THE GALLI GROUP
Engineering Consulting

GEOTECHNICAL DESIGN REPORT
BEACH LOOP SUBDIVISION
T28S, R15W, SEC 36C
TAX LOTS 219, 400, 500, 600, 700 & 1500
BEACH LOOP ROAD
BANDON, OREGON

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1.0 INTRODUCTION

This report presents the results of our geotechnical investigation and geologic hazard evaluation of the site for the proposed commercial development and residential subdivision on the approximately 25-acre parcel (combined lots) located near the intersection of Beach Loop Road and Face Rock Road, in Bandon, Oregon. Please see *Figure 1, Vicinity Map*, for a more precise site location.

The purpose of our investigation and this report was to evaluate surface and subsurface conditions at the site with a series of 4 exploratory borings, 9 permeability borings, and 16 test pits, in order to provide general geotechnical recommendations and a geologic hazard evaluation for design and construction of the proposed mixed-use site development; including site grading, new access roadways and structure foundations.

2.0 SITE AND PROJECT DESCRIPTION

The subject site encompasses an irregularly shaped parcel consisting of multiple adjoining lots totaling approximately 25 acres (easements inclusive). The site is located on the south side of Face Rock Drive and the east side of Beach Loop Road SW. The majority of the eastern portion of the site has generally mild slopes (approximately 1% to 3%). Portions of the western side of the site extend across the top of an existing, undulating foredune area, which contains slope inclinations of up to 20%. The parcel sits at elevations ranging between 70 and 115 feet above mean sea level.

Based on preliminary conceptual plans provided by Parametrix, we understand that the current proposed development of the site will include subdividing the eastern portion of the parcel into smaller, residential tax lots. Construction of a hotel with underground parking and associated clubhouse and pool area are planned for the west side of the parcel. Associated development will include an access road network in and out the project from Face Rock Drive, Beach Loop Road and Carter Street. There will likely be undeveloped portions of the site in existing delineated wetlands areas, which will be designated as “open space” and will likely be utilized for stormwater detention.

As part of the feasibility study for the development of the property, we understand the geologic hazards that might be associated with the site due to the proposed development must be evaluated for possible impacts on this or adjacent parcels. Additionally, geotechnical recommendations are needed to mitigate any observed or anticipated geologic hazards areas and/or subsurface conditions which may limit the use of conventional design and construction techniques. To accomplish this development, we anticipate the following geotechnical-related design and construction work will be required: general cut and fill road construction, retaining wall construction, site grading, preparation of building pad areas for the structures and installation of utilities.

3.0 FIELD EXPLORATION

We accomplished the following subsurface site explorations during the field investigation of the project site.

3.1 EXPLORATORY BORINGS

On August 29, 2022, our senior engineer/geologist, Dennis Duru, PE, CEG, RG, visited the site to conduct exploratory drilling operations. On August 30, 2022, our project engineer Lyn Chand, P.E., also visited the site for completion of the planned drilling exploration. A total of four (4) exploratory borings were drilled throughout the site at the locations shown on *Figure 2, Site Plan with Exploration Locations*. The drilling was accomplished using a truck mounted CME 75 drill rig and crew provided by Western States Soil Conservation, Inc.

Borings were advanced with sample collection and testing being accomplished at various depths. Standard Penetration Testing (SPT) was accomplished in each boring. This entails driving a 1½ inch I.D, 2-inch O.D., steel split spoon sampler by dropping a 140-pound weight for a 30-inch drop. The total number of blows it takes to drive the sampler the last 12 inches of an 18-inch drive is called the SPT N-value. These can be correlated with soil strength and density parameters from testing on thousands of other projects.

Borings penetrated to depths of between 25.0 and 46.5 feet, terminating in the dense to very dense, Sand or Gravel soils beneath the site. All holes were backfilled with drill spoils and bentonite chips after drilling.

Our representative identified the exploration locations away from utilities, logged subsurface soils and water conditions and obtained soil samples for transport to our laboratory. Visual classifications of the soils were made in the field and are presented in the *Boring Logs in Appendix A*, at the end of this report.

3.2 EXPLORATORY TEST PITS

On September 9, 2022, our senior engineer/geologist, Dennis Duru, PE, CEG, RG., visited the site to conduct the test pit exploration. A total of sixteen (16) exploratory test pits were excavated at various locations across the site, to depths of between 3.0 and 12.0

feet below the existing ground surface. The test pits were excavated at the locations shown on *Figure 2*.

The test pits were excavated using a Kubota KX057-5 excavator outfitted with a 24” wide, toothed bucket, provided by Bandon Concrete LLC. Our representative logged subsurface soils and water conditions and obtained soil samples. Shelby tube and grab samples were obtained in some of the test pits and transported to The Galli Group laboratory for further soil characterization and laboratory testing.

Visual classifications of the soils were made in the field and are presented in the *Test Pit Logs in Appendix B*, at the end of this report.

3.3 FIELD PERMEABILTY TESTING

In order to evaluate on-site soil permeability, the Galli Group performed field falling-head permeability tests on nine (9) augured borings in the locations shown on *Figure 2*. Borings of 8-inch diameter were augured to depths of 36 inches into the sandy soils. The augured holes were pre-soaked, then filled with water to near the top. Falling head measurements were then obtained at various time intervals, with a record of the depth of water and elapsed time since the last reading.

The field data was “reduced” by our firm and yielded the calculated permeability rates of between 3.34 X 10^{-3} and 4.37 X 10^{-3} inches per second, or 12.01 in./hr. and 15.73 in./hr., for holes located on the west side of the project, and between 1.57 X 10^{-4} and 5.60 X 10^{-4} inches per second, or 0.566 in./hr. and 2.02 in./hr., for holes located on the east side of the project. See *Appendix C* for additional detail regarding the field data and permeability calculations and results.

4.0 LABORATORY TESTING

The soil samples collected during our investigation were tested for natural moisture content. To characterize the subsurface and evaluate liquefaction potential of the site, fifteen (15) washed sieves (ASTM D-6913) and five (5) hydrometer analyses (ASTM D-7928) were performed on representative soil samples obtained at various depths during the site subsurface exploration. In addition, three (3) direct shear tests (ASTM D-3080), and one (1) consolidation test (ASTM D-2435) were performed on soft, clayey and silty samples to determine shear strength parameters and consolidation potential.

Two (2) California Bearing Ratio, CBR, tests (ASTM D-1883) and one Standard Proctor test (ASTM D-698) were accomplished on representative soil samples collected at locations near the proposed roadways. These tests were accomplished in order to determine an approximate CBR value for these soils, which are anticipated as native subgrade soils, for new roadway pavement design.

See *Appendix D* for more detail of each laboratory test conducted.

5.0 SURFACE AND SUBSURFACE CONDITIONS

5.1 SOIL

In general, our site observation and subsurface exploration showed that nearly the entire site is covered with surficial layers of organic topsoil and peat of varying thicknesses.

The west side of the site consists of an irregular pattern of sandy foredune deposits. The foredunes are hummocky and most of the surface is covered with a minimum of 12-inches of organic topsoil. The dune side slopes in this area appear unstable and show patterns of erosional instability. Depending on the elevation, the soils subsurface consist of loose to very loose, fine Sand to depths ranging between 15 feet (elevation 70 feet) and 30 feet (elevation 63 feet) below the ground surface. Below these fine sands are layers of dense, gravelly Sands and sandy Gravels.

The east side of the site is relatively flat. Between 1 and 2 feet of highly organic/peat topsoil covers this area. Beneath the topsoil, the subsurface borings and test pits encountered medium dense sand to approximately 4 feet. This is then underlain by dense and weakly cemented Sand. TP-14 encountered a deeper organic layer (with strong organic odor) to the depth of excavation of 8.5 feet.

Please see more specific soils information in the *Boring Logs in Appendix A* and *Test Pit Logs in Appendix B,* and Cross-section view of the subsurface provided in *Figure 3A Subsurface Cross-Section A-A'* and *Figure 3B Subsurface Cross-Section B-B*'. Please note that the soils are shown as distinct layers in the Logs, while in nature they may change more gradually. Soils/rock conditions may also change somewhat between the locations investigated.

5.2 GROUNDWATER

Static groundwater levels were measured at between 20 feet (elevation 65 feet) and 34 feet (elevation 59 feet) on the west side of the site and at approximately 10 feet (elevation 64 and 67 feet) on the east side. We do not anticipate the groundwater to rise much higher on the west side of the site. However, based on the color variations in the soil profile, it is very likely that groundwater may rise to within 2 feet of the ground surface on the east side of the site.

6.0 GEOLOGIC HAZARDS AND SEISMIC DESIGN PARAMETERS

6.1 SITE GEOLOGY

The project, located in southwestern Oregon, in the City of Bandon, is within Oregon's Coast Range Physiographic Province. Turbidite (Tul) sedimentary rock units, which are part of the "Eocene" sequence of the Tenmile Formation, and the Greenstone (Jov) volcanic rock units, which are part of the Jurassic/Cretaceous period of the Sixes River

Terrane, form the bedrock units in the project area (OGDC-6, 2015). These bedrock units beneath the site are overlain by the Quaternary Marine Terrace deposits (Qmt) and associated coastal terrace deposits. These upper, mixed-grained sediments appear to be on the order of 30 to 50 feet deep before encountering the bedrock unit, based on deep site borings. However, from OGDC-6, 2015 mapping, it appears the volcanic bedrock unit may be outcropped (at or near the surface) in some locations on the west side of the project. However, we did not encounter this outcrop during our limited surface and subsurface site investigation.

The Coast Range province experienced regional uplift into Miocene times. This continuing coastal uplift, combined with Pleistocene sea level changes, produced a series of marine terraces along the Southern Oregon Coast.

Active faults and folding, associated with a convergent plate boundary offshore of the project, are mapped in the Bandon area. The northwest/southeast trending axis of the active Coquille Anticline is approximately a mile north of the northern edge of the project. However, no Quaternary fault activity, which could produce surface fault rupture, has been established within the immediate project area of Bandon (Hazvu, 2018; USGS; 2018a;).

6.2 GEOLOGIC HAZARDS

6.2.1 Flooding

The project is not within any designated FEMA Special Flood Hazard Area (“100-year” flood), as shown on online mapping (OregonRiskMap, 2018). Risk of flood damage to the project site is considered to be very low.

6.2.2 Landslides / Slope Instability

The east side of the site is relatively flat with mild slopes in the range of 1% to 3 %. The foredune area located across part of the west side of the site contains areas with slopes of up to 20%. Soil thickness at the project site appears to be relatively deep (30 to 50 feet) before encountering weathered bedrock. Site slopes appear to be stable in their current natural condition with the exception of some apparent surface slides which can be attributed to erosion. Based on the subsurface investigation, the sands which constitute these slopes are dry and loosely deposited and will become unstable if site conditions change during construction. The sandy soils on the west side of the site will not support cut slopes at typically recommended inclinations and may require temporary or permanent retaining walls in areas where very flat cut slope inclinations are not feasible.

The project site is not within an existing deep-seated Quaternary landslide area (Qls), according to the air photos (Google Earth, 2016), and Lidar imagery (bare earth and highest hit imagery) of the Bandon Quadrangle (DOGAMI, 2021). The State Landslide Information Database for Oregon (SLIDO 4.2, 2020) mapped some areas on the west side of the project site as having moderate susceptibility for a landslide.

***Note**: Recommendations for site grading and proper methods of cut-and-fill construction will be provided in the geotechnical recommendations section of this report, and it is essential these recommendations be followed closely in order to minimize slope instability both during and after construction. Similarly, recommendations addressing surface and subsurface drainage in the project area, as well as erosion control measures, will be also be provided and must be followed during, and in some cases, after construction, to maintain slope stability in the project area. In-progress grading inspections should be made during construction to note any adverse conditions which could negatively affect cut slopes or general site grading.*

6.2.3 Expansive Soils

Soils encountered during the subsurface exploration at the project site are not expansive. Risk to the constructed project due to expansive soils is zero.

6.2.4 Liquefaction

General Evaluation. A general screening of liquefaction hazard includes evaluation of the following: seismic source potential to cause liquefaction, historic occurrence of liquefaction, depth to the water table, geologic age and composition of subsurface material, including density of material.

- A seismic source potential for liquefaction certainly exists on the nearby Cascadia Subduction zone with distances between 15km and 20km from the project site, and regarded as being capable of producing over M 9.0 earthquakes.
- No evidence of historical or paleoseismic liquefaction has been established for the Quaternary surficial deposits at the site. This may be attributed to lack of substantive research and to the fact that the last such great earthquake is predicted to have occurred over 300 years ago; enough time for such evidence to no longer be apparent. A reconnaissance screening report by DOGAMI considered the west side of the project to have “high liquefaction hazard” potential and the east side to have “moderate liquefaction hazard” potential. This reconnaissance report is intended for a basic “screening” of liquefaction potential, and not to be used for site specific design.
- Groundwater was measured at an approximate depth of between 20 and 34 feet below ground surface, or elevation 59 and 65 feet above MSL, on the west side of the project site. On the east side of the project site, groundwater was measured at a depth of approximately 10 feet below ground surface, or elevation 64 and 67 feet above MSL.
- The loose, fine sands on the west side of the site extend to an approximate depth of between 15 feet to 40 feet, and could be thicker in areas of higher ground. The densities of these sands are loose, with SPT N-values in the range of 0 to 8 to depths of between 15 and 34 feet, and greater than 30 SPT N-Value below this depth. On the east side of the site, the sand layers have thicknesses extending to the entire depth explored. Densities are higher in this area, with SPT N-values

ranging from 18 to over 40, to a depth of 20 feet, and greater than 30 SPT N-value below this depth.

From the evaluation above, the west side of the site has loose to very loose, fine sand. However, these loose sands are above the groundwater table. We did not see any evidence that groundwater fluctuates much and therefore do not anticipate the groundwater rising up to the loose sands. Therefore, the likelihood of liquefaction of the site soils on the west side of the site is considered low. However, due to the loose nature of these sands, excessive settlement is expected during the design earthquake event.

On the east side of the site, the sands below the water table are medium dense to dense. Clean, sandy soils with densities such as those on the east side of the project site have moderate susceptivity to liquefaction. Therefore, site liquefaction and associated seismic settlements have been assessed using Simplified Methods by Seed and Idriss, 1971. The results of the evaluation are presented below.

Liquefaction Settlement Assessment. A liquefaction analysis using the Simplified Method (Seed and Idriss, 1971) was completed for two modeled subsurface conditions and SPT N-values, each representing observed subsurface profiles of the west and east sides of the site. All settlement calculations were performed using the SHAKE 2000 (Ordonez, 2012) software using Tokimatsu & Seed (1987) and Ishihara & Yoshimine (1992) methods for both conditions above the water table, and a saturated condition below the water table. This shake analysis was accomplished using a site modified peak ground acceleration (PGA_M) of 1.116g. Table 1, below, summarizes the subsurface data used in the shake analysis for the liquefaction and settlement evaluation of the west side of the site.

Table 1: Summary of Subsurface Data-WEST

Depth	Thickness	SPT N-Value	Unit Weight
feet	feet		pcf
7	7	0	100
11	4	18	100
27	16	40	120
52	25	69	130
77	25	100	140
100	23	100	140

Table 2: Summary of Seismic Settlement Calculations-WEST

PGA (g)	Tokimatsu & Seed (1987)	Ishihara & Yoshimine (1992)
	Total Settlement – (Inches)	Total Settlement – (Inches)
1.116	6.5	6.5

The analysis for the west side assumes that the footing elevation for the hotel structure is at 70 feet above MSL. This puts the groundwater at approximately 11 feet below the bottom of the footings (B-2 profile), which was used for this analysis. Based on the analysis, the site would not liquefy during a seismic event. However, estimated seismic induced settlement of the loose soils above the water table is 6.5 inches on the west side of the site (see Table 2).

Table 3: Summary of Subsurface Data-EAST

Depth	Thickness	SPT N-Value	Unit Weight
feet	feet		pcf
4	4	7	110
10	6	18	110
12.5	2.5	23	110
20	7.5	14	110
25	5	75	120
30	5	21	120
35	5	30	120
40	5	50	140
60	20	75	140
100	40	100	140

Table 4: Summary of Seismic Settlement Calculations-EAST

PGA (g)	Tokimatsu & Seed (1987)	Ishihara & Yoshimine (1992)
	Total Settlement – (Inches)	Total Settlement – (Inches)
1.116	4.4	6.1

The analysis for the east side assumes that the footing elevation for structures will be near the existing ground surface. Due to the evidence of fluctuating groundwater level, the groundwater is set at approximately 2.0 feet (B-4 profile) for this analysis. The soil profile used for the analysis is represented in *Table 3* above. Based on the analysis, the site subsurface layers with SPT N-values less than 30 would liquefy during a seismic event. The Liquefaction will induce settlements estimated on the order of 4.4 to 6.1 inches on the east side of the site as shown in Table 4.

Lateral Spread. Given the site topography and the nearly level layering of the alluvial deposits, we do not anticipate lateral spread large enough to negatively impact the project. We utilized the Multiple regression analysis Models of Bartlett & Youd (2002) and Zhang & Zhao (2005) to predict the permanent ground deformation that could result from lateral spread during a magnitude Mw 8.0 (maximum for the models) earthquake and a site to source distance of 17 km (closest source distance). The result shows that a maximum lateral spread of 7 feet may occur. This potential lateral spread will be limited to soil layers/pockets across the site where sand below the water table is loose to medium dense.

6.2.5 Ground Rupture

No Quaternary faults are shown to cut across the project site, based on geologic mapping (OGDC, 2017; USGS; 2017). There are Cascadia faults and folds within 15 km (9 miles) west (offshore) of the property, in the pacific ocean. The Coquille anticline is within 2 miles of the northeast area of the project site. Therefore, this site is considered a near fault site and must be accounted for in the design of the project structures.

6.2.6 Ground Shaking

Project structures, retaining walls, and fills should be designed according to the Oregon Structural Specialty Code (OSSC; 2019). Based on obtained subsurface data during subsurface exploration and desk study, a Site Class of E should be used for structures on this project site. Seismic design recommendations are provided in Table 1 of *Section 6.5*, below.

6.2.7 Seismic Ground Amplification or Resonance

No unexpectedly hazardous amplification or resonance effects from seismic waves have been associated with the soil subsurface conditions in the project area. Potential amplification or resonance effects in the project area are accounted for in the ASCE 7-16 seismic design methods, as prescribed in OSSC, 2019. The risk of damage at the site from unexpectedly severe shaking due to seismic wave amplification is low.

6.2.8 Tsunami and Seiche

The project site is located approximately 80 miles inland, and is therefore not subject to inundation from a tsunami. The site is not located downstream of any dams, reservoirs, lakes, or any significant body of water. Therefore, the risk of damage to the site due to hazard from seiche or seismic-induced flooding is very low.

6.3 SITE SPECIFIC GROUND MOTION HAZARD ANALYSIS

Site Specific Ground Motion Hazard Analysis was carried out in order to meet the requirements of the new ASCE 7 (2016), as specified in (OSSC, 2019); that a site-specific study is required for structures on sites with a Site Class D or E with S_1 greater than or equal to 0.2g, and all sites with Site Class F. Based on our site reconnaissance, desk study and subsurface exploration, the subject site was determined to have a Site Class E, based on the Site Classification Procedures for Seismic Design set forth in the ASCE 7-16 Chapter 20. Therefore, a Ground Motion Hazard Analysis is required to determine the design acceleration parameters for structures constructed in these areas.

A probabilistic and deterministic ground motion hazard analyses were carried out in accordance with ASCE 7-16 Section 21.2. Careful consideration was given to the requirements of Section 21.3 of the ASCE 7-16 in choosing the final Design Response Spectrum.

6.3.1 Probabilistic Ground Motion Hazard Analysis

The probabilistic ground motion hazard analysis was accomplished consistent with method 1 in the ASCE 7-16 Section 21.2.1.1. We utilized the US Geologic Survey (USGS, 2014) Unified Hazard Tool to compute the Uniform Hazard Response Spectrum that has a 2% probability of exceedance within a 50-year period for the site. The site Risk Coefficients, C_{Rs} and C_{R1} were obtained from Figures 22-18 and 22-19, respectively, of the ASCE 7-16. The C_R was then calculated as described in the Section 21.2.1.1. The scale factor recommended in Section 21.2 was introduced to scale the response spectrum to the maximum response. The probabilistic site specific MCE_R at any period was determined as the product of the Uniform Hazard Response Spectrum, the C_R and the scale factor at that period. The site specific MCE_R spectral accelerations from the probabilistic ground motion hazard analysis are shown in Diagram 1 (following).

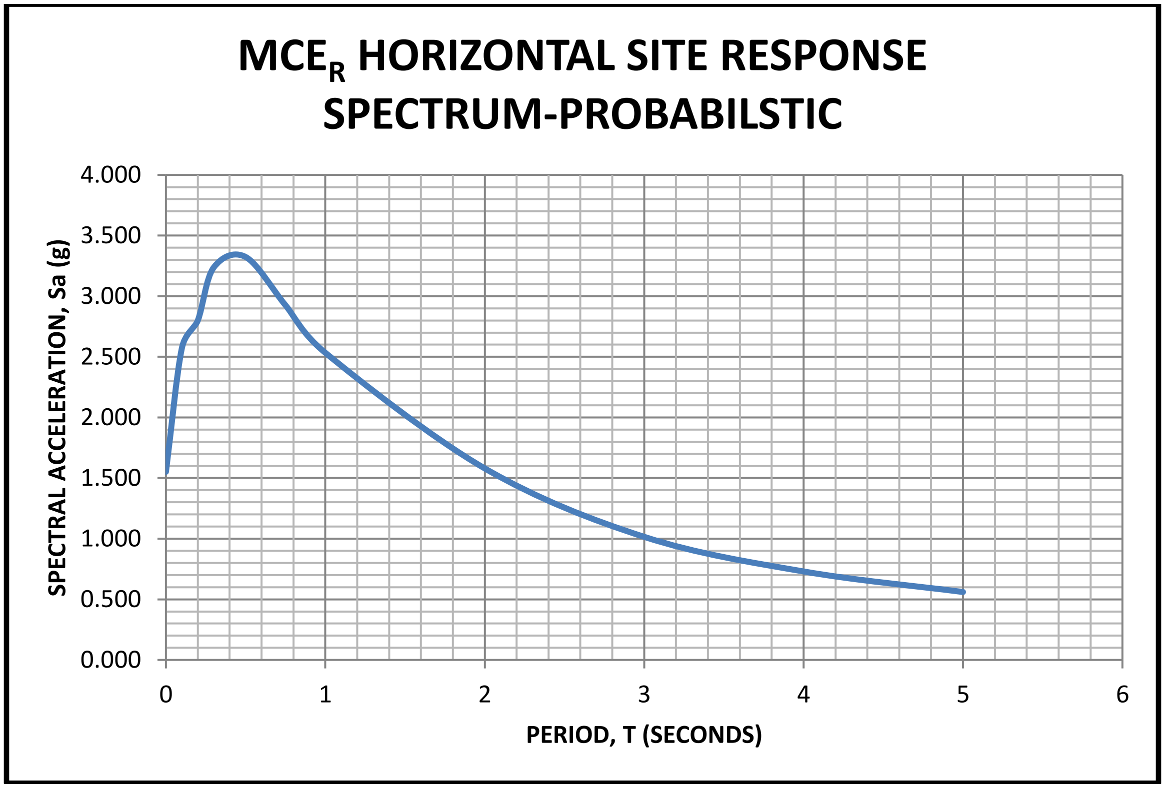

Diagram 1. Site specific MCE_R spectral accelerations from the probabilistic ground motion hazard analysis.

6.3.2 Deterministic Ground Motion Hazard Analysis

The deterministic ground motion hazard analysis was accomplished consistent with ASCE 7-16 21.2.2. The de-aggregation option of the US Geologic Survey (USGS, National Seismic Hazard Mapping 2014) national seismic hazard mapping website was used to evaluate the predominant earthquake source, magnitude and distance to the subject site contributing to the probabilistic (2% in 50 years) ground motion hazard. It was determined that the predominant earthquake source is from the Cascadian Subduction Zone, capable of >9.08Mw earthquake, with a mean source-to-site distance of 17 km.

Six ground motion prediction equations developed for Cascadian Subduction and Interface fault systems were used to calculate the 5% damped Horizontal Spectral Acceleration for the site. The deterministic MCE_R was determined as the geometric mean of the calculated spectral response accelerations at the 84th percentile. The following prediction equations were used; (Atkinson and Macias 2009), Atkinson & Boore, 2003, BC Hydro, (2016), BC Hydro, (2018), Montalva et al., (2017), (Grego, et al. 2002). The source-to-site distance of 50km and a moment magnitude Mw=9.0 were used in the equations. The site specific MCE_R spectral accelerations from the deterministic ground motion hazard analysis are given in Diagram 2.

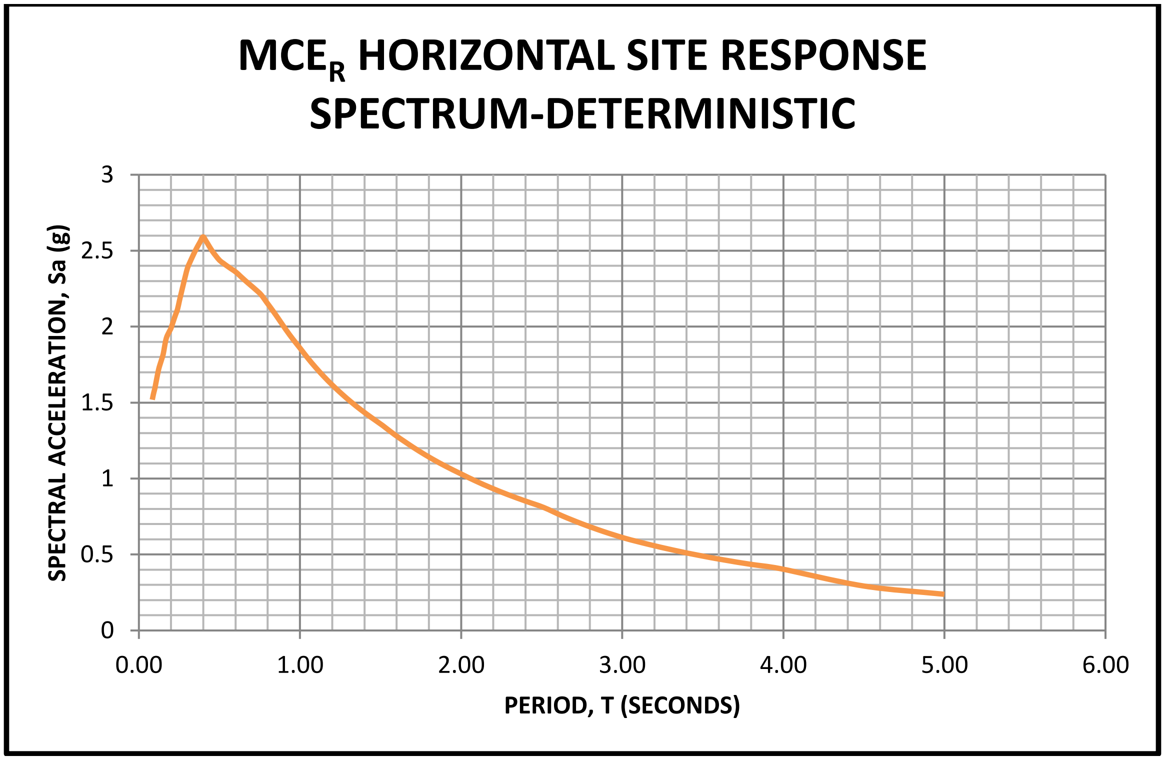

Diagram 2. Site specific MCE_R spectral accelerations from the Deterministic ground motion hazard analysis.

