

TALL TREE
ENGINEERING
CIVIL & GEOTECHNICAL

PRELIMINARY GEOTECHNICAL REPORT

Pyle Cabin

159/161 Cove View Drive
Shelter Cove, CA 95589

Date: July 14, 2025

Prepared For: Christina Pyle
100 Helmich St.
McCall, Id 83638

Prepared By: Dustin S. Fredricey, PE
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99 South Main Street
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Attachments:

Figure 1 – Boring Location Plan
Figure 2 - Geologic Map
Figure 3 - Boring Log (BH1)
Figure 4 - Boring Log (BH2)
Figure 5 - Boring Log (BH3)
Figure 6 - Soil Classification Chart
Figure 7 - Bedrock Classification Chart
Figure 8 - Drilled Pier Subdrain Detail

Dear Ms. Pyle,

This report presents the results of a geotechnical investigation completed by Tall Tree Engineering, Inc. for a proposed new residence located at 159 and 161 Cove View Drive in Shelter Cove, CA and corresponding with latitudinal and longitudinal coordinates of 40.027717 and -124.061563, respectively. This report was prepared for the sole use of you and your design consultants for the project specified herein. The conclusions and recommendations presented in this report are based on a review of historic records, data obtained from our field investigation and results of laboratory testing performed on select soil samples collected from exploratory boreholes.

Structural loading information was not available at the time of this report. For our analysis, we assume that dead plus live continuous wall loads will be less than two kips per lineal foot and dead plus live isolated column loads less than 50 kips. If these assumed loads vary significantly from the actual loads, we should be consulted to review the actual loading conditions and, if necessary, revise the recommendations of this report.

Based on the results of our investigation, it is our opinion that the project is feasible from a geotechnical standpoint provided the recommendations and criteria presented in this report are incorporated in to the design and implemented throughout construction.

1.0 INTRODUCTION

The purpose of our investigation was to evaluate subsurface conditions and provide geotechnical recommendations for the proposed new residence. As outlined in our proposal dated January 29, 2025 our scope of our services was limited to:

- reviewing readily available documents regarding site geology and potential geologic hazards,
- exploring subsurface conditions,
- performing testing of select soil/rock samples, as appropriate, and
- performing the necessary engineering evaluations to develop geotechnical recommendations for the proposed foundation

1.1 Project History

The site has been investigated several times over the past 35 five years. In 1990 an engineering geology report was prepared by LACO Associates. Follow up investigations were performed by Busch Geotechnical Consultants (BGC) in 2005 (BGC, 2006).

A test pit was excavated as part of the investigation by LACO in 1990. The test pit was excavated to a depth of 11 feet and was described as a loose to "firm" silty clay with angular rock fragments. The

percent of rock fragments were listed as approximately 30% at 2.5 feet below ground surface (bgs) and increased with depth. Pocket penetrometer testing was performed during test pit excavation and are reported on the log at 2.0 tons per square feet (TSF) and 2.5 TSF in the upper two and a half feet. A setback distance of 40 feet was recommended based on an erosion rate of 1 ft/yr – the project was never developed.

The investigation performed by BGC in 2005 included a detailed inspection of site geomorphology, a qualitative review of geologic hazards (fault rupture, earthquake shaking and slope stability) and a topographic survey but did not include a subsurface investigation. A copy of the test pit excavation log from LACO was included in the report from BGC. Site soils were also classified through visual observation of exposed soils in the project vicinity. Potential failure planes of 35 and 40 degrees were projected on site cross sections and a hazard rating was given based on where the failure planes daylight the building area. A recommendation was provided for a shallow foundation with a 16-foot minimum setback distance but the project was never developed.

2.0 SITE EXPLORATION

Our exploration consisted of a geologic site reconnaissance, a review of geologic and topographic maps, a subsurface investigation and a review of previous project reports.

2.1 Field Investigation

On February 21, 2025, we explored subsurface conditions by advancing three geotechnical boreholes to depths of 26.0 (BH1), 20.0 (BH2) and 13.5 (BH3) feet below ground surface (bgs) at the approximate locations shown on Figure 1. Boreholes were advanced by Stapleton Engineering and Exploration using a limited access bobcat equipped with a 4-inch flight auger. A 2.43-inch inside diameter (ID) California Modified Sampler and a 1.375-inch ID Standard Penetration sampler were driven into the underlying soil at selected depths using a 140-lb hammer falling 30 inches to obtain an indication in the field of soil densities and to allow for visual examination of a portion of the soil column. Relatively undisturbed and disturbed samples were obtained from the exploratory boreholes. Soils encountered in the boreholes were logged and field classified in general accordance with ASTM standard D2488, Standard Practice for Description and Identification of Soils. Soil boring logs are presented in the Appendix (Figures 3-5). Our principal civil engineer (Dustin S. Fredricey, PE) and senior geotechnical engineer (Patrick J. Conway, GE, CE, CEG, PG) were on site to observe drilling operations, prepare geologic logs of encountered materials, obtain soil/rock samples for visual and laboratory classification and complete a review of site geomorphic features.

2.1.1 Pocket Penetrometer

Pocket penetrometer tests were performed on the clayey stratum during drilling operations. The test estimates the unconfined compressive strength by measuring the materials resistance to penetration using a calibrated, spring loaded, cylinder with a maximum capacity of 4.5 tons per square foot.

2.2 Laboratory Investigation

Relatively undisturbed and disturbed samples were transported from the field to the laboratory for testing and classification. Laboratory testing was carried out employing, whenever practical, the currently accepted test procedures of the American Society for Testing and Materials (ASTM). Our laboratory investigation included determining Atterberg Limits, natural water content and dry density of select samples. The results from the laboratory investigation are shown on the soil boring logs presented in the Appendix (Figures 3-5).

2.2.1 Atterberg Limits

Atterberg Limits were determined for samples of the clayey subsurface soils to estimate the range of moisture content over which these materials exhibit plasticity. Liquid and plastic limits were determined in general accordance with ASTM D4318. These values are used to classify the soils in accordance with United Soil Classification System and to evaluate expansion potential and approximate strength.

2.2.2 Natural Water Content and Dry Density

Natural water content was determined on selected relatively undisturbed and disturbed samples to evaluate variations in retained moisture of in-situ soils. The samples were visually classified and weighed to obtain wet weight. The wet densities were obtained by measuring the length of the collected sample and multiplying by the density factor of the sample tube. The samples were then dried, in accordance with ASTM D2216, for a period of 24 hours in an oven maintained at a temperature of 100° Celsius. After drying, the weight of each sample was determined and the moisture content calculated. The dry density was determined after adjusting for natural water content. The dry density tests evaluate variations in the materials density and saturation. The tests are useful for estimating relative soil strengths and for comparing densities and moisture contents of the in-place materials.

