GEOTECHNICAL STUDY AND REPORT
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MYRTLE POINT, OREGON

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A. EXECUTIVE SUMMARY

It is our opinion, supported by field investigations, laboratory tests and geotechnical analysis, that the existing and proposed site work, soils and geological conditions at the project site are suitable for the proposed structure, provided the recommendations of our report are incorporated during design and construction.

Special attention will be required during site preparation, construction of the building foundations and drainage features and other associated improvements. Subsequent sections of this report provide geotechnical recommendations for design and construction of the planned project.

- Local deposits of unsuitable soils may be encountered and would require excavation and disposal.
- **Construction Materials Engineering and Testing (CoMET) services of site cuts and fills, compaction testing and observation of construction of slopes and drainage features is recommended.**
- **CoMET of structural fill and MSE or other retaining walls is required.**
- **Review of site and foundation design by the geotechnical engineer is recommended prior to beginning construction.**

The following sections of this report provide geotechnical recommendations for design and construction of the planned project.

B. INTRODUCTION

B.1. Purpose and Scope

The property owner plans to construct a small single story addition to a home on the property. Construction of the addition is to consist of typical wood frame construction. Guerin Lane is located on a moderately to steeply sloped, generally westerly descending parcel located near the southern boundary of the City of Myrtle Point. Pinnacle Engineering, Inc. (PEI) has been engaged to prepare this report for use in site design and construction, to develop recommendations for embankment (cut and fill slope) construction and to recommend design parameters for foundations for the proposed structures.

Field investigations were conducted on 10 May 2019, and included a geologic reconnaissance of the site and immediate surrounding area, observation, sampling and testing of the underlying soils encountered in one test boring.

Soil samples were retrieved during site exploration for laboratory testing and other studies necessary to develop recommendations for design and construction of the foundations for the proposed structures, to evaluate potential complications that may occur during construction, to assess probable long term performance of the structures and for use in monitoring soil compaction.

B.2. Site and Project Description

The proposed site is located in Section 19, Township 27S, Range 5W, W.M., in Myrtle Point, Oregon. The site is bounded to the south by Guerin Lane and is otherwise surrounded by undeveloped property on other sides. The site is located within 1000 feet of the Coquille River.

The project will result in the addition to a small home.

B.3. Previous Geotechnical Report

PEI is not aware that a prior geotechnical study report has been prepared for the above referenced site. Accordingly, this report (PEI) should be regarded as a residential geotechnical study and recommendations, consistent with the complexity of the proposed site and residential structure.

C. TOPOGRAPHIC MAPPING

Development of topographic mapping was beyond the scope of this study. Site slope measurements were measured on site by other means, where required.

D. GEOLOGIC SITE CHARACTERIZATION

Geologic and geotechnical terms used in this report are defined in Figure 3. Surface geologic mapping of the site is presented as Figure 4.

D.1 Regional Geology

The project site is located approximately 60 miles east of the Cascadia Subduction Zone. The Cascadia Subduction Zone reflects subduction of the Juan de Fuca plate beneath the western edge of the North American continental shelf. [1]

D.2 Project Area Geology

The site is located within the Oregon Coast Range Geological province. Site soils consist of a thin layer of top soil overlying a thicker layer of silty clay. A layered matrix of residual turbidite sandstone and mudstone is intermixed and underlies the surface soils. The soil matrix overlies Early Eocene and late Paleocene Submarine Basalt flows at the project site.

	**Otter Point Formation (Jurassic) -** A tectonically sheared assemblage of rocks including pervasively sheared sedimentary rocks (Jop) now prone to regional mass \movement and subordinate amounts of sheared to intact volcanic rock (JOV), isolated blocks of thinly bedded tightly folded chert (Jc), exposures of serpentinite (Jsp), and isolated blocks of resistant blucschist (Js), a medium metamorphic rock. Soil types, thicknesses, and properties highly variable; major hazards include mass movement, slope erosion, stream bank erosion, and variable bearing strength.

Bulletin 87 – Environmental Geology of Western Coos and Douglas Counties, Oregon Geologic Map of the Coquille Quadrangle Oregon, 1975, R.E. Corcoran, State Geologist, Oregon department of Geology and Mineral Industries.

1 Geology of the Pacific Northwest, 1999, Orr, Elizabeth L, and William N., Kendall/Hunt Publishing Company.

D.3. Seismicity and Seismotectonic Considerations

Local faults generally trend from northeast to southwest, and include both normal and thrust type events. Inactive fault locations relative to the project site are depicted on Figure 4.

D.3.a. Area and Site Seismicity

Extensive seismotectonic studies continuing since 1990 have concluded that western Oregon is subject to a much greater likelihood of both random and plate-subduction seismic events of far greater magnitude and far more frequently than was formerly 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 5.75.

D.3.b. Site Stability

The soils underlying the project site are likely to be very stable during seismic events having a reasonable probability of occurrence. There is no likelihood of liquefaction, *tsunami* or *seiche*. Seismically induced landslides are possible but does not appear to be a concern.

D.3.c. Site Classification

The soil underlying the residential pad is consistent with Site Class C, as defined by the 2017 Oregon Residential Specialty Code (OSSC).

D.3.d. Seismic Refraction Survey

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

- Underlying the root zone, the low plasticity SILT material can be expected to transmit lateral accelerations typical of a lower velocity range of 800 to 1,200 ft/sec.
- Beneath the surface soils, the SILT grades to SANDSTONE, which can be expected to transmit lateral accelerations typical of a medium velocity range of 1,200 to 1,500 ft/sec.