6.3.3 Site Specific MCE_R

The Site Specific MCE_R spectral response acceleration at any period, S_{am}, is taken as the lesser of the spectral response accelerations from the probabilistic ground motion hazard analysis and the deterministic ground motion hazard analysis, according to Section 21.2.3 of ASCE 7-16. The recommended site specific MCE_R is given in Diagram 3 (on the following page).

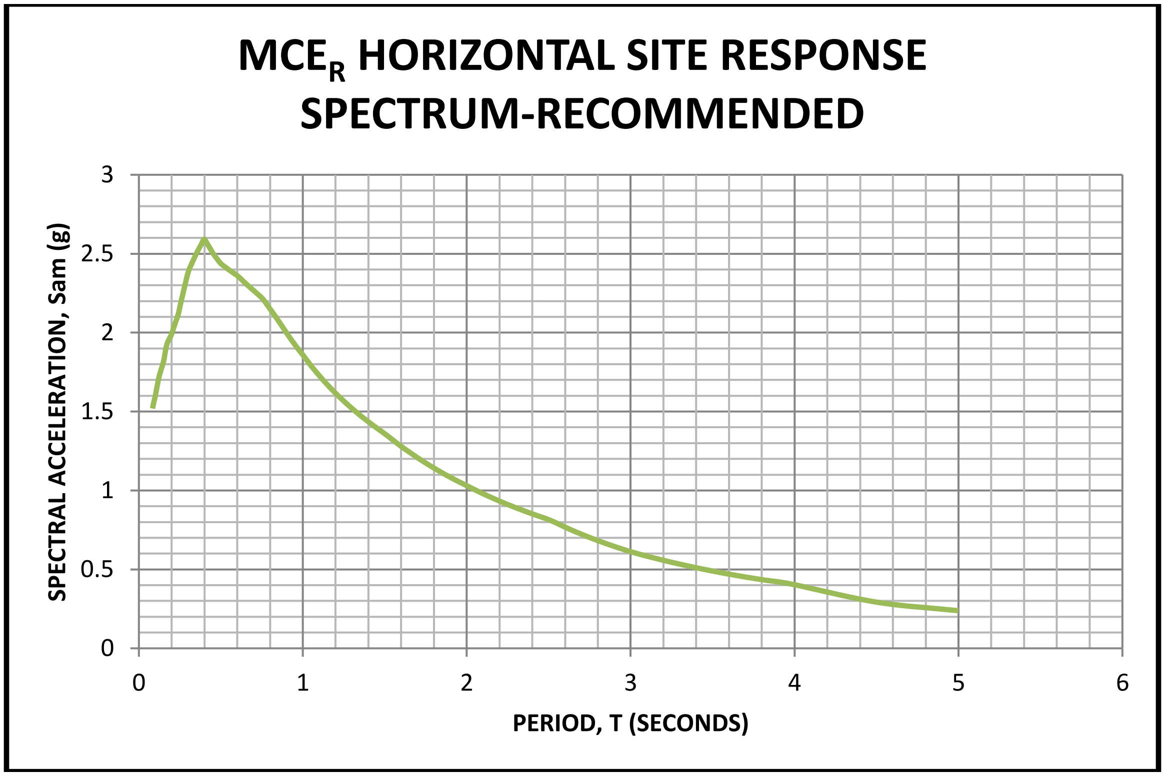

Diagram 3. Recommended MCE$_R$ Response Spectrum from the Site-Specific Ground Motion Hazard Analysis

6.3.4 Recommended Design Acceleration Parameters

The design earthquake parameters for the project area are based upon the methodology set forth in 2019 Oregon Structural Specialty Code and ASCE 7-16 Sections 21.3 and 21.4 and on the results of the ground motion hazard analysis. Recommended design acceleration parameters are provided in Table 5.

Table 5: Recommended Design Acceleration Parameters.

SITE SPECIFIC GROUND MOTION HAZARD ANALYSIS: Beach Loop Subdivision (02-6151-02)	
Project Area: 0 Beach Loop Road, Oregon	Latitude: 43.104276
	Longitude: -124.430659
Risk Category	III
Site Class	E
S_S=MCER Ground Motion (period=0.2s)	2.038 g
S_1=MCER Ground Motion (period = 1.0s	0.972 g
Design Spectral Acceleration **S_{DS}**, Short Period (ASCE 7-16 Section 21.4)	1.555 g
Design Spectral Acceleration **S_{D1}**, 1 sec Period (ASCE 7-16 Section 21.4)	1.228 g

Spectral Response Acceleration, **S_{MS}**, Short Period S_{DS}*1.5 (ASCE 7-16 Section 21.4)	2.332 g
Spectral Response Acceleration, **S_{M1}**, 1 sec Period (ASCE 7-16 Section 21.4)	1.842 g
MCE_G adjusted for site class effects, **PGA_M** (Probabilistic PGA Section 21.5 ASCE 7-16)	1.115 g
Seismic Design Category (Table 11.6-1 and 11.6-2 ASCE 7-16)	**E**
Per the requirements of Section 11.6 of the ASCE 7-16 code, SD1>0.75, therefore **SEISMIC DESIGN CATEGORY** is **E**	

7.0 CONCLUSIONS

In our professional opinion, based on our field investigation, laboratory testing and office review, the soils conditions at the site are suitable for the proposed development, provided the recommendations of our report are incorporated in the design and construction of the project. The project is in close proximity to the Cascadia Subduction Zone (CSZ) and would be subjected to severe shaking during a CSZ earthquake event. Special attention must be paid to the design and construction of the roadways, foundations and structures due to the presence of loose, sandy soils on the west side of the site. The following sections provide geotechnical recommendations for the design and construction of the planned improvements.

8.0 GEOTECHNICAL RECOMMENDATIONS

The subject site soils are marginal for support of proposed structural developments. The following sections provide methods for proper site preparation/grading, foundation support and related items.

8.1 SITE PREPARATION AND GRADING

The project site is generally covered with thick grasses, brush and trees. There is also thick organic debris near the ground surface of the project site. Therefore, normal methods of clearing, grubbing and stripping for organic removal and subgrade soil preparation will apply.

8.1.1 Clearing, Grubbing and Stripping

All areas proposed for roadways, structures, driveways, parking and walkways 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. It appears that a stripping depth of between 12 and 24 inches will be required to remove the topsoil. Additional stripping may be required to remove the deeper roots of some of the trees. The stripped materials should be hauled from the site or stockpiled for use in landscape areas only. This material must not be used in structural fill or trench backfill on this project.

Abandoned utility lines, storm drains, underground tanks or other items which provide void space beneath the surface must be removed or effectively plugged. Movement of surface and/or groundwater through these old conduits can create the potential for piping of soils (the removal of soil fines by water seeping into the void spaces or through conduits), resulting in subsidence of the surface or settlement of structures and paved areas.

Holes or depressions resulting from the removal of underground obstructions and old excavations, including test pits that extend below the finish subgrade, which will be beneath roadways or structures, shall be cleared of all loose material and dished to provide access for compaction equipment. These areas shall then be backfilled and compacted to grade with structural fill, as described later in this report.

It is recommended that stripping of the site and backfill and compaction of depressions below finish subgrade be observed by the geotechnical engineer or his representative.

8.1.2 Subgrade Densification

After removal of all vegetation, organic soil and deleterious materials and when the subgrade has been cut to grade, the exposed subgrade shall be redensified by numerous passes with a heavy vibratory roller. Due to the presence of loose sandy soils on the site, the subgrade shall be redensified by several passes and must achieve at least 93% of the Maximum Dry Density as determined by laboratory test method ASTM D-698 (Standard Proctor) or must successfully pass a proofroll, as described in the following section. This densification shall be accomplished under all areas of the site, including outside of the structure, and including concrete walkways and slabs and asphalt areas. *Redensification shall be discontinued if it starts to "pump up" the subgrade. Care must be used to not disturb prepared subgrade areas.*

8.1.3 Subgrade Preparation and Proofrolling

The exposed subgrade throughout the site which will support structures, roadways, exterior slabs, fills, driveways and sidewalks shall be proofrolled (after grubbing and stripping and over-excavation, where required) under the observation of a representative from The Galli Group. The proofrolling may be accomplished with a loaded to partially loaded dump truck, water truck or large heavy roller (no vibration). Proofrolling shall be discontinued if it appears the operation is pumping moisture up to the surface or otherwise disturbing the in-place soils. *When proofrolling, a successful test is when the tires of a loaded or partially loaded truck do not deflect the soils more than ⅜ inch.*

Where subgrade soils are disturbed or do not demonstrate a firm, unyielding condition when proofrolled, the soil should be redensified or aerated and redensified, or replaced with imported granular fill. The imported fill material shall be compacted to a minimum of 95 percent of the maximum dry density as determined by ASTM Test Method D-698 (Standard Proctor). All soft and/or unstable areas shall be over-excavated and backfilled with granular structural fill.

Note: This proofrolling and subgrade verification shall be accomplished on the exposed subgrade of over-excavated areas as well as the layers of structural fill and the finish subgrade surface.

We recommend our firm observe proofrolling of the subgrade after excavations are complete and prior to placement of structural fill. After completion of site stripping, excavation to subgrade, redensification and proofrolling, the contractor must take care to protect the subgrade from disturbance due to construction equipment, especially during very wet weather.

8.2 STRUCTURAL FILL PLACEMENT AND COMPACTION

Beneath Structures and Roadways. Structural fill is defined as any fill placed and compacted to specified densities and used in areas that will be under roadways, structures, driveways, sidewalks and other load-bearing areas. It appears that the footings, floor slabs, garage slabs, roadways, sidewalks, parking and other structures will be founded over fill. The subgrade needs to be prepared properly and the fill must be placed and compacted correctly for proper long-term performance.

Structural Fill Materials. Ideally, and particularly for wet weather construction, structural fill must consist of a free-draining granular material (non-expansive) with a maximum particle size of six inches. The material shall be reasonably well-graded with less than 5 percent fines (silt and clay size passing the No. 200 mesh sieve). During dry weather, any organic-free, non-expansive, compactable granular material, meeting the maximum size criteria (6" diameter), is typically acceptable for this purpose. Locally available crushed rock and jaw-run crushed "shale" has performed adequately for most applications of structural fill.

***Note:** It is the contractor's responsibility to understand the impending weather and plan for use of structural fill that will be capable of being compacted properly and remain stable under the expected construction traffic in all weather that could arise during the project construction.*

Structural Fill Placement. Structural fill shall be placed in horizontal lifts not exceeding 8 inches loose thickness (less, if necessary, to obtain proper compaction) for heavy compaction equipment and four inches or less for light and hand-operated equipment. Each lift must be compacted to a minimum of 98 percent of the maximum dry density, unless otherwise specified, as determined by the Standard Proctor test, (ASTM D698/AASHTO T99). Compaction shall be by mechanical means; "jetting" or water settling will not be allowed.

Beneath Footings. When structural fill is used beneath footings or other structural elements it must extend beyond all sides of such elements a distance equal to at least ½ the total depth of the structural fill beneath the structural element in question for vertical support (i.e., for 2 feet of structural fill beneath footings, extend the fill at least 1 foot past all edges of the footing).

To facilitate the earthwork and compaction process, the earthwork contractor must place and compact fill materials at or slightly above their optimum moisture content. If fill soils are on the wet side of optimum, they can be dried by continuous windrowing and aeration or by intermixing lime or Portland Cement to absorb excess moisture and improve soil properties. If soils become dry during the summer months, a water truck must be available to help keep the moisture content at or near optimum during compaction operations.

Fill Placement Observation and Testing Methods. The required construction monitoring of the structural fill utilizing standard nuclear density gauge testing and standard laboratory compaction curves (ASTM D-698 specified) is not applicable to larger jaw run shale (2" or above) or larger crushed rock. The high percentage of rock particles greater than ¾'s of an inch in these materials causes laboratory and field density test results to be erratic and does not provide an adequate representation of the density achieved. Therefore, construction specifications for this type of material typically specify method of placement and compaction coupled with visual observation during the placement and compaction operations.

Observation of Fill Placement. For larger rock materials, we recommend the 8-inch lift (after being "worked in") be compacted by a minimum of 3 passes with a heavy vibratory roller. One "pass" is defined as the roller moving across an area once in both directions. The placement and compaction must be observed by our representative. After compaction, as specified above, is completed the entire area shall be proofrolled with a loaded dump truck to verify density has been achieved. *All areas which exhibit movement or compression of the rock material more than ¼ inch, under proofrolling, must be reworked or removed and replaced as specified above.*

Nuclear Density Testing of Fill. Field density testing by nuclear density gage would be adequate for verifying compaction of 2-inch to ¾-inch minus crushed rock aggregate base sand/gravel soils, Decomposed Granite and other materials 2 inches or smaller in size. Therefore, typical % compaction specifications would suffice. Testing must be accomplished in a systematic manner on all lifts as they are placed. Testing only the upper lifts is not adequate.

8.2.1 Non-Structural Fill

Any waste soil, organic strippings or other deleterious soil would be considered non-structural fill. These materials may make excellent landscape soils and lawn topsoil material. This material may be placed in landscape areas and waste soil areas. It should not be placed as part of a structural fill slope. It is recommended that when these soils are used they be given a moderate level of compaction (90 to 92 percent) to help seal them from surface water. When utilized in berms less than 10 feet in height we recommend side slopes no greater than 3.5H:1.0V and the soils be compacted to at least 92%. Some downslope soil creep may be expected of the surface soil layer.

8.3 SITE EXCAVATIONS

During construction on the west side of the project, deep excavations below the exiting surface grades will be required to create a parking garage below the exterior finish grade elevations of the proposed hotel. In addition, excavations will be required for construction of utility lines and roadways within the project. These excavations will encounter the loose fine Sand and possibly some of the Sand and Gravel below the water table.

Excavators of all sizes should have no difficulty excavating the loose sand on the west side of the project site. Only moderate size and larger excavators will be able to excavate on the east side to depths of below 4 feet, if required. Deeper excavations, such as the one required to create the underground parking garage, if accomplished without shoring support or at the recommended temporary cut slope inclination will be unstable and would have significant failure that could impact adjacent properties and create unsafe conditions during construction. These will likely require shoring especially along the west property boundary.

Note: *Do not pile trench spoils within 8 feet of the excavation sides. This will likely cause sidewall collapse.*

Site Dewatering

During the project's subsurface exploration, free groundwater was encountered at between 20 and 34 feet from the existing ground surface on the west side of the site and at approximately 10 feet on the east side. Depending on the final elevation of the underground structure’s footings, ground water may be encountered during the construction of the hotel. On the east side, we expect seasonal fluctuations in the groundwater level. The highest groundwater tables may coincide with high tides on the pacific ocean. We anticipate groundwater levels to fluctuate and come close to 2 feet below the ground surface during the wetter months of very wet years, and deeper following most late summer months. Depending on the time of the year the excavations underneath the building footprint and the utility excavations are carried out, these excavations will likely encounter the free groundwater. Therefore, for construction to go on below the observed water table (if still observed prior to construction), site dewatering will be necessary.

Due to the hydraulic conductivity of the site soils, dewatering can be achieved by installing a series of sump pumps strategically around the site. The sump pumps may be installed at different depth intervals and on an as-needed basis. The contractor is responsible for lowering the water table prior to construction.

Note: There will be recharge from around the excavation once the excavation level drops below the water table. However, it appears that such recharge will decrease with time. Pumping from several open sumps that penetrate to below where excavation is taking place could work well. It must be noted also that when water is flowing/seeping out of the soil layers beneath the water level and into the excavation, these slopes can become unstable. The stability will improve after the site areas have drained of water. Shoring with drainage behind will likely be required in these areas in order to keep the cut faces and areas behind stable during construction.

Excavation Shoring

During the excavation of the underground garage for the hotel, a slope failure is probable on the west property boundary if a cut slope cannot be constructed at the recommended safe inclinations (see *Section 8.4.1*). Such failure would impact adjacent properties and pose a threat to the life, health and safety of workers/pedestrians in the area. Mitigation measures will be necessary to protect against the probable failure.

We recommend installation of shoring prior to excavation, in order to mitigate the possibility of excavation wall failure. This can be accomplished in the form of cantilever shoring utilizing H piles with lagging in between or sheet piling. The piles must be designed to provide the strength necessary to bear against the expected earth pressure that would be imposed on it. Detailed design and recommendations for the shoring is beyond our current scope of services. If requested, we will gladly provide these additional design services.

8.4 CUT AND FILL SLOPES

Cuts and fills will be required for the roadways and structures. These should be designed as described below.

8.4.1 Temporary Cut Slopes

The west side of the project site has sandy subsurface soils that are dry and loose. It will be difficult for temporary slopes to stand at typically recommended inclinations. We recommend temporary cut slopes in this area of the site be cut at 4H:1V or flatter for stability.

Temporary cut slopes in unsaturated medium dense sandy soils on the east side of the site may be cut at 1.0H:1V or flatter in dry weather conditions. The contractor must be prepared to flatten temporary cut slopes to 2.0H:1V for any cut slope locations subjected to emerging groundwater seepages, or during extended periods of heavy rains.

8.4.2 Permanent Cut Slopes

Permanent cut slopes are very likely to be constructed on portions of the west side of the project site as part of the proposed hotel and roadways grading operation. Such permanent cut slopes will encounter the loose sandy soils, and must be constructed at inclination of 5H:1V (20%). We do not anticipate permanent cut slopes on the relatively flat east side of the project.

8.4.3 Fill

Fills may be required for site grading, building pad construction and roadway base. Fills may be constructed of imported rock/shale or from the native beach sand. All fill slopes shall be constructed at the following inclinations or flatter;

Native fine Sand (Roadway Only)	3.5H:1V
Imported Angular Crushed Rock	1.75H:1V
Imported Jaw Run Shale	2.0H:1V

We recommend, in order to decrease sloughing and erosion of the fill slope, that all fills be over-built and the face cut back to a compacted fill face. This would not be required of slopes constructed of imported rock fill materials. It is critical that these fills be placed and compacted properly to decrease long-term settlements beneath the roadways (if these are to be placed over new fill). We recommend periodic density testing of all fills as they are being built. Density testing only on the top lift is not adequate.

8.4.4 Fill on Slopes

Fill placed on sloping areas of the site (slope angle of underlying native slope 10% or greater) must incorporate additional precautionary measures. To assure that these fills remain in place due to static and seismic loads, or do not fail due to hydrostatic pressure of trapped water, we recommend the following:

Key Trench. The toe of all fills placed on slopes must be keyed into the slope by use of a key trench. The depth of embedment should be 2 feet into densified native soils for fill slopes up to 8 feet high, 3 feet for fill slopes up to 15 feet high and 4 feet for fills over 15 feet high. The key trench should run parallel to contour lines, be wide enough to accommodate excavation and compaction equipment (9 to 10 feet minimum), and have the base flat or sloped back into the hillside somewhat. All loose or soft soil must be removed from the key trench prior to placement of structural fill.

Benching. The underlying native slope should be cut into flat benches (into undisturbed native soil) back up the slope above the key trench, prior to placement of the fill slope. These benches should run parallel to the contour lines and be flat or tipped back slightly into the hillside. Please see *Figure 4* for graphic representation of these details.

Drainage. All noticeable seepage or wet zones observed during the keying and benching excavation process (or other areas which may seep during wet weather) should be provided with subdrains. At the discretion of the geotechnical engineer, at a minimum, the key trench and upslope end bench may require a subdrain section. Where wet conditions exist, the other benches may also require subdrain sections to remove subsurface flow from behind the new fill. Please note that fills placed on slopes typically have a much lower lateral permeability than the native soils. Therefore, seepage through the native soil can become trapped behind these fills causing fill slope stability problems, if proper subdrains are not installed. We recommend that our engineer observe all keys and benches during construction to verify where subdrains should be installed.

8.5 UTILITY LINE RECOMMENDATIONS

Below we have provided general recommendations for utility construction for the project. Recommendations are based upon observations from our field investigation and experience on other projects in similar soils conditions.

8.5.1 Trench Excavation

Trenches will be required across the site for utility installation of various kinds. Trench excavation should be relatively easy in all areas of the site. However, temporary cut slope inclinations will be required for trench excavations deeper than 1 foot on the west and 4 feet on the east side of the site.

8.5.2 Trench Backfill and Compaction

The new utility lines will require trench backfill and compaction along the entire alignment. The pipes need to be adequately supported and the trenches need to be backfilled and compacted properly to prevent subsidence of the surface or damage to utility lines or the potential overlying pavement section.

In our experience, utility trench backfill has been the source of the majority of post-construction fill settlement problems in paved areas. Therefore, we recommend trench backfill be placed as structural fill.

Pipe Bedding. The bottom of the trench must be shaped out of acceptable bedding materials (refer to manufacturer’s recommendations) to fit the pipe base prior to placement of the pipe. It is critical to the long-term performance of the pipe that the bottom and haunches be fully supported by a dense bedding which decreases pipe distortion from load. If the native soils can be cut with a clean, undisturbed, rounded bottom, that fits the pipe size, they would support the pipe. Otherwise, crushed rock or lean mix cement slurries usually make good bedding materials.

Pipe bedding should be compacted to 95% of ASTM D-698 (Standard Proctor) or to that specified by the pipeline designer. Cement-treated pea gravel or sand/cement slurry (with at least 200 pounds of cement per cubic yard) will solidify and would typically not require compaction after placement and makes good bedding material. Care must be taken to make sure the pipe does not “float” up in the fluid mix.

Pipe Zone Material. All of the lines should be backfilled around and to approximately 12-inches above the pipe with an acceptable "pipe zone" material. This may consist of finer crushed rock, cement-treated pea gravel, sand/cement slurry, coarse sand with fine gravel, or other material acceptable to the City and pipeline designers. The pipe zone material should be well compacted on each side of the pipe, and to within at least 12-inches above the pipe. Mechanical means will be required to densify these materials to the required densities (unless a cement-treated material is used).

Density requirements for "pipe zone" backfill should be per the manufacturer's specifications for the type of pipe being used (we recommend using 95% of ASTM D-698). Care should be taken when compacting close to and immediately above the pipe so as to not damage the pipe.

General Trench Backfill. Above the "pipe zone" the backfill materials may consist of any compactable material that does not have excessive voids (such as gap-graded large gravels and cobbles), organics, expansive clay, debris or other deleterious material. The soft rock material (if open graded) should be mixed with other fine-graded materials to achieve a material that does not have excessive voids (which could allow piping of soil fines resulting in surface subsidence). Crushed rock, jaw-run shale, onsite sandy soils and sandy, decomposed granite will work well for general trench backfill.

Where laterals of any kind, or valving, extend upward from the lines, we recommend the trench areas adjacent to these items be backfilled with the "pipe zone" backfill materials. This will prevent the larger pieces of other backfill materials from damaging the valves and/or other equipment.

We strongly recommend that all general trench backfill be placed and compacted in the same manner as for general structural fill. Trench backfill beneath asphalt pavements but not under structures should be compacted to at least 98 percent of the maximum dry density, as determined by ASTM Test Method D-698 (Standard Proctor) for the upper 36 inches. Below 36 inches the trench backfill should be compacted to between 95 and 98 percent of the maximum dry density. Trench backfill in landscape areas, that are not part of a cut or fill slope, may be compacted to at least 93 percent of the maximum dry density. The trench backfill should also be systematically tested during construction. This should include observation and testing periodically along utility trenches on all lifts.

8.6 STRUCTURE SUPPORT RECOMMENDATIONS

Typically, a hotel structure with underground parking, such as proposed on the west side, with moderate to large foundation loads, could be supported on conventional spread footings founded in medium dense to dense, underlying soil zones. However, the site has surface and deeper layers of very loose to loose Sand. Column loads may exceed 250 kips. Wall loads may range from 1 to 4.5 kips per linear foot. As a result, total and differential settlement anticipated when utilizing conventional spread footings, will almost certainly be more than these structures can typically tolerate. There is also the possibility of up to 6.5 inches of seismic settlement during the design seismic event. Therefore, we recommend structural loads be supported utilizing a structural slab over grade beams and deep foundations. Alternatively, the use of geo-aggregate piers or aggregate columns (a soil improvement method that can improve support and decrease settlement) may be able to provide support for the proposed structures on the west side of this project.

For the structures proposed on the east side, we recommend a conventional foundation over geogrid reinforced crushed rock building pad or structural slab over conventional spread footing foundation be utilized to mitigate liquefaction related settlements associated with the site soils on the east side.

8.6.1 Conventional Foundations (EAST ONLY)

8.6.1.1 Building pad Construction

Liquefaction below the water table is likely during a seismic event. As a result we recommend full support of structures with a geogrid and crushed rock pad. This support method will help mitigate the impact of liquefaction and liquefaction induced settlement during the design seismic event.

For residential structures with conventional footing and crawl space foundation, support required will be a 24 inch thick building pad constructed of angular compacted crushed rock and geogrids, all below the bottom of footings. The base of this pad must be kept at least 1.5 feet above the groundwater level at the time of construction to allow for proper compaction and support of construction equipment. Please see *Figure 5, Building Support Pad* and *Figure 6, Geogrid Placement Pattern* for how this must be constructed.

Construction of this building support pad shall be as listed below.

1. Overexcavate down to at least 24 inches below the base of the deepest footing. Size of excavation base to be at least 3 feet beyond the outside edge of all exterior footings.
2. Redensify the excavation subgrade.
3. Place Geogrids as shown on *Figures 5 and 6*. Geogrids must be pulled tight prior to rock placement. Overlap and position as described on Figure 6.
4. The geogrids should be pulled tight (no wrinkles) and should have full 12 foot overlap at the corners and where cross geogrid laps over the long sides. Geogrid material shall be Fortrac 80/30-20, Miragrid 7XT, or equivalent. Overlap all edges at least 2 feet and ends at least 8 feet. Place 2" to 3" of 1½ inch minus crushed rock between all overlapped geogrids.
5. Place and compact the crushed rock structural fill in 8" lifts, compacted to 98% of ASTM D-698 (Standard Proctor). **Note:** First lift may be 12" of crushed rock.
6. **Note:** Crushed rock must be well graded when placed against the native sands. If not it must be separated from the sand by a filter fabric.
7. Place and compact crushed rock up to footing subgrade. Thickness of the crushed rock over the geogrids and beneath the footings must be at least 24 inches.

> **Note:** It is acceptable to bring the structural rock fill up to footing subgrade, construct the footings and then construct the remainder of the structural fill pad adjacent to the footings and stem walls (if any) up to floor section subgrade.

All portions of the subgrade densification, placement of the geogrid and compaction of the structural fill above must be inspected and tested by our representatives.

8.6.1.2 Foundation Recommendations

Footings placed on the building pad constructed as described above may be designed as listed below.

1. Redensify any loose crushed rock on the surface.
2. Footings placed on the structural building pad may be designed for an allowable bearing pressure of 2,000 pounds per square foot. A 1/3 increase in this allowable bearing pressure may be used when considering short-term transitory wind and seismic loads.
3. Spread footings shall have their base buried a minimum of 12 inches below finish grade in order to provide lateral support and frost protection.
4. We recommend minimum lateral dimensions of 12 inches for continuous load bearing footings and 24 inches for isolated piers constructed in this manner.

Anticipated Settlements. For properly constructed foundations founded on the redensified, geogrid supported pad, we anticipate maximum total and differential settlement to be approximately 3/4-inch and 3/8-inch, respectively. These should occur fairly rapidly given the granular nature of the site soils. .

NOTE: Seismic Settlement. Additional settlement will take place during a seismic event. Our computations indicate liquefaction settlement at the site could be between 4 inches and 6 inches. Given the proposed rock and geogrid pad, the seismic settlement will be even, mitigating severe differential settlement during such a seismic event.

8.6.1.3 Interior Floor Slabs

The properly prepared building pad area discussed above would provide very good support for the concrete slab-on-grade floors such as in the garage.