3.0 SITE CONDITIONS

The following sections outline the geologic setting, surface conditions and subsurface conditions encountered during the investigation.

3.1 Surface Conditions

The site is located in the western vicinity of two adjoining parcels (APN 111-211-010 and 111-211-011) having a combined area of approximately 0.33 acres. The parcels are situated along the eastern slope

of a north-south trending ridgeline. According to a USGS 7.5 Minute Quadrangle (Shelter Cove) the site is situated at approximately 420-feet above mean sea level and is located on mild to very steeply sloped terrain. The eastern third of the project area descends steeply to unnamed tributary draining directly to the Pacific Ocean. The property is accessed from a paved cul-de-sac at the end of Cove View Drive. Existing site vegetation consists of native grasses, poison oak and fir trees.

3.2 Geologic Setting

The Site is located within the Coast Ranges geomorphic province which has northwest trending ridges and valleys that run subparallel to the San Andreas Fault Zone. The province is bound on the east by the Great Valley and on the west by the Pacific Ocean. It extends north into Oregon and south to the Transverse Ranges in Ventura County (CGS, 2002). The coastal belt of the Franciscan Complex is a geologically complex region driven primarily by the San Andreas fault zone. The site has been mapped regionally Quaternary landslide (Qls). These landslide materials were deposited by mass wasting of King Range to the east (Maclaughin et al., 2000) and likely occurred at time when the climate was much wetter than current conditions. Below the mass wasting deposits are argillite and mélange of the Miocene to Late Cretaceous King Range terrane (krk1) (Busch, 2006).

3.3 Subsurface Conditions

Our investigation indicates that the site surface soils consist primarily of soft to stiff, low to medium plasticity clay (CL) with gravels extending up to 12 feet bgs. A highly weathered, thinly interbedded siltstone and claystone mélange was encountered below the clay surface soils to depths up to 26 feet bgs. The dry densities and moisture contents of the encountered materials were found to range from 86.9 pcf to 111.5 pcf and 14% to 25%, respectively. Free groundwater was not encountered in any of our boreholes. Soil boring logs are presented in the Appendix (Figures 3-4).

4.0 GEOLOGIC HAZARDS

Infrastructure in California is exposed to geologic hazards such as earthquake induced ground shaking, faulting, liquefaction, expansive soils, soil creep and slope instability.

4.1 Faulting and Earthquake Induced Ground Shaking

The site is located in a seismically active area that is likely to experience strong shaking from earthquakes during the expected lifetime of the project (Branum, et al., 2008). The closest mapped fault zone is the Shelter Cove section of the San Andreas Fault Zone. The site is located within the mapped Alquist-Priolo Zone; however, the Shelter Cove Subdivision was created prior to the adoption of the Alquist-Priolo Act in Humboldt County and the proposed project is expected to be exempt from the requirements of a site-specific fault trench study. Traces of the San Andreas fault system were

mapped in the project vicinity during the 1906 San Francisco Earthquake. The closest of which, an en echelon trace, was mapped cross-cutting the ridge about 200 feet to the north of the project site. North-South trending fault lines have been mapped, roughly following the drainage courses that bound the ridge to the west (300ft) and east (200ft) (Bryant and Lundberg, 2002). The risks associated with faulting and earthquake ground shaking are moderate to high.

4.2 Liquefaction

Liquefaction is a phenomenon in which loosely deposited granular soils generally less than 50-feet below ground surface, with silt and clay contents of less than 35 percent and non-plastic silts located below the groundwater table, undergo rapid loss of shear strength when subjected to strong earthquake induced ground shaking. Ground shaking of sufficient duration results in the loss of grain-to-grain contact due to a rapid rise in pore water pressure thus causing the soil to behave as a fluid for a short period of time. Factors known to influence liquefaction potential include composition and thickness of soil layers, grain size distribution, relative density, groundwater level, degree of saturation, and both intensity and duration of ground shaking (Kramer and Stewart, 2024). The primary indicators for liquefaction potential are the presence of low-density granular soils and a high groundwater table, neither of which were observed during our investigation. Liquefaction potential for site soils during extreme earthquake events is considered low.

4.3 Expansive Soils

Expansive soils tend to undergo volume changes (shrink and swell) with changes in moisture content. Our investigation revealed soils with low expansion potential ($PI=14.8$). Provided the recommendations found in this report are followed, the risks associated with expansive soil and/or bedrock is considered low.

4.4 Slope Instability

Soils in northern California are subject slope instability risks due to creep and ground movement. Creep is a slow, seasonal, downslope movement of upper soils, on or near slopes, resulting from the combination of gravity forces and expansion/contraction of the upper soils due to seasonal variations in moisture content. Our investigation revealed a potential creep zone containing soft compressible clays with trace organics to depths up to five feet below ground surface. Slope instability risk is increased by high moisture content and can be exacerbated by erosion (raveling, rilling, etc.). Much of the fine-grained site soils and soils in the greater project vicinity are inherently subject to minor erosion. The risks associated with slope instability are moderate to high.

5.0 CONCLUSIONS

The results of our exploration and analysis indicate the project is feasible from a geotechnical engineering standpoint provided the recommendations contained in this report are incorporated into design and implemented throughout construction. The primary geotechnical constraints identified during this study include the presence of over steepened slopes, proximity to known fault systems and potential for strong earthquake ground shaking. The site is known to be subject to inherent risks that cannot be fully mitigated and it is imperative the owners acknowledge the presence of these risks.

The proposed residence will be supported on drilled cast-in-place concrete piers and a setback back distance of at least 20-feet should be maintained from descending slopes.

6.0 RECOMMENDATIONS

The following sections outline our recommendations for the proposed building foundation and site improvements.

6.1 Earthwork and Grading

We anticipate minimal site grading activities will be required to meet project design grades. We anticipate cuts of approximately three feet and less to provide positive gradients for site drainage. Temporary cut slopes up to six feet and less are anticipated. Based on subsurface findings we generally recommend that temporary cut slopes should not exceed one-half horizontal to one vertical (1/2H:1V). However, steeper temporary cut slopes may be considered acceptable depending on conditions encountered during construction. Temporary cut slopes should not be left exposed longer than absolutely necessary. All site preparation should be observed by a representative of Tall Tree Engineering, Inc. or other qualified geotechnical professional. Excavated material should not be stockpiled on site and should be removed from the project area as soon as practical after excavation.