Note, that areas of the site contain a thick layer of root zone and underlying organic soil that will require removal.

E. FIELD STUDIES

E.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.

E.2. Surface Hydrology

The subject site is located on a northerly trending margin of a moderately sloped hillside inclining north of Guerin Lane. The moderate slopes of the subject site can receive significant inflow from the slopes to the north and can facilitate a relatively rapid runoff of surface waters during rainfall events.

Cutting and filling activities will disturb the shallow perched water in a number of locations. This disturbance must be closely managed in order to avoid raising the phreatic surface within the embankment mass, possibly resulting in a decrease in stability.

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

E.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.

E.4. Site Exploration and Field Testing

Field investigations conducted on May 10, 2019 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 one test boring.

The test boring was advanced by use of a Little Beaver drill mounted on an all-terrain vehicle, which advanced 6 inch diameter continuous flight hollow stem auger at the location depicted on Figure 2. The test boring extended to a depth of 7 feet below ground surface. Soil samples were obtained using a split spoon sampler by method of Standard Penetration Test (SPT). Samples were sealed and stored for laboratory testing.

Samples were retrieved in the test boring at approximate 4 foot intervals and at visible soil horizon changes.. Bulk samples were retrieved at the depths and locations indicated on the test boring log.

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

The test boring was left unfilled for a brief time to allow groundwater levels to stabilize if present. Groundwater was not encountered in the test boring.

Please note that shear strengths and estimated bearing capacities 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 G to confirm applicability and that the designer's interpretation is consistent with our understanding of design properties.</u>

E.5. Geotechnical Characterization

Surface soils consist of high plasticity clayey SILT and extends to a depth of 3 feet below ground surface. Beneath the clayey SILT material, a layer of SILT containing traces of sand exists to a depth of approximately 7 feet below ground surface. Beneath the surface soils the SANDSTONE layer is estimated to be 10 feet below ground surface.

The shallow soils are compactible after removal of the vegetative component and may be used as non-structural or site fills if construction occurs during dry weather. The vegetative component is suitable for use as landscaping material or for sculpting wetlands mitigation areas.

The silt soils can be excavated with light to moderate effort by moderate energy excavation equipment. These materials should stand at relatively steep angles in shallow utility trenches. Deeper trenches will also stand at relatively steep angles initially, but seepage and wet weather combined with occasional weaker zones of soil increase the likelihood of sloughing.

Please note that 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.

E.6. Groundwater

Groundwater (the phreatic surface) was not encountered during the field investigation. 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 approximately 8 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.

E.7. Soil Permeability

Although permeability tests were not performed for this study, experience indicates that flow velocities within the native shallow soils 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 directly to weathered formational material. After compaction of the fills, permeability will likely decrease to range between 10^{-5} and 10^{-6} cm/sec. Where sandy layers exist, their permeability will be on the order of 10^{-3} cm/sec.

F. 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, physical characteristics, including bearing capacity, consolidation, unconfined compressive strength, natural moisture/density relationship, plasticity indices and sieve analyses were determined for the fine grained portion of all samples. Samples were then classified in conformance with the Unified Soil Classification System (USCS) per ASTM D-2487.

F.1. Soil Classification

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

Table F 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 per cent	L
Peat	Pt	w_H > 50 per cent	H

F.2. Electro-Chemical Parameters

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

F.3. Strength Parameters

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

Table F 2
Strength Parameters

Normal Load	Soil Type	Phi	Cohesion
500 psf	Low plasticity clayey SILT	30 degrees	120 #/ft^2
	Low plasticity sandy SILT	30 degrees	120 #/ft^2
	Imported ABC FILL @ 90% density per D 1557	33 degrees	0#/ft^2
3,000 psf	Low plasticity clayey SILT	34 degrees	360 #/ft^2
	Low plasticity sandy SILT	34 degrees	240 #/ft^2
	Imported ABC FILL @ 90% density per D 1557	37 degrees	200 #/ft^2

Note that the above values are based on historic, typically 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

F.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 F 3
Performance Parameters

Pressure	Consolidation (Swell)	Swell pressure	Location	Remarks
225 psf	(0.4%)	-	TB 1 @ 4 feet	Low plasticity tan SILT
1,500 psf	3.2%	-	TB 1 @4 feet	Low plasticity tan SILT

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

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

G. ENGINEERING STUDIES AND RECOMMENDATIONS

G.1. General

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

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 is assumed to consist of conventional light wood framing.

Pertinent geotechnical factors that may influence design and construction include;

- Control of both ground and surface water will be required during construction to facilitate constructability and during the life of the project to assure satisfactory long term performance.
- Stability of excavations during construction of all structures and trenches will require careful monitoring by the contractor.

G.2. Site Preparation and Grading

G.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 6 to 8 inches. The removed material will consist of root zone.

PEI should be contacted to verify suitable subgrade.

G.2.b Removal of Unsuitable Soil

The top 24 inches of soil should be removed beneath the footings and replaced with a minimum of 12" of structural backfill.

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 Engineer.

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

After stripping, 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 be replaced with imported structural fill. Effective re-compaction of the fine grained soil will require moisture conditioning and will require less effort if compacted with a pneumatic or static sheepsfoot roller. Moisture conditioning and re-compaction beneath pavement or slabs should extend to a depth of between 10 and 12 inches. The re-compaction 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 re-compaction is not required. We should be contacted to perform *in situ* strength tests of subgrade soils and to advise regarding moisture conditioning and compaction.