Slab Section. The following recommendations are provided for slabs constructed on the compacted crushed rock building pad.

1. Backfill around and above footings with compacted structural fill up to slab section subgrade. This may consist of the excavated sand soils or rock products. Compact in lifts to at least 98% of ASTM D-698 (Standard Proctor).
2. Place 8 inches of compacted 3/4-inch minus crushed rock for slab support.
3. A tough impermeable membrane, such as Stego Industries 10 mil or 15-mil Stego Wrap vapor barrier (or an equivalent product) should be placed over the rock

layer to further retard upward migration of moisture vapor into and through the concrete slab. Seal all seams well with manufacturers recommended method.

The building pad area beneath the interior slab shall be accomplished as described earlier in this report (*Section 8.5.1*).

Note: If it appears water may pond in the rock below the slab, a series of slab subdrains should be installed. These shall be constructed as shown on *Figure 9* and as described later in report *Section 8.9*.

8.6.2 Structural Slab Foundation Alternative (EAST ONLY)

Alternatively, the residences may be supported on a monolithic structural slab. This will require excavation into the underlying, undisturbed medium dense to dense sandy soils. If this method of support is utilized, the foundation must be prepared as follows.

1. Excavate the entire footprint of the structure to expose the underlying, undisturbed, medium dense to dense sandy soils. The base of the excavation should extend to a minimum of 6 inches below the base of the foundation and should be excavated with a smooth bucket (no teeth) to the subgrade to minimize disturbance to the soils. All areas excavated beneath the foundations must extend beyond all edges of the exterior footing a distance equal to at least 2 feet.
2. Excavate all footings both exterior and interior. Allow for at least 6 inches of crushed rock below all footing base.
3. Recompact any loosened areas of the subgrade.
4. Place 6 inches of compacted crushed rock fill on both the footing and structural slab section. Place and compact to at least 98% per ASTM D-698 (Standard Proctor).
5. Construct a structural slab over the footing in a monolithic concrete pour. The slab must be structurally tied to the footing. The rebar size, spacing and placement for the slab must be designed by a structural engineer.

Anticipated Settlements. For properly constructed foundations, we anticipate maximum total and differential settlement to be less than 3/4-inch and 3/8-inch, respectively.

Foundation Drains. We typically recommend all footings be installed with a footing drain to intercept groundwater seepage. Footing drains shall consist of a rigid, smooth-wall perforated pipe surrounded by drain rock (sides and above), all wrapped in a non-woven geotextile fabric and shall be placed adjacent to the footings. See *Figures 7 and 8* for details. This is addressed more fully later in this report (*Section 8.9*).

Note: We recommend each structure be reviewed to verify that the proposed foundation scheme is consistent with our geotechnical recommendations and the specific conditions on the subject lot.

Note: If it appears water may pond in the rock below the slab, a series of slab subdrains should be installed. These shall be constructed as shown on *Figure 9* and as described later in report *Section 8.9*.

8.6.3 Deep Foundations (WEST)

Due to the very loose condition of the subsurface soils, deep foundations must be used for support of the hotel and all structures on the west side of the project site. Estimates of settlements exceeded 5 inches for static condition only. Typical deep foundation methods include driven piles, drilled auger cast piers or geo aggregate piers. These deep foundation supports must be embedded into the very dense sands and sandy Gravel soils which were encountered beneath the site. We recommend the driven piles, drilled piers or geo-aggregate piers be located in a grid pattern around the perimeter and across the interior of the structure. Surface grade beams must then be used at the tops of piles/piers in order to provide structural connections in both directions for load transfer and to help minimize potential for foundation separation.

The final length of the deep foundations will be determined after the final grades and lowest finish floor elevations had been determined. We assumed an elevation of 70 feet for the bottom of the lowest footing and deep foundation length of 25 feet for our analyses and recommendations below.

Driven, Small Diameter, Steel Pipe Piles

Several types of driven piles can be used to support or resist such loads. Small diameter pipe piles are typically the least expensive and easiest to install option for deep foundations on projects such as this. Design capacity of various piles are typically as follows:

Piles Type and Size (in)	Pile Bearing Capacity (Kips)	Uplift Capacity (Kips)
3” Pipe	10	4.5
4” Pipe	15	6.5
6” Pipe	30	9.5
12”	90	19

1. These estimated design capacities use a factor of safety of 2.0 for compression and 3.0 for uplift. Actual capacities will be based on pile load testing prior to pile installation. We recommend a minimum of 3 pile tests for the project.
2. We anticipate use of Sch 40 galvanized steel pipe piles driven to refusal into the very dense sandy Gravel.
3. Length of pile will depend on the final footing base elevation for the hotel and other associated structures. Capacity estimation assumes 25 feet pile length.
4. Utilize a pile hammer/vibratory driver, appropriately sized for the piles to be installed.

5. Final set/refusal criteria will likely require that the piles be driven until less than 1 inch of advancement in 10 to 15 seconds of continuous driving has been attained (must be verified when pile size and hammer/driver type and size have been selected).
6. Piles shall be placed at least 5 pile diameters apart (minimum of 2 feet).
7. Piles may be driven at a batter up to 30° off the vertical for added lateral resistance (verify maximum possible batter with pile driving subcontractor).
8. Piles will require a steel top cap/reaction plate, embedded up into the new footings/grade beams. These shall be designed/verified by the structural engineer.
9. Pile splices are allowed. However, they must be at least 6 feet below the top of the piles. Friction couplers/splices cannot be used for uplift. Welded caps and welded splice connections must be used where piles are designed for uplift capacity. This must be verified by the structural engineer.

Anticipated Settlement. For steel pipe piles driven to “refusal” as described above, total and differential settlements are anticipated to be less than 0.4 inches and 0.3 inches, respectively.

Reinforced Concrete Drilled Piers

Reinforced concrete piers may also be used to provide needed support and to limit settlement. They provide higher capacity than driven piles but may be more expensive to construct. Design capacity of various pier diameter are typically as follows:

Piles Type and Size (in)	Pile Bearing Capacity (Kips)	Uplift Capacity (Kips)
12” Pier	45	9.5
18” Pier	90	14.5
24” Pier	150	19.5
30” pier	230	24

1. Estimated design capacities use factor of safety of 2.0 for compression and 3.0 for uplift. Actual capacities will be based on pile load testing following pile installation. We recommend a minimum of 2 pile tests for the project.
2. Piers shall be located at least 3 pier diameters apart. Alternate diameters and spacing of the drilled piers may also be used for the vertical and lateral loads if deemed acceptable by the structural designer. We recommend a minimum 4-foot center-to-center spacing.
3. All drilled piers must be embedded at least 5 feet into the underlying dense sandy Gravel soils encountered beneath the site. Hole and embedment depths must be verified by the geotechnical engineer at the time of drilling.
4. All corners and ends of footings must have piers for support.
5. Keep base of hole clean of all loose materials and water until steel reinforcement and concrete is placed.
6. We recommend at least an 18” deep footing/grade beam over the top of the piers.

Note: Due to the **presence of groundwater**, the drilled pier option may not be the best. Contractor must be ready to deal with the groundwater challenges this option will present with construction options such as auger cast method of installation.

Ground Improvement by Geo-Piers

Ground improvement methods such as geo-piers are a good alternative to driven or auger-cast piles if overall site conditions are suitable for this option. The method considered is the Geo-Pier Impact Displacement system. This consists of using a hollow mandrel with an internal compaction surface that is forced into the ground. This is accomplished by using both a static down force and dynamic vertical impact energy. When the mandrel is driven to the design depth the mandrel then serves as a conduit for aggregate placement. The mandrel is raised and then re-driven downward with thin lifts of rock below. This creates rock layers that are densified vertically and horizontally. This is repeated until this densified aggregate "pier" is formed.

The geo-piers are installed on an offset grid pattern throughout and beyond the footprint of the structure. This would normally include rows of geo-piers directly under the foundations where allowable bearing capacities of up to 3,500 psf can usually be attained. There would then be a grid pattern under the entire building and at least one row extending around the building.

Besides providing footing support, the geo-piers will also provide soil densification and short drainage paths to ensure soil liquefaction in the soil around them does not take place. With liquefaction mitigation included, the spacing is usually tight enough to provide good lower floor slab support. In this way the lower floor can be designed as a conventional slab-on-grade instead of a structural slab.

Typical depths for the Rammed Aggregate Piers (RAP) created by this process are between 10 and 40 feet. This depth could work well for the proposed new hotel. The entire system of installation, structure support and liquefaction mitigation would be designed by the contractor's professional engineers well familiar with their methods and results.

Foundations supported by the geo-pier soil improvement would have to use the mass of the foundations only for uplift resistance. Where spread footings extend beyond the foundation stem walls or columns, are buried with site fill and structurally connected to the columns, the overlying mass of soil may also be used for uplift resistance.

The geo-pier method should be able to limit static consolidation settlement to a maximum of 1 inch for total settlement and ½ inch for differential settlement. This would also increase the allowable bearing capacity to 3,500 psf. This allows for smaller/narrower footings that then have a stress influence that is shallower.

We recommend this method of structure support be considered. It could result in cost savings over conventional deep foundation methods.

Seismic Impacts on Geo-Pier Support. Geo-Pier support through the loose sands and into the dense sandy Gravel will help mitigate/eliminate seismic induced settlement. This would then create a deep zone of densified soil and RAP beneath to just outside the entire structure. During the design seismic event this mass of soil would provide adequate support to the structure.

Note: Number and location of driven piles, drilled piers or RAP will be determined by the structural engineer, based on the structure loads and design capacities provided above. We recommend we be allowed to review the foundation plan and pile/pier locations prior to final plans generation.

8.7 LATERAL LOAD RESISTANCE

8.7.1 Foundation Members

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

We recommend the use of passive equivalent fluid pressures of the following values for portions of the structure and foundations embedded into the native soils.

• Native Soils (below 12”)	250 pcf
• Dense Compacted Crushed Rock (below 18”)	450 pcf
• Improved Ground	550 pcf

A coefficient of friction of 0.55 can be used for elements poured neat against angular crushed rock structural fill. These should be reduced to 0.20 for areas over a vapor barrier or 0.35 over native soils.

8.7.2 Piles and Piers

All Piles or Piers installed will also provide added lateral resistance. These were computed for the upper portion of the support members and are presented below.

Pile Diameter (in)	Lateral Resistance (kips)
3	2.5
4	3.75
6	5.85
12	10.0

Pier Diameter (in)	Lateral Resistance (kips)
12	9.25
18	10.15
24	11.05
30	12.15

(1) These were computed with a factor of safety of 2.0. Actual capacities will be based on pile testing following pile installation.
(2) Assumes approximate maximum of 0.75-inch lateral movement at top.
(3) Assumes full embedment and fixed top of pile/pier into foundation to prevent rotation at the top.

8.8 LATERAL EARTH PRESSURES

Lateral earth pressures will be imposed on all below ground and backfilled structures or walls, including foundations which do not have uniform heights of fill on both sides and grade separation retaining walls. The following recommendations are provided for design and construction of conventional reinforced concrete or CMU block retaining walls as well as for mechanically stabilized earth (MSE) walls:

- We recommend walls which are free to rotate at the top (unrestrained) when backfilled, be designed for the following loads.

Native Sandy Soils EFP	48 pcf
Low Grade Angular Rock/Shale EFP	40 pcf
Crushed Rock EFP	35 pcf
Seismic coefficient (up to 10 feet tall)	0.556

- Walls that are fixed at the top (restrained) when backfilled should be designed for the following loads.

Native Sandy Soils EFP	70 pcf
Low Grade Angular Rock/Shale EFP	50 pcf
Crushed Rock EFP	45 pcf
Seismic (up to 10 feet tall)	1.115

- The walls all must have full drainage as described in *Section 8.9* and as shown on *Figure 10*.
- These equivalent fluid pressures are to be used for the soil through which the anticipated failure plane will develop (assume envelope beginning 4 feet behind base of wall and rising up and away from wall at 60 degrees off the horizon).
- A wet soil unit weight of 140 pcf should be used for design of retaining walls which are backfilled with crushed rock or jaw-run “shale”. Use 130 pcf for native soil backfill.
- These values are for properly compacted, free draining walls. The onsite sandy soils as well as imported crushed rock or clean jaw-run “shale” work well for wall backfill materials.
- These design values assume the wall or structure is fully drained, has a flat backfill and has no surcharge loads from traffic or other structures. The structural designer should include surcharge loading from traffic, building loads and/or sloped backfill.

- We recommend designing retaining walls to resist seismic loading. A horizontal acceleration coefficient (k_h) of at least 1.11g (for rigid retaining walls that cannot accommodate any lateral displacements) and 0.55 (for retaining wall systems that can accommodate 1-2 inches of lateral displacement) should be applied to the mass of an enlarged active wedge of soil behind the walls and utilized in a pseudo-static analysis. The wedge length back from the wall along the ground surface may be taken to be 0.8H, where H is the height of the wall.
- The backfill should be placed in lifts at near the optimum moisture content (at 2% to 3% above optimum) and compacted to between 93 and 95 percent of the maximum dry density as determined by laboratory procedure ASTM D-698 (Standard Proctor). Loosely placed backfill will exert greater pressures on the wall than the pressures provided above and must be avoided.
- To prevent damage to the wall, backfill and compaction against walls or embedded structures should be accomplished with lighter, hand-operated equipment within a distance of 1/2 h (h being the vertical distance from the level being compacted down to the surface on the opposite side of the wall). Outside this distance, normal compaction equipment may be used.

While proper compaction of wall backfill is critical to the proper performance of the walls, care should be taken to not over-compact the backfill materials. Over-compaction can induce greater lateral loads on the wall or structure than the design pressures given above.

Wall Loading Considerations. All retaining structures are acted upon by lateral earth pressures from the wall backfill, surcharge loads from sloping backfill, vehicles, structures and dynamic loads during seismic events.

The static lateral earth pressure exerted on the wall is highly dependent upon compaction of the backfill materials. Looser backfill will exert greater pressures. They are also more susceptible to a decrease in shear strength during wet weather. Therefore, proper compaction must be accomplished during construction for the above-listed soil strength parameters to be valid.

All of the design recommendations assume the following:

- The wall and backfill are placed on level benches with proper toe embedment on the downslope side.
- All walls are fully drained and any benches cut into the slope to facilitate backfill are fully drained (Please see the attached *Figure 8*).

8.9 FOUNDATION AND RETAINING WALL DRAINS

All exterior foundations and embedded structures should have proper drainage.

Footing Drains. Foundation drainage shall consist of a rigid, smooth-wall perforated pipe surrounded by at least 6 inches of drain rock on the top and outside edge, all wrapped in a non-woven geotextile designed as a filter fabric (such as Mirafi 140N or equivalent). The perforated pipe shall be located on the footing next to the stem wall (or beside the footing), provided this is at least 12 inches below underslab drain rock (if applicable). Please see *Figures 7 and 8*.

Floor Sub Drains. Where the drain rock layer below slabs will be lower than the adjacent exterior grades, water will tend to accumulate in this low area. To remove the water, include a series of subdrains at the bottom of the drain rock layer beneath the slab. The subdrain lines typically consist of 3-inch diameter, smooth interior, solid wall, perforated pipe at spacing of 10 feet (or less) across the structure (and around the interior perimeter). The perforated pipe is placed in a deepened zone of the drain layer as shown on *Figure 9*. The pipes are sloped to drain and collected by a tightline which leads to the stormwater disposal system. We recommend we be allowed to review the subdrain system design prior to final plan submittal or construction bidding.

Retaining Wall Drainage. Wall drains should also have a minimum 12-inch-wide drainage zone of drain rock wrapped in non-woven filter fabric immediately behind the wall extending up from the drainage section to within 12 to 18 inches of the surface. A preformed, fabric-wrapped, polymer sheet drain, such as Amerdrain, Linq Drain or Enkamat must be placed against the wall. Exterior wall drains, which will not be sealed on top by asphalt or concrete, should have the upper 12 inches backfilled with compacted onsite silty clay soils to minimize intrusion of surface waters into the wall drain system. Please see *Figure 10*.

Walls that should not pass water vapor (for aesthetics or livable space) must be fully sealed (with a bitumen-based sealer that will not harden or crack) before the sheet drain is attached. Wall seal such as MasterBlend HLM5000 or equivalent, shall be used and applied per the manufacturer's recommendations. Multiple coats are preferred.

All drains should be tightlined and positively sloped to an approved stormwater disposal location into the public right-of-way. **Note:** In no case shall water be collected and/or directed or discharged close to the foundations. Such improper water discharge can cause added water related problems.

We strongly recommend against connecting roof drains or surface area drains to foundation or wall drains unless it is to a common discharge line far away from the structure. All drains must consist of rigid, smooth-wall pipe. The rigid smooth-wall pipe can be cleaned out by means of a “roto-rooter” type system should it become plugged with sediment or fine roots. We recommend cleanouts be placed periodically by the designer to facilitate cleaning and maintenance of the drains.

8.10 ASPHALTIC PAVEMENTS

Based on the preliminary conceptual design of the proposed development, it is our understanding that multiple roadways and parking areas are currently proposed for this development. These include a minor collector, multiple residential streets and driveways, which will likely consist of Hot Mix Asphaltic Concrete (HMAC) paved surface. The following sections provide recommendations for asphaltic concrete section design and construction. All constructed roadway design sections must also meet all City of Bandon requirements.

8.10.1 Pavement Subgrade & Traffic Loading

In order to determine the asphaltic section design, our firm conducted a California Bearing Ratio (CBR) test on the subgrade soils that would typically be encountered throughout the project. These included the loose sandy soil on the west side of the project and the dense sand with some silt on the east side of the project. Based on these tests (see *Appendix D*), an R-value of 35 was selected for the subgrade soil.

The following asphalt sections were designed utilizing a Crushed Rock Equivalent (CRE) method. Sufficient thickness of asphaltic concrete and rock materials are used to provide the computed crushed rock equivalent needed to protect the subgrade soils and successive rock layers from anticipated traffic loads.

Based on the typical design values in most cities similar to Bandon, the following traffic indices were used for each of the roadway asphalt section designs.

Project Area	Traffic Index (TI)
Major Collectors	8.73
Minor Collectors	8.21
Standard Residential Streets	7.22
Residential Lanes & Minor Residential Streets	6.33
Private Driveways	4.0

The successful performance of pavement structures is a function of subgrade material properties, traffic conditions, drainage conditions, the pavement material properties and design, careful construction, and ongoing maintenance.

8.10.2 Asphaltic Concrete Pavement Design

Below, we have provided asphaltic concrete pavement section designs for the anticipated traffic indices for the expected usage of the roadways. When final traffic conditions are known, the proper section design could be selected for all portions of the roadways based on the following recommendations.

Based on these TI’s and R-values of 35, 50 or 70 and 100, (subgrade soil, low-grade subbase or 4”-minus crushed rock, and 3/4" or 1" minus crushed rock), we have computed asphalt design sections (utilizing the Crushed Rock Equivalent Method) with the following results.

Major Collectors (TI = 8.73) - Traffic Lanes
(CRE = 22 inches) Crushed Rock Equivalent

4" AC (two 2" lifts or single lift with vibratory rollers)
6" AB (3/4" or 1" minus Crushed Rock)
16" ASB (Jaw-Run Shale or 12” of 4" minus Crushed Rock)
Geotextile Support Fabric (ACF 180, S200 or Equivalent)
Redensified Subgrade

OR

4" AC (two 2" lifts or single lift with vibratory rollers)
14" AB (3/4" or 1" minus Crushed Rock)
Geotextile Support Fabric (ACF 180, S200 or Equivalent)
Redensified Subgrade

Minor Collectors (TI = 8.21) - Traffic Lanes
(CRE = 21 inches) Crushed Rock Equivalent

4" AC (two 2" lifts or single lift with vibratory rollers)
6" AB (3/4" or 1" minus Crushed Rock)
14" ASB (Jaw-Run Shale or 10” of 4" minus Crushed Rock)
Geotextile Support Fabric (ACF 180, S200 or Equivalent)
Redensified Subgrade

OR

4" AC (two 2" lifts or single lift with vibratory rollers)
13" AB (3/4" or 1" minus Crushed Rock)
Geotextile Support Fabric (ACF 180, S200 or Equivalent)
Redensified Subgrade

Standard Residential Streets (TI = 7.22) - Traffic Lanes
(CRE = 19 inches) Crushed Rock Equivalent

4" AC (two 2" lifts or single lift with vibratory rollers)
12" AB (3/4" or 1" minus Crushed Rock)
Geotextile Support Fabric (ACF 180, S200 or Equivalent)
Redensified Subgrade

Residential Lanes & Minor Residential Streets (TI = 6.33) - Traffic Lanes
(CRE = 16 inches) Crushed Rock Equivalent

3" AC (single lift with vibratory rollers)
10" AB (3/4" or 1" minus Crushed Rock)
Geotextile Support Fabric (ACF 180, S200 or Equivalent)
Redensified Subgrade

Driveways (TI = 4.0) – Parking
(CRE = 10 inches) Crushed Rock Equivalent
2" AC (single lift with vibratory rollers)
6" AB (3/4" or 1" minus Crushed Rock)
Geotextile Support Fabric (ACF 180, S200 or Equivalent)
Redensified Subgrade

Note: When final subgrade conditions are known and the actual carrying capacity of the streets are determined, the proper section design could then be selected for the roadways.

8.10.3 General Recommendations

Subgrade Preparation. Subgrade preparation should begin with removal of debris and loose and disturbed soils. All debris and organic material should be disposed of properly and is not permitted as subgrade or fill material.

All finish subgrades should be shaped to a uniform surface running reasonably true to established line and grade described in the contract documents. Areas so specified must be redensified and/or backfilled with structural fill. It is important that dense, stable conditions of the subgrade be maintained until the subgrade is covered with the subbase aggregate. Subgrade preparation should include cleaning, redensification to at least 95% of ASTM D-698, and proofrolling (as described earlier in this report) to identify soft and disturbed subgrade areas.

After subgrade preparation is completed, the upper 10 inches of exposed subgrade prepared for the pavement structure should demonstrate a firm and unyielding condition as shown by proofrolling.

Soft or loose materials disturbed during the site preparation process, incapable of achieving the compaction criteria, should be removed to appropriate bearing materials prior to replacing with structural fill. Where loose or softened subgrade areas are identified, the area should be over-excavated and replaced with imported granular fill with less than 10 percent passing the number 200 sieve.

Geotextile Fabric Placement. When the subgrade soils have been properly prepared, the described subgrade areas shall be covered with the woven geotextile support fabric prior to placement and compaction of structural fill. We recommend a fabric such as ACF 180, S200 or equivalent. The fabric shall be laid longitudinally with the direction of traffic. All ends and edges should be overlapped a minimum of 5 and 2 feet, respectively. Fabric layout shall be such that it “runs” aligned with the lane traffic directions. Care must be taken to not damage the fabric. In no case shall track vehicles be allowed on the fabric.

It should be noted that construction trucks should not be allowed to “run” directly on top of the on-site native subgrade soils until they are covered with rock. This could result in the disturbance of the subgrade soils due to the heavily loaded vehicles (which would result in additional over-excavation to remove softened soils). We recommend covering the subgrade soils with at least 12 inches of crushed rock or "shale" over the woven fabric, during construction, prior to light construction truck traffic traversing the area. Therefore, construction traffic must be carefully coordinated in order to minimize disturbance to the underlying Silt soils.

Wet Weather Construction. We recommend that for construction during very wet weather or on wet subgrades all construction roads and drive lane subgrades should be covered with a woven geotextile support fabric (ACF 180, S200 or equivalent) and a minimum of 12 inches of imported granular 4-inch minus crushed rock. Compaction of the fill should not begin until a minimum of 8 inches of rock is placed above the fabric. Compact carefully so as not to disturb the subgrade. This should provide an adequate working surface and help protect the subgrade from damage from construction traffic. Construction traffic should not be allowed to traverse the area until the minimum of 12 or more inches of compacted material has been placed and compacted over the support fabric.

Drainage. Adequate provision should be made to direct surface water away from the pavement section and subgrade. Ponded water adjacent to the asphalt areas can saturate the subgrade resulting in loss of support. Therefore, we recommend the areas along the edge of the asphalt be well drained. All paved areas should be sloped and drainage gradients maintained to carry surface water to catch basins or ditches for transmission off the roadway and parking areas. Excessive landscape watering can also saturate the subgrade and decrease pavement life. Deep curbs, drip irrigation and/or use of dry-land plants will mitigate these affects.

Maintenance. Pavement life can be extended by providing proper maintenance and overlays as needed. Cracks in the pavement should be filled to prevent intrusion of surface water into the subbase. Asphalt pavements typically require seal coats or overlays after 10 to 12 years to maintain structural performance and aesthetic appearance.

9.0 MATERIALS SPECIFICATIONS

The following materials specifications shall apply to the materials used on this project. **Note:** All such materials to be used on the project must be submitted for compliance testing or review, at least two weeks prior to use at the site.

Aggregate Base Rock (AB) Structural Fill

- Angular Crushed Rock (¾ or 1" Minus); R=85 or greater; Well Graded (No Gaps and at least 60% retained on the No. 4 sieve).
- Exceeds the fracture, durability and sand equivalent requirements outlined in Section 00641 of the Oregon Standard Specifications for Construction.
- Maximum passing the No. 200 sieve ≤ 5% Total; ≤ 2% Clay Size.

- Compacted to 98% of the maximum dry density as determined by ASTM D698 or AASHTO T-99.

Aggregate Subbase Rock (ASB) Structural Fill

- Angular Clean Crushed (jaw run) hard “Shale” (4" Minus Jaw-Run) or Crushed Rock (2" to 4” Minus); R=50 or greater; Angular and Reasonably Well Graded.
- At Least 60% retained on the No. 4 Sieve.
- Exceeds the fracture, durability and sand equivalent requirements outlined in Section 00641 of the Oregon Standard Specifications for Construction.
- Maximum passing the No. 200 sieve ≤ 10% Total; ≤ 3% Clay Size.
- <u>During wet weather; passing No. 200 sieve ≤ 5%.</u>
- Compacted to 95% of the maximum dry density as determined by ASTM D698 or AASHTO T-99; initial lift may not attain 95% due to soft subgrade; Engineer to decide in the field.
- Care must be taken to avoid very silty subbase that will not support construction loads, especially when wet (will not meet specifications).

Embankment Fill (Acceptable for Structural Fill During <u>Dry</u> Weather)

- Reasonably well graded (not open work).
- Has at least 60% retained on the No. 4 sieve.
- Has no more than 30% passing No. 200 sieve.
- Passing No. 200 sieve must have less than 20% clay size.

On-Site Soil Structural Fill

- Sandy Soils - For roadway Subbase.
- Others Use – with approval of geotechnical engineer

Note: Some fill materials will be difficult to nearly impossible to compact during wet weather. *The contractor <u>must</u> select the type of structural fill that will be able to be placed and compacted to specified conditions during the weather conditions that may take place during the construction schedule.*

Sand

- Clean washed sand or sand and gravel, less than 1% passing No. 200.
- Gravel to be rounded or subrounded (no fracture faces), 1" or less.
- Must have less than 30% gravel by weight.

Drain Rock (For Drainage Sections)

- Clean, <u>washed,</u> rounded or angular openwork drain rock.
- Gradation to be 1/4" and greater, sized to not move into and through perforations in the pipe.
- 1/4" to 3/4" clean crushed, 3/4" to 1" clean rounded rock, 1" to 2" clean angular rock are all acceptable.
- Clean means washed rock with <u>NO</u> coating of silt, clay or sand.

Geotextile Filter Fabric

- Non-woven geotextile filter fabric for wrapping drainage sections and separation of openwork rock from sands or soils fines.
- Meet specifications as per Mirafi 140N or equivalent.
- Overlap all edges at least 24 inches (12" for drainage section envelope).
- Secure in place such that overlaps will not move during covering operation.

