (typically 10 feet on center) to achieve effective drainage. The free draining backfill should be protected from plugging by encapsulating with drainage geotextile as recommended above.
- Allowable bearing capacities should be as recommended for Building Structures.

F.9. Trenching and Piping

Additional underground piping will be constructed. Excavation can be accomplished by normal means throughout the site. Depending on when construction occurs, dewatering of the trench may be necessary to facilitate construction.

- Pipe should be cradled in coarse aggregate compacted to 90% density per ASTM D 1557, having a minimum thickness equal to 1/4 pipe diameter below bottom of pipe and extending upward to the pipe spring line.
- The trench backfill should consist of clean excavated material, compacted to 90% density per ASTM D 1557.

- Beneath paved areas, full depth granular backfill is recommended as a minimum, and use of lean cement slurry should be considered.
- The top 12 inches of the trench backfill should be compacted to a density of 92% per ASTM D 1557. Loads on pipe will vary with depth and width of trench.
- For pipe design, an effective pressure of 130 $\#/ft^3$ per foot of depth is recommended.
- Underground pipes located beneath paved areas and having shallow cover should be designed to withstand vehicular loads.

G. ADDITIONAL SERVICES AND LIMITATIONS OF REPORT

G.1. Additional Services

Additional services by the geotechnical engineer are recommended to help insure that design recommendations are correctly interpreted during final project design and to help verify compliance with project specifications during construction. Additional services could include, but not be limited to:

- Review of final construction plans and specifications for compliance with geotechnical recommendations.
- Review of proposed cuts and fills, fills on slopes, surface and subdrains, swale drains, foundation support, and basement or rock fill subdrains.
- Site observation and/or CoMET services, i.e., observation of over excavated areas below keys, benches and footings and slabs, subgrade proof rolling, placement and compaction testing of structural fill, fill subdrains, swale subdrains, foundation drains, wall drains, subgrade proof rolling, pavement subgrade and aggregate base placement, site grading, surface drainage, etc.

G.2. Limitations

Where used herein, the terms "Special Inspector, Inspector and Special Inspection" are understood to be for services contemplated, prescribed and as defined by the International Building Code and the Oregon Structural Specialty Code.

The analyses, conclusions and recommendations contained in this report are based on site conditions and development plans as they existed at the time of the study, and assume that soils and groundwater conditions encountered, observed or inferred during our exploration are representative of soils and groundwater conditions throughout the site. If, during construction, subsurface conditions are found to be different or design parameters change, we should be advised at once so that we can review this report and reconsider our recommendations, as appropriate. If there is a significant lapse of time between submission of this report and the start of work at the site, if the project is changed, or if site conditions have changed, we recommend that this report be reviewed to verify continued applicability.

This report was prepared for the use of the owner and design team for the subject project. It is only for this site and construction project. No third party beneficiaries are intended. Potential users of the report should be so notified.

It should be made available to other contractors for information and factual data only, such as test boring or test pit logs, measured water levels, samples, sample classifications and laboratory test results. The report is interpretive in nature and shall not be used for contractual purposes, such as warranting that subsurface conditions will be consistent with, or as indicated by the formal boring or test pit logs and subsurface profiles contained or inferred herein and/or discussions of subsurface conditions. It is not to be used for extensions of this project or for other projects without our express written consent. We should be contacted to review both plans and specifications for compatibility with this report before finalization. **CoMET services, compaction testing and periodic observation during construction are recommended.**

We have performed these services in conformance with generally accepted engineering and geotechnical engineering practices in southern Oregon at the time the study was accomplished. No other warranty is either expressed or implied.

Since test borings represent only the conditions at those discrete locations, unanticipated soil conditions may be and, in fact, are commonly encountered on projects of similar size. Unanticipated conditions cannot be precluded by practical field studies. Since such unexpected conditions frequently result in budget increases to attain a properly constructed project, we recommend that a reasonable contingency account be established sufficient to fund possible extra costs.

We appreciate the opportunity to assist you on your project. If you have any questions, or if we may be of further assistance, please do not hesitate to contact us.

Sincerely,

Pinnacle Engineering, Inc.