G.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, and specifically includes all site fills more than 4 feet thick.

G.3.a. Structural Fill Materials

Structural fill should consist of a free-draining granular material with a maximum particle size of 8 inches or 2/3 of the un-compacted lift thickness, whichever is lesser. The material should be well graded with less than 5 percent 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.

G.3.a.1. SILT Fill Soil - Where natural or imported SILT soil will be used to construct the building pad, driveway embankment or yard, they should be placed and compacted at 2% above optimum moisture and thoroughly worked in order to create a homogeneous fill. Some shrinkage cracks and long-term creep will likely occur on the surface of these SILT fill slopes during the life of the project.

G.3.b. Structural Fill Placement

Structural fill should be placed in horizontal lifts not exceeding 12 inches loose thickness, or thinner if necessary to obtain specified density. Each lift should be compacted to 90% of the maximum density. The lift thickness may be increased if specified density is consistently being exceeded and approved by the Engineer.

In order to accomplish effective compaction for the full fill footprint, we recommend that fills be over built by five feet, then the face cut back to achieve the design fill face.

Structural fill placed beneath footings or other structural elements should be centered on the footing. Thickness of the structural fill will vary depending on the depth of suitable bearing conditions. The width of structural fill should be equal to the width of footing plus twice the depth of the structural fill beneath the footing.

G.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 at 2% to 4% over optimum and compacted to 88% of maximum density. Swell properties should be determined by laboratory testing prior to use as structural fill.

G.3.c.1. Fill Observation and Testing Methods - Field density testing by nuclear methods is appropriate for compaction of 2½ - inch to ¾ - inch minus crushed base rock, fine grained soils, decomposed granite and other materials 2½ 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.

G.3.d. Non-Structural Fill

All natural clayey SILT, 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 be utilized in berms less than 10 feet in height having slopes no steeper than 3 ½ H to1.0V.

G.4. Slopes

Temporary cut and low, permanent fill slopes will be required for construction of the site fill and structure building pads.

G.4.a. Cut Slopes

Permanent cut slopes will 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 may be excavated at steeper angles than listed above. The SILT soil 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
CLAY and SILT Soils	Temporary Cuts	1½ H to 1V
CLAY and SILT Soils	Permanent Cuts	2½ H to 1V
Tan SILT w/ SHALE	Temporary Cuts	1 H to 1V
Tan SILT w/ SHALE	Permanent Cuts	2 H to 1V
Intact SHALE	Temporary Cuts	½ H to 1V
Intact SHALE	Permanent Cuts	1 H to 1V

G.4.b. Fill Slopes

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

Table G 2 - Fill Slopes

Soil Classification	Type of Fill	Inclination
Fine grain soils (CLAY and SILT)	All	2 H to 1 V
Tan SILT w/ SHALE	All	1¾ H to 1V
Compacted, crushed base course	All	1½ H to 1V

All 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 is recommended to integrate each lift with the one below. Rubber-tired rollers can also achieve this result, but smooth-drum rollers should not be used. Care should be exercised when placing dried hard clay to avoid leaving voids within the fill mass, which voids may allow the soil to lose strength when wetted.

G.4.c. Slope Creep

It is likely that surface creep will occur at locations where the organic SILT soils are utilized to construct 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.

G.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 descending slopes and edge of footing should be 5 feet.

The minimum recommended separation between the ascending slopes and edge of footing should be 5 feet. Note that this is not a stability concern, but to provide access for future maintenance activities.

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. Engineer should be contacted to verify suitable setback prior to placement of footings.

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. PEI can provide a required depth for deepened footings upon request.

G.5. Paved Areas and Non-Structural Slabs on Grade

G.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 ½" 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 1 for soaked subgrade in fill areas to in excess of 20 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 2018 edition of the ODOT Standard Specifications for Highway Construction.

G.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.

G.6. Site Drainage and Erosion Control

G.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 10%. 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.

G.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.

G.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.

G.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.

G.6.e. Erosion Control

Site soils are moderately susceptible to erosion if unprotected. The site grades are such that erosion and sediment transport during construction are not 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 not be allowed to flow directly into streams or off-site drainage systems.

Typical project landscaping should be adequate for long-term erosion control.

G.7. Building Foundations

G.7.a. General

A combination of spread and continuous footings is recommended for residential structures. To compensate for swell pressures, footings should bear on non-swelling imported structural fill.

G.7.b. Spread Footings

G.7.b.1. Fill – See Section G.2 and G.3 of this report.

G.7.b.2. Footing Embedment - Spread footings 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.

G.7.b.3. Allowable Bearing Pressure – To calculate allowable bearing capacity, we assumed that the footing will be embedded 1 foot below the adjacent surface, yielding a D_f/B ratio of 0.75. Footings placed in this configuration may be designed for an allowable bearing pressure of 1,600 #/ft^2.

G.7.b.3.a Load Duration and Shape Increases – Allowable bearing pressure may be increased by one-third for short term loads. Allowable bearing pressures on square spread footings may be increased by 20%.

G.7.b.4. Minimum Dimensions - The minimum recommended width for continuous footings is 1'- 4" and the minimum recommended dimension for spread footings is 1'-6".

G.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 G.7.f.

G.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 up to 1 inch. Differential movement could be as much as 0.3 inches.

G.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;

G.7.e.1. Aggregate Base Course (ABC) - A 6 inch thick layer of clean (less than 2% passing the No. 200 sieve) ¾" 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 ¼ inch thickness of clean sand should be placed on top of the ABC.