6.2 Foundation: Drilled Cast-In-Place Concrete Piers

We judge that the proposed residence can be supported on drilled cast-in-place concrete piers constructed a **minimum distance of 20 feet** from descending slopes provided the following recommendations are incorporated into the design and implemented throughout construction.

6.2.1 Vertical Loads

The new residence should be supported on drilled, cast-in-place concrete piers a minimum of 18-inches in diameter and spaced at least three pier diameters center to center. The piers will derive their support

through peripheral skin friction. Piers should extend **at least 20 feet** below the existing ground surface, regardless of structural loads.

All perimeter piers and piers supporting continuous loads should be tied together with grade or tie beams. The grade beams should be designed to span between the piers in accordance with the structural requirements. The tie or grade beams should not be spaced greater than 20 feet apart in both directions and designed to resist stresses imposed by creep forces.

The portion of the piers extending at least five feet beneath the finished ground surface may be designed using an allowable dead plus live skin friction of **500 pounds per square foot**. This value may be increased by one-third for short duration wind and seismic loads. End bearing should be neglected because of the difficulty of cleaning out small diameter pier holes and the uncertainty of mobilizing end bearing and skin friction simultaneously. A value equal to one-half the vertical capacity of the pier may be used to resist uplift forces.

6.2.2 Lateral Loads

Lateral loads resulting from wind, lateral soil pressures or earthquakes can be resisted by the piers through a combination of cantilever action and passive resistance of the soils surrounding the pier. A passive equivalent fluid pressure of **250 pounds per square foot per foot of depth (psf/ft)** acting on two pier diameters should be used. The upper five feet should be neglected for passive resistance.

The piers should be designed and reinforced to resist lateral pressures due to soil creep. The soil creep pressures should be equal to **65 psf/ft** acting on two pier diameters. The height of the creep zone should be considered to be **five** feet.

Table 2. Geotechnical Design Criteria.

Allowable Skin Friction (Dead + Live) ^{1,2}	500 psf
Allowable Lateral Passive Resistance ^{1,2,3}	250 psf/ft
Maximum Limit of Allowable Lateral Passive Pressure at Depth	2,000 psf
Minimum Pier Depth Below Lowest Adjacent Soil Grade	20 feet
Minimum Pier Diameter	18 inches
Minimum Pier Spacing	3 Pier Diameters
Lateral Soil Creep ³	65 psf/ft
Soil Creep Zone	5 feet

¹ Value may be increased by one-third for Dead + Live + Short Term Dynamic Loads (wind and seismic).

² Neglect upper five feet of soil.

³ Acting on two pier diameters.

6.2.3 Settlement

Provided the recommendations contained in this report are followed, the total and differential settlements are estimated to be small and within tolerable limits. The total settlement will vary depending on the final geometry of foundation elements and the actual load supported. Foundation settlements have been estimated based on assumed foundation loads and geotechnical design criteria provided.

6.3 Retaining Walls

Retaining walls free to rotate on top should be designed to resist active lateral pressures. If walls are restrained by rigid elements to prevent rotation or to support compacted engineered fill they should be designed for “at rest” lateral earth pressures. A summary of recommended earth equivalent fluid pressures for retaining walls with level backfill are provided in Table 3.

Table 3. *Lateral Soil Loading for Retaining Walls.*

Active Pressure	65 psf/ft
At-Rest Pressure	85 psf/ft

We recommend that a backdrain be provided behind all retaining walls or that the walls be designed for full hydrostatic pressures. The backdrains should consist of four-inch diameter SDR-35 perforated pipe sloped to drain to outlets by gravity, and of clean, free-draining, three-quarter to one and one-half inch crushed rock or gravel. The crushed rock or gravel should extend 12 inches horizontally from the back face of the wall and extend to one foot below the finished ground surface. A Mirafi 140N filter cloth should be placed between the on-site native material and the drain rock to prevent clogging. The upper 12 inches should be backfilled with compacted fine-grained soil to exclude surface water. For retaining walls taller than six feet, a horizontal pseudostatic force acting upon the retaining wall during a seismic event should be considered.

6.4 Concrete Slabs-on-Grade

We anticipate both structural and non-structural slabs-on-grade will be used.

6.4.1 Structural

Slab-on-grade floors should be at least six inches thick and reinforced with bars to reduce cracking and reduce spreading of cracks that do appear. The floor slabs should be structurally designed and should be underlain with a 15 mil vapor barrier and 4-inch thick capillary moisture break consisting of free-draining clean angular gravel or crushed rock between ¼- and ¾-inches. We recommend that gravel be placed as soon as possible after preparation of the subgrade to prevent drying of the subgrade. If the subgrade is allowed to dry out prior to slab-on-grade construction, the subgrade soil should be thoroughly moisture conditioned before slab on grade construction.

6.4.2 Non-structural

We anticipate that concrete slabs will be used for exterior flatwork. Non-structural slabs-on-grade should be supported on a blanket of low to non-expansive engineered fill at least 18 inches thick that extends at least three feet beyond the perimeter of the exterior slabs. Non structural concrete slabs-on-grade should be cast and maintained separate from adjacent foundations. Concrete slabs-on-grade for exterior flatwork should not be placed within the project slope setback.

6.5 Seismic Design

The minimum design criteria presented herein (Table 3) is based on the 2022 edition of the California Building Code (CBC) and American Society of Civil Engineers (ASCE) Standard 7-16 ASCE/SEI, Minimum Design Loads and Associated Criteria for Buildings and Other Structures.

Table 4. Minimum Seismic Design Criteria. *

Site Class: D	
Mapped Acceleration Parameters	$S_S=2.188$ $S_1=0.91$
Spectral Response Acceleration Parameters	$S_{MS}=2.188$ $S_{M1}=N/A$
Design Spectral Acceleration Parameters	$S_{DS}=1.459$ $S_{D1}=N/A$

*Assumes the seismic response coefficient is determined as outlined in ASCE 7-16, 11.4.8 (Exception 2).

6.6 Site Drainage

Poor surface and/or subsurface drainage can often lead to foundation problems. Care should be taken to intercept and divert concentrated surface flows and subsurface seepage away from the foundation

elements and surface slopes. Surface runoff should be controlled and diverted to the west towards Cove View Drive and not allowed to run over the steep slopes to the east. Downspouts should not be discharged adjacent to foundations but rather connected in closed conduits and discharged away from the structure and slopes.