Geotextile Support Fabric

- Woven geotextile support fabric designed for separation of crushed rock and subgrade soil and for rock section support.
- Meet specifications as per ACF180 woven support fabric.
- Overlap edges at least 2 feet and ends at least 5 feet.
- Align roll lengthwise with direction of traffic in all drive lanes.
- Pull tight full length and keep tight during placement of crushed rock above fabric.
- Do not drive on the fabric until it is covered with rock.

Perforated Pipe

- 3", 4" or 6" rigid wall, smooth interior perforated pipe.
- Secure all joints with solvent weld glue. <u>DO NOT</u> use only compression push together fittings.
- Slope to drain per specifications in report or on plan sheets.
- Align perforations in the downward direction.
- <u>Must</u> always be placed within filter fabric wrap unless specifically specified otherwise.
- Protect from construction traffic until buried at least 2 times pipe diameter (minimum 8 inches) of angular rock fill.

Wall Sheet Drain

- Polymer sheet drain with filter fabric attached 1 or 2 sides, designed for drainage of vertical embedded foundation or retaining walls.
- For walls up to 10 feet tall. Must meet specifications as for American Wick Drain's AMERDRAIN 200 or 220.
- Install and splice and patch per manufacturer's recommendations.
- Install with fabric side towards the backfill.
- Attach to wall per manufacturer's recommendations.
- Extend down wall all the way to bottom of drainage section around perforated pipe.
- Protect from damage when backfilling with crushed rock larger than 2-inch minus.
- Repair all damaged areas prior to final backfill.

10.0 SITE DRAINAGE

<u>The site shall be graded during construction such that surface water does not pond within or around structures.</u> Surface runoff shall be controlled during construction and with final site grading. All areas adjacent to structures shall have a permanent slope away from the foundations at an inclination of at least 6 inches in eight (8) feet. This surface water shall be channeled into landscape area drains or catch basins, or other approved discharge location. Where items such as landscape areas and walkways block the flow of surface water, small area drains shall be installed to collect the surface runoff. Good site design accommodates all site runoff and conveys it away from the structures and off the site to an acceptable disposal location.

All roof downspouts must be connected to a sealed tightline system, which discharges to an acceptable disposal location.

11.0 EROSION CONTROL

The site soils have low to moderate susceptible to erosion. However, the site grades are steep on the west and mild on the east sides of the site. Therefore, site erosion should generally be low to moderate on the east side of the site and high on the west side of the site. Erosion control measures must be implemented prior to and during construction to prevent sediment movement off site.

Construction Erosion Control. All disturbed areas shall have the low side surrounded by a silt fence with the bottom edge embedded in the soil at least two (2) inches. At select locations settling ponds of hay-bale backed silt fence should be established to decrease silt content of surface water flowing off site. Hay bale “V’s” may be needed in ditches to stop silt migration for up to 200 feet from the site.

The site will also require crushed rock (or shale) construction entrances to prevent "tracking" of mud by construction vehicles onto the roads. These are typically required to be at least 50 feet long and be constructed of a 12" section of angular, open-work rock over a woven fabric (more if needed to protect the subgrade soils).

Permanent Erosion Control. Permanent project landscaping and paving, as required by the City/County, will meet most needs of long-term erosion control. All disturbed areas on the site, but outside the developed area of the project, must be reseeded with local native grasses for erosion prevention. Ideally, these areas would be graded reasonably smooth and the surface scarified to 1/2 inch deep, then hydroseeded with a combination of erosion control grass seed, fertilizer and mulch. Alternatively, and at a minimum, these areas should be covered with a thin layer of crushed rock.

12.0 ADDITIONAL SERVICES AND LIMITATIONS

12.1 ADDITIONAL SERVICES

Additional services by the geotechnical engineer are recommended to help ensure that design recommendations are correctly interpreted in final project design and to help verify compliance with project specifications during the construction process. For this project we anticipate additional services could include the following:

- Review of final construction plans and specifications for compliance with geotechnical recommendations. This needs to be done to ensure that the structures, and roadway designs meet the recommendations in our report.
- Possible project team meetings to clarify issues and proceed smoothly into and through the construction process.
- Observation of on-site excavations to verify stability is acceptable
- Observation of cut and fill areas for drainage placement and rock evaluation.
- Observation and/or testing of overexcavated areas below footings and slabs, structural fill placement, foundation drains, subgrade proofrolling, pavement subgrade and aggregate base placement, site grading and surface drainage.
- Density testing of all structural fill, trench backfill and wall backfill.
- Observation/documentation of deep foundation installation.
- Periodic construction field reports, as requested by the client and required by the building department.

We would provide these additional services on a time-and-expense basis in accordance with our Standard Fee Schedule and General Conditions at the time of construction. If we are not retained to provide these services, we cannot be held responsible for the decisions by others.

12.2 LIMITATIONS

The analyses, conclusions and recommendations contained in this report are based on site conditions and assumed development plans as they existed at the time of the study, and assume soils, rock and groundwater conditions exposed and observed in the test pits and borings during our investigation are representative of soils and groundwater conditions throughout the site. If during construction, subsurface conditions or assumed design information is found to be different, we should be advised at once so that we can review this report and reconsider our recommendations in light of the changed conditions. 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 conditions have changed due to acts of God or construction at or adjacent to the site, it is recommended that this report be reviewed in light of the changed conditions and/or time lapse.

This report was prepared for the use of the owner and his design and construction team for the design and construction of the project. It should be made available to contractors for information and factual data only. This report should not be used for contractual purposes as a warranty of site subsurface conditions. It should also not be used at other sites or for projects other than the one intended.

We have performed these services in accordance with generally accepted geotechnical engineering practices in Oregon, at the time the study was accomplished. No other warranties, either expressed or implied, are provided.

THE GALLI GROUP
GEOTECHNICAL CONSULTING

Dennis Duru

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Project Engineer/Geologist

Mel Galli III

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REGISTERED PROFESSIONAL ENGINEER
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OREGON
January 12, 2016
MELVIN JOHN GALLI III
EXPIRES: 06/30/23

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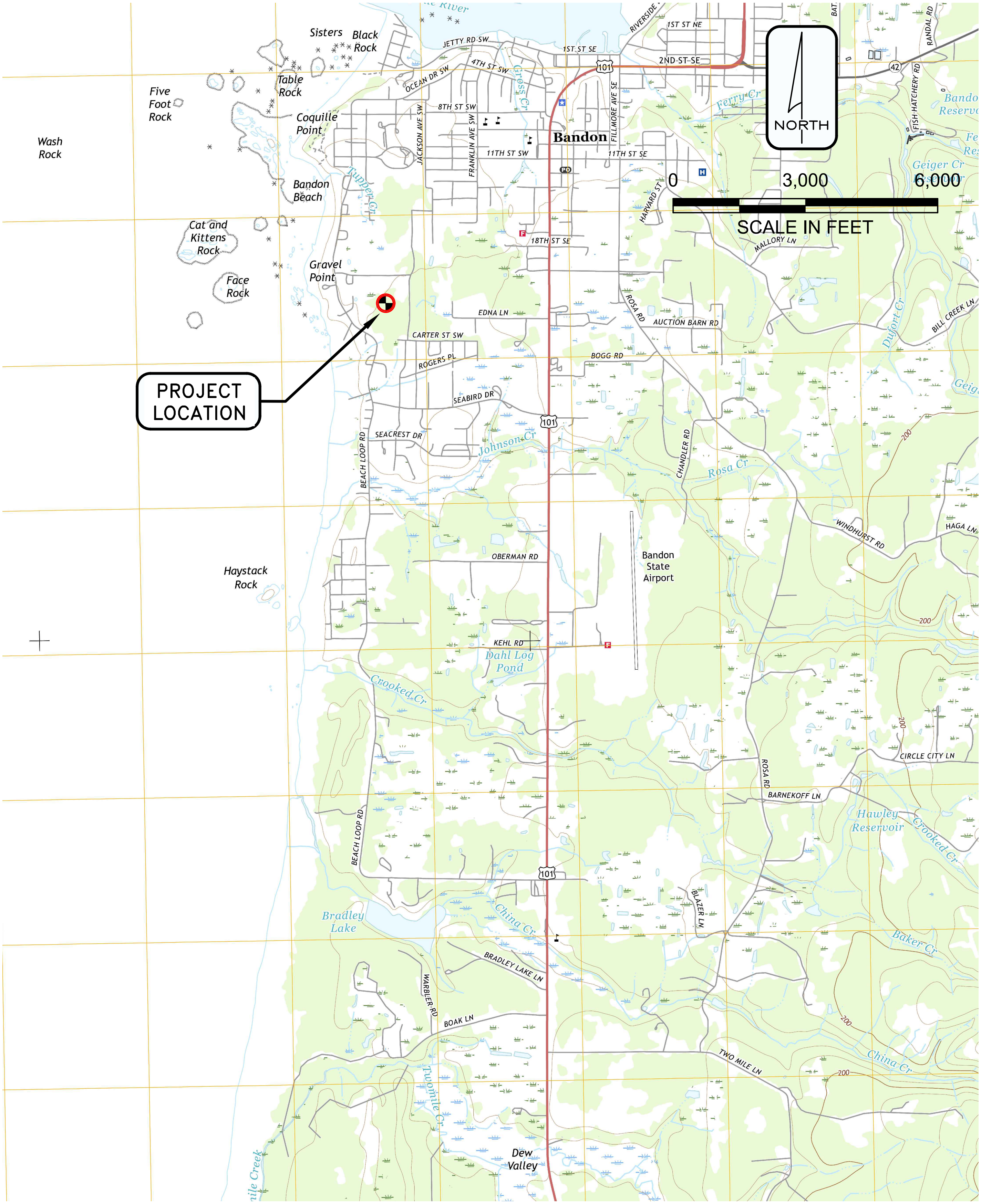

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612 NW 3rd Street
Grants Pass, OR 97526

VICINITY MAP

BEACH LOOP SUBDIVISION
BANDON, OREGON

DATE: OCTOBER 2022
JOB NO: 02-6151-02
REV: 10/13/2022 3:01 PM
PREPARED BY: BD
6151 Beach Loop Bandon - 01 - Vicinity.dwg

FIGURE: 1

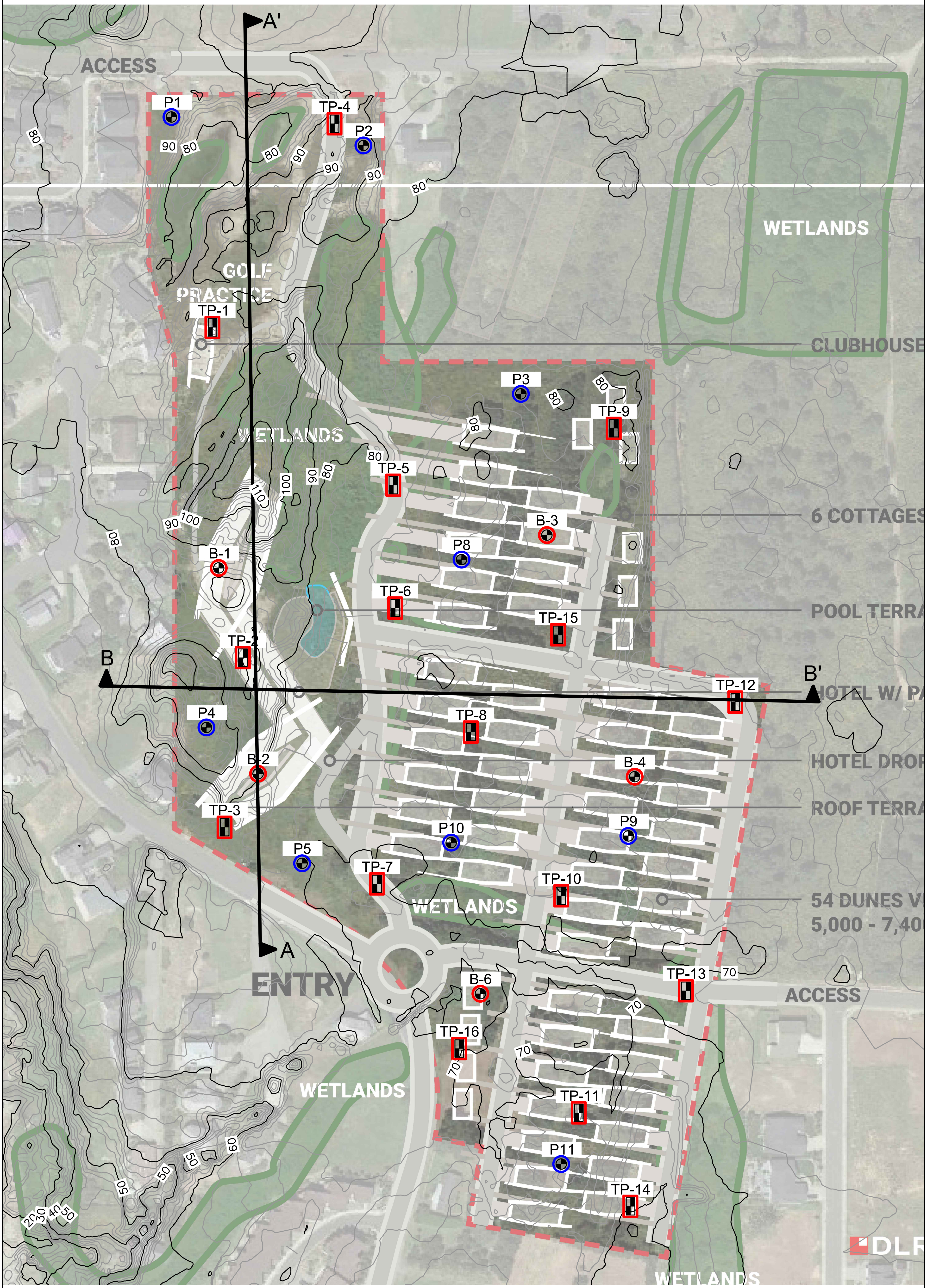
GENERAL NOTES:

1. SITE PLAN PROVIDED BY PARAMETRIX
2. TOPOGRAPHIC INFORMATION
   HORIZONTAL DATUM: NAD83 (HARN)
   PROJECTION: OREGON STATEWIDE LAMBERT CONFORMAL CONIC
   VERTICAL DATUM: NAVD88 (GEOID03)
   UNITS: INTERNATIONAL FEET
   AUTOCAD BASED SURFACE CREATION

APPROX NORTH

0 150 300

SCALE IN FEET



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612 NW 3rd Street
Grants Pass, OR 97526

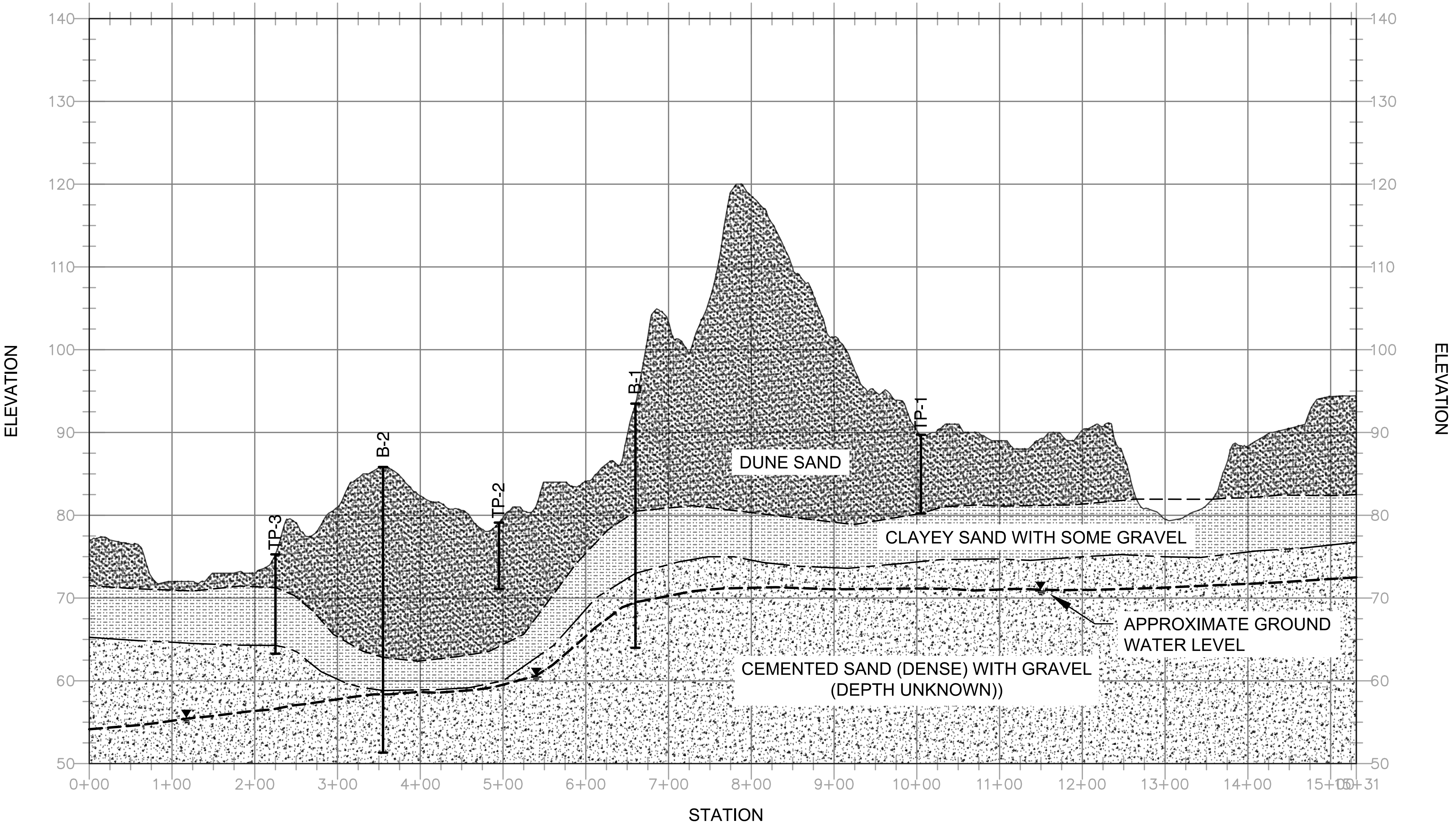
SITE PLAN WITH EXPLORATION LOCATIONS

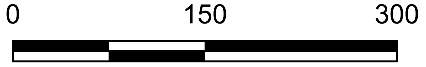
BEACH LOOP SUBDIVISION
BANDON, OREGON

DATE: OCTOBER 2022
JOB NO: 02-6151-02
REV: 10/17/2022 4:00 PM
PREPARED BY: BD
6151 Beach Loop - 02 - Site Plan.dwg

FIGURE: 2

CROSS SECTION A-A'





HORIZONTAL SCALE IN FEET
10X VERTICAL SCALE

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Grants Pass, OR 97526

CROSS SECTION A-A'

BEACH LOOP SUBDIVISION
BANDON, OREGON

DATE: OCTOBER 2022
JOB NO: 02-6151-02
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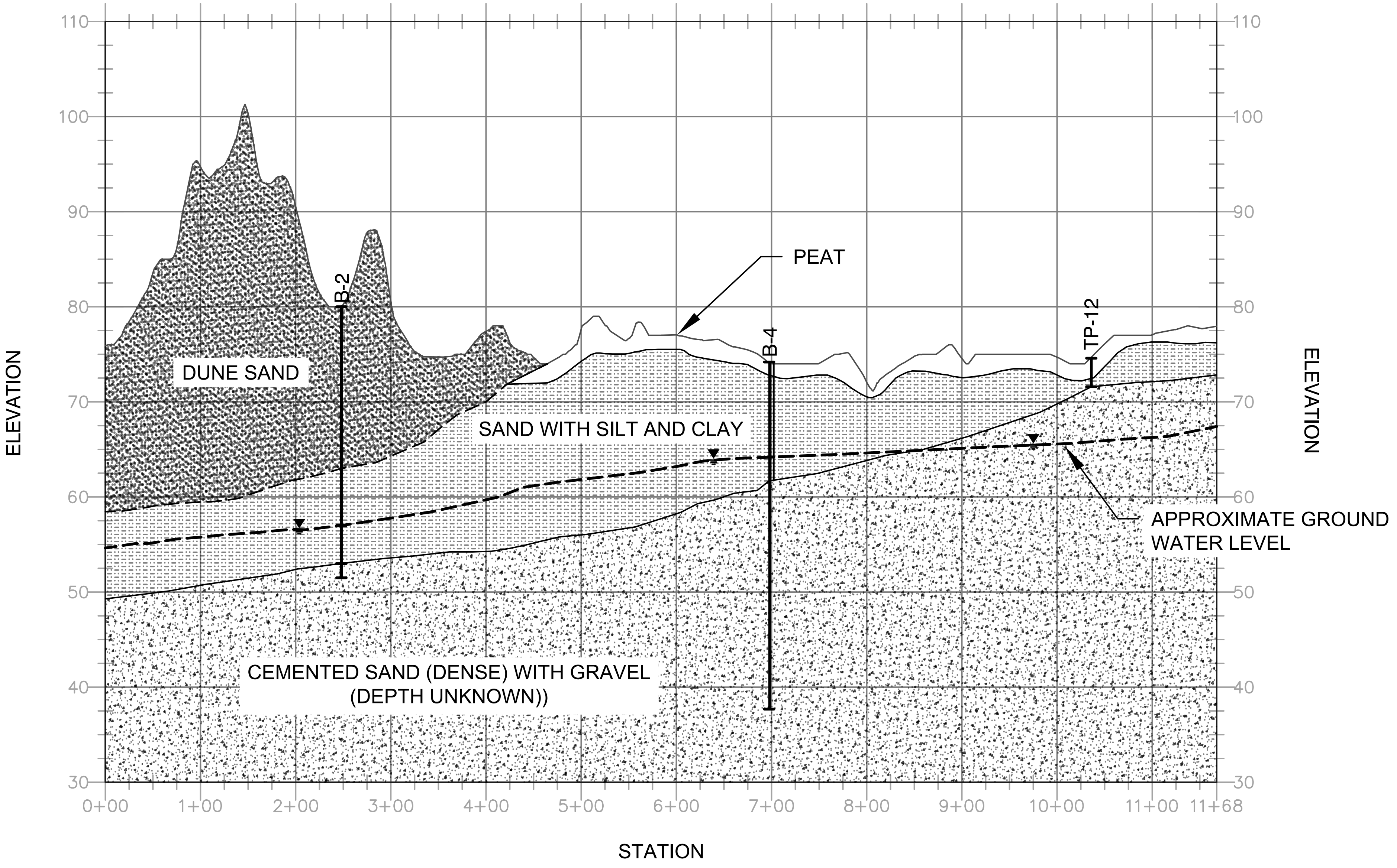
PREPARED BY: BD
6151 Beach Loop - 3A 3B - Cross-sections.dwg

FIGURE:
3A

CROSS SECTION B-B'



HORIZONTAL SCALE IN FEET
10X VERTICAL SCALE

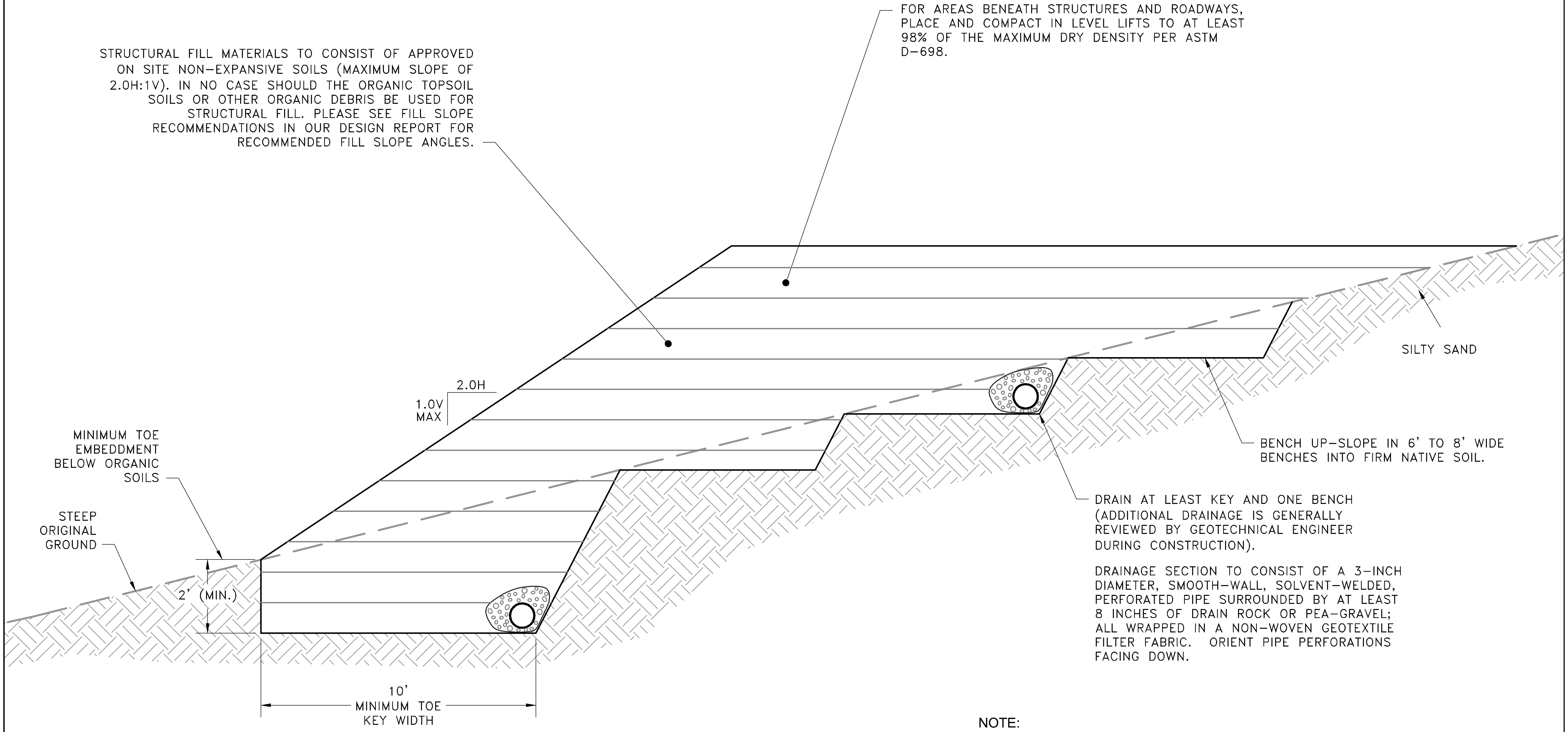
THE GALLI GROUP
GEOTECHNICAL CONSULTING
612 NW 3rd Street
Grants Pass, OR 97526

CROSS SECTION B-B'

BEACH LOOP SUBDIVISION
BANDON, OREGON

DATE: OCTOBER 2022
JOB NO: 02-6151-02
REV: 10/17/2022 4:07 PM
PREPARED BY: BD
6151 Beach Loop - 3A 3B - Cross-sections.dwg

FIGURE:
3B



NOTE:

THIS IS A GENERAL CROSS-SECTION FOR FILL PLACED ON SLOPES.
IT IS NOT INTENDED AS A SPECIFIC DESIGN FOR THIS PROJECT.