G.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 ¼ inch thickness of clean sand should be placed on top of the membrane.

G.7.e.3. Minimum Slab Thickness - Minimum recommended slab thickness is 5 inches to allow sufficient cover over the reinforcing steel. **Note that all slabs should be designed for the actual use and equipment anticipated.**

G.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.

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

G.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.

G.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.

G.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.

G.7.f. Footing and Floor Drains

G.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.

G.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.

G.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.

G.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.***

G.8. Lateral Earth Pressures and Drainage

G.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 F.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.

G.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, soil 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. 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 0.2g 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.

G.9. Trenching and Piping

Additional underground piping will be constructed. Excavation can be accomplished by normal means above the rock elevations projected to underlie the site at an average depth of 2 to 6 feet. 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, 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.
- Beneath paved areas, full depth granular backfill is recommended as a minimum, and use of lean cement slurry should be considered.
- The top 12" of trench backfill should be compacted to a density of 92%. 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.

H. ADDITIONAL SERVICES AND LIMITATIONS OF REPORT

H.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.
- Attend project team meetings to clarify issues raised during the construction process.
- Review and/or design of swale, fill and basement subdrain systems.
- 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.
- Special Inspection as defined by the OSSC may be required for certain of the components.
- Periodic construction field reports, as requested by the client and required by the building department

H.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.

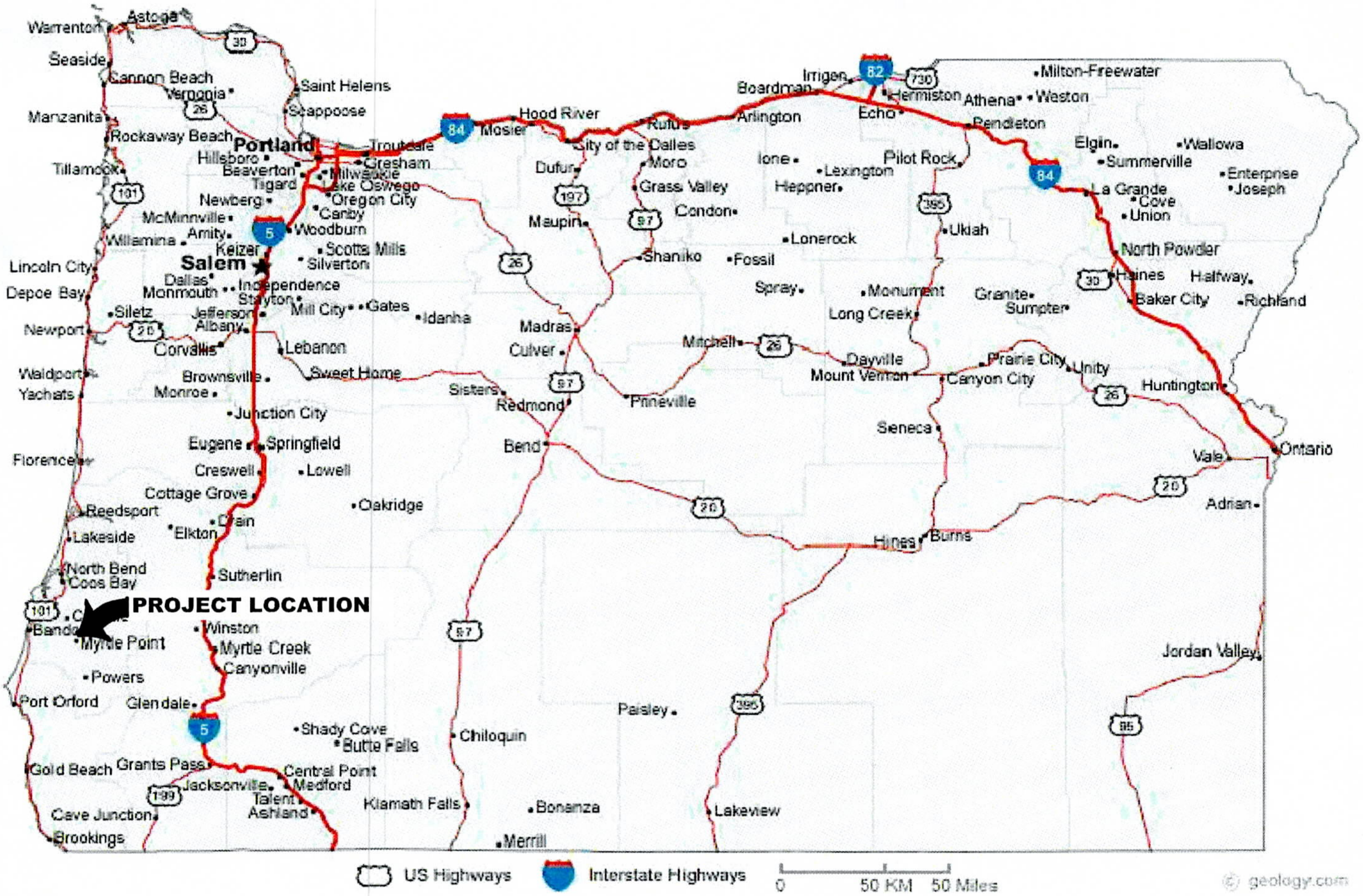
This report was prepared for the use of the owner and design team for the subject project. It is only for this site and construction project. No third party beneficiaries are intended. Potential users of the report should be so notified.