Matt Keller, P.E., CSI
Registered Geotechnical Engineer
President

REGISTERED PROFESSIONAL ENGINEER
OREGON
SEPT. 12, 2017
MATTHEW RYAN KELLER
EXPIRES: 6-30-25

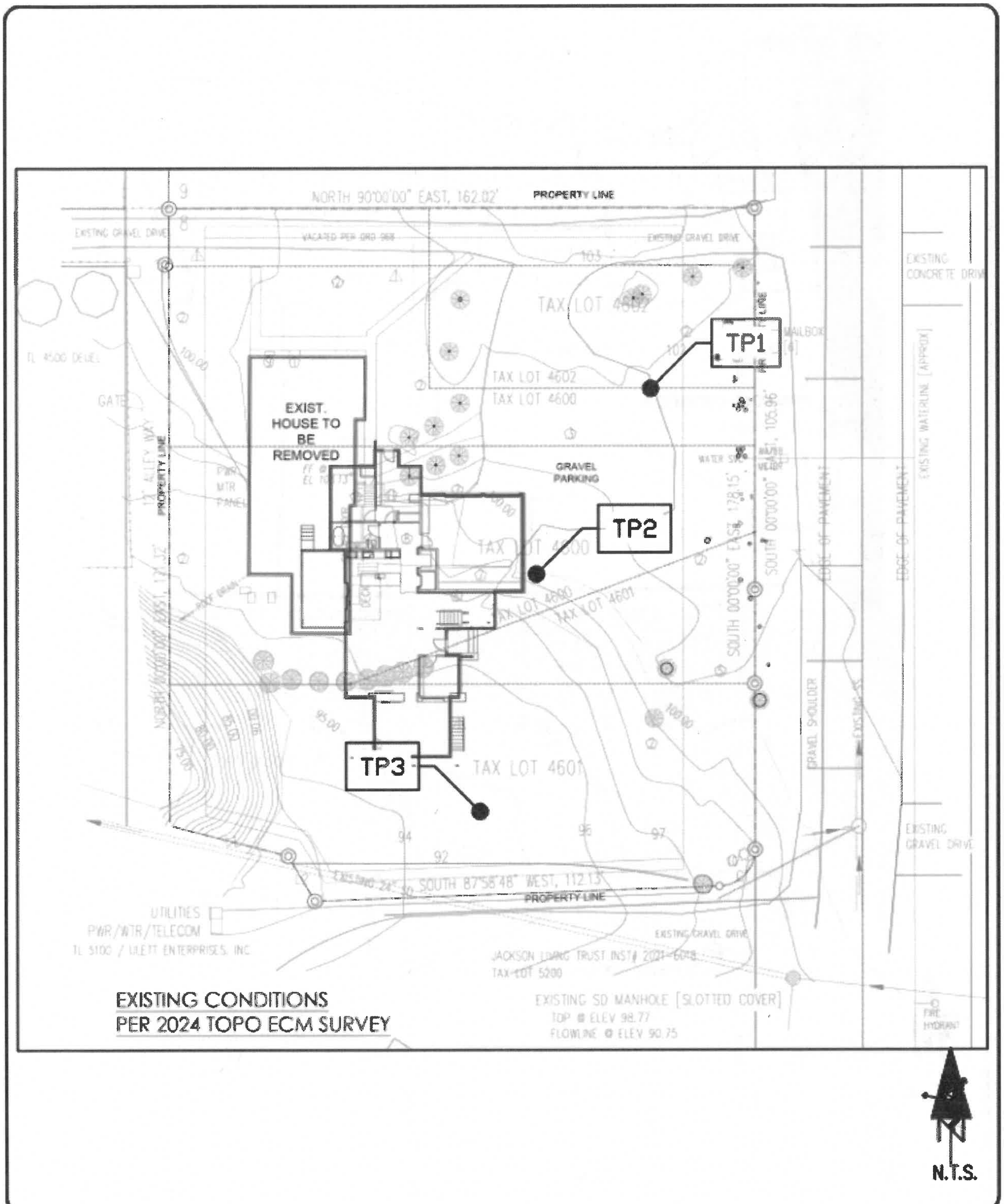
FIGURES

PINNACLE ENGINEERING, INC.

SUBSURFACE SOILS INVESTIGATION
VICINITY MAP

PROJECT: 30725 - 1190 Beach Loop Dr SW, Bandon
CLIENT: Vine Maple Design

FIG. 1



SUBSURFACE SOILS INVESTIGATION
SITE MAP
PROJECT: 30725 - 1190 Beach Loop Dr SW, Bandon
CLIENT: Vine Maple Design

FIG. 2

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SOIL TYPES (Ref. 1)

Boulders:	Particles of rock that will not pass a 12 inch screen.
Cobbles:	Particles of rock that will pass a 12 inch screen, but not a 3 inch sieve.
Gravel:	Particles of rock that will pass a 3 inch sieve, but not a #4 sieve.
Sand:	Particles of rock that will pass a #4 sieve, but not a #200 sieve.
Silt:	Soil that will pass a #200 sieve, that is non-plastic or very slightly plastic, and exhibits little pr no strength when dry.
Clay:	Soil that will pass a #200 sieve, that can be made to exhibit plasticity within a range of water contents, and that exhibits considerable strength when dry.

MOISTURE AND DENSITY

Moisture condition:	An observational term; moist, wet.
Moisture content:	The weight of water in a sample divided by the weight of dry soil in the sample, expressed as a percentage.
Dry Density:	The pounds of dry soil in a cubic foot of soil

DESCRIPTORS OF CONSISTENCY (Ref. 3)

Liquid Limit:	The water content at which a - #200 soil is on the boundary between exhibiting liquid and plastic characteristics. The consistency feels like soft butter.
Plastic Limits:	The water content at which a - #200 soil is on the boundary between exhibiting plastic and semi-solid characteristics. The consistency feels like stiff putty.
Plasticity Index:	The difference between the liquid limit and the plastic limit, i.e. the range in water contents over which the soil is in a plastic state.

MEASURES OF CONSISTENCY OF COHESIVE SOILS (CLAYS) (Ref's 2&3)

Very soft	N=0-1*	C=0-250 psf	Squeezes between fingers
Soft	N=2-4	C=250-500 psf	Easily molded by finger pressure
Medium stiff	N=5-8	C=500-1000 psf	Molded by strong finger pressure
Stiff	N=9-15	C=1000-2000 psf	Dented by strong finger pressure
Very stiff	N=16-30	C=2000-4000 psf	Dented slightly by finger pressure
Hard	N>30	C>4000 psf	Dented slightly by pencil point

*N= Blows per foot in the Standard Penetration Test. In cohesive soils, with the 3 inch diameter sampler. 140-pound weight, divide the blow count by 1.2 to get N (Ref. 4).

MEASURES OF RELATIVE DENSITY OF GRANULAR SOILS (GRAVELS, SANDS, SILTS) (Ref's 2 & 3)

Very Loose	N=0-4**	RD=0-30	Easily push a ½ inch reinforcing rod by hand
Loose	N=5-10	RD=30-50	Push a ½ inch reinforcing rod by hand
Medium Dense	N=11-30	RD=50-70	Easily drive a ½ inch reinforcing rod
Dense	N=31-50	RD=70-90	Drive a ½ inch reinforcing rod 1 foot
Very Dense	N>50	RD=90-100	Drive a ½ inch reinforcing rod a few inches

**N= Blows per foot in the Standard Penetration Test. In granular soils, with the 3 inch diameter sampler. 140 pound weight, divide the blow count by 2 to get N (Ref 4). RD = Relative Density.