6.7 Construction Considerations

Our investigation did not reveal free groundwater in any of the boreholes. However, it is well known that localized groundwater levels can vary widely in northern California. To minimize costs construction should be completed when groundwater levels are lowest, typically in summer or early fall. If the pier excavations encounter water or if water has accumulated in the piers, the concrete may be placed by tremie method.

The bottom of the pier excavations should be reasonably free of loose cuttings and soil/mud debris prior to installing reinforcing steel and placing concrete. All pier holes should be securely covered after drilling to minimize subsequent cleaning and for safety reasons. We do not anticipate the need for pier casings as our investigation did not reveal caving soils. The drilling contractor should review this report so he/she may select suitable equipment and determine the need for casing or dewatering.

7.0 FUTURE GEOTECHNICAL SERVICES

To check for conformance with the recommendations contained within this report we should be retained to perform the following:

- Review project plans and specifications upon completion to determine that the design is consistent with the recommendations of this report.
- Observe site earthwork and pier drilling operations, to review the actual conditions exposed, and provide modifications in the field, if necessary.
- Provide recommendations for subdrainage installations at the time of construction.

These services and associated fees are not included in our current scope of services. We can provide a scope and fee estimate for these services when construction documents are finalized and project construction schedules are known.

8.0 REFERENCES

American Society of Civil Engineers (ASCE), 2024, ASCE 7 Hazard Tool. Accessed July 09, 2024.
<https://asce7hazardtool.online/>

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9.0 LIMITATIONS

This report has been prepared for the exclusive use of Ms. Christina Pyle, her contractors and consultants, and appropriate public authorities for the specific development described in this document. We have exercised a standard of care equal to that generated for this industry to ensure that the information contained in this report is current and accurate. Any alteration, unauthorized distribution, or deviation from this description will invalidate this report. The data presented should not be utilized by any third-party to represent data for any other time or location, and we assume no responsibility for any third-party reliance on the data presented.

Data generated for this report represents information gathered at the time and locations indicated. Subsurface conditions may change with time and under natural and anthropogenic influences. As such, the recommendations included in this report are based, in part, on assumptions about subsurface conditions that may only be observed and/or tested during subsequent project earthwork. Accordingly, the validity of these recommendations is contingent upon review of the subsurface conditions exposed during construction in order to verify that they are consistent with the conditions characterized in this report. Upon request, we can discuss the extent and fee for the observations and tests required to check the validity of the recommendations presented in this report during construction.

In addition, changes in applicable standards of practice can occur, whether from legislation or the broadening of knowledge. Accordingly, the opinions presented in the report may be invalidated, wholly or partially, by changes outside our control. Therefore, this report is subject to review and should not be relied upon after a period of three years, nor should it be used, or is it applicable, for any property other than that evaluated.

10. APPENDIX

Figures

Figure 1 – Boring Location Plan

Figure 2 - Geologic Map

Figure 3 - Boring Log (BH1)

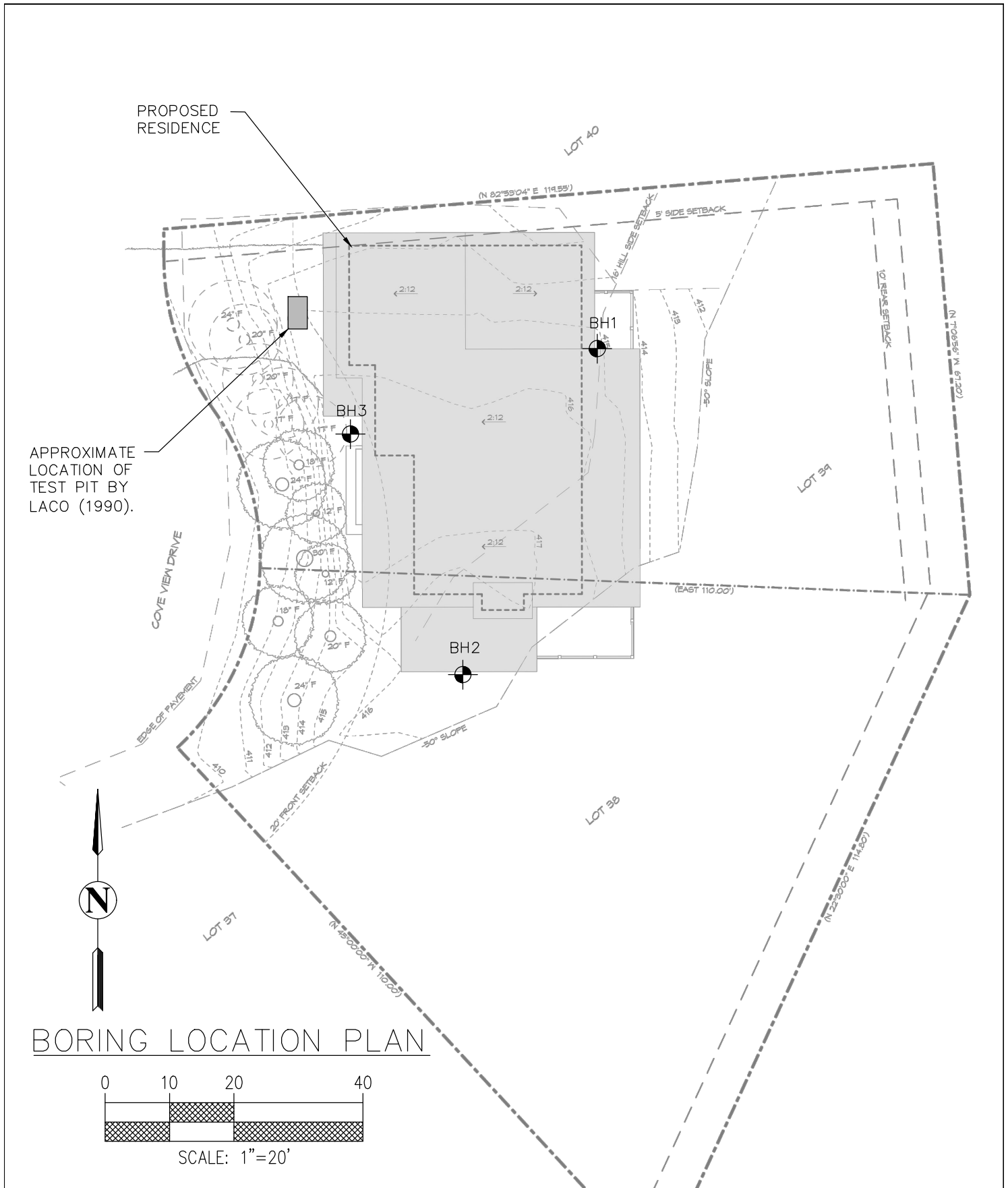
Figure 4 - Boring Log (BH2)