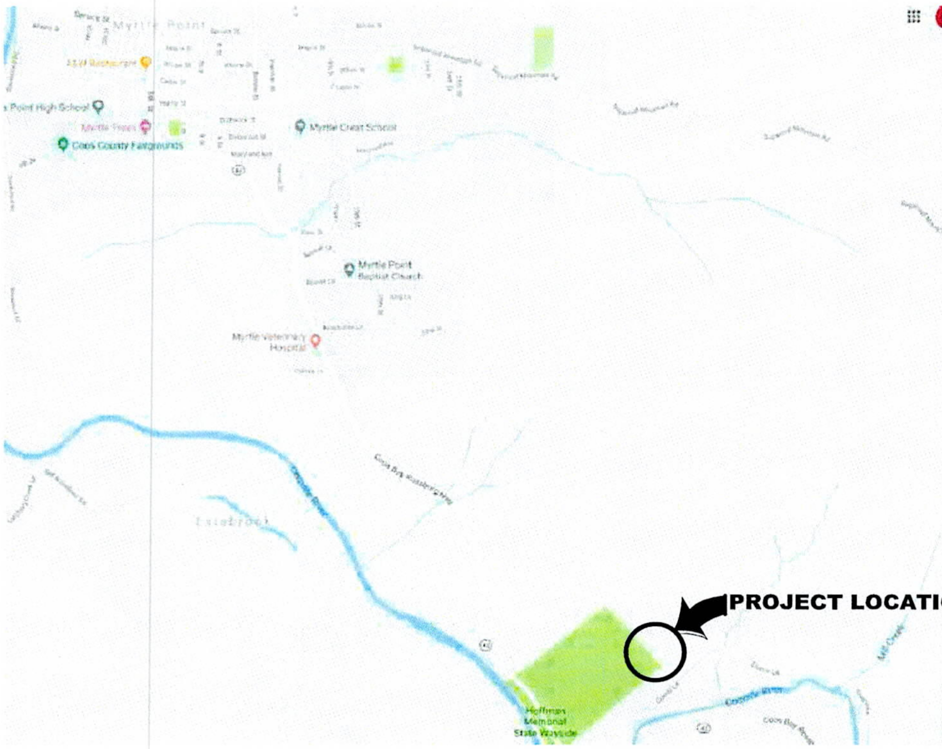
It should be made available to other contractors for information and factual data only, such as test boring or test pit logs, measured water levels, samples, sample classifications and laboratory test results. The report is interpretive in nature and shall not be used for contractual purposes, such as 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 pits and 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.

FIGURES





N.T.S.

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SUBSURFACE SOILS INVESTIGATION VICINITY MAP

PROJECT: 30112.03 - 95667 GUERIN LANE - GEOTECH

CLIENT: RON LAFRANCHI

FIG. 1

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SUBSURFACE SOILS INVESTIGATION SITE MAP

PROJECT: 30122.03 - 95667 GUERIN LANE - GEOTECH

CLIENT: RON LAFRANCHI

FIG. 2

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 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

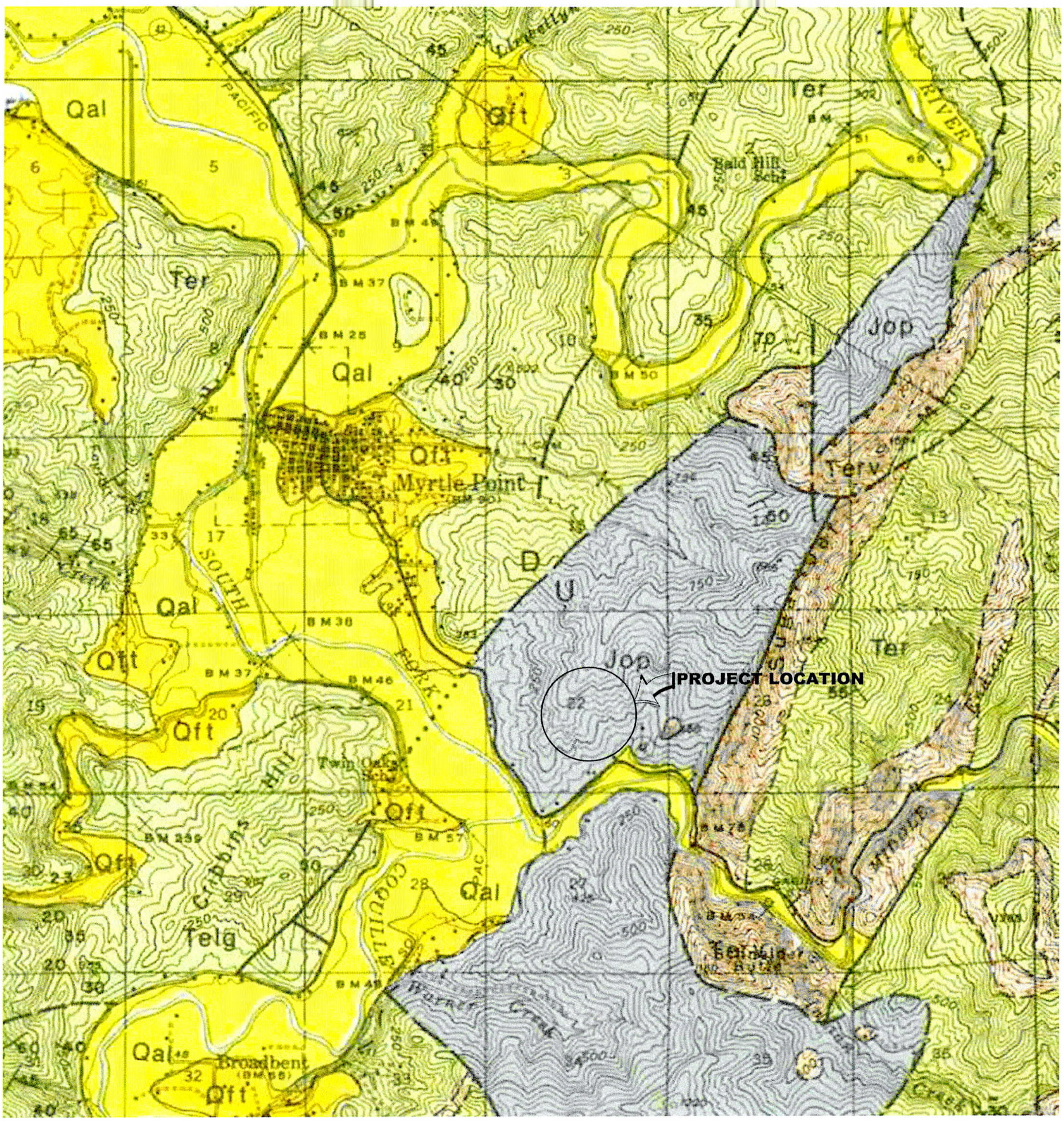
*N= Blows per foot in the Standard Penetration Test. In cohesive soils, with the 3 inch diameter sampler. 140-pound weight, divide the blow count by 1.2 to get N (Ref. 4).