Ref. 1: ASTM Designation: D 2487-93, Standard Classification of Soils for Engineering Purposes(Unified Soil Classification system).
Ref.2: Terzaghi, Karl, and Peck, Ralph B., Soil Mechanics in Engineering Practice, John Wiley & Sons, New York, 2nd Ed., 967, pp. 30, 341, 347.
Ref.3: Sowers, George F., Introductory Soil Mechanics and Foundations: Geotechnical Engineering, Macmillan Publishing Company, New York, 4th Ed., 1979, pp. 80,81, and 312.
Ref.4: Lowe, John III, and Zaccheo, Phillip F., Subsurface Explorations and Sampling Chapter 1 in Foundation Engineering Handbook, Hsai-Yang Fang, Editor , Van Nostrand Reinhold Company, New, 2nd Ed. 1991, p.39/

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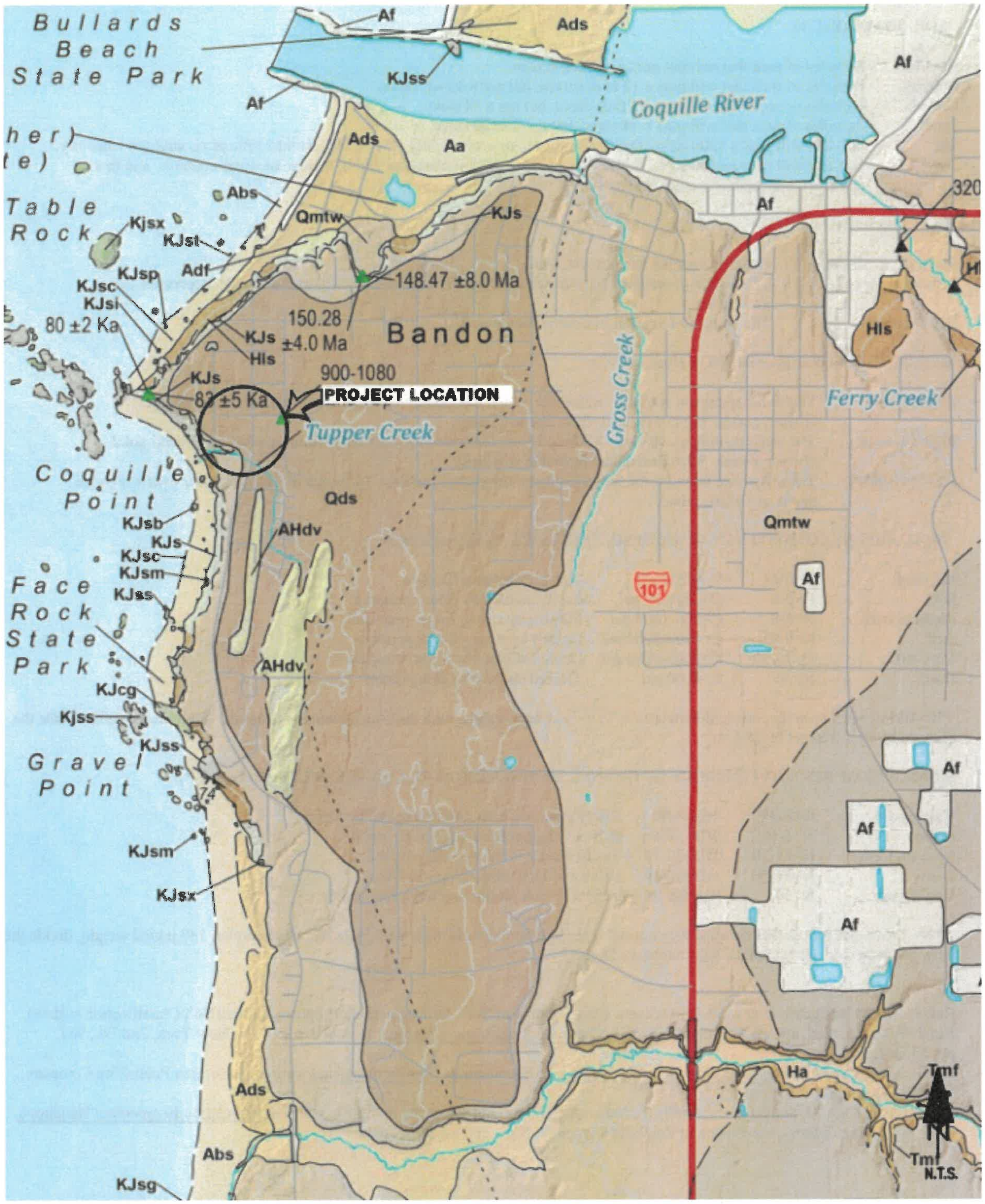
SUBSURFACE SOILS INVESTIGATION
GEOLOGIC REFERENCE

PROJECT: 30725 - 1190 Beach Loop Dr SW, Bandon
CLIENT: Vine Maple Design

FIG. 3



PINNACLE ENGINEERING, INC.

SUBSURFACE SOILS INVESTIGATION GEOLOGIC MAP

PROJECT: 30725 - 1190 Beach Loop Dr SW, Bandon
CLIENT: Vine Maple Design

FIG. 4

Geosynthetics and Slope Protection
Section 02320 - Geosynthetics

Description

02320.00 Scope - This section includes the requirements for geosynthetics used in various applications.

02320.01 Definitions - Geosynthetic terms are defined in 00350.01

Materials

02320.10 Acceptance:

(a) General Requirements - Furnish all geosynthetics meeting the following requirements:

- Free of defects, cuts or tears.
- Resistant to ambient temperatures, acid and alkaline conditions, micro-organisms and insects.
- For the intended purpose and have dimensional stability.

(1) Geotextiles - Furnish woven or nonwoven geotextiles meeting the following requirements:

- Fibers used in manufacture of geotextiles, and threads used in joining geotextiles by sewing, shall consist of long-chain synthetic polymers, composed of at least 95 percent by weight of polyolefins or polyester. They shall be formed into a stable network such that the filaments or yarns retain their dimensional stability to each other, including selvages.
- Meet or exceed the properties specified in 02320.20.
- Be free of any chemical treatment or coating which might significantly reduce permeability.

(2) Geogrids - Furnish geogrids meeting the following requirements:

- Geogrid reinforcements approved as Type 1 MSEW Geogrid on the QPL.
- Geogrid for Subgrade reinforcement approved as Subgrade Reinforcement Geogrid on the QPL.

(b) Acceptance Requirements - The actual minimum average roll values furnished by the manufacturer shall be based on representative test results from the manufacturing plant which produced the geosynthetic, and shall meet or exceed each of the specified minimum values. All geosynthetics shall be clearly labeled as being part of the same production run certified as meeting all applicable requirements.