FOR ILLUSTRATION PURPOSES ONLY
NOT TO SCALE

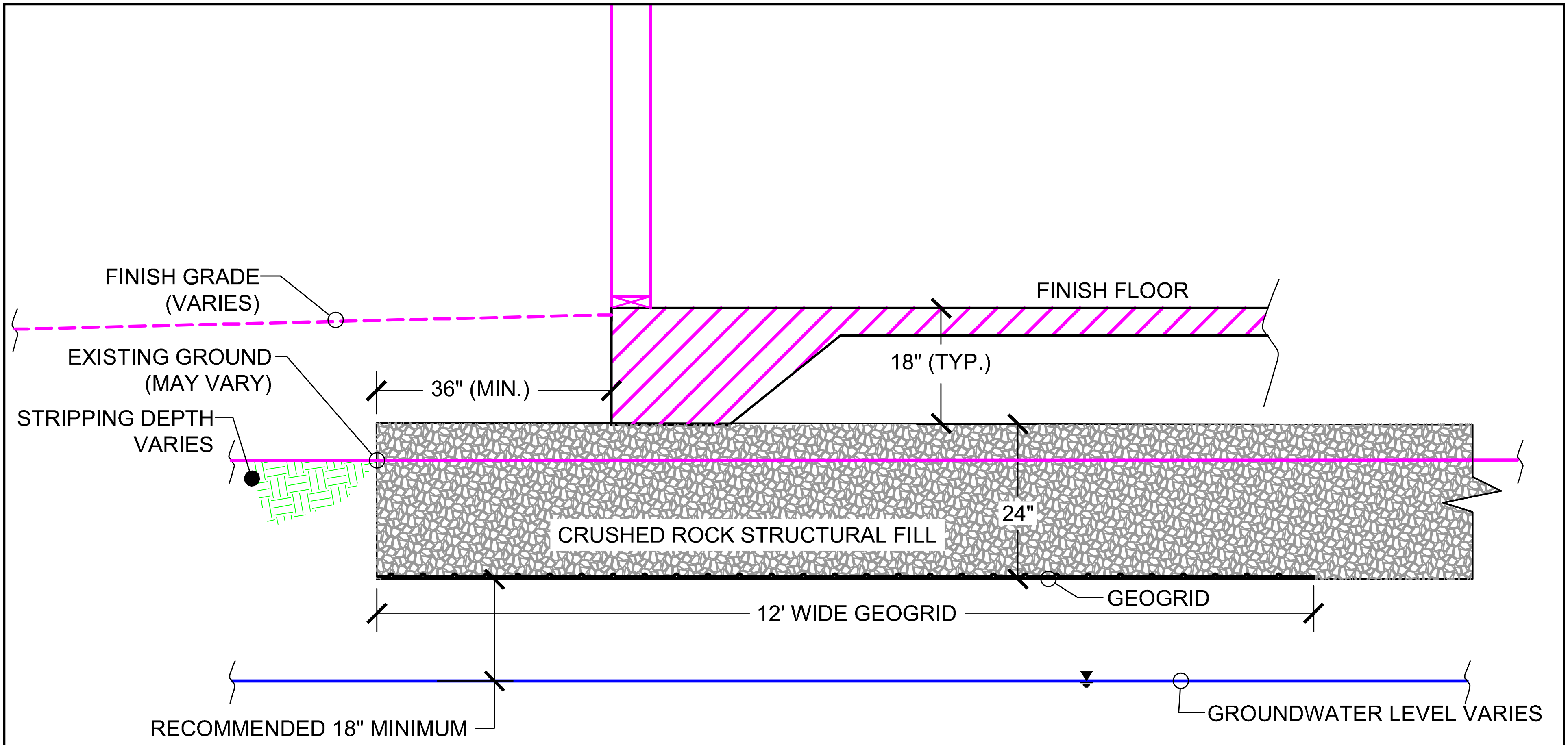
THE GALLI GROUP
GEOTECHNICAL CONSULTING
612 NW 3rd Street
Grants Pass, OR 97526

FILL ON STEEP SLOPE CROSS-SECTION

BEACH LOOP SUBDIVISION
BANDON, OREGON

DATE: OCTOBER 2022
JOB NO: 02-6151-02
REV: 10/17/2022 5:12 PM
PREPARED BY: BD
6151 Beach Loop - 04 - Slope Fill.dwg

FIGURE: 4



NOTE: THIS IS THE GENERAL LAYOUT OF PROPOSED GEOGRID SUPPORT. THIS COULD BE ALTERED TO MEET FINAL STRUCTURE DESIGN WHEN THAT IS AVAILABLE

0 2 4

SCALE IN FEET
(NO VERTICAL EXAGGERATION)

THE GALLI GROUP
GEOTECHNICAL CONSULTING

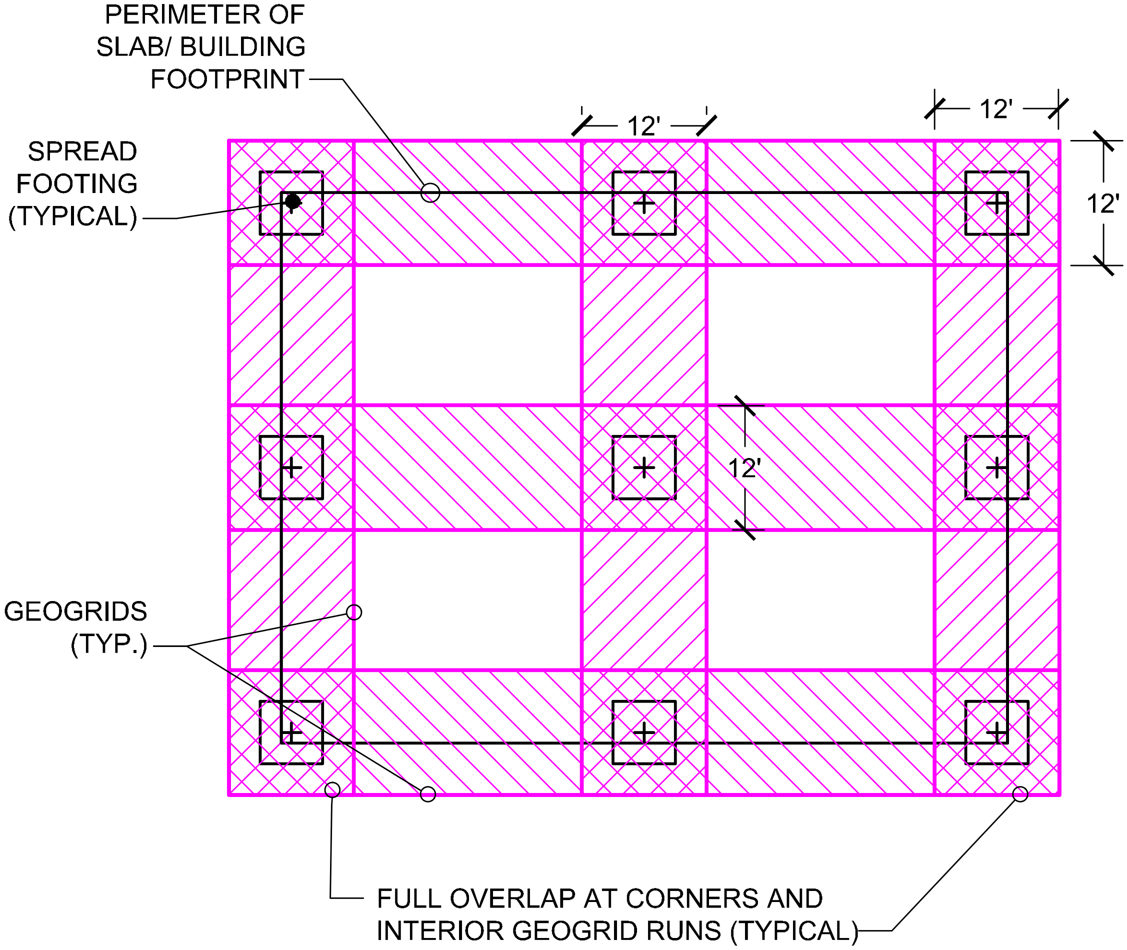
612 NW 3rd Street
Grants Pass, OR 97526

BUILDING SUPPORT PAD

BEACH LOOP SUBDIVISION
BANDON, OREGON

DATE: OCTOBER 2022
JOB NO: 02-6151-02
REV: 10/17/2022 5:04 PM
PREPARED BY: BD
6151 Beach Loop - 05 - Building Support Pad.dwg

FIGURE: 5



NOTE: THIS IS THE GENERAL LAYOUT OF PROPOSED GEOGRID SUPPORT. THIS COULD BE ALTERED TO MEET FINAL STRUCTURE DESIGN WHEN THAT IS AVAILABLE

THE GALLI GROUP
GEOTECHNICAL CONSULTING

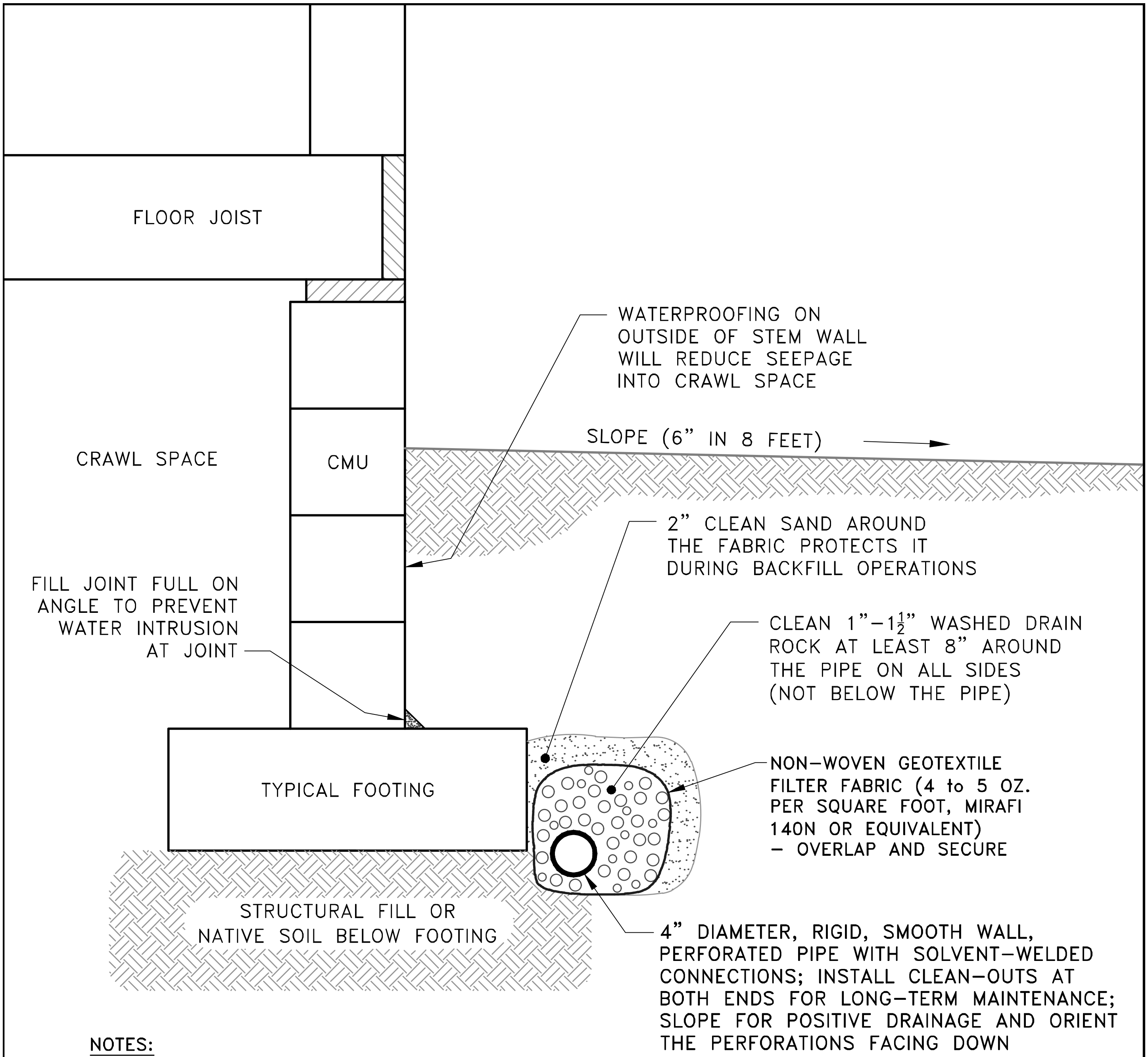
612 NW 3rd Street
Grants Pass, OR 97526

TYPICAL GEOGRID
PLACEMENT PATTERN

BEACH LOOP SUBDIVISION
BANDON, OREGON

DATE: OCTOBER 2022
JOB NO: 02-6151-02
REV: 10/17/2022 4:34 PM
PREPARED BY: MG3
6151 Beach Loop - 06 - Geogrid Placement Pattern.dwg

FIGURE:
6



NOTES:

(1) VAPOR BARRIER TO BE STEGO INDUSTRIES 15mil STEGO WRAP OR EQUIVALENT. OWNER MY CHOOSE TO USE 6mil VISQUENE, UNDERSTANDING IT WILL NOT WORK AS WELL.

(2) CAPILLARY BREAK ROCK BELOW VAPOR BARRIER TO BE 1/4" TO 3/4" CLEAN CRUSHED ROCK OR EQUIVALENT.

FOR ILLUSTRATION PURPOSES ONLY - NOT FOR CONSTRUCTION
NOT TO SCALE

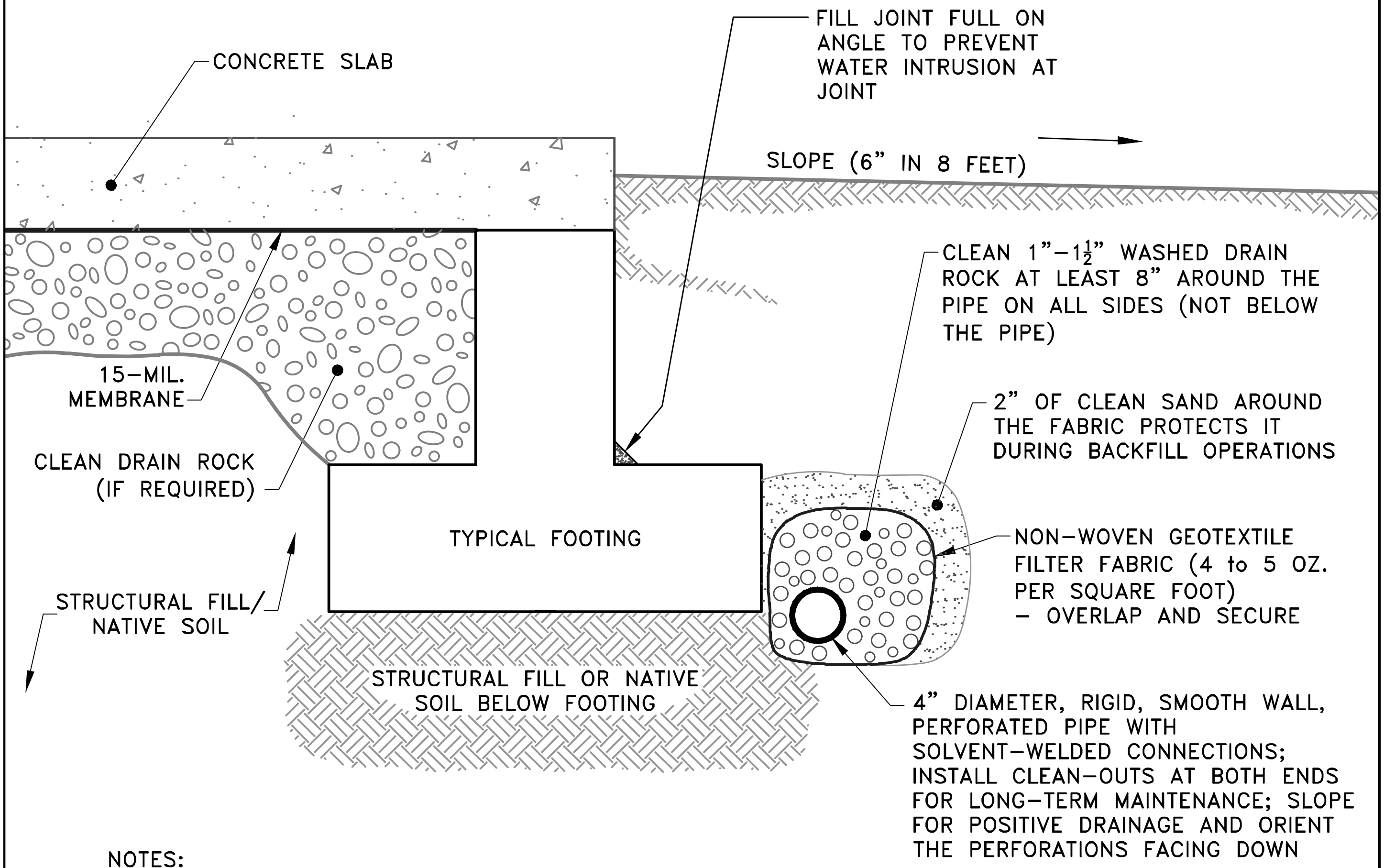
THE GALLI GROUP
GEOTECHNICAL CONSULTING
612 NW 3rd Street
Grants Pass, OR 97526

TYPICAL FOUNDATION DRAIN
WITH CRAWL SPACE

BEACH LOOP SUBDIVISION
BANDON, OREGON

DATE: OCTOBER 2022
JOB NO: 02-6151-02
REV: 10/13/2022 3:23 PM
PREPARED BY: BD
6151 Beach Loop - 07 - Crawl drn.dwg

FIGURE:
7



NOTES:

(1) VAPOR BARRIER TO BE STEGO INDUSTRIES 15mil STEGO WRAP OR EQUIVALENT.

(2) CAPILLARY BREAK ROCK BELOW VAPOR BARRIER TO BE 1/4" TO 3/4" CLEAN CRUSHED ROCK OR EQUIVALENT.

FOR ILLUSTRATION PURPOSES ONLY
NOT TO SCALE

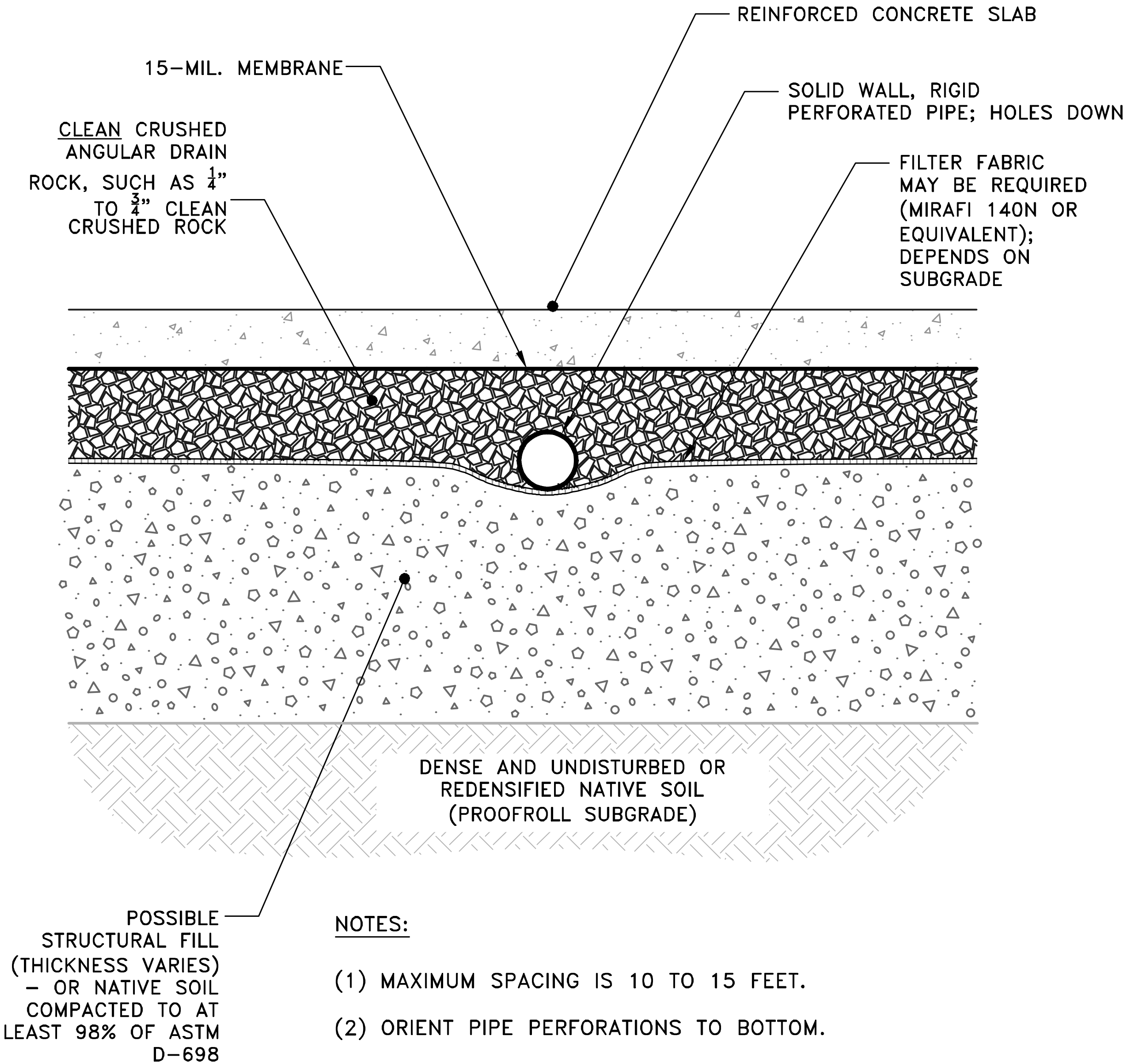
THE GALLI GROUP
GEOTECHNICAL CONSULTING
612 NW 3rd Street
Grants Pass, OR 97526

TYPICAL FOUNDATION DRAIN
SLAB ON GRADE FLOOR

BEACH LOOP SUBDIVISION
BANDON, OREGON

DATE: OCTOBER 2022
JOB NO: 02-6151-02
REV: 10/17/2022 3:57 PM
PREPARED BY: BD
6151 Beach Loop - 08 - Slab OG.dwg

FIGURE:
8



NOTES:

(1) MAXIMUM SPACING IS 10 TO 15 FEET.

(2) ORIENT PIPE PERFORATIONS TO BOTTOM.

(3) ASSEMBLE PIPE USING SOLVENT-WELDED CONNECTIONS.

(4) DO NOT DRIVE OVER DRAIN LINES.

(5) DRAIN ROCK AND STRUCTURAL FILL TO MEET SPECS. IN REPORT BODY - SLOPE PIPE TO DRAIN.

(6) MAY REQUIRE FILTER FABRIC ON NATIVE SUBGRADE OR IF STRUCTURAL FILL IS VERY SILTY OR SANDY.

(7) BEST VAPOR BARRIER IS STEGO INDUSTRIES 15mil STEGO WRAP (OR EQUIVALENT).

FOR ILLUSTRATION PURPOSES ONLY
NOT TO SCALE

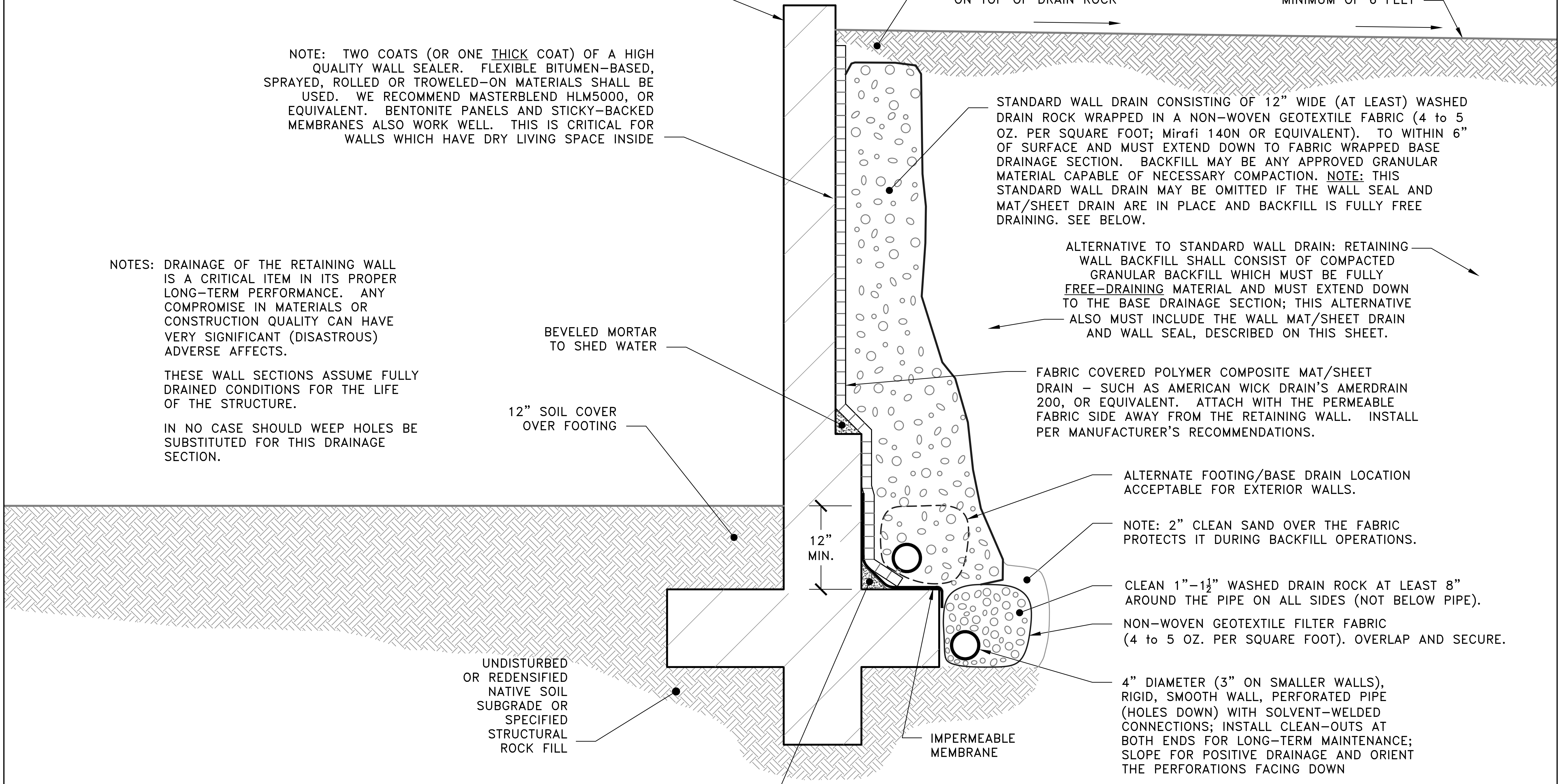
THE GALLI GROUP
GEOTECHNICAL CONSULTING
612 NW 3rd Street
Grants Pass, OR 97526

FLOOR SUBDRAIN DETAIL

BEACH LOOP SUBDIVISION
BANDON, OREGON

DATE: OCTOBER 2022
JOB NO: 02-6151-02
REV: 10/17/2022 3:57 PM
PREPARED BY: BD
6151 Beach Loop - 09 - Slab sub.dwg

FIGURE:
9



TYPICAL RETAINING WALL CROSS-SECTION

NOTE: TWO COATS (OR ONE <u>THICK</u> COAT) OF A HIGH QUALITY WALL SEALER. FLEXIBLE BITUMEN-BASED, SPRAYED, ROLLED OR TROWELED-ON MATERIALS SHALL BE USED. WE RECOMMEND MASTERBLEND HLM5000, OR EQUIVALENT. BENTONITE PANELS AND STICKY-BACKED MEMBRANES ALSO WORK WELL. THIS IS CRITICAL FOR WALLS WHICH HAVE DRY LIVING SPACE INSIDE

CLAYEY SOIL SEAL OR PLASTIC SHEETING ON TOP OF DRAIN ROCK

BACKSLOPE EXTERIOR SURFACES AT LEAST 2% TO 5% FOR A MINIMUM OF 6 FEET

STANDARD WALL DRAIN CONSISTING OF 12" WIDE (AT LEAST) WASHED DRAIN ROCK WRAPPED IN A NON-WOVEN GEOTEXTILE FABRIC (4 to 5 OZ. PER SQUARE FOOT; Mirafi 140N OR EQUIVALENT). TO WITHIN 6" OF SURFACE AND MUST EXTEND DOWN TO FABRIC WRAPPED BASE DRAINAGE SECTION. BACKFILL MAY BE ANY APPROVED GRANULAR MATERIAL CAPABLE OF NECESSARY COMPACTION. <u>NOTE:</u> THIS STANDARD WALL DRAIN MAY BE OMITTED IF THE WALL SEAL AND MAT/SHEET DRAIN ARE IN PLACE AND BACKFILL IS FULLY FREE DRAINING. SEE BELOW.