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 MAP

PROJECT: 30122.03 - 95667 GUERIN LANE GEOTECH
CLIENT: RON LAFRANCHI

FIG. 4

APPENDIX A
TEST BORING LOG AND TESTS

TEST LOG

TB1

File: 60321 Date Printed: 5/16/2019

PROJECT: 95667 GUERIN LANE
PROJECT NO.: 60321.22
CLIENT: PINNACLE ENGINEERING, INC.
DATE: 5/10/19
LOCATION: 43°2'15"N, 124°6'39"W
ELEVATION: 114
DRILLER: TWS/TSK
LOGGED BY: TSK
DRILLING METHOD: RTV 900X MOUNTED DRILL WITH 4" AUGER
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	ROOT ZONE								
	MH-clayey SILT, dark brown to black, very moist to wet, medium stiff								
2					35055		BAG		
	ML-SILT w/ some rounded gravel and traces of sand, dark gray, moist, stiff				35056		BAG		
4	Qc=15	8	18		35057		SPT		3-6-7 (N=13)
6									
	very hard drilling Qc=17	6	18		35058		SPT		3-4-11 (N=15)
8	END TEST BORE AT 7 FEET. DRILL REFUSAL clayey SAND at bottom of hole								
10									
12									
14									

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

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Coast Office
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NATURAL MOISTURE DENSITY REPORT

PROJECT: 95667 GUERIN LN. MYRTLE POINT

PROJECT NO: 60321.2

CONTRACTOR: PEI

DATE: 5/13/19

SUBJECT: NATURAL MOISTURE AND DENSITY

Sun | Mon | Tues | Wed | Thurs | Fri | Sat

Tested By: TJB

Testing Date: 05/13/19

BORE HOLE	TB 1@ 4'							
SAMPLE NO.	35057							
LENGTH 1 (in.)	4.3							
LENGTH 2 (in.)	4.1							
LENGTH 3 (in.)	4.2							
AVG LENGTH (in.)	*4.20*							
DIAMETER 1 (in.)	1.9							
DIAMETER 2 (in.)	1.9							
AVG DIAMETER (in.)	*1.90*							
VOLUME (ft^3)	0.01							
TARE (gram)	32.5							
WET + TARE (gram)	401.7							
DRY + TARE (gram)	355.4							
DRY WEIGHT (gram)	322.9	0	0	0	0	0	0	0
WATER (gram)	46.3	0	0	0	0	0	0	0
% MOISTURE	*14.3%*							
DENSITY (PCF)	*103.6*							

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)								