(c) Manufacturer's Test Certification - Furnish test result certificates according to 00165.35 from the geosynthetic manufacturer, and the following:

(1) Geotextiles - For geotextiles, include the following:

- Manufacturer's name, lot number, roll number, production facility address, and full product information (style, brand, name, etc.)
- Chemical composition of filaments and yarns, including polymer(s) used.
- Minimum average roll values for each of the specifies properties from the same lot of geotextiles as the delivered material.

(2) Geogrids - For mechanically stabilized earth retaining wall geogrid, include the following:

- Average roll values for each of the specified properties from the same production run as the delivers material.
- Production run number, production plant name, and location.
- Manufacturer's name and address.
- Full product name and information.
- QPL Product Category and the Standard Specification Subsection number.
- Retaining wall location referencing the drawing name, detail, and structure number.
- Polymer types for geogrid and coating, if present.
- Primary resin type, class, grade, and category for HDPE (ASTM D1248) and PP (ASTM D4101)

For subgrade reinforcement geogrid, include the following:

- Minimum average roll values and average roll values for each of the specified properties from the same production run as the delivered material.
- Production run number, production plant name and location.

(d) Manufacturer's Sampling/Testing - The manufacturer's reported property values shall be based on the following sampling and testing requirements:

(1) Sampling - Sample all geosynthetics according the ASTM D4354. The production unit used for sampling shall be a roll or sheet.

(2) Geotextile Testing - Perform the specified tests to determine geotextile properties for the intended applications. The tensile strength requirements shall be tested in both machine and cross-machine directions.

(3) Geogrid Testing - For mechanically stabilized earth retaining wall geogrid, provide laboratory test results the demonstrate the average roll value for each geogrid product is greater than or equal to the geogrid ultimate wide width tensile strength reported for the initial geogrid product evaluation and approval on the QPL. Determine the ultimate wide width tensile strength (T_{ult}) according to ASTM D6637. If the average roll value for each geogrid reinforcement product is less than the geogrid ultimate wide width tensile strength identified on the QPL, the entire production run will be rejected.

(e) Agency Check Tests - The Agency reserves the right to sample and test products for compliance with pertinent requirements, according to 00165.02. When the Agency performs check tests, the entire production run will be accepted or rejected according to 00150.25, if any of the average roll values of tested rolls are less than the specified minimum values.

02320.11 Seam Testing and Acceptance:

(a) Factory Seams - Where factory seams are made, the sheets of geotextile shall:

- Be sewn together using a lock type stitch Type 301 or 401 as shown.
- Be sewn with polymeric thread that is at least 95 percent, by weight, polyolefin or polyester, and as resistant to deterioration as the geotextile being sewn.
- Have test results showing that the seams meet or exceed 90 percent of the specified tensile strength minimum values for intended application.
- Nylon thread will not be allowed.

(b) Field Seams - Where field sewn seams will be used, furnish:

- The manufacturer's test result certificate, according to 00165.35, that includes wide strip, tensile strength test results and verifies that seams tensile strength and seam grab tensile strength meet or exceed 90 percent of the minimum specified tensile strength values for the geotextile.
- A field-stitched seam test sample.

PINNACLE ENGINEERING, INC.

SLOPE STABILIZATION GEOSYNTHETIC NOTES

PROJECT: 30725 - 1190 Beach Loop Dr SW, Bandon

CLIENT: Vine Maple Design

FIG. 5a

Geotextile Property Values: Summary of Tables 02320-1 - 02320-6 (Oregon Standard Specifications for Construction 2024)

GEOTEXTILE PROPERTY	TEST METHOD	UNITS	FILTRATION							SEPARATION	REINFORCEMENT	
			DRAINAGE (1) GEOTEXTILE		RIPRAP (1) GEOTEXTILE		SEDIMENT FENCE GEOTEXTILE			SUBGRADE GEOTEXTILE	EMBANKMENT GEOTEXTILE	PAVEMENT OVERLAY GEOTEXTILE
							SUPPORTED	UNSUPPORTED				
			TYPE 1	TYPE 2	TYPE 1	TYPE 2	—	Elongation >/= 50% (2)	Elongation </= 50% (2)			
Grab Tensile Strength (minimum) Machine Direction Cross Machine Direction	ASTM D 4632	lb	180 115	250 160	250 160	315 200	90 90	120 100	120 100	180 113	315 200	100
Grab Elongation (minimum)	ASTM D 4632	%	50		50		—	—		50	50	50
Tear Strength (minimum)	ASTM D 4533	psi	67 40	90 56	90 56	110 80	—	—		68 41	110 80	—
Puncture Strength (minimum)	ASTM D 6241	lb	370 220	495 310	495 310	620 430	—	—		371 223	620 430	—
Apparent Opening Size (AOS) (maximum) U.S. Standard Sieve	ASTM D 4751	in	NO. 40		NO. 40		NO. 30	NO. 30	NO. 30	NO. 30	NO. 30	—
Permittivity (minimum)	ASTM D 4491	s^{-1}	0.5		0.5		0.05	0.05	0.05	0.05	0.02	—
Ultraviolet Stability (Retained Strength) (minimum)	ASTM D 4355 @ 500 hours	%	50		70		70 After 500 hours exposure			50	50	—
Asphalt Retention (minimum)	ODOT TM 817	oz/ft^2	—		—		—	—		—	—	2.8
Melting Point (minimum)	ASTM D 276	°F	—		—		—	—		—	—	300

(1) Slit film or slit tape fabrics are not acceptable.
(2) As measured according to ASTM 4632.