Figure 5 - Boring Log (BH3)

Figure 6 - Soil Classification Chart

Figure 7 - Bedrock Classification Chart

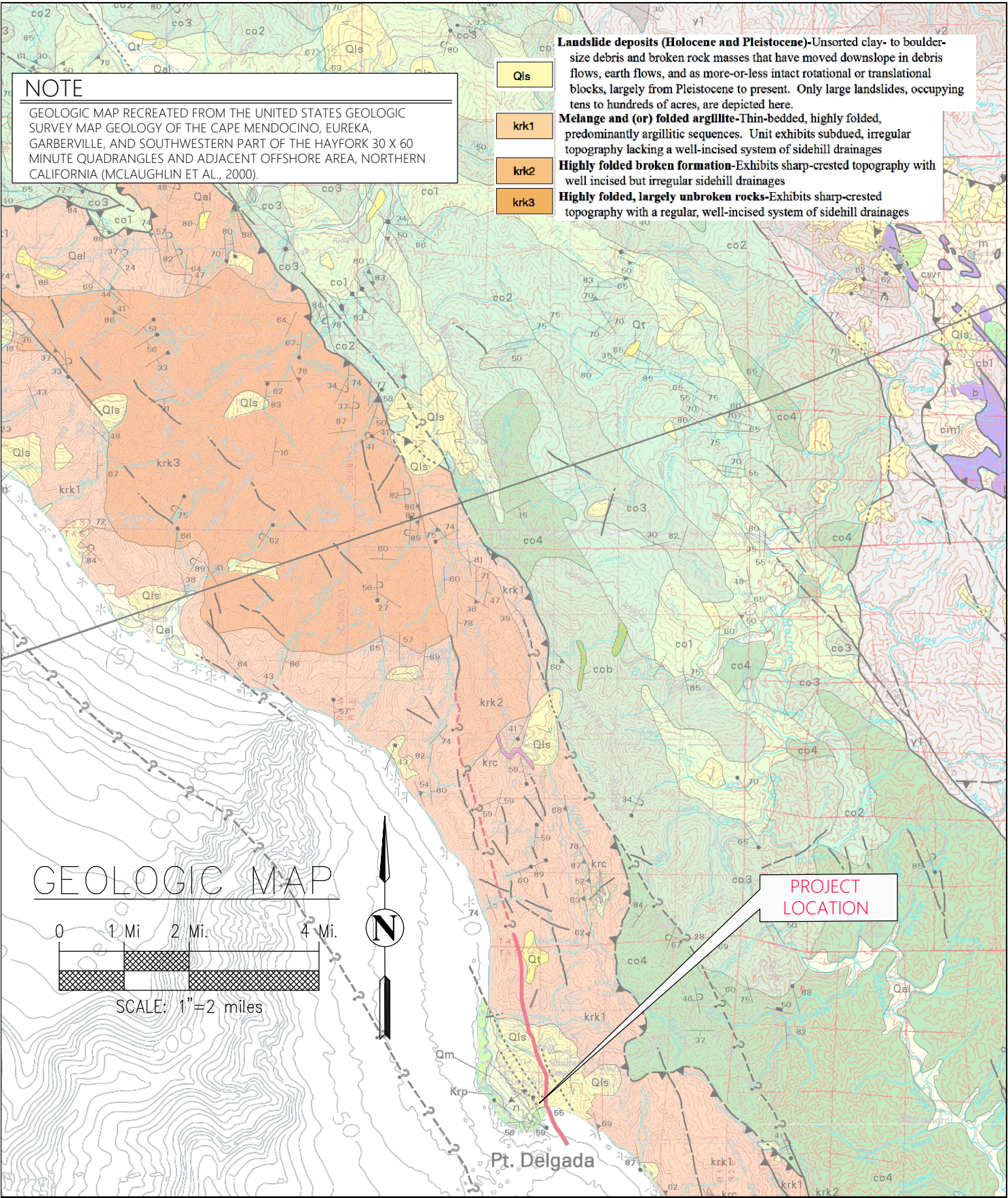
Figure 8 - Drilled Pier Subdrain Detail



BORING LOCATION PLAN
 Pyle Cabin
 159 & 161 Cove View Drive
 Shelter Cove CA, 95589

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FIGURE:	1
PROJECT:	25-10
DATE:	07/14/25
SCALE:	1"=20'



GEOLOGIC MAP
 Pyle Cabin
 159 & 161 Cove View Drive
 Shelter Cove CA, 95589

TALL TREE
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FIGURE:	2
PROJECT:	25-10
DATE:	07/14/25
SCALE:	1"=2 Mi.

Project Name: Pyle Residence GI
 Drilling Date: 02/21/2025
 Logged By: dsf
 Borehole Location: See Figure 1
 Ground Elevation: ~415'

Drilling Equipment: Limited access with flight auger and 140 lb hammer
 Drilling Operator: Stapleton Engineering and Exploration
 Borehole Diameter: 4-inches
Groundwater Levels
 Time of Drilling: none End of Drilling: none After Drilling: none

BORING NUMBER BH-1
 PAGE 1 OF 2

DEPTH (ft)	GRAPHIC LOG	U.S.C.S.	MATERIAL DESCRIPTION	SAMPLE TYPE	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS			
									LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	
0.0												
1.0		CL	[0.0' - 2.0'] Dark brown sandy CLAY (CL), moist, medium plasticity, soft, trace organics									
2.0												
3.0		CL	[2.0'-12.0'] Yellowish brown CLAY (CL) with gravels, moist, medium to high plasticity, stiff									
4.0			Yellowish brown and light gray mottling at 3.5'									
5.0			Stiffness increases with depth	MC	7-10-11 (21)							
6.0												
7.0												
8.0												
9.0												
10.0				MC	5-5-5 (10)	>4.5	96.9	22.6				
11.0												
12.0												
13.0		KJfm	[12.0'-26.0'] Reddish yellow sandy clay with light brown mottling and thinly interbedded siltstone and clay stone, highly weathered, weak, moderately hard									
14.0				MC	5-8-9 (17)		111.5	13.9				
15.0												

GEOTECHNICAL BOREHOLE
 Pyle Cabin
 159 & 161 Cove View Drive
 Shelter Cove CA, 95589