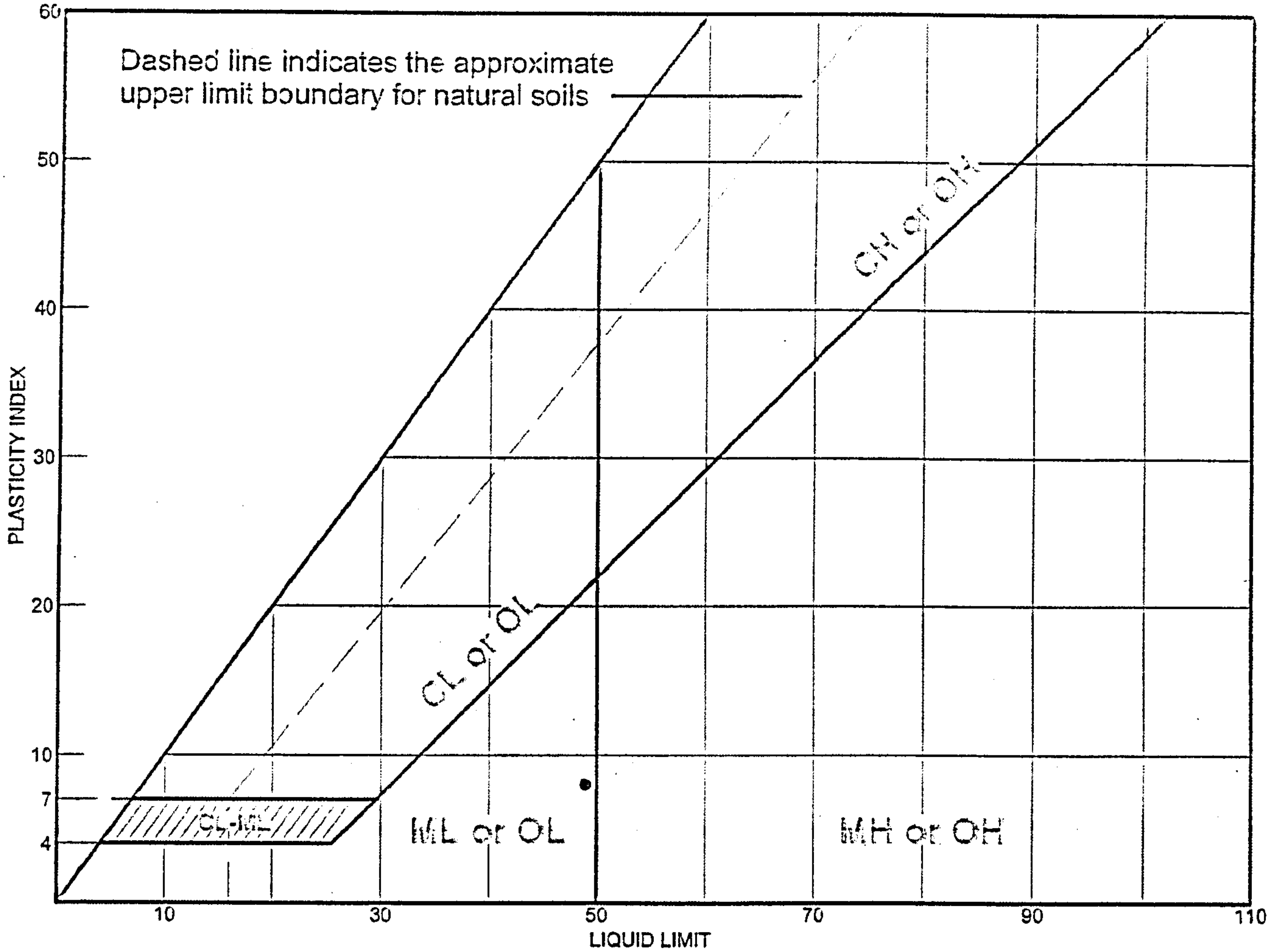
REMARKS:

Reviewed By:

Date:

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

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	ML-SILT w/ some rounded gravel and traces of sand, dark gray, moist, stiff	49	41	8			

Project No. 60321.22 **Client:** PINNACLE ENGINEERING, INC.

Project: 95667 GUERIN LANE

● **Source:** TB1 **Sample No.:** 35056 **Elev./Depth:** 3

Remarks:

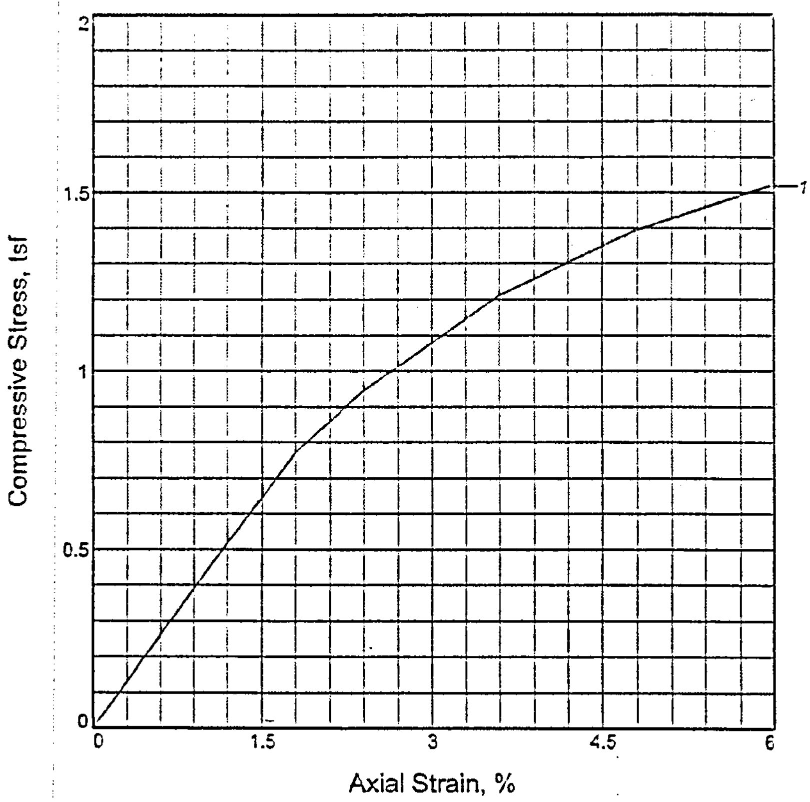
● TEST RAN BY TJB ON 5/15/19 PER ASTM D4318

Western Testing, LLC

Roseburg, Oregon

Figure

UNCONFINED COMPRESSION TEST



Sample No.	1			
Unconfined strength, tsf	1.519			
Undrained shear strength, tsf	0.760			
Failure strain, %	6.0			
Strain rate, in./min.	0.01			
Water content, %	14.3			
Wet density, pcf	118.0			
Dry density, pcf	103.2			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.90			
Specimen height, in.	4.20			
Height/diameter ratio	2.21			

Description: Qc=15

LL =	PL =	PI =	Assumed GS=	Type: UNDISTURBED

Project No.: 60321.22
Date Sampled:

Remarks:
TEST RAN BY TJB ON 5/13/19 PER ASTM D2166. SAMPLE FAILURE AT .276 INCHES.