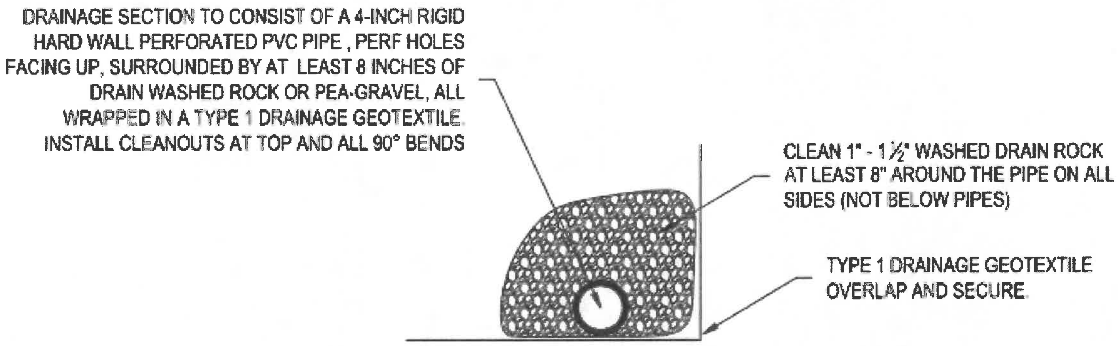
NOTE: CONTRACTOR SHALL SUBMIT TO ENGINEER MATERIAL VENDOR DOCUMENTATION OF COMPLIANCE WITH GEOTEXTILE SPECIFICATIONS

PINNACLE ENGINEERING, INC.

SLOPE STABILIZATION GEOSYNTHETICS TABLE

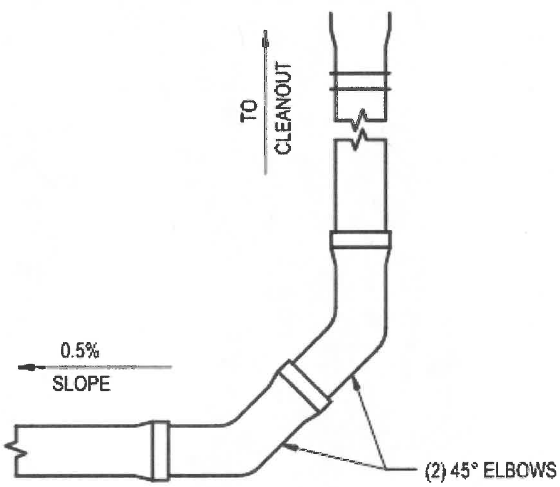
PROJECT: 30725 - 1190 Beach Loop Dr SW, Bandon
CLIENT: Vine Maple Design

FIG. 5b



1/5c PERIMETER DRAIN DETAIL

SCALE: N.T.S



2/5c CLEANOUT DETAIL

SCALE: 1" = 1'-0"

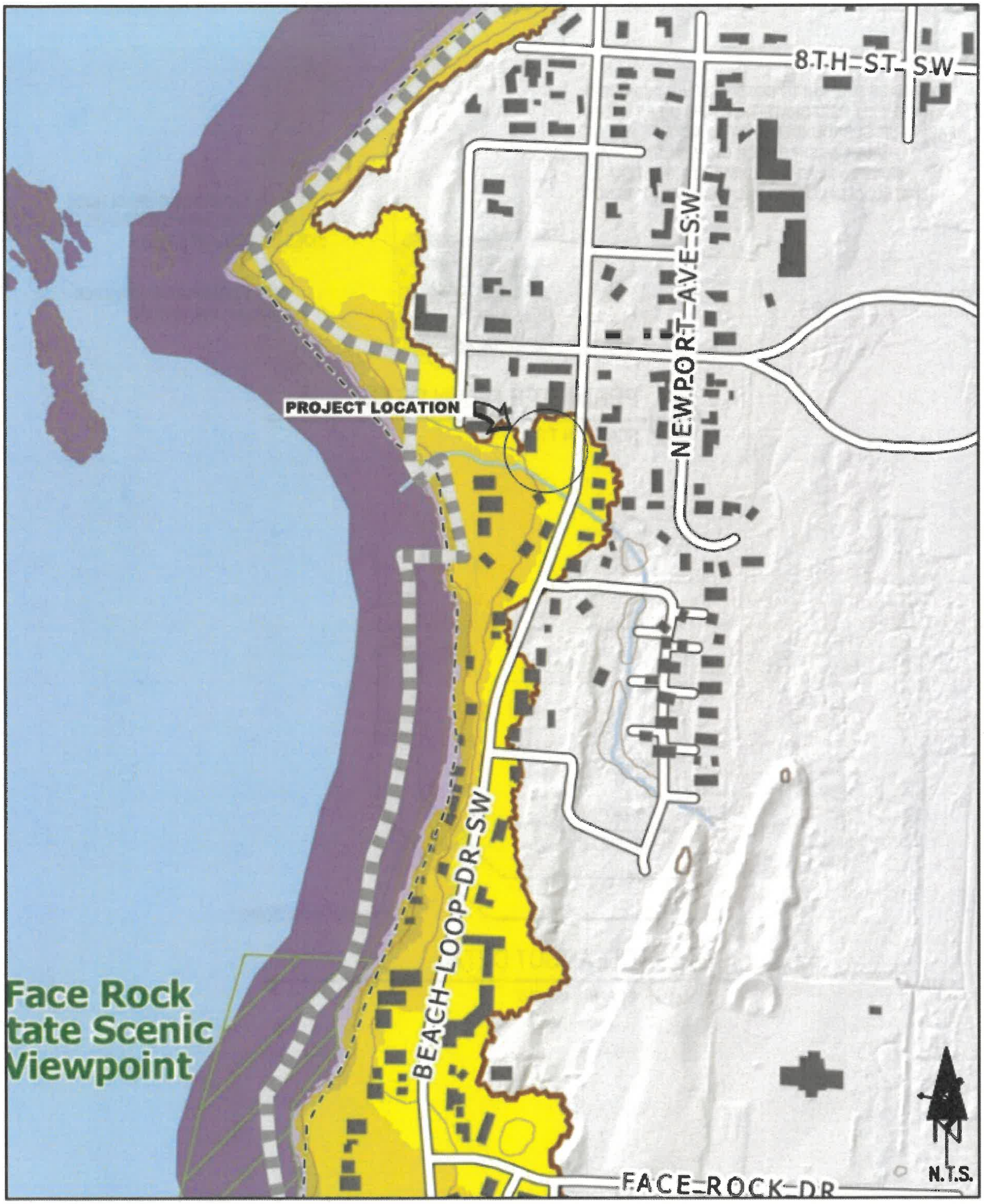
PINNACLE ENGINEERING, INC.