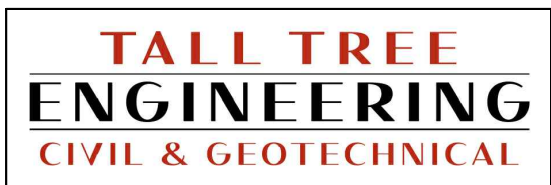


FIGURE: 3a
 PROJECT: 25-10
 DATE: 07/14/25
 SCALE: NTS

Project Name: Pyle Residence GI
 Drilling Date: 02/21/2025
 Logged By: dsf
 Borehole Location: See Figure 1
 Ground Elevation: ~415'

Drilling Equipment: Limited access with flight auger and 140 lb hammer
 Drilling Operator: Stapleton Engineering and Exploration
 Borehole Diameter: 4-inches
Groundwater Levels
 Time of Drilling: none End of Drilling: none After Drilling: none

BORING NUMBER BH-1
 PAGE 2 OF 2

DEPTH (ft)	GRAPHIC LOG	U.S.C.S.	MATERIAL DESCRIPTION	SAMPLE TYPE	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS														
									LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX												
15.0																							
16.0		KJfm	[12.0'-26.0'] Reddish yellow sandy clay with light brown mottling and thinly interbedded siltstone and clay stone, highly weathered, weak, moderately hard		7-8-9 (17)		107.2	14.6	33.8	19.0	14.8												
17.0																							
18.0																							
19.0													10-12-15 (27)										
20.0																							
21.0																							
22.0																							
23.0																							
24.0																							
25.0																							
26.0					7-8-8 (16)																		
27.0	Bottom of borehole at 26.0 feet below grade surface. No free groundwater encountered.																						
28.0																							
29.0																							
30.0																							

GEOTECHNICAL BOREHOLE
 Pyle Cabin
 159 & 161 Cove View Drive
 Shelter Cove CA, 95589

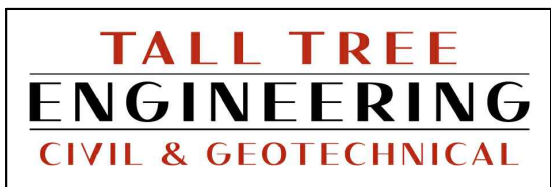

















FIGURE: 3b
 PROJECT: 25-10
 DATE: 07/14/25
 SCALE: NTS

Project Name: Pyle Residence GI
 Drilling Date: 02/21/2025
 Logged By: dsf
 Borehole Location: See Figure 1
 Ground Elevation: ~417'

Drilling Equipment: Limited access with flight auger and 140 lb hammer
 Drilling Operator: Stapleton Engineering and Exploration
 Borehole Diameter: 4-inches
Groundwater Levels
 Time of Drilling: none End of Drilling: none After Drilling: none

BORING NUMBER BH-2
 PAGE 1 OF 2

DEPTH (ft)	GRAPHIC LOG	U.S.C.S.	MATERIAL DESCRIPTION	SAMPLE TYPE	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS				
									LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX		
0.0													
1.0		CL	[0.0'-1.0'] Dark brown sandy CLAY (CL), moist, medium plasticity, soft, trace organics										
2.0		CL	[1.0'-6.5'] Yellowish brown CLAY (CL) with gravels, moist, medium to high plasticity, stiff										
3.0				MC	3-5-4 (9)								
4.0													
5.0													
6.0				MC	6-8-8 (16)	4.0	99.5	20.7					
7.0		KJfm	[6.5'-20.0'] Reddish yellow sandy clay with light brown mottling and thinly interbedded siltstone and clay stone, highly weathered, weak, moderately hard										
8.0													
9.0													
10.0													
11.0				MC	3-3-4 (7)	4.0	91.3	17.7					
12.0													
13.0													
14.0													
15.0													

GEOTECHNICAL BOREHOLE
 Pyle Cabin
 159 & 161 Cove View Drive
 Shelter Cove CA, 95589

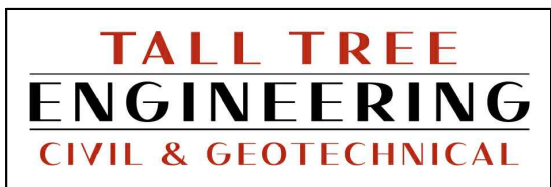


FIGURE: 4a
 PROJECT: 25-10
 DATE: 07/14/25
 SCALE: NTS

Project Name: Pyle Residence GI
 Drilling Date: 02/21/2025
 Logged By: dsf
 Borehole Location: See Figure 1
 Ground Elevation: ~417'

Drilling Equipment: Limited access with flight auger and 140 lb hammer
 Drilling Operator: Stapleton Engineering and Exploration
 Borehole Diameter: 4-inches
Groundwater Levels
 Time of Drilling: none End of Drilling: none After Drilling: none

BORING NUMBER BH-2
 PAGE 2 OF 2

DEPTH (ft)	GRAPHIC LOG	U.S.C.S.	MATERIAL DESCRIPTION	SAMPLE TYPE	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS			
									LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	
15.0												
16.0		KJfm	[6.5'-20.0'] Reddish yellow sandy clay with light brown mottling and thinly interbedded siltstone and clay stone, highly weathered, weak, moderately hard	MC	7-14-16 (30)		102.5	17.8				
17.0												
18.0				SPT	7-9-9 (18)							
19.0												
20.0				SPT	7-9-9 (18)							
21.0	Bottom of borehole at 20.0 feet below grade surface. No free groundwater encountered.											
22.0												
23.0												
24.0												
25.0												
26.0												
27.0												
28.0												
29.0												
30.0												

GEOTECHNICAL BOREHOLE
 Pyle Cabin
 159 & 161 Cove View Drive
 Shelter Cove CA, 95589

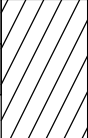














FIGURE: 4b
 PROJECT: 25-10
 DATE: 07/14/25
 SCALE: NTS

Project Name: Pyle Residence GI
 Drilling Date: 02/21/2025
 Logged By: dsf
 Borehole Location: See Figure 1
 Ground Elevation: ~416'

Drilling Equipment: Limited access with flight auger and 140 lb hammer
 Drilling Operator: Stapleton Engineering and Exploration
 Borehole Diameter: 4-inches
Groundwater Levels
 Time of Drilling: none End of Drilling: none After Drilling: none