ALTERNATIVE TO STANDARD WALL DRAIN: RETAINING WALL BACKFILL SHALL CONSIST OF COMPACTED GRANULAR BACKFILL WHICH MUST BE FULLY <u>FREE-DRAINING</u> MATERIAL AND MUST EXTEND DOWN TO THE BASE DRAINAGE SECTION; THIS ALTERNATIVE ALSO MUST INCLUDE THE WALL MAT/SHEET DRAIN AND WALL SEAL, DESCRIBED ON THIS SHEET.

NOTES: DRAINAGE OF THE RETAINING WALL IS A CRITICAL ITEM IN ITS PROPER LONG-TERM PERFORMANCE. ANY COMPROMISE IN MATERIALS OR CONSTRUCTION QUALITY CAN HAVE VERY SIGNIFICANT (DISASTROUS) ADVERSE AFFECTS.

THESE WALL SECTIONS ASSUME FULLY DRAINED CONDITIONS FOR THE LIFE OF THE STRUCTURE.

IN NO CASE SHOULD WEEP HOLES BE SUBSTITUTED FOR THIS DRAINAGE SECTION.

BEVELED MORTAR TO SHED WATER

FABRIC COVERED POLYMER COMPOSITE MAT/SHEET DRAIN – SUCH AS AMERICAN WICK DRAIN'S AMERDRAIN 200, OR EQUIVALENT. ATTACH WITH THE PERMEABLE FABRIC SIDE AWAY FROM THE RETAINING WALL. INSTALL PER MANUFACTURER'S RECOMMENDATIONS.

12" SOIL COVER OVER FOOTING

ALTERNATE FOOTING/BASE DRAIN LOCATION ACCEPTABLE FOR EXTERIOR WALLS.

NOTE: 2" CLEAN SAND OVER THE FABRIC PROTECTS IT DURING BACKFILL OPERATIONS.

CLEAN 1"–1½" WASHED DRAIN ROCK AT LEAST 8" AROUND THE PIPE ON ALL SIDES (NOT BELOW PIPE).

NON-WOVEN GEOTEXTILE FILTER FABRIC (4 to 5 OZ. PER SQUARE FOOT). OVERLAP AND SECURE.

UNDISTURBED OR REDENSIFIED NATIVE SOIL SUBGRADE OR SPECIFIED STRUCTURAL ROCK FILL

IMPERMEABLE MEMBRANE

4" DIAMETER (3" ON SMALLER WALLS), RIGID, SMOOTH WALL, PERFORATED PIPE (HOLES DOWN) WITH SOLVENT-WELDED CONNECTIONS; INSTALL CLEAN-OUTS AT BOTH ENDS FOR LONG-TERM MAINTENANCE; SLOPE FOR POSITIVE DRAINAGE AND ORIENT THE PERFORATIONS FACING DOWN

BEVELED MORTAR TO SHED WATER

FOR ILLUSTRATION PURPOSES ONLY

NOT TO SCALE

THE GALLI GROUP
GEOTECHNICAL CONSULTING

612 NW 3rd Street
Grants Pass, OR 97526

EXTERIOR RETAINING WALL DRAINAGE CROSS-SECTION

BEACH LOOP SUBDIVISION
BANDON, OREGON

DATE: OCTOBER 2022
JOB NO: 02-6151-02
REV: 10/17/2022 3:50 PM
PREPARED BY: BD
6151 Beach Loop - 010 - Ext Ret Wall.dwg

FIGURE:
10

APPENDIX A

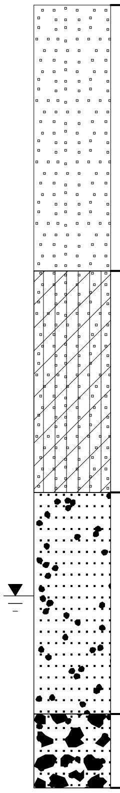
BORING LOGS

THE GALLI GROUP
GEOTECHNICAL CONSULTANTS

BORING LOG
B-1

Project: Beach Loop Subdivision
Client: Bandon Beach Ventures, LLC
Location: See Site Plan
Driller: Western States (Shane)
Drill Rig: CME-55 track rig #2
Depth To Water> Initial ∇ : 20 At Completion ▼ : 20

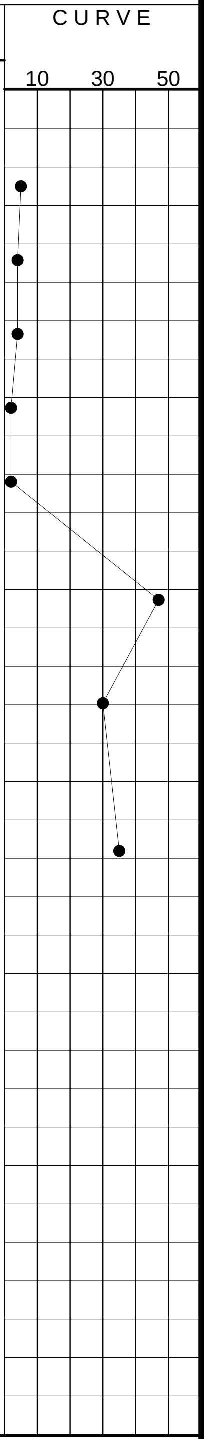
Project No.: 02-6151-02
Date: 8/29/2022
Elevation: 85
Logged By: Dennis Duru



USCS	Description	Depth
SP	Loose, brown, fine Sand; damp to moist.	9.0
SM/SC	Very loose, brown-black, silty, clayey Sand; wet.	16.5
SW	Dense, brown, Sand; trace gravel, moist to saturated.	24.0
SW/GW	Dense, brown, sandy Gravel; saturated.	26.5
	Bottom of boring at 26.5 feet. Static groundwater at 20 feet.	

Depth: 0, 6.5, 13, 19.5, 26, 32.5, 39, 45.5

Sample No. and Type	NMC	Standard Penetration Test N
S-1 ◩	2.2%	5
S-2 ◩	7.3%	4
S-3 ◩	3.6%	4
S-4 ◩	25%	2
S-5 ◩	22%	2
S-6 ‖		
S-7 ◩	25%	47
S-8 ◩	17%	30
S-9 ◩	23%	35



Legend of Samplers: ▯ Grab sample ◩ SPT sample ‖ Shelby tube sample

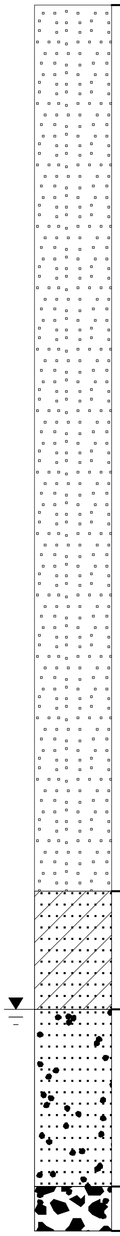
This information pertains only to this boring and should not be interpreted as being indicative of the site.

THE GALLI GROUP
GEOTECHNICAL CONSULTANTS

BORING LOG
B-2

Project: Beach Loop Subdivision
Client: Bandon Beach Ventures, LLC
Location: See Site Plan
Driller: Western States (Shane)
Drill Rig: CME-55 track rig #2
Depth To Water> Initial ▽: 34

Project No.: 02-6151-02
Date: 8/29/2022
Elevation: 93
Logged By: Dennis Duru
At Completion ▼: 34

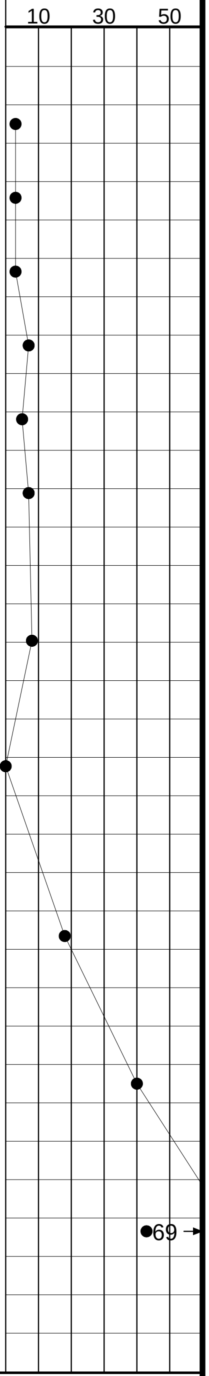


USCS	Description	Depth
SP	Very loose to loose, brown, fine Sand; damp to moist.	0
		30.0
SW	Medium dense, orange-brown, Sand; trace clay, moist	
		34.0
SW	Dense, brown-orange, Sand; thin fine gravel interbed, saturated.	
		40.0
GW	Very dense, brown, Gravel; some sand, saturated. Top of weathered mudstone bedrock.	
		41.5
	Bottom of boring at 41.5 feet. Static groundwater at 34.0 feet.	

Depth scale: 0, 6.5, 13, 19.5, 26, 32.5, 39, 45.5

Sample No. and Type	NMC	Standard Penetration Test N
S-1	1.3%	3
S-2	2.6%	3
S-3	2.8%	3
S-4	3.5%	7
S-5	3.9%	5
S-6	4.8	7
S-7	4.0%	8
S-8		0
S-9	18%	18
S-10	22%	40
S-11	11%	69

Standard Penetration Test CURVE



Legend of Samplers: ▯ Grab sample ◩ SPT sample ⌶ Shelby tube sample

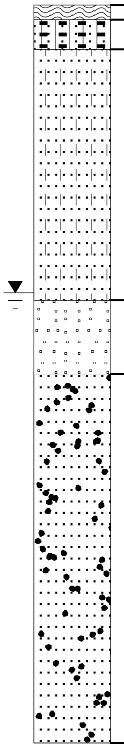
`This information pertains only to this boring and should not be interpreted as being indicative of the site.`

THE GALLI GROUP
GEOTECHNICAL CONSULTANTS

BORING LOG
B-3

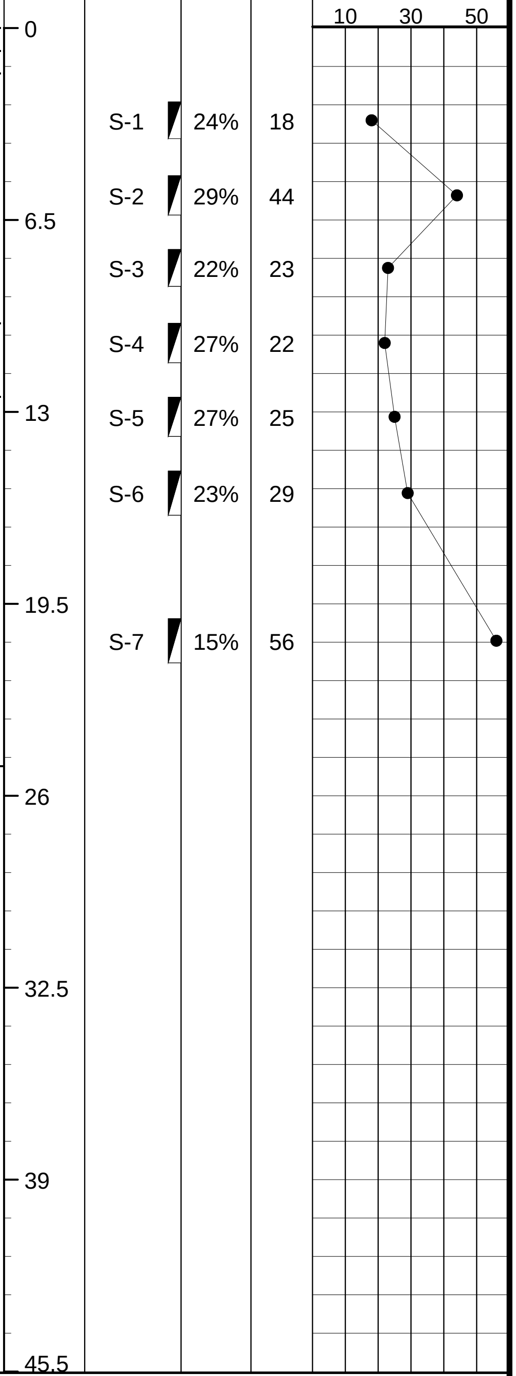
Project: Beach Loop Subdivision
Client: Bandon Beach Ventures, LLC
Location: See Site Plan
Driller: Western States (Shane)
Drill Rig: CME-55 track rig #2
Depth To Water> Initial ▽ : 9.75

Project No.: 02-6151-02
Date: 8/30/2022
Elevation: 77
Logged By: Lyn Chand
At Completion ▼ : 9.75



USCS	Description	Depth	Sample No. and Type	NMC	Standard Penetration Test N
OL	Wood debris/peat.	0.5			
SM	Loose, dark brown, silty Sand; with organics, moist.	1.5			
SW-SM	Medium dense, orange-brown, Sand; with silt, moist.		S-1 ◢	24%	18
SW-SM	Dense, mottled orange-gray, Sand with silt; moist.		S-2 ◢	29%	44
SW-SM	Medium dense, orange, Sand; with silt, moist to wet.	10.0	S-3 ◢	22%	23
SP	Medium dense, brown, Sand; saturated.	12.5	S-4 ◢	27%	22
SW	Medium dense to very dense, brown, gravelly Sand; trace clay, saturated.		S-5 ◢	27%	25
			S-6 ◢	23%	29
		25.0	S-7 ◢	15%	56

Bottom of boring at 25.0 feet.
Excessive heaving at the bottom of boring
Static groundwater at 9.75 feet.



Legend of Samplers: ▯ Grab sample ◢ SPT sample ‖ Shelby tube sample

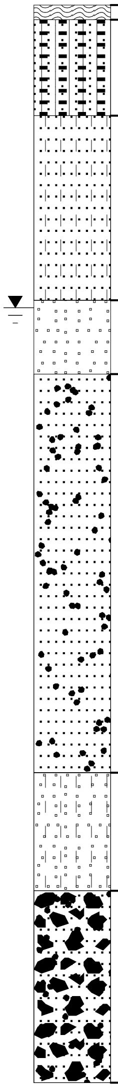
`This information pertains only to this boring and should not be interpreted as being indicative of the site.`

THE GALLI GROUP
GEOTECHNICAL CONSULTANTS

BORING LOG
B-4

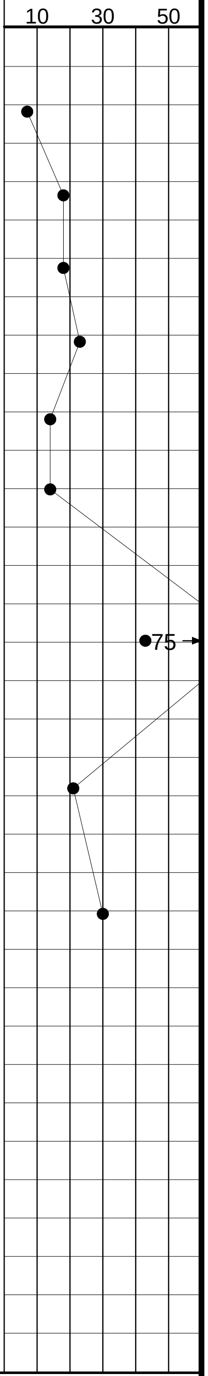
Project: Beach Loop Subdivision
Client: Bandon Beach Ventures, LLC
Location: See Site Plan
Driller: Western States (Shane)
Drill Rig: CME-55 track rig #2
Depth To Water> Initial ▽ : 10.25

Project No.: 02-6151-02
Date: 8/30/2022
Elevation: 74
Logged By: Lyn Chand
At Completion ▼ : 10.25



USCS	Description	Depth	Sample No. and Type	NMC	N
OL	Wood debris/peat. 0.5	0			
SM	Loose, orange, silty Sand; trace clay, moist. 3.75		S-1 ◪	6.8%	7
				19%	
SW-SM	Dense, orange-gray, Sand; with silt, moist.		S-2 ◪	21% 19%	18
		6.5			
SW-SM	Medium dense, brown-gray, Sand; with silt, moist. 10.0		S-3 ◪	20%	18
SP	Medium dense, orange, Sand; saturated. 12.5		S-4 ◪	18%	23
SW	Medium dense to very dense, orange-brown, Sand; with some gravel, trace clay, saturated.	13	S-5 ◪	25%	14
			S-6 ◪	26%	14
		19.5			
			S-7 ◪	19%	75
	26.0	26	S-8 ◪	31% 28%	21
SP-SM	Medium dense, gray, Sand; with silt, saturated. 30.0			16%	30
SW/GW	Dense, gray, cemented, sandy Gravel; saturated.	32.5			
	36.5				
	Bottom of boring at 36.5 feet. Excessive heaving at the bottom of boring. Static groundwater at 10.25 feet.	39			
		45.5			

Standard Penetration Test



Legend of Samplers: ▯ Grab sample ◪ SPT sample ⌶ Shelby tube sample

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

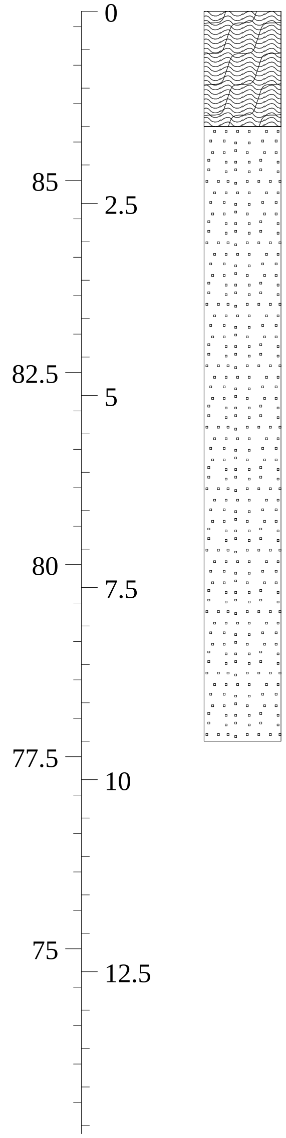
APPENDIX B

TEST PIT LOGS

TEST PIT LOG
Test Pit No.: TP-1

PROJECT	PROJECT NO.
Beach Loop Subdivision	02-6151-02
CLIENT	DATE
Bandon Beach Ventures, LLC	9/9/2022
LOCATION	ELEV.
See Site Plan	87.2 ft
EXCAVATION METHOD	LOGGER
Kubota KX057-5, 24" toothed bucket	Dennis Duru

DEPTH TO - Water: None When checked: None Caving: Moderate Caving



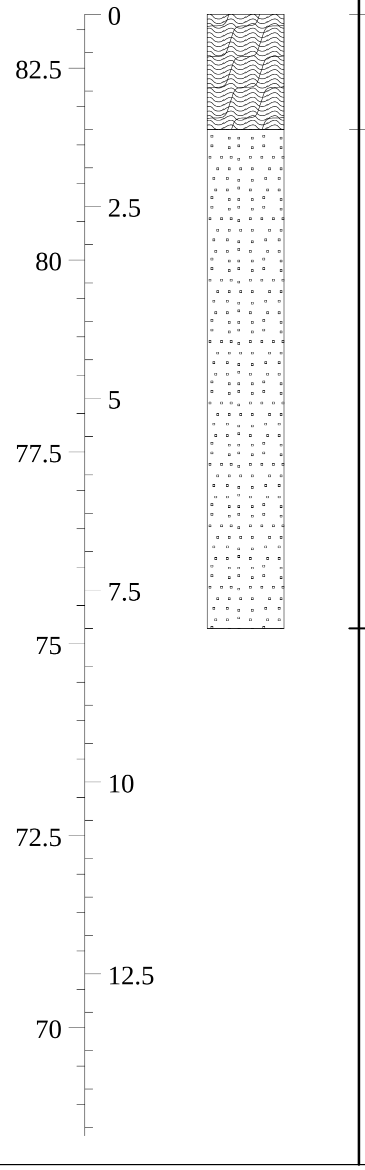
ELEVATION/ DEPTH	SOIL SYMBOLS AND SAMPLERS: GRAPHIC	BULK	DRIVEN	USCS	DESCRIPTION	DENSITY pcf	MOISTURE %
0				OL	Peat/organic zone.		
				SP	Very loose, brown, Sand; damp. Moderate caving.		
					Bottom of test pit at 9.5 feet. No free groundwater was encountered.		

Notes:

TEST PIT LOG
Test Pit No.: TP-2

PROJECT Beach Loop Subdivision	PROJECT NO. 02-6151-02
CLIENT Bandon Beach Ventures, LLC	DATE 9/9/2022
LOCATION See Site Plan	ELEV. 83.2 ft
EXCAVATION METHOD Kubota KX057-5, 24" toothed bucket	LOGGER Dennis Duru

DEPTH TO - Water: None When checked: None Caving: Excessive Caving



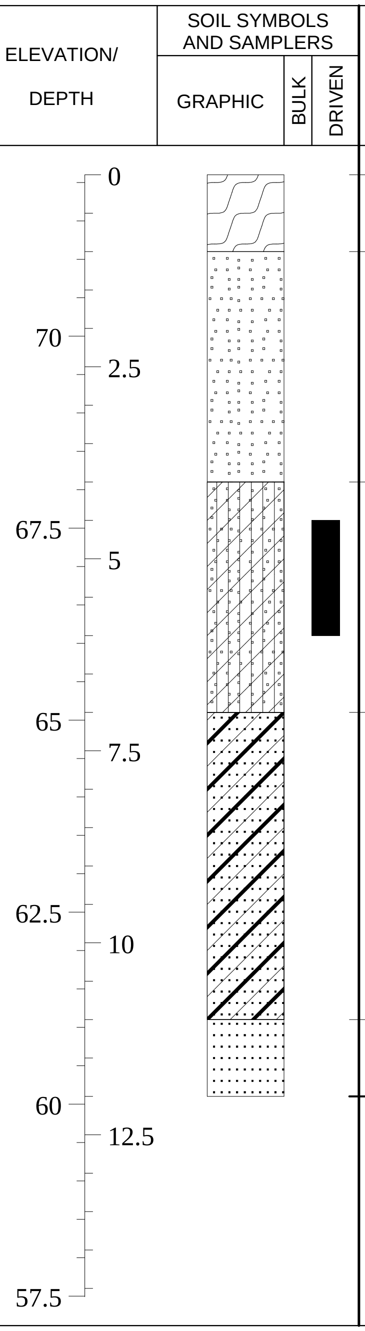
USCS	DESCRIPTION	DENSITY pcf	MOISTURE %
OL	Peat/organic zone.		
SP	Very loose, brown, Sand; dry. Excessive caving.		
	Bottom of test pit at 8.0 feet. No free groundwater was encountered.		

Notes:

TEST PIT LOG
Test Pit No.: TP-3

PROJECT	PROJECT NO.
Beach Loop Subdivision	02-6151-02
CLIENT	**DATE**
Bandon Beach Ventures, LLC	9/9/2022
LOCATION	**ELEV.**
See Site Plan	72.1 ft
EXCAVATION METHOD	**LOGGER**
Kubota KX057-5, 24" toothed bucket	Dennis Duru

DEPTH TO - Water: None When checked: None Caving:

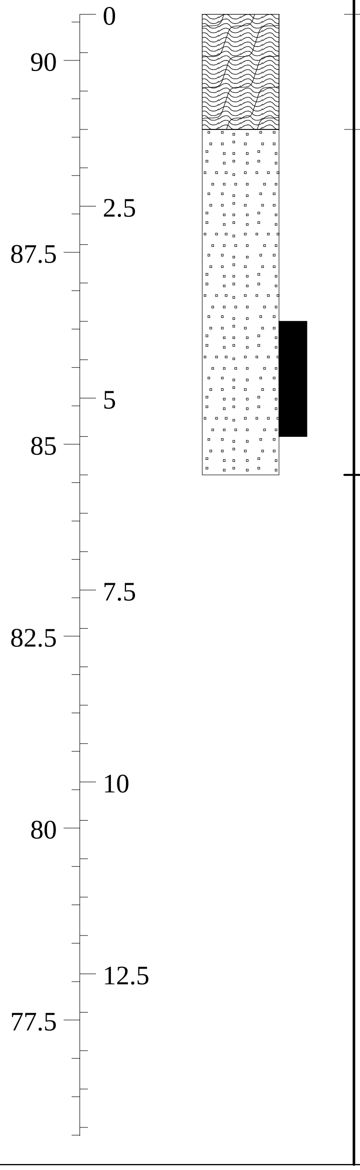


USCS	DESCRIPTION	DENSITY pcf	MOISTURE %
OL	Organic topsoil/grass rootzone.		
SP	Loose, brown, Sand; trace rootlets, damp.		
SM/SC	Loose, dark brown, silty clayey Sand; some rootlets, moist.	118.7	15.3%
CH/SC	Medium stiff, gray to mottled gray-brown, sandy Clay; moist, high plasticity.		
SW	Medium dense, brown, cemented Sand; moist.		
	Bottom of test pit at 12.0 feet. No free groundwater was encountered.		

Notes:

TEST PIT LOG
Test Pit No.: TP-4

PROJECT: Beach Loop Subdivision
PROJECT NO.: 02-6151-02
CLIENT: Bandon Beach Ventures, LLC
DATE: 9/9/2022
LOCATION: See Site Plan
ELEV.: 90.6 ft
EXCAVATION METHOD: Kubota KX057-5, 24" toothed bucket
LOGGER: Dennis Duru
DEPTH TO - Water: None When checked: None Caving: Excessive Caving



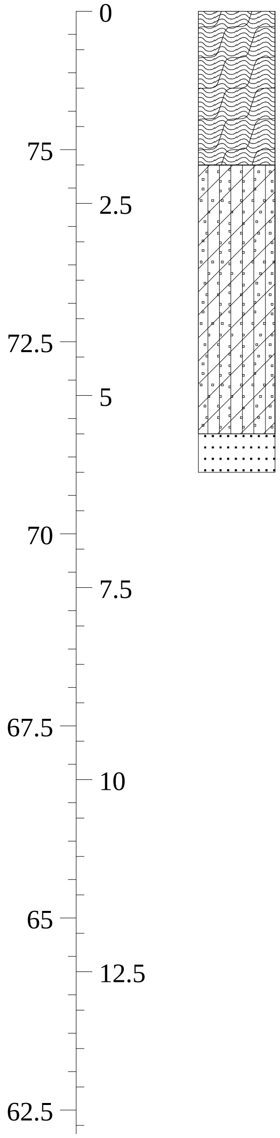
USCS	DESCRIPTION	DENSITY pcf	MOISTURE %
OL	Peat/organic zone.		
SP	Very loose, brown, Sand; dry. excessive caving.	102.2	5.7%
	Bottom of test pit at 6.0 feet. No free groundwater was encountered.		