Figure ______

Client: PINNACLE ENGINEERING, INC.

Project: 95667 GUERIN LANE

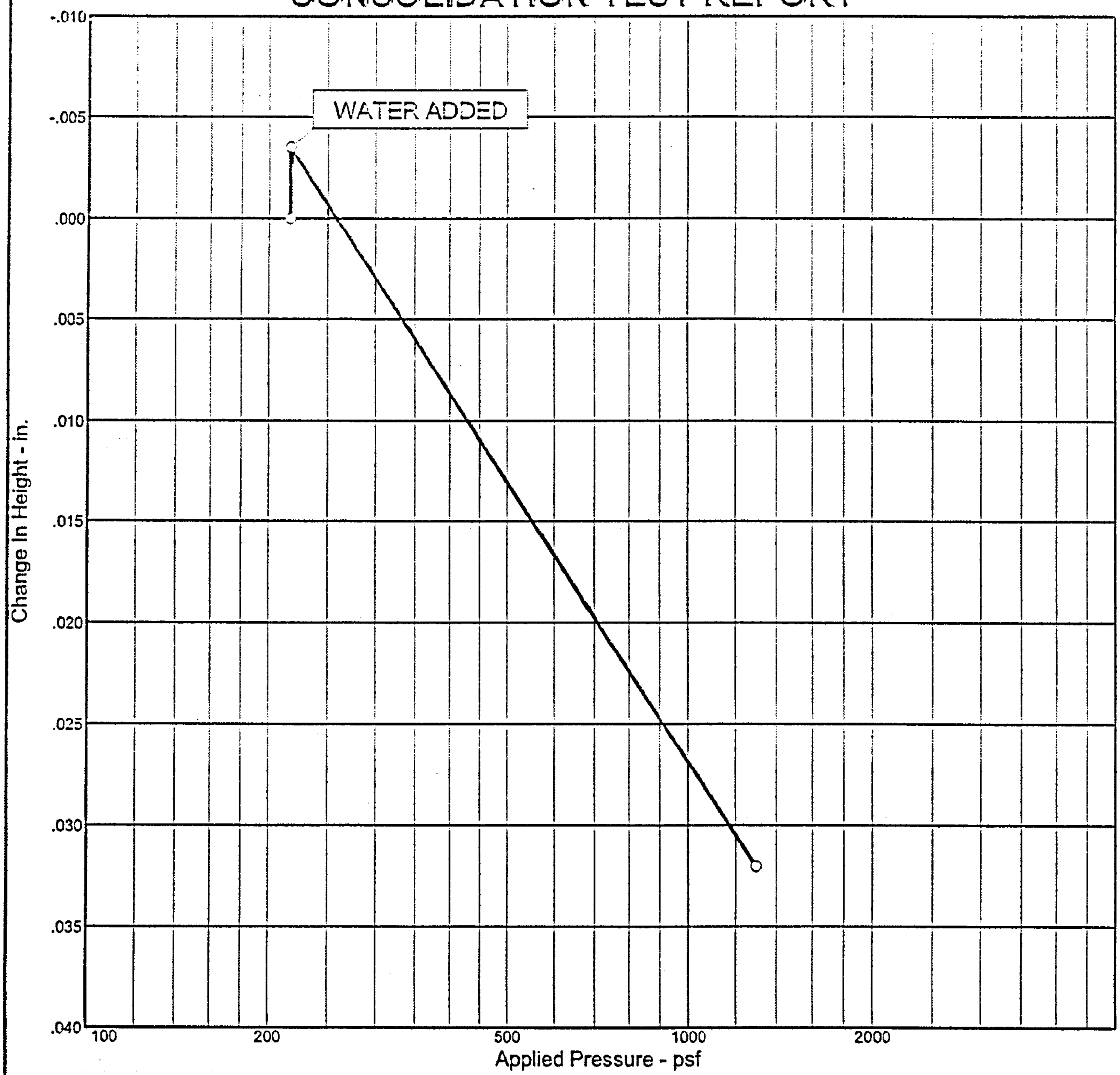
Source of Sample: TB1 **Depth:** 4
Sample Number: 35057

UNCONFINED COMPRESSION TEST
Western Testing, LLC
Roseburg, Oregon

Test ran per ASTM D 2166

Tested By: TJB

CONSOLIDATION TEST REPORT



Natural Sat.	Natural Moist.	Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (psf)	P_c (psf)	C_c	C_r	Swell Press. (psf)	Swell %	e_o
50.3 %	10.2 %	111.3			2.8	414				260	0.4	0.570

MATERIAL DESCRIPTION	USCS	AASHTO
ML-SILT w/ some rounded gravel and traces of sand, dark gray, moist, stiff Qc=15		

Project No. 60321.22 **Client:** PINNACLE ENGINEERING, INC.

Project: 95667 GUERIN LANE

Source: TB1 **Sample No.:** 35057 **Elev./Depth:** 4

Western Testing, LLC

Roseburg, Oregon

Remarks:

TEST RAN BY TSK ON 5/15/19
PER ASTM D2435 MODIFIED.
PP=1.5tsf, TV=4tsf, pH=8.7

Figure