BORING NUMBER BH-3
 PAGE 1 OF 1

DEPTH (ft)	GRAPHIC LOG	U.S.C.S.	MATERIAL DESCRIPTION	SAMPLE TYPE	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS				
									LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX		
0.0													
1.0		CL	[0.0'-1.5'] Dark brown sandy CLAY (CL), moist, medium plasticity, soft, trace organics										
2.0		CL	[1.5'-7.5'] Yellowish brown CLAY (CL) with gravels, moist, medium to high plasticity, stiff										
3.0				MC	4-5-6 (7)	4.0	86.9	24.5					
4.0													
5.0				MC	5-4-4 (8)								
6.0													
7.0													
8.0		KJfm	[7.5'-13.5'] Reddish yellow sandy clay with light brown mottling and thinly interbedded siltstone and clay stone, highly weathered, weak, moderately hard										
9.0				MC	5-8-9 (17)		95.3	17.5					
10.0													
11.0													
12.0													
13.0				MC	4-9-9 (18)		98.3	23.2					
14.0		Bottom of borehole at 13.5 feet below ground surface. No free groundwater encountered.											
15.0													

GEOTECHNICAL BOREHOLE
 Pyle Cabin
 159 & 161 Cove View Drive
 Shelter Cove CA, 95589

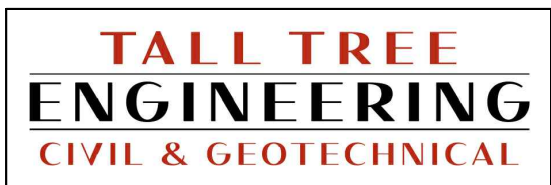

















FIGURE: 5
 PROJECT: 25-10
 DATE: 07/14/25
 SCALE: NTS

UNIFIED SOIL CLASSIFICATION AND SYMBOL CHART

COARSE GRAINED SOILS (more than 50% of coarse fraction larger than No. 200 sieve size)		
Clean gravels (less than 5% fines)		
GRAVELS More than 50% of coarse fraction larger than No. 4 sieve size	 GW	Well-graded gravels, gravel-sand mixture, little or no fines
	 GP	Poorly-graded gravels, gravel-sand mixture, little or no fines
Gravels with fines (more than 12% fines)		
	 GM	Silty gravels, gravel-sand-silt mixtures
	 GC	Clayey gravels, gravel-sand-clay mixtures
Clean Sands (less than %5 fines)		
SANDS 50% or more of coarse fraction smaller than NO. 4 sieve size	 SW	well- graded sands, gravelly sands, little to no fines
	 SP	Poorly graded sands, gravelly sands, little to no fines
Sands with fines (More than 12% fines)		
	 SM	Silty sands, sand-silt mixtures
	 SC	Clayey sands, sand silt mixtures
FINE GRAINED SOILS (50% or more of material is smaller than No. 200 sieve size.)		
SILTS AND CLAYS Liquid limit less than 50%	 ML	Inorganic silts and very fine sands, rock flour, silty of clayey fine sands or clayey silts with slight plasticity
	 CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
	 OL	Organic silts and organic silty clays of low plasticity
SILTS AND CLAYS Liquid limit 50% or greater	 MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts
	 CH	Inorganic clays of high plasticity, fat clays
	 OH	Organic clays of medium to high plasticity, organic silts
HIGHLY ORGANIC SOILS	 PT	Peat and other highly organic soils

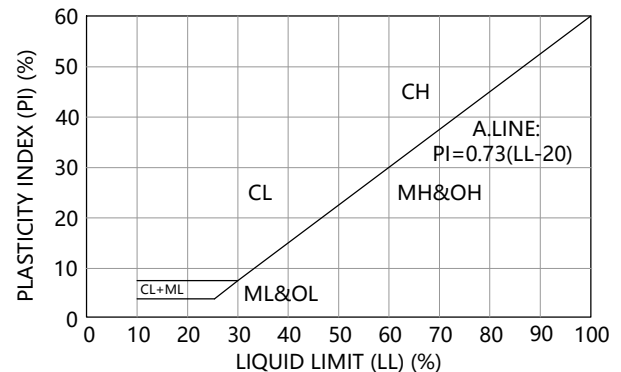
LABORATORY CLASSIFICATION CRITERIA

Not meeting all gradation requirements for GW		
GW	$C_u = \frac{D_{60}}{D_{10}}$ greater than 4 ; $C_c = \frac{D_{30}}{D_{10} \times D_{60}}$ between 1 and 3	
GP Not meeting all gradation requirements for GW		
GM	Atterberg limits below "A" line with P.I. less than 4	Above "A" line with P.I. between 4 and 7 are borderline cases requiring use of dual symbols
GC	Atterberg limits above "A" line with P.I. greater than 7	
SW	$C_u = \frac{D_{60}}{D_{10}}$ greater than 4 ; $C_c = \frac{D_{30}}{D_{10} \times D_{60}}$ between 1 and 3	
SP Not meeting all gradation requirements for GW		
SM	Atterberg limits below "A" line or P.I. less than 4	Limits plotting in shaded zone with P.I. between 4 and 7 borderline cases requiring use of dual symbols.
SC	Atterberg limits Above "A" line with P.I. greater than 7	

Determine percentages of sand and gravel from grain-size curve. Depending on percentages of fines (fraction smaller than No. 200 sieve size), coarse-grained soils are classified as follows:

less than 5 percent GW, GP, SW, SP
 More Than 12 percent GM, GC, SM, SC
 5 to 12 percent Borderline cases requiring dual symbols

PLASTICITY CHART



SOIL CLASSIFICATION CHART

Pyle Cabin
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TALL TREE
ENGINEERING
 CIVIL & GEOTECHNICAL

FIGURE: 6

PROJECT: 25-10

DATE: 07/14/25

SCALE: NONE

ROCK TYPES



Conglomerate



Metamorphic Rocks
Hydrothermally Altered Rocks



Shale



Sandstone



Sheared Shale Mélange



Meta-Sandstone



Igneous Rocks



Chert

BEDDING THICKNESS JOINT, FRACTURE OR SHEAR SPACING

Massive	> 6'	Very Widely Spaced	> 6'
Thickly Bedded	2'-6'	Widely Spaced	2'-6'
Medium Bedded	8"-24"	Moderately Widely Spaced	8"-24"
Thinly Bedded	2.5"-8"	Closely Spaced	2.5"-8"
Very Thinly Bedded	0.75"-2.5"	Very Closely Spaced	0.75"-2.5"
Closely Laminated	0.25"-0.75"	Extremely Closely Spaced	< 0.75"
Very Closely Laminated	< 0.25"		

HARDNESS

Soft - Pliable, can be dug by hand

Slightly Hard - Can be gouged deeply or carved with a pocket knife

Moderately Hard - Can be readily scratched with a pocket knife; Scratch leaves heavy trace of dust and is readily visible after the powder has been blown away