SLOPE STABILIZATION
DRAIN AND CLEANOUT

PROJECT: 30725 - 1190 Beach Loop Dr SW, Bandon

CLIENT: Vine Maple Design

FIG. 5c



PINNACLE ENGINEERING, INC.

TSUNAMI INUNDATION MAP

PROJECT: 30725 - 1190 Beach Loop Dr SW, Bandon
CLIENT: Vine Maple Design

FIG. 6

APPENDIX A
TEST BORING LOG AND TESTS

DRILL HOLE LOG

TP1

File 61194 Date Printed: 2/11/2025

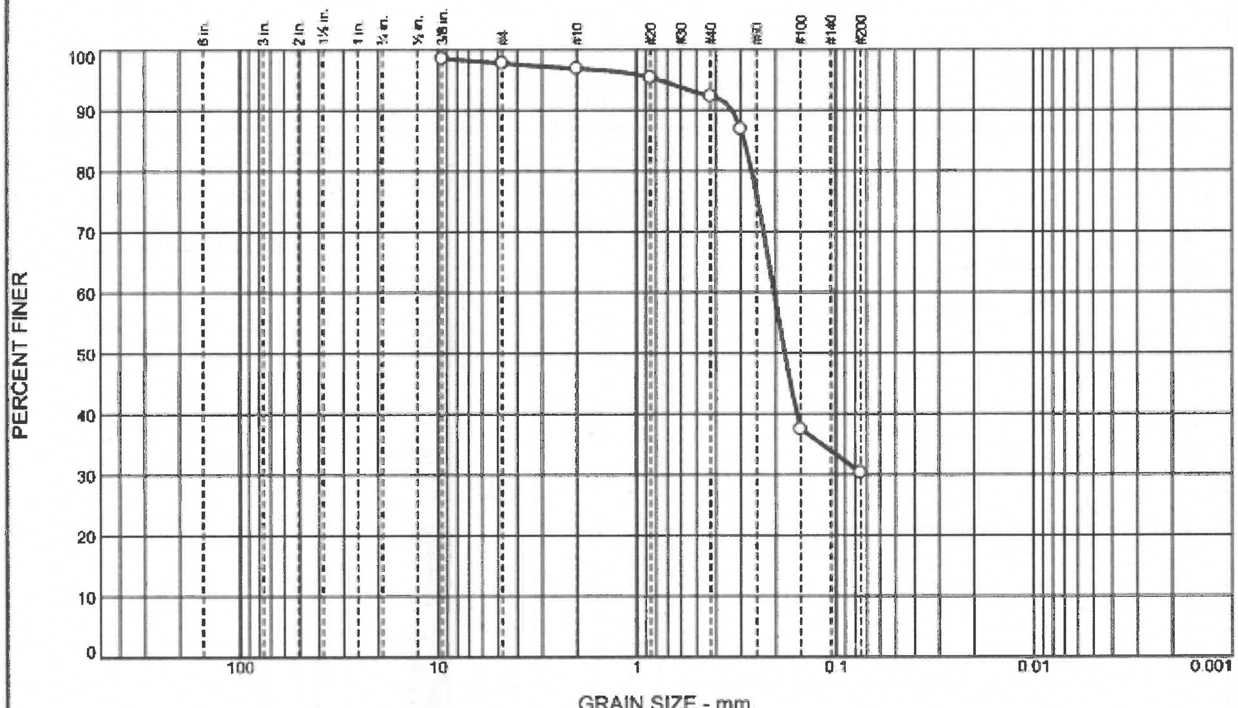
PROJECT: 1190 BEACH LOOP	PROJECT NO.: 61194
CLIENT: PEI	DATE: 1/17/24
LOCATION: 43.11269, -124.43208	ELEVATION: 94'
DRILLER: TWS	LOGGED BY: JBE
DRILLING METHOD: TAKEUCHI 235 18" BKT	
DEPTH TO - WATER> INITIAL:	AFTER DRILLING:
SEEPAGE: 8.5'	

DEPTH (feet)	Description	Recov (in)	Driven (in)	PID ppm	Sample#	Soil Type	Sampler	Symbol	TEST RESULTS (Plastic Limit / Liquid Limit; Water Content; 10 20 30 40 50)
0	GW-GRAVEL.								
	SM-silty SAND:Light brown to orange, moist, loose.								
1.5									
					36640		cal		
3									
	Medium to dense, Qc = 25.				36643 36642 36641		bkt bag cal		
4.5									
6									
7.5									
	Water seepage, end of test pit @ 8.5'.								
9									
10.5									

This information pertains only to this boring and should not be interpreted as being indicitive of the site.

Figure PAGE 1 of 1 **Western Testing, LLC**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
			0.9	4.6	62.0	30.3	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)

* (no specification provided)

Material Description

SM-silty SAND:Light brown to orange, moist, loose.

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.3314 D_{85}= 0.2877 D_{60}= 0.2039
D_{50}= 0.1804 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= AASHTO=

Remarks

TEST RAN PER AASHTO TM-27 BY JBE ON 1/28/25

Source of Sample: TP1 **Depth:** 4
Sample Number: 36643

Date: 1/28/25

Western Testing, LLC Roseburg, Oregon	**Client:** PEI **Project:** 1190 BEACH LOOP **Project No:** 61194 **Figure**