Notes:

TEST PIT LOG
Test Pit No.: TP-5

PROJECT	PROJECT NO.
Beach Loop Subdivision	02-6151-02
CLIENT	DATE
Bandon Beach Ventures, LLC	9/9/2022
LOCATION	ELEV.
See Site Plan	76.8 ft
EXCAVATION METHOD	LOGGER
Kubota KX057-5, 24" toothed bucket	Dennis Duru

DEPTH TO - Water: None When checked: None Caving:



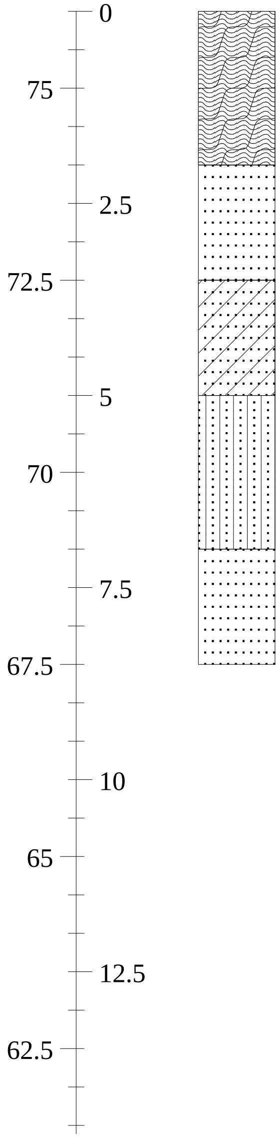
ELEVATION/ DEPTH	SOIL SYMBOLS AND SAMPLERS GRAPHIC	BULK	DRIVEN	USCS	DESCRIPTION	DENSITY pcf	MOISTURE %
				OL	Peat/organic zone.		
				SM/SC	Medium dense, gray-red-brown, silty clayey Sand; moist.		
				SW	Dense, red brown, cemented Sand; moist.		
					Bottom of test pit at 6.0 feet. No free groundwater was encountered.		

Notes:

TEST PIT LOG
Test Pit No.: TP-6

PROJECT Beach Loop Subdivision	PROJECT NO. 02-6151-02
CLIENT Bandon Beach Ventures, LLC	DATE 9/9/2022
LOCATION See Site Plan	ELEV. 76.0 ft
EXCAVATION METHOD Kubota KX057-5, 24" toothed bucket	LOGGER Dennis Duru

DEPTH TO - Water: None When checked: None Caving:



USCS	DESCRIPTION	DENSITY pcf	MOISTURE %
OL	Peat/organic zone.		
SW	Dense, orange-brown, cemented Sand; moist.		
SC	Dense, gray-red-brown, silty clayey Sand; moist.		
SM	Dense, gray-brown, silty Sand; moist.		
SW	Medium dense, brown, cemented Sand; moist.		
	Bottom of test pit at 8.5 feet. No free groundwater was encountered.		

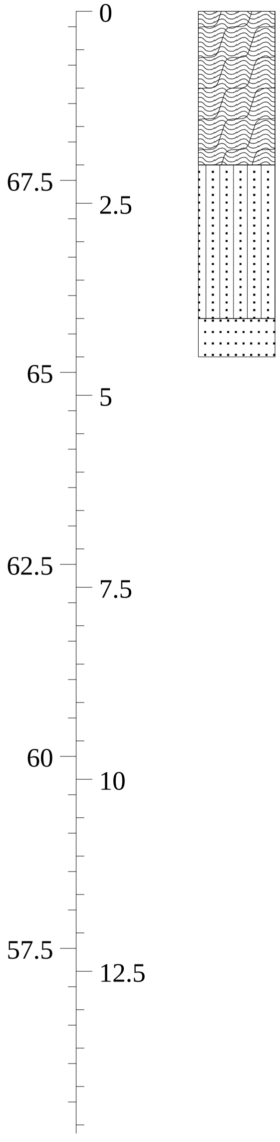
Notes:

TEST PIT LOG

Test Pit No.: TP-7

PROJECT Beach Loop Subdivision	PROJECT NO. 02-6151-02
CLIENT Bandon Beach Ventures, LLC	DATE 9/9/2022
LOCATION See Site Plan	ELEV. 69.7 ft
EXCAVATION METHOD Kubota KX057-5, 24" toothed bucket	LOGGER Dennis Duru

DEPTH TO - Water: None When checked: None Caving:



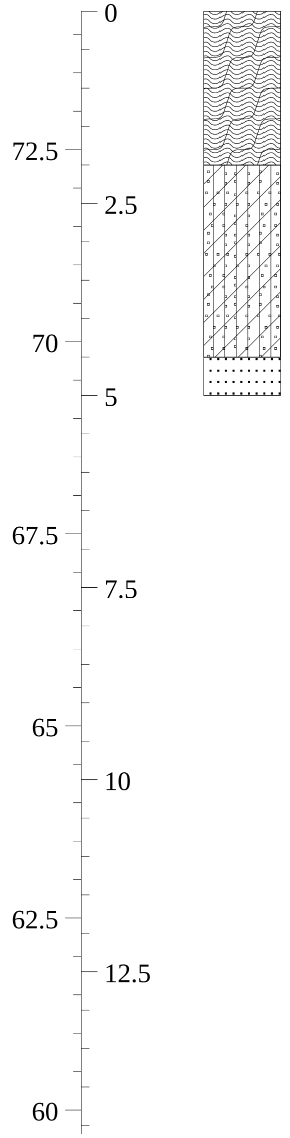
ELEVATION/ DEPTH	SOIL SYMBOLS AND SAMPLERS GRAPHIC	BULK	DRIVEN	USCS	DESCRIPTION	DENSITY pcf	MOISTURE %
				OL	Organic rootzone.		
				SM	Medium dense, gray, silty Sand; moist.		
				SW	Dense, light brown, cemented Sand; moist.		
					Bottom of test pit at 6.0 feet. No free groundwater was encountered.		

Notes:

TEST PIT LOG
Test Pit No.: TP-8

PROJECT Beach Loop Subdivision	PROJECT NO. 02-6151-02
CLIENT Bandon Beach Ventures, LLC	DATE 9/9/2022
LOCATION See Site Plan	ELEV. 74.3 ft
EXCAVATION METHOD Kubota KX057-5, 24" toothed bucket	LOGGER Dennis Duru

DEPTH TO - Water: None When checked: None Caving:

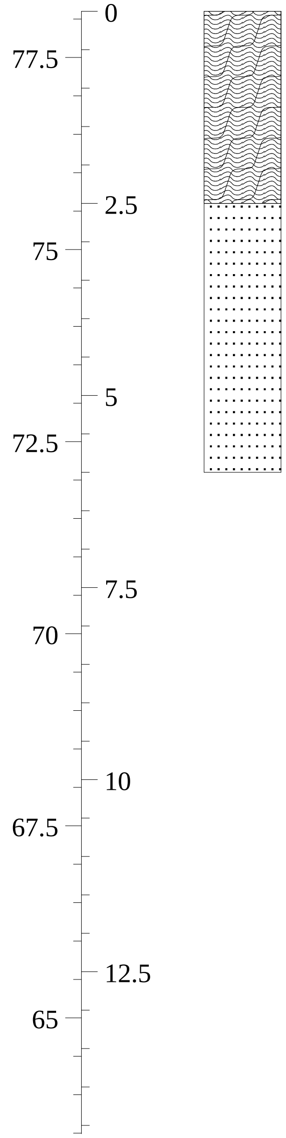


ELEVATION/ DEPTH	SOIL SYMBOLS AND SAMPLERS GRAPHIC	BULK	DRIVEN	USCS	DESCRIPTION	DENSITY pcf	MOISTURE %
				OL	Organic rootzone.		
				SM/SC	Medium dense, gray-brown, silty clayey Sand; moist.		
				SW	Dense, light brown, cemented Sand; moist.		
					Bottom of test pit at 6.0 feet. No free groundwater was encountered.		

Notes:

TEST PIT LOG
Test Pit No.: TP-9

PROJECT: Beach Loop Subdivision
PROJECT NO.: 02-6151-02
CLIENT: Bandon Beach Ventures, LLC
DATE: 9/9/2022
LOCATION: See Site Plan
ELEV.: 78.1 ft
EXCAVATION METHOD: Kubota KX057-5, 24" toothed bucket
LOGGER: Dennis Duru
DEPTH TO - Water: None When checked: None Caving:



ELEVATION/ DEPTH	SOIL SYMBOLS AND SAMPLERS: GRAPHIC	BULK	DRIVEN	USCS	DESCRIPTION	DENSITY pcf	MOISTURE %
				OL	Peat/Organic zone.		
				SW	Dense, light brown-gray, cemented Sand; moist.		
					Bottom of test pit at 6.0 feet. No free groundwater was encountered.		

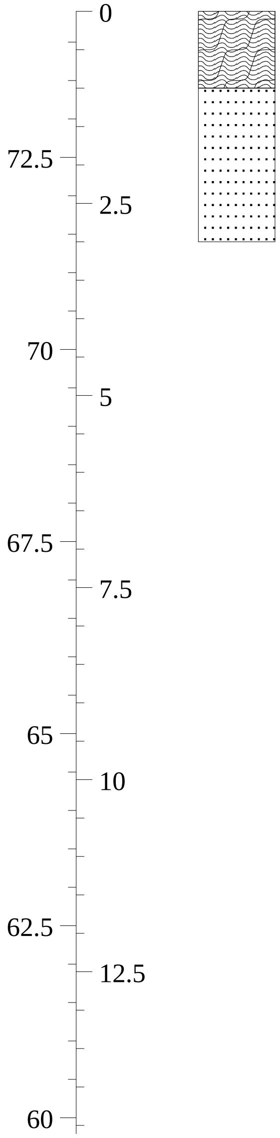
Notes:

TEST PIT LOG

Test Pit No.: TP-10

PROJECT Beach Loop Subdivision	PROJECT NO. 02-6151-02
CLIENT Bandon Beach Ventures, LLC	DATE 9/9/2022
LOCATION See Site Plan	ELEV. 74.4 ft
EXCAVATION METHOD Kubota KX057-5, 24" toothed bucket	LOGGER Dennis Duru

DEPTH TO - Water: None When checked: None Caving:



ELEVATION/ DEPTH	SOIL SYMBOLS AND SAMPLERS: GRAPHIC	BULK	DRIVEN	USCS	DESCRIPTION	DENSITY pcf	MOISTURE %
				OL	Peat/Organic zone.		
				SW	Dense, light brown-red, cemented Sand; moist.		
					Bottom of test pit at 3.0 feet. No free groundwater was encountered.		

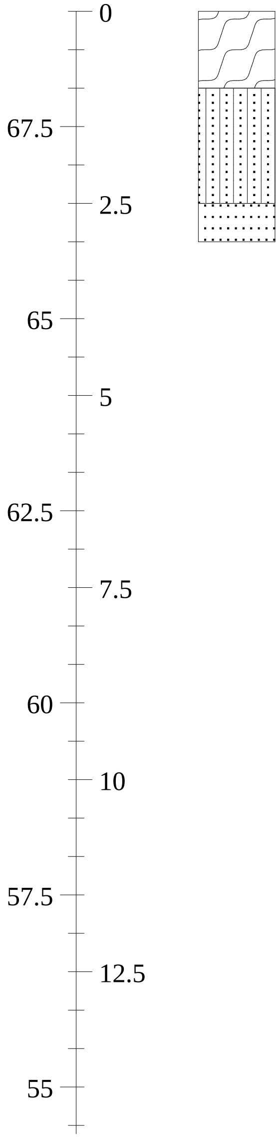
Notes:

TEST PIT LOG

Test Pit No.: TP-11

PROJECT Beach Loop Subdivision	PROJECT NO. 02-6151-02
CLIENT Bandon Beach Ventures, LLC	DATE 9/9/2022
LOCATION See Site Plan	ELEV. 69.0 ft
EXCAVATION METHOD Kubota KX057-5, 24" toothed bucket	LOGGER Dennis Duru

DEPTH TO - Water: None When checked: None Caving:



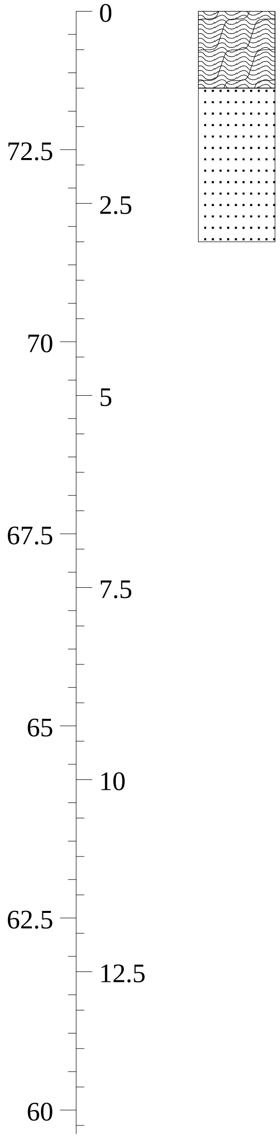
ELEVATION/ DEPTH	SOIL SYMBOLS AND SAMPLERS			USCS	DESCRIPTION	DENSITY pcf	MOISTURE %
	GRAPHIC	BULK	DRIVEN				
0				OL	Organic rootzone.		
				SM	Medium dense, brown, silty Sand; damp.		
2.5				SW	Very dense, light brown, cemented Sand; moist.		
					Bottom of test pit at 3.0 feet. No free groundwater was encountered.		

Notes:

TEST PIT LOG
Test Pit No.: TP-12

PROJECT Beach Loop Subdivision	PROJECT NO. 02-6151-02
CLIENT Bandon Beach Ventures, LLC	DATE 9/9/2022
LOCATION See Site Plan	ELEV. 74.3 ft
EXCAVATION METHOD Kubota KX057-5, 24" toothed bucket	LOGGER Dennis Duru

DEPTH TO - Water: None When checked: None Caving:



ELEVATION/ DEPTH	SOIL SYMBOLS AND SAMPLERS GRAPHIC	BULK	DRIVEN	USCS	DESCRIPTION	DENSITY pcf	MOISTURE %
				OL	Organic topsoil/grass rootzone.		
				SW	Dense, light brown, cemented Sand; moist.		
					Bottom of test pit at 3.0 feet. No free groundwater was encountered.		

Notes:

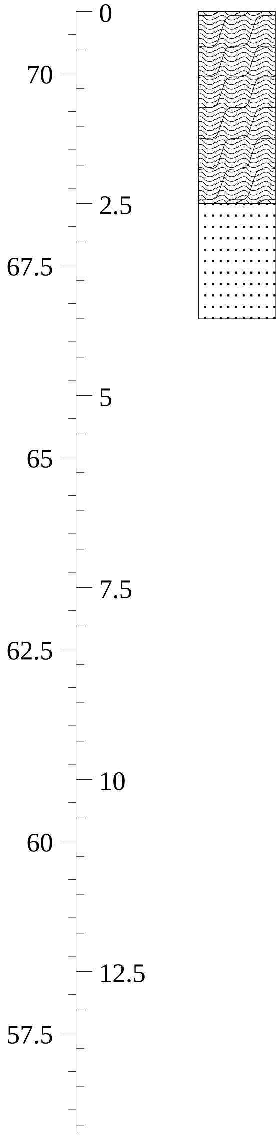
TEST PIT LOG

Test Pit No.: TP-13

PROJECT	PROJECT NO.
Beach Loop Subdivision	02-6151-02
CLIENT	**DATE**
Bandon Beach Ventures, LLC	9/9/2022
LOCATION	**ELEV.**
See Site Plan	70.8 ft
EXCAVATION METHOD	**LOGGER**
Kubota KX057-5, 24" toothed bucket	Dennis Duru

DEPTH TO - Water: None When checked: None Caving:

ELEVATION/ DEPTH	SOIL SYMBOLS AND SAMPLERS GRAPHIC	BULK	DRIVEN	USCS	DESCRIPTION	DENSITY pcf	MOISTURE %
0				OL	Grass rootzone.		
2.5				SW	Dense, light brown, cemented Sand; moist.		
4.0					Bottom of test pit at 4.0 feet. No free groundwater was encountered.		



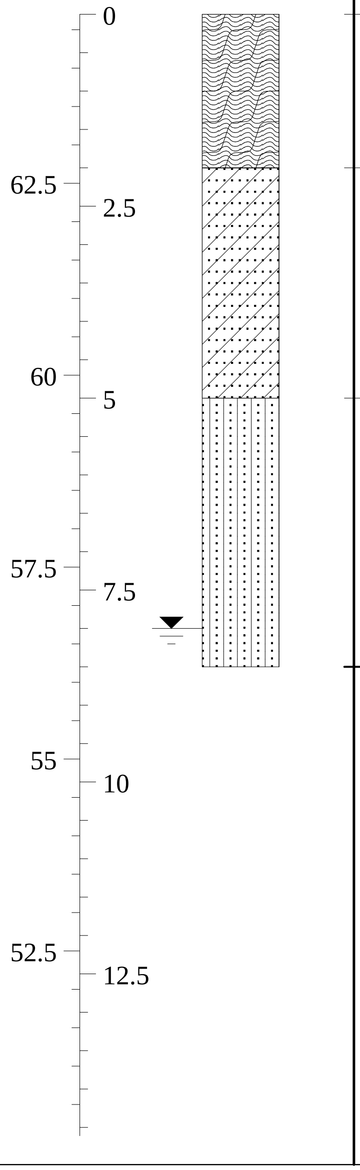
Notes:

TEST PIT LOG
Test Pit No.: TP-14

PROJECT Beach Loop Subdivision	PROJECT NO. 02-6151-02
CLIENT Bandon Beach Ventures, LLC	DATE 9/9/2022
LOCATION See Site Plan	ELEV. 64.7 ft
EXCAVATION METHOD Kubota KX057-5, 24" toothed bucket	LOGGER Dennis Duru

DEPTH TO - Water: 8.0 When checked: 8.0 Caving: minor caving below 8

ELEVATION/ DEPTH — SOIL SYMBOLS AND SAMPLERS: GRAPHIC, BULK, DRIVEN



USCS	DESCRIPTION	DENSITY pcf	MOISTURE %
OL	Peat/organic zone.		
SC	Loose, tan-gray, clayey Sand; moist. organic odor.		
SM	Loose, gray, silty Sand; wet. Organic odor.		
SM	Loose, gray, gravelly Sand; wet. Organic odor, moderate seepage, organic wood debris.		
	Bottom of test pit at 8.5 feet. Moderate seepage at base of excavation.		

Notes:

TEST PIT LOG

Test Pit No.: TP-15

PROJECT Beach Loop Subdivision	PROJECT NO. 02-6151-02
CLIENT Bandon Beach Ventures, LLC	DATE 9/9/2022
LOCATION See Site Plan	ELEV. 75 ft
EXCAVATION METHOD Kubota KX057-5, 24" toothed bucket	LOGGER Dennis Duru

DEPTH TO - Water: None When checked: None Caving:

ELEVATION/ DEPTH	SOIL SYMBOLS AND SAMPLERS			USCS	DESCRIPTION	DENSITY pcf	MOISTURE %
	GRAPHIC	BULK	DRIVEN				
75 — 0				OL	Organic rootzone.		
				SM	Medium dense, brown-gray, silty Sand; dry.		
72.5 — 2.5							
				SW	Very dense, light brown-orange, cemented Sand; moist.		
					Bottom of test pit at 3.5 feet. No free groundwater was encountered.		
70 — 5							
67.5 — 7.5							
65 — 10							
62.5 — 12.5							

Notes:

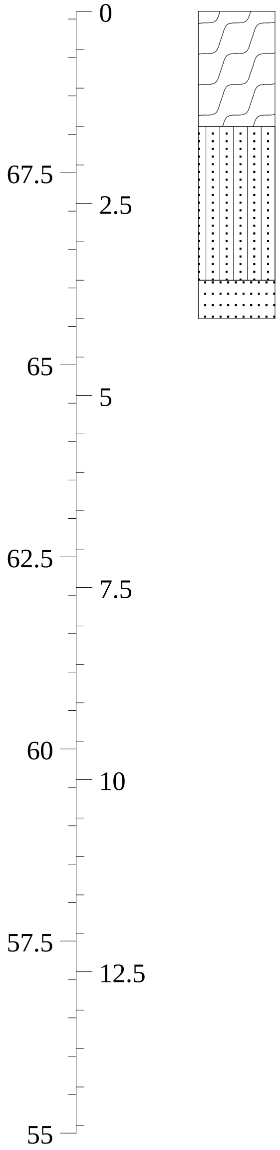
TEST PIT LOG

Test Pit No.: TP-16

PROJECT Beach Loop Subdivision	PROJECT NO. 02-6151-02
CLIENT Bandon Beach Ventures, LLC	DATE 9/9/2022
LOCATION See Site Plan	ELEV. 69.6 ft
EXCAVATION METHOD Kubota KX057-5, 24" toothed bucket	LOGGER Dennis Duru

DEPTH TO - Water: None When checked: None Caving:

ELEVATION/ DEPTH — SOIL SYMBOLS AND SAMPLERS: GRAPHIC, BULK, DRIVEN



USCS	DESCRIPTION	DENSITY pcf	MOISTURE %
OL	Organic rootzone.		
SM	Medium dense, brown, silty Sand; dry.		
SW	Very dense, light brown, cemented Sand; moist.		
	Bottom of test pit at 4.0 feet. No free groundwater was encountered.		

Notes:

APPENDIX C

PERMEABILITY TESTING RESULTS

FALLING HEAD PERMEABILITY TESTING

THE GALLI GROUP
Geotechnical Consulting

Client: Bandon Beach Ventures, LLC
Project: Beach Loop Subdivision
Job No.: 02-6151-02
Test Date: September 8, 2022

Test Hole No.: **P-1**
Depth of Hole (FT): 3.00
Diameter (IN): 8.0

Note: Falling head test was conducted in fine Sand (from 0' to 3').

Actual Time	**Lapsed Time (s)**	**Water Depth (ft)**	(1) **Dh (in)**	(2) **Dt (s)**	(3) **V (in^3)**	(4) **Q (in^3/s)**	(5) **L (in)**	(6) **Hc (in)**
15:32:00	0	0.00	0.0	0	0.0	0.000	0.0	0.0
15:34:00	120	1.25	15.0	120	753.6	6.280	28.5	14.3
15:35:00	60	2.00	9.0	60	452.2	7.536	31.5	15.8
15:36:00	60	2.33	4.0	60	201.0	3.349	34.0	17.0
15:39:00	180	3.00	8.0	180	401.9	2.233	32.0	16.0

Actual Time	(7) **mL/D**	(8) **(1+(mL/D)^2)^0.5**	(9) **ln (7)+(8)**	(10) **2pLHc**	(11) **(4)/(10)**	(12) **k (in/s)**	**k (in/hr)**
15:34:00	3.56	3.70	1.98	2551.8	2.46E-03	4.880E-03	17.567
15:35:00	3.94	4.06	2.08	3117.2	2.42E-03	5.027E-03	18.098
15:36:00	4.25	4.37	2.15	3631.7	9.22E-04	1.986E-03	7.150
15:39:00	4.00	4.12	2.09	3217.0	6.94E-04	1.454E-03	5.234

Average Permeability Coefficient (k): **3.34E-03** **in/sec**

Average Permeability Coefficient (k): **12.012** **in/hr**

FALLING HEAD PERMEABILITY TESTING

Client: Bandon Beach Ventures, LLC
Project: Beach Loop Subdivision
Job No.: 02-6151-02
Test Date: September 8, 2022

Test Hole No.: **P-2**
Depth of Hole (FT): 3.00
Diameter (IN): 8.0

Note: Falling head test was conducted in fine Sand (from 0' to 3').

Actual Time	**Lapsed Time (s)**	**Water Depth (ft)**	(1) **Dh (in)**	(2) **Dt (s)**	(3) **V (in^3)**	(4) **Q (in^3/s)**	(5) **L (in)**	(6) **Hc (in)**
15:43:00	0	0.00	0.0	0	0.0	0.000	0.0	0.0
15:44:00	60	1.17	14.0	60	703.4	11.723	29.0	14.5
15:45:00	60	1.75	7.0	60	351.7	5.861	32.5	16.3
15:47:00	120	3.00	15.0	120	753.6	6.280	28.5	14.3

Actual Time	(7) **mL/D**	(8) **(1+(mL/D)^2)^0.5**	(9) **ln (7)+(8)**	(10) **2pLHc**	(11) **(4)/(10)**	(12) **k (in/s)**	**k (in/hr)**
15:44:00	3.63	3.76	2.00	2642.1	4.44E-03	8.872E-03	31.938
15:45:00	4.06	4.18	2.11	3318.3	1.77E-03	3.727E-03	13.416
15:47:00	3.56	3.70	1.98	2551.8	2.46E-03	4.880E-03	17.567

Average Permeability Coefficient (k): **5.83E-03** **in/sec**

Average Permeability Coefficient (k): **20.973** **in/hr**

FALLING HEAD PERMEABILITY TESTING

THE GALLI GROUP
Geotechnical Consulting

Client: Bandon Beach Ventures, LLC
Project: Beach Loop Subdivision
Job No.: 02-6151-02
Test Date: September 8, 2022

Test Hole No.: **P-3**
Depth of Hole (FT): 3.00
Diameter (IN): 8.0

Note: Falling head test was conducted in fine Sand (from 0' to 3').

Actual Time	**Lapsed Time (s)**	**Water Depth (ft)**	(1) **Dh (in)**	(2) **Dt (s)**	(3) **V (in^3)**	(4) **Q (in^3/s)**	(5) **L (in)**	(6) **Hc (in)**
13:33:00	0	0.00	0.0	0	0.0	0.000	0.0	0.0
15:55:00	8520	2.50	30.0	8520	1507.2	0.177	21.0	10.5
15:56:00	60	0.25	-27.0	60	-1356.5	-22.608	49.5	24.8
16:01:00	300	0.42	1.0	300	50.2	0.167	35.0	17.5
16:06:00	300	0.58	2.0	300	100.5	0.335	35.0	17.5
16:11:00	300	0.75	2.0	300	100.5	0.335	35.0	17.5
16:16:00	300	0.83	2.0	300	100.5	0.335	35.5	17.8

Actual Time	(7) **mL/D**	(8) **(1+(mL/D)^2)^0.5**	(9) **ln (7)+(8)**	(10) **2pLHc**	(11) **(4)/(10)**	(12) **k (in/s)**	**k (in/hr)**
15:55:00	2.63	2.81	1.69	1385.4	1.28E-04	2.161E-04	0.778
16:01:00	4.38	4.49	2.18	3848.5	4.35E-05	9.494E-05	0.342
16:06:00	4.38	4.49	2.18	3848.5	8.70E-05	1.899E-04	0.684
16:11:00	4.38	4.49	2.18	3848.5	8.70E-05	1.899E-04	0.684
16:16:00	4.44	4.55	2.20	3959.2	8.46E-05	1.857E-04	0.669

Average Permeability Coefficient (k): 1.75E-04 in/sec

Average Permeability Coefficient (k): 0.631 in/hr

FALLING HEAD PERMEABILITY TESTING

Client: Bandon Beach Ventures, LLC
Project: Beach Loop Subdivision
Job No.: 02-6151-02
Test Date: September 8, 2022

Test Hole No.: **P-4**
Depth of Hole (FT): 3.00
Diameter (IN): 8.0

Note: Falling head test was conducted in fine Sand (from 0' to 3').