Hard - Can be scratched with difficulty; Scratch produces little powder and is faintly visible

Very Hard - Cannot be scratched with pocket knife, leaves metallic streak

STRENGTH

Plastic - Capable of being molded by hand

Friable - Crumbles by rubbing with fingers

Weak - An unfractured specimen will crumble under light hammer blows

Moderately Strong - Specimen will withstand a few heavy hammer blows before breaking

Strong - Specimen will withstand a few heaving ringing hammer blows and usually yields large fragments

Very Strong - Rock will resist heavy ringing hammer blows and will yield with difficulty only dust and small flying fragments

DEGREE OF WEATHERING

Highly Weathered - Abundant fractures coated with oxides, carbonates, sulphates, mud, etc., through discoloration, rock disintegration, mineral decomposition

Moderately Weathered - Some fracture coating, moderate or localized discoloration, little to no effect on cementation, slight mineral decomposition

Slightly Weathered - A few stained fractures, slight discoloration, little to no effect on cementation, no mineral decomposition

Fresh - Unaffected by weathering agents, no appreciable change with depth

BEDROCK CLASSIFICATION KEY

Pyle Cabin
159 & 161 Cove View Drive
Shelter Cove CA, 95589

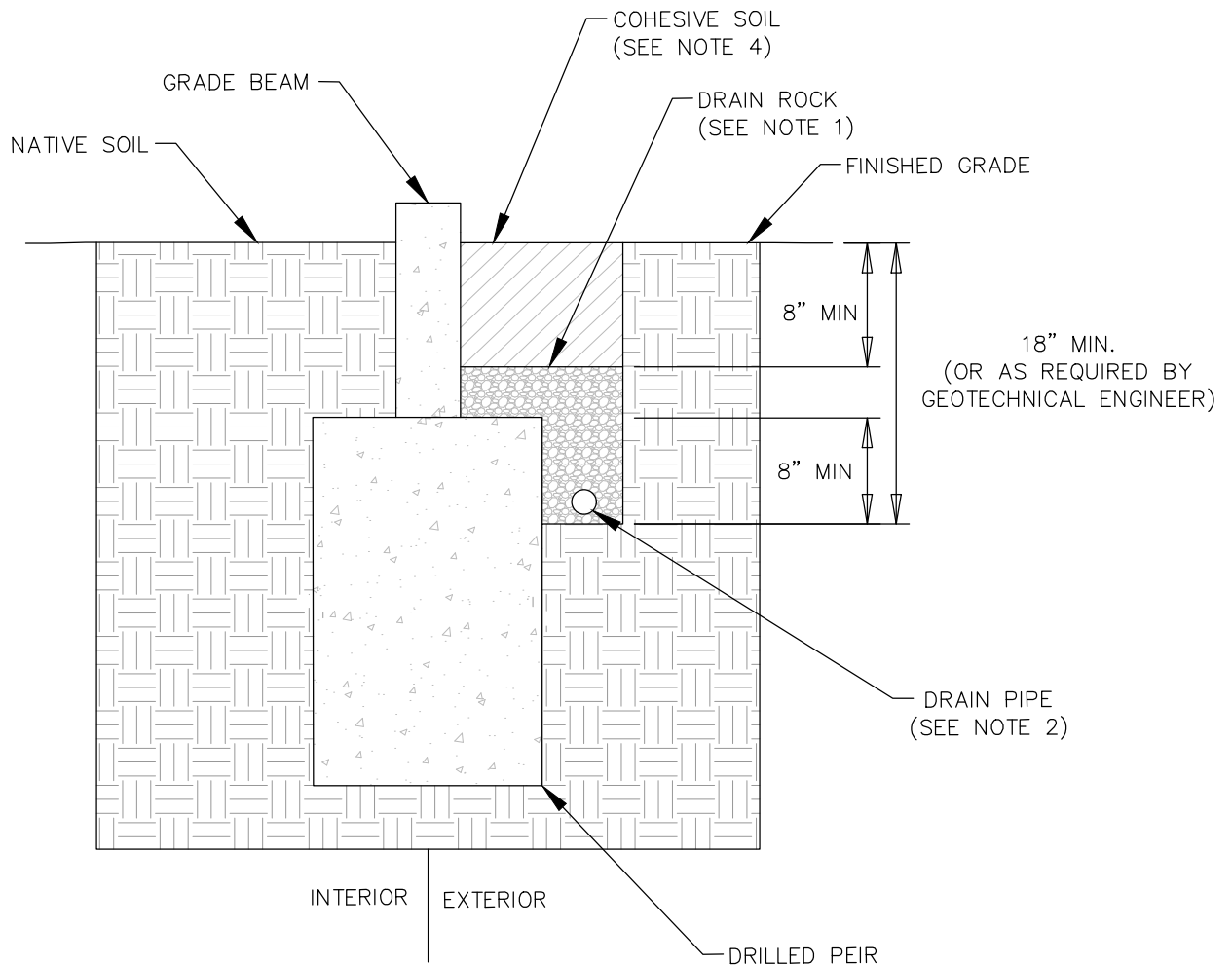
TALL TREE
ENGINEERING
CIVIL & GEOTECHNICAL

FIGURE: 7

PROJECT: 25-10

DATE: 07/14/25

SCALE: NONE



NOTES:

- (1) DRAIN ROCK SHOULD MEET THE CLASS 2 PERMEABLE MATERIAL REQUIREMENTS IN THE LATEST VERSION OF THE CALTRANS STANDARD SPECIFICATIONS.
- (2) DRAIN PIPE SHOULD BE SDR 35 OR EQUIVALENT, PLACED WITH PERFORATION DOWN, AND SLOPED AT A MINIMUM OF 1% TO DRAIN BY GRAVITY TO OUTLET.
- (3) A CLEAN-OUT PIPE WITH CAP SHOULD BE INSTALLED AT THE UP-SLOPE END OF PERFORATED PIPE, AND PIPE ELBOWS SHOULD BE 45 DEGREES OR LESS (FOR "SNAKE" ACCESS)
- (4) TOP 8 INCHES OF FILL ABOVE BACK DRAIN SHOULD CONTAIN AT LEAST 20 PERCENT CLAY TO LIMIT SURFACE WATER FROM FLOWING INTO THE DRAIN ROCK.

DRILLED PIER SUBDRAIN DETAIL

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 Shelter Cove CA, 95589

TALL TREE
ENGINEERING
 CIVIL & GEOTECHNICAL

FIGURE: 8
PROJECT: 25-10
DATE: 07/14/25
SCALE: NONE