Tested By: JBE

DRILL HOLE LOG

TP2

File: 61194 Date Printed: 1/26/2025

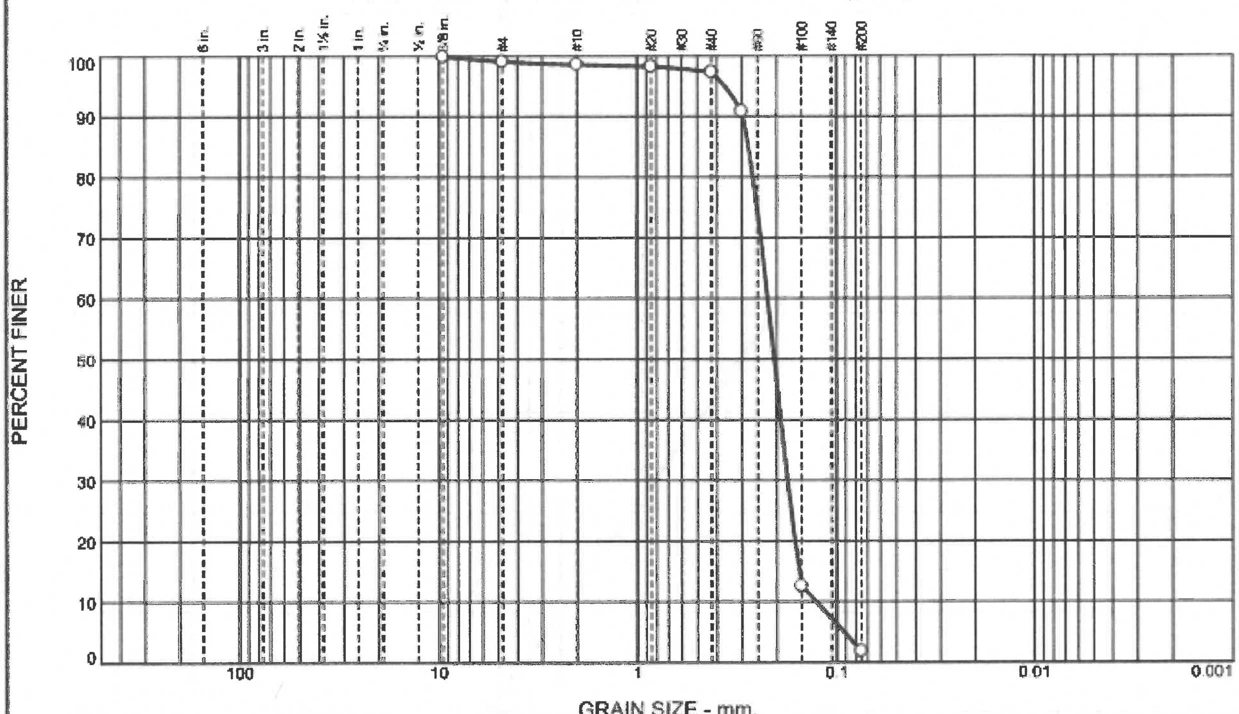
PROJECT: 1190 BEACH LOOP	PROJECT NO.: 61194
CLIENT: PEI	DATE: 1/17/25
LOCATION: 43.11259, -124.43216	ELEVATION: 92'
DRILLER: TWS	LOGGED BY: JBE
DRILLING METHOD: TAKEUCHI 235 18" BKT	
DEPTH TO - WATER > INITIAL:	AFTER DRILLING:	SEEPAGE:

DEPTH (feet)	Description	Recov (in)	Driven (in)	PID ppm	Sample#	Soil Type	Sampler	Symbol	TEST RESULTS (Plastic Limit / Liquid Limit; Water Content; 10 20 30 40 50)
0	GW-Gravel.								
	SP-SAND: brown to orange, moist . loose.				36644 36645		cal bag		
1.5									
	SM-silty SAND:Light brown to orange . moist. loose.								
3									
	Medium to dense. Qc = 30.				36646		cal		
4.5									
6									
7.5									
9									
10.5	End of test pit @ 10.5'.								

This information pertains only to this boring and should not be interpreted as being indicative of the site.

Figure PAGE 1 of 1 **Western Testing, LLC**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
			0.5	1.3	95.5	1.8	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)

* (no specification provided)

Material Description

SP- SAND:Light brown to orange , moist, loose.

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.2961 D_{85}= 0.2775 D_{60}= 0.2223
D_{50}= 0.2061 D_{30}= 0.1767 D_{15}= 0.1541
D_{10}= 0.1270 C_u= 1.75 C_c= 1.11

Classification

USCS= SP AASHTO=

Remarks

TEST RAN PER AASHTO TM-27 BY JBE ON 1/28/25

Source of Sample: TP2 **Depth:** 2
Sample Number: 36645

Date: 1/28/25

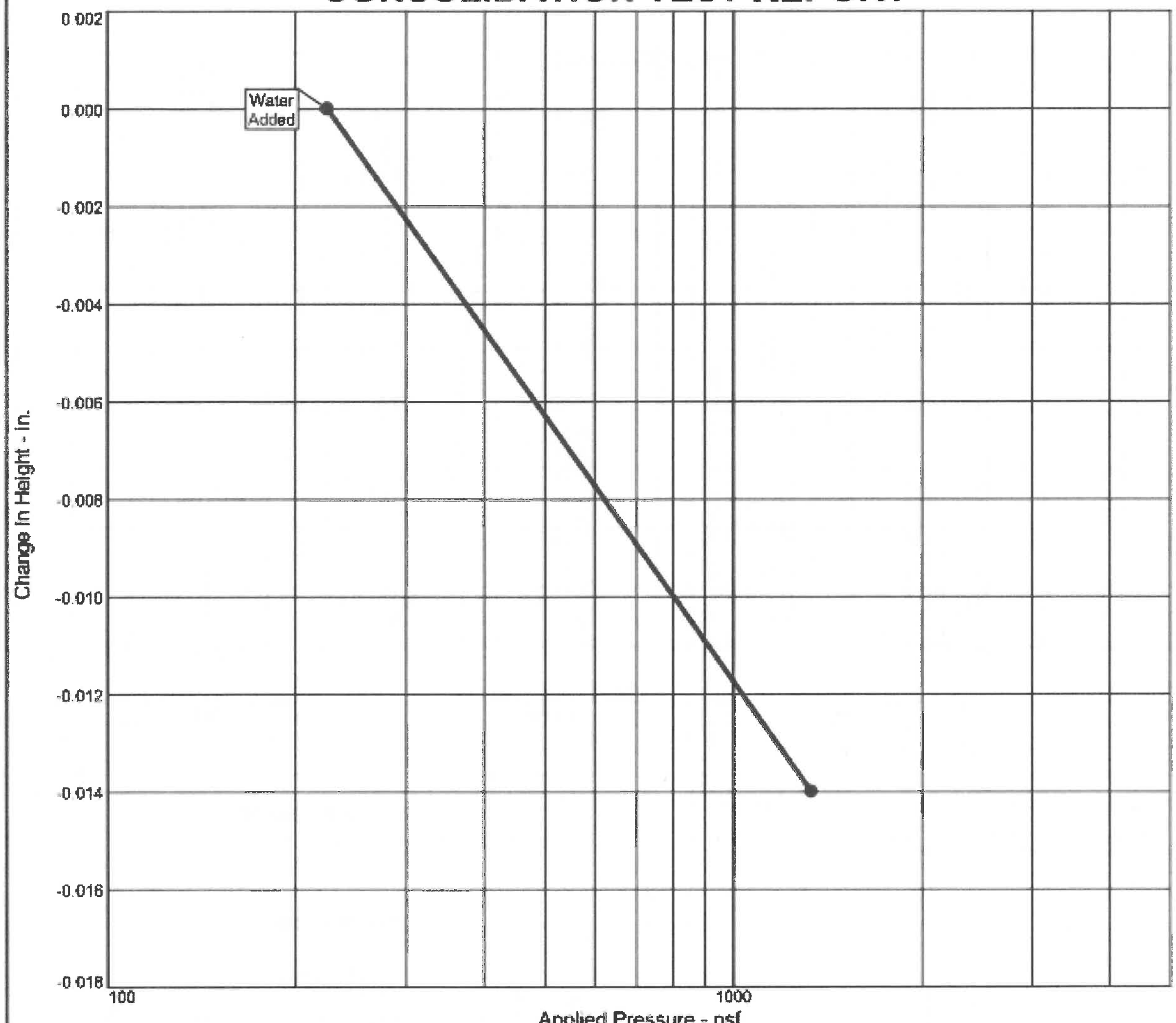
Western Testing, LLC

Roseburg, Oregon

Client: PEI
Project: 1190 BEACH LOOP
Project No: 61194 **Figure**

Tested By: JBE

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (psf)	P_c (psf)	C_c	C_r	Swell Press. (psf)	Swell %	e_o
Sat.	Moist.											
32.0 %	11.2 %	79.3			2.8	317.2					0.0	0.982

MATERIAL DESCRIPTION	USCS	AASHTO
SM-silty SAND:Light brown to orange , moist, loose		

Project No. 61194 **Client:** PEI
Project: 1190 BEACH LOOP

Source of Sample: TP2 **Depth:** 4 **Sample Number:** 36646

Western Testing, LLC

Roseburg, Oregon

Remarks:
TEST RAN PER D2345 ON 1/23/25 BY JBE.

Figure

Tested By: JBE

DRILL HOLE LOG

TP3

File: 61194 Date Printed: 2/12/2025

PROJECT: 1190 BEACH LOOP **PROJECT NO.:** 61194
CLIENT: PEI **DATE:** 1/17/25
LOCATION: 43.11243, -124.4323 **ELEVATION:** 88'
DRILLER: JBE **LOGGED BY:** TWS
DRILLING METHOD: TAKEUCHI 235 18" BKT
DEPTH TO - WATER> INITIAL: ___ **AFTER DRILLING:** ___ **SEEPAGE:** 10.5'

DEPTH (feet)	Description	Recov (in)	Driven (in)	PID ppm	Sample#	Soil Type	Sampler	Symbol	TEST RESULTS: Plastic Limit ├──┤ Liquid Limit; Water Content - ● (10 20 30 40 50)
0	GRASS/ROOTZONE								
	SP-SAND:Brown to orange, moist, loose.								
1.5									
3	SM- silty SAND light brown to orange, moist, medium to dense. qc = 26.				36647		cal		●
					36648		cal		
4.5									
6									
7.5					36649		bag		
9									
10.5	Water seepage, end of test pit @ 10.5'.								

This information pertains only to this boring and should not be interpreted as being indicitive of the site.

Figure PAGE 1 of 1 **Western Testing, LLC**

Western Testing, LLC
Construction Materials Testing
Quality Results. Always.

Roseburg Office
4276 Old Hwy 99 South
Roseburg, OR 97471
Ph: (541) 957-1233

NATURAL MOISTURE DENSITY REPORT

PROJECT: 1190 Beach Loop **PROJECT NO:** 61194

CONTRACTOR: **DATE:** 1/20/25

SUBJECT: NMD

Sun	Mon	Tues	Wed	Thurs	Fri	Sat

Tested By: TJB Testing Date: 01/20/25

BORE HOLE	TP3-3'							
SAMPLE NO.	36647							
LENGTH 1 (in.)	4							
LENGTH 2 (in.)	4							
LENGTH 3 (in.)	4							
AVG LENGTH (in.)	*4.00*							
DIAMETER 1 (in.)	1.9							
DIAMETER 2 (in.)	1.9							
AVG DIAMETER (in.)	*1.90*							
VOLUME (ft^3)	0.01							
TARE (gram)	22.54							
WET + TARE (gram)	354.07							
DRY + TARE (gram)	321.6							
DRY WEIGHT (gram)	299.06	0	0	0	0	0	0	0
WATER (gram)	32.47	0	0	0	0	0	0	0
% MOISTURE	*10.9%*							
DENSITY (PCF)	*100.8*							

BORE HOLE								
SAMPLE NO.								
LENGTH 1 (in.)								
LENGTH 2 (in.)								
LENGTH 3 (in.)								
AVG LENGTH (in.)								
DIAMETER 1 (in.)								
DIAMETER 2 (in.)								
AVG DIAMETER (in.)								
VOLUME (ft^3)								
TARE (gram)								
WET + TARE (gram)								
DRY + TARE (gram)								
DRY WEIGHT (gram)	0	0	0	0	0	0	0	0
WATER (gram)	0	0	0	0	0	0	0	0
% MOISTURE								
DENSITY (PCF)								

REMARKS:

Reviewed By: Date: 6/9/23

* "Special Inspection", "Inspection" and "Inspector" are terms as defined by the International Building Code