Actual Time	**Lapsed Time (s)**	**Water Depth (ft)**	(1) **Dh (in)**	(2) **Dt (s)**	(3) **V (in^3)**	(4) **Q (in^3/s)**	(5) **L (in)**	(6) **Hc (in)**
14:54:00	0	0.00	0.0	0	0.0	0.000	0.0	0.0
14:55:00	60	1.25	15.0	60	753.6	12.560	28.5	14.3
14:58:00	180	2.08	10.0	180	502.4	2.791	31.0	15.5
15:00:00	120	3.00	11.0	120	552.6	4.605	30.5	15.3

Actual Time	(7) **mL/D**	(8) **(1+(mL/D)^2)^0.5**	(9) **ln (7)+(8)**	(10) **2pLHc**	(11) **(4)/(10)**	(12) **k (in/s)**	**k (in/hr)**
14:55:00	3.56	3.70	1.98	2551.8	4.92E-03	9.759E-03	35.133
14:58:00	3.88	4.00	2.06	3019.1	9.24E-04	1.908E-03	6.869
15:00:00	3.81	3.94	2.05	2922.5	1.58E-03	3.228E-03	11.619

Average Permeability Coefficient (k): **4.97E-03** **in/sec**

Average Permeability Coefficient (k): **17.874** **in/hr**

FALLING HEAD PERMEABILITY TESTING

THE GALLI GROUP
Geotechnical Consulting

Client: Bandon Beach Ventures, LLC
Project: Beach Loop Subdivision
Job No.: 02-6151-02
Test Date: September 8, 2022

Test Hole No.: **P-5**
Depth of Hole (FT): 3.00
Diameter (IN): 8.0

Note: Falling head test was conducted in fine Sand (from 0' to 3').

Actual Time	**Lapsed Time (s)**	**Water Depth (ft)**	(1) **Dh (in)**	(2) **Dt (s)**	(3) **V (in^3)**	(4) **Q (in^3/s)**	(5) **L (in)**	(6) **Hc (in)**
15:06:00	0	0.00	0.0	0	0.0	0.000	0.0	0.0
15:07:00	60	0.25	3.0	60	150.7	2.512	34.5	17.3
15:09:00	120	0.58	4.0	120	201.0	1.675	34.0	17.0
15:11:00	120	0.75	1.0	120	50.2	0.419	35.0	17.5
15:14:00	180	1.00	3.0	180	150.7	0.837	34.5	17.3
15:17:00	180	1.17	2.0	180	100.5	0.558	35.0	17.5
15:21:00	240	1.42	2.0	240	100.5	0.419	34.5	17.3
15:26:00	300	1.58	2.0	300	100.5	0.335	35.0	17.5

Actual Time	(7) **mL/D**	(8) **(1+(mL/D)^2)^0.5**	(9) **ln (7)+(8)**	(10) **2pLHc**	(11) **(4)/(10)**	(12) **k (in/s)**	**k (in/hr)**
15:07:00	4.31	4.43	2.17	3739.3	6.72E-04	1.456E-03	5.243
15:09:00	4.25	4.37	2.15	3631.7	4.61E-04	9.931E-04	3.575
15:11:00	4.38	4.49	2.18	3848.5	1.09E-04	2.374E-04	0.855
15:14:00	4.31	4.43	2.17	3739.3	2.24E-04	4.854E-04	1.748
15:17:00	4.38	4.49	2.18	3848.5	1.45E-04	3.165E-04	1.139
15:21:00	4.31	4.43	2.17	3739.3	1.12E-04	2.427E-04	0.874
15:26:00	4.38	4.49	2.18	3848.5	8.70E-05	1.899E-04	0.684

Average Permeability Coefficient (k): **5.60E-04** **in/sec**

Average Permeability Coefficient (k): **2.017** **in/hr**

FALLING HEAD PERMEABILITY TESTING

THE GALLI GROUP
Geotechnical Consulting

Client: Bandon Beach Ventures, LLC
Project: Beach Loop Subdivision
Job No.: 02-6151-02
Test Date: September 8, 2022

Test Hole No.: **P-8**
Depth of Hole (FT): 3.00
Diameter (IN): 8.0

Note: Falling head test was conducted in fine Sand (from 0' to 3').

Actual Time	**Lapsed Time (s)**	**Water Depth (ft)**	(1) **Dh (in)**	(2) **Dt (s)**	(3) **V (in^3)**	(4) **Q (in^3/s)**	(5) **L (in)**	(6) **Hc (in)**
10:52 AM	**0:00**	0.00	**0**	0	0.0	**0**	**0**	0.0
11:50:00	3480	1.50	18.0	3480	904.3	0.260	27.0	13.5
13:40:00	6600	1.92	5.0	6600	251.2	0.038	33.5	16.8
13:40:00	0	3.00	13.0	0	653.1	#DIV/0!	29.5	14.8
16:27:00	10020	0.58	-16.0	10020	-803.8	-0.080	44.0	22.0
4:27:00	#NUM!	3.00	29.0	#NUM!	1457.0	#NUM!	21.5	10.8
16:29:00	120	0.17	1.0	120	50.2	0.419	38.5	19.3
16:34:00	300	0.25	1.0	300	50.2	0.167	35.5	17.8
16:39:00	300	0.42	2.0	300	100.5	0.335	35.0	17.5
16:44:00	300	0.50	2.0	300	100.5	0.335	35.5	17.8
16:49:00	300	0.58	2.0	300	100.5	0.335	35.5	17.8

Actual Time	(7) **mL/D**	(8) **(1+(mL/D)^2)^0.5**	(9) **ln (7)+(8)**	(10) **2pLHc**	(11) **(4)/(10)**	(12) **k (in/s)**	**k (in/hr)**
11:50:00	3.38	3.52	1.93	2290.2	1.13E-04	2.191E-04	0.789
13:40:00	4.19	4.31	2.14	3525.7	1.08E-05	2.309E-05	0.083
16:29:00	4.81	4.92	2.27	4656.6	8.99E-05	2.045E-04	0.736
16:34:00	4.44	4.55	2.20	3959.2	4.23E-05	9.287E-05	0.334
16:39:00	4.38	4.49	2.18	3848.5	8.70E-05	1.899E-04	0.684
16:44:00	4.44	4.55	2.20	3959.2	8.46E-05	1.857E-04	0.669
16:49:00	4.44	4.55	2.20	3959.2	8.46E-05	1.857E-04	0.669

Average Permeability Coefficient (k): **1.57E-04** **in/sec**

Average Permeability Coefficient (k): **0.566** **in/hr**

FALLING HEAD PERMEABILITY TESTING

THE GALLI GROUP
Geotechnical Consulting

Client: Bandon Beach Ventures, LLC
Project: Beach Loop Subdivision
Job No.: 02-6151-02
Test Date: September 8, 2022

Test Hole No.: **P-9**
Depth of Hole (FT): 3.00
Diameter (IN): 8.0

Note: Falling head test was conducted in silty Sand (from 0' to 3').

Actual Time	**Lapsed Time (s)**	**Water Depth (ft)**	(1) **Dh (in)**	(2) **Dt (s)**	(3) **V (in^3)**	(4) **Q (in^3/s)**	(5) **L (in)**	(6) **Hc (in)**
9:59:00	0	0.00	0.0	0	0.0	0.000	0.0	0.0
11:21:00	4920	1.67	20.0	4920	1004.8	0.204	26.0	13.0
11:21:00	0	0.00	-20.0	0	-1004.8	#DIV/0!	46.0	23.0
13:06:00	6300	1.58	19.0	6300	954.6	0.152	26.5	13.3
13:06:00	0	0.00	-19.0	0	-954.6	#DIV/0!	45.5	22.8
14:35:00	5340	1.25	15.0	5340	753.6	0.141	28.5	14.3
14:35:00	0	0.00	-15.0	0	-753.6	#DIV/0!	43.5	21.8
14:40:00	300	0.17	2.0	300	100.5	0.335	35.0	17.5
14:45:00	300	0.33	2.0	300	100.5	0.335	35.0	17.5
14:50:00	300	0.50	2.0	300	100.5	0.335	35.0	17.5
14:53:00	180	0.67	2.0	180	100.5	0.558	35.0	17.5

Actual Time	(7) **mL/D**	(8) **(1+(mL/D)^2)^0.5**	(9) **ln (7)+(8)**	(10) **2pLHc**	(11) **(4)/(10)**	(12) **k (in/s)**	**k (in/hr)**
11:21:00	3.25	3.40	1.89	2123.7	9.62E-05	1.822E-04	0.656
13:06:00	3.31	3.46	1.91	2206.2	6.87E-05	1.314E-04	0.473
14:35:00	3.56	3.70	1.98	2551.8	5.53E-05	1.097E-04	0.395
14:40:00	4.38	4.49	2.18	3848.5	8.70E-05	1.899E-04	0.684
14:45:00	4.38	4.49	2.18	3848.5	8.70E-05	1.899E-04	0.684
14:50:00	4.38	4.49	2.18	3848.5	8.70E-05	1.899E-04	0.684
14:53:00	4.38	4.49	2.18	3848.5	1.45E-04	3.165E-04	1.139

Average Permeability Coefficient (k): 1.87E-04 in/sec

Average Permeability Coefficient (k): 0.673 in/hr

FALLING HEAD PERMEABILITY TESTING

Client: Bandon Beach Ventures, LLC
Project: Beach Loop Subdivision
Job No.: 02-6151-02
Test Date: September 8, 2022

Test Hole No.: **P-10**
Depth of Hole (FT): 3.00
Diameter (IN): 8.0

Note: Falling head test was conducted in silty Sand (from 0' to 3').

Actual Time	**Lapsed Time (s)**	**Water Depth (ft)**	(1) **Dh (in)**	(2) **Dt (s)**	(3) **V (in^3)**	(4) **Q (in^3/s)**	(5) **L (in)**	(6) **Hc (in)**
13:00:00	0	0.00	0.0	0	0.0	0.000	0.0	0.0
14:17:00	4620	2.33	28.0	4620	1406.7	0.304	22.0	11.0
14:17:00	0	0.00	-28.0	0	-1406.7	#DIV/0!	50.0	25.0
14:22:00	300	0.50	6.0	300	301.4	1.005	33.0	16.5
14:27:00	300	0.67	2.0	300	100.5	0.335	35.0	17.5
14:32:00	300	0.83	2.0	300	100.5	0.335	35.0	17.5

Actual Time	(7) **mL/D**	(8) **(1+(mL/D)^2)^0.5**	(9) **ln (7)+(8)**	(10) **2pLHc**	(11) **(4)/(10)**	(12) **k (in/s)**	**k (in/hr)**
14:17:00	2.75	2.93	1.74	1520.5	2.00E-04	3.477E-04	1.252
14:22:00	4.13	4.24	2.12	3421.2	2.94E-04	6.240E-04	2.246
14:27:00	4.38	4.49	2.18	3848.5	8.70E-05	1.899E-04	0.684
14:32:00	4.38	4.49	2.18	3848.5	8.70E-05	1.899E-04	0.684

Average Permeability Coefficient (k): <u>3.38E-04</u> <u>in/sec</u>

Average Permeability Coefficient (k): <u>1.216</u> <u>in/hr</u>

FALLING HEAD PERMEABILITY TESTING

THE GALLI GROUP
Geotechnical Consulting

Client: Bandon Beach Ventures, LLC
Project: Beach Loop Subdivision
Job No.: 02-6151-02
Test Date: September 8, 2022

Test Hole No.: **P-11**
Depth of Hole (FT): 3.00
Diameter (IN): 8.0

Note: Falling head test was conducted in silty Sand (from 0' to 3').

Actual Time	**Lapsed Time (s)**	**Water Depth (ft)**	(1) **Dh (in)**	(2) **Dt (s)**	(3) **V (in^3)**	(4) **Q (in^3/s)**	(5) **L (in)**	(6) **Hc (in)**
13:52:00	0	0.00	0.0	0	0.0	0.000	0.0	0.0
13:59:00	420	0.83	10.0	420	502.4	1.196	31.0	15.5
14:04:00	300	1.17	4.0	300	201.0	0.670	34.0	17.0
14:09:00	300	1.67	6.0	300	301.4	1.005	33.0	16.5
14:14:00	300	1.83	2.0	300	100.5	0.335	35.0	17.5

Actual Time	(7) **mL/D**	(8) **(1+(mL/D)^2)^0.5**	(9) **ln (7)+(8)**	(10) **2pLHc**	(11) **(4)/(10)**	(12) **k (in/s)**	**k (in/hr)**
13:59:00	3.88	4.00	2.06	3019.1	3.96E-04	8.178E-04	2.944
14:04:00	4.25	4.37	2.15	3631.7	1.84E-04	3.972E-04	1.430
14:09:00	4.13	4.24	2.12	3421.2	2.94E-04	6.240E-04	2.246
14:14:00	4.38	4.49	2.18	3848.5	8.70E-05	1.899E-04	0.684

Average Permeability Coefficient (k): **4.08E-04** **in/sec**

Average Permeability Coefficient (k): **1.468** **in/hr**

APPENDIX D

LABORATORY TEST RESULTS

THE GALLI GROUP
Geotechnical Consulting
612 Northwest Third Street,
Grants Pass, Oregon 97526

California Bearing Ratio (CBR)
ASTM D1883

Client: Bandon Beach Ventures, LLC
Project: Beach Loop Rd.
Test Details: 1 Point CBR at optimal moisture content
Soil Type: light brown, silty Sand

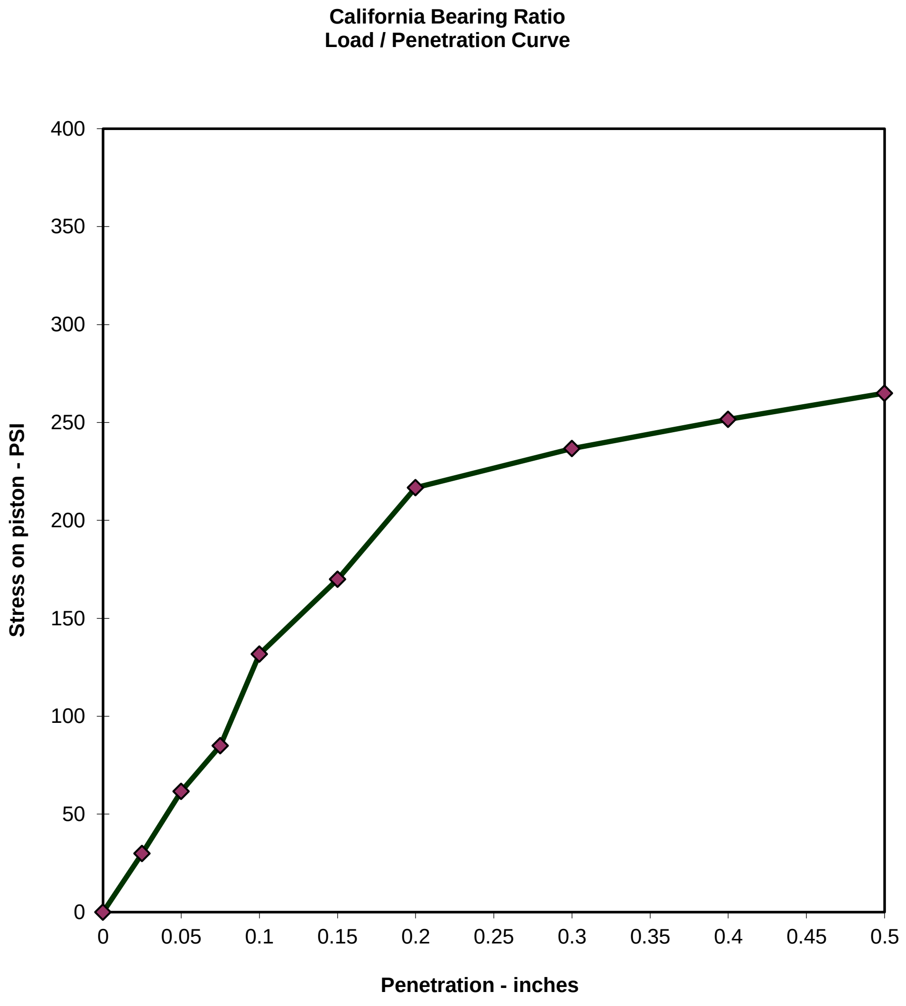
Date: 9/16/2022
Date Sampled: 9/9/2022
Job No.: 02-6151-02
Sample: TP-4

CBR Value: 13 **EQUIVALENT TO R-VALUE = 44**

Compacted utilizing ASTM D698 (Standard Proctor) methods

Tested Dry Density:	103.4	pcf (@ 97% of Maximum Dry Density, est.)
M/C:	19.7	% (existing)
Surcharge on sample:	12.6	pounds
Sample soaked:	112.5	hours
Swell:	0.0	%

Average moisture of sample after soaking: 18.8%
Moisture after soaking- upper 1" of sample: 17.6%



By: Ken Perry/Lyn Chand

THE GALLI GROUP
Geotechnical Consulting
612 Northwest Third Street,
Grants Pass, Oregon 97526

California Bearing Ratio (CBR)

ASTM D1883

Client: Bandon Beach Ventures, LLC
Project: Beach Loop Rd.
Test Details: 1 Point CBR at natural moisture content
Soil Type: light brown, silty Sand

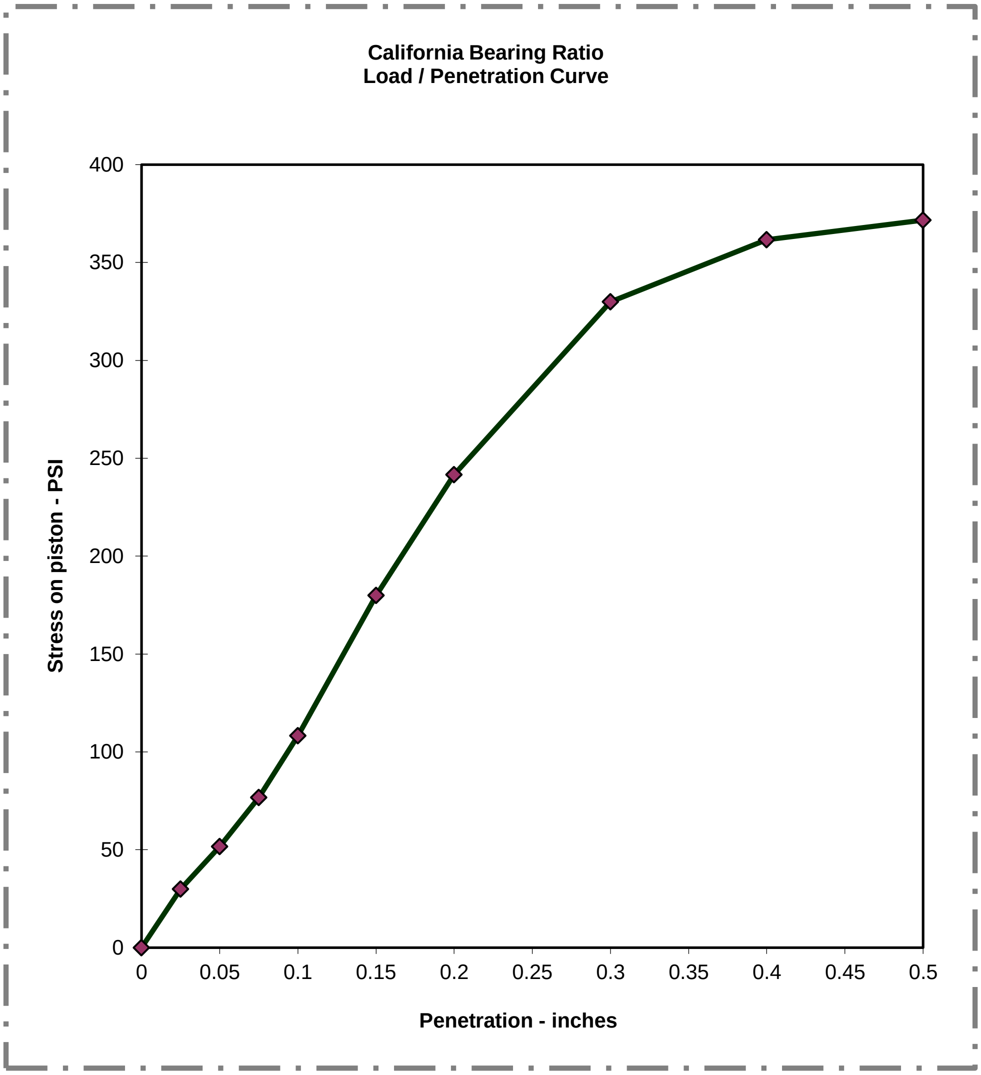
Date: 9/16/2022
Date Sampled: 9/9/2022
Job No.: 02-6151-02
Sample: TP-12

CBR Value: 11 **EQUIVALENT TO <u>R-VALUE = 40</u>**

Compacted utilizing ASTM D698 (Standard Proctor) methods

Tested Dry Density:	116.1	pcf	(@ 95% of Maximum Dry Density, est.)
M/C:	7.0	%	(existing)
Surcharge on sample:	12.6	pounds	
Sample soaked:	81	hours	
Swell:	0.0	%	

Average moisture of sample after soaking: 20.5%
Moisture after soaking- upper 1" of sample: 20.4%

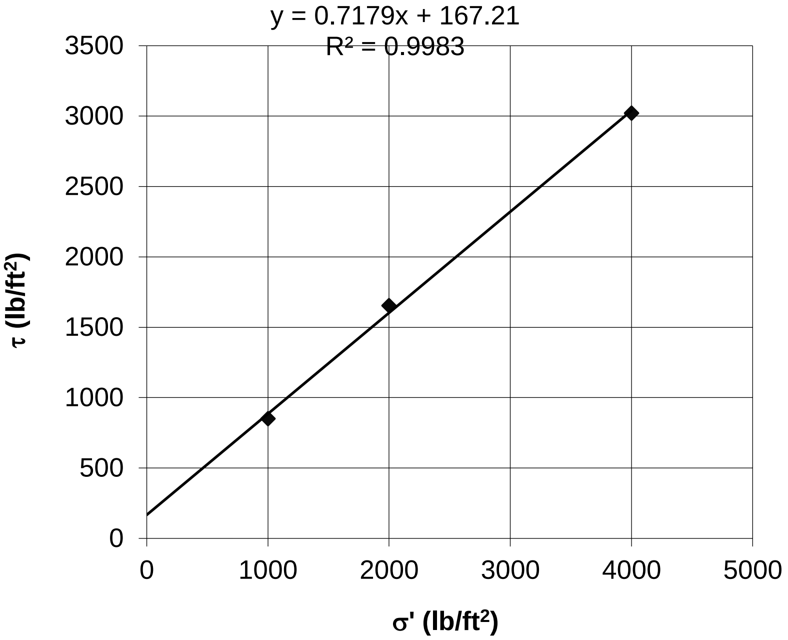


By: Ken Perry/Lyn Chand

Client: Bandon Beach Ventures, LLC
Project: Beach Loop Rd.
Date: 9/1/2022
Job No: 02-6151-02
Location: B1 S6 15'-16.5'
Sample Date: 8/29/2022
Description of Soil: silty Sand

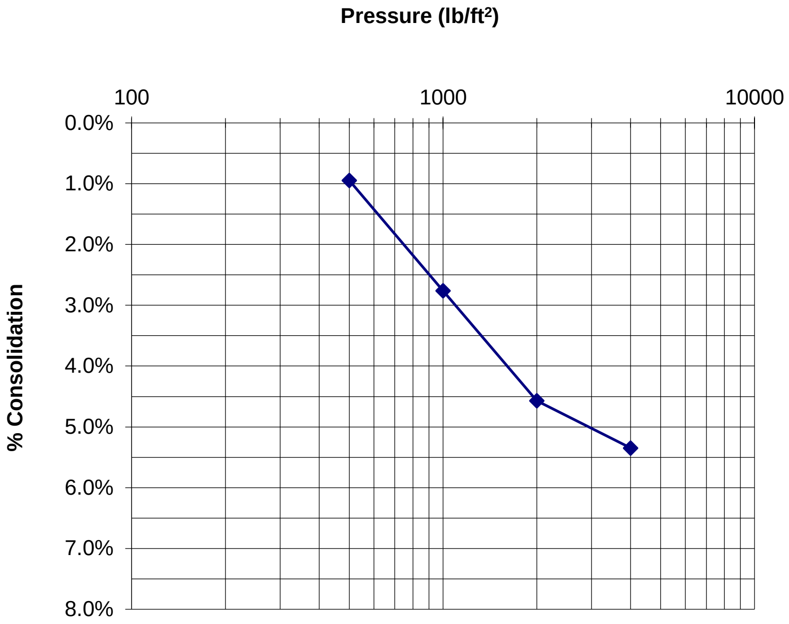
ϕ= 35
C= 167

σ'_1:	1000	τ_1:	851
σ'_2:	2000	τ_2	1655
σ'_3:	4000	τ_3:	3022



Percent Consolidation

Client: Bandon Beach Ventures, LLC
Project: Beach Loop Rd.
Date: 9/15/2022
Method of Test: ASTM D2435, Method A
Job No: 02-6151-02
Location: TP-3 @ 5.2'
Sample Date: 9/9/2022
Description of Soil: dark brown, silty Sand

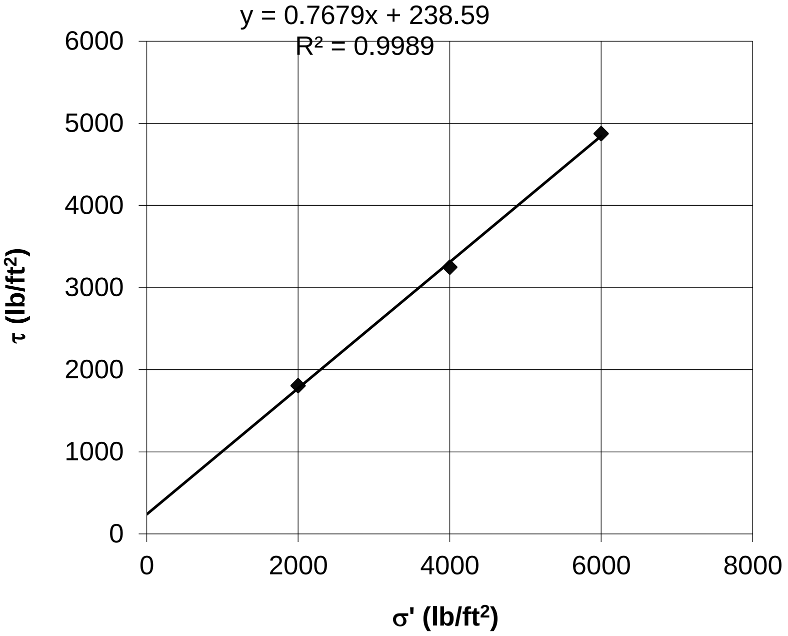


THE GALLI GROUP
Geotechnical Consulting

Client: Bandon Beach Ventures, LLC
Project: Beach Loop Rd.
Date: 9/15/2022
Job No: 02-6151-02
Location: TP-3 5.5'-6' @5.7
Sample Date: 9/9/2022
Description of Soil: Gray Light Brown silty clayey Sand

ϕ= 37.5
C= 238

σ'_1:	2000	τ_1:	1804
σ'_2:	4000	τ_2	3250
σ'_3:	6000	τ_3:	4876



THE GALLI GROUP
Geotechnical Consulting

Unit Weight

ASTM D7263

Client:	Bandon Beach Ventures, LLC
Project	Beach Loop Rd
Job No:	02-6151-02
Date:	9/16/2022
Sample Location:	TP-4, 4.0' - 6.0'
Sample Date:	9/9/2022
Description of Soil:	Light brown, silty Sand

Before Soak:		After Soak:	
Sample Wet Weight + Mold (lb)	24.84	Sample Wet Weight + Mold (lt	24.88
Weight of Mold (lb):	15.56	Weight of Mold (lb):	15.56
Weight of Soil (lb):	9.28	Weight of Soil (lb):	9.32
Volume of sample (Ft^3):	0.075	Volume of sample (Ft^3):	0.075
Sample Unit Wt. (PCF):	**123.7**	Sample Unit Wt. (PCF):	**124.3**
Sample Dry Unit Wt. (PCF):	**103.4**	Sample Dry Unit Wt. (PCF):	**103.4**

Trimmed Moisture Content		Soaked Moisture Content:	
can no.	G10	weight of dry soil (lb):	7.75
wet weight of soil + can (g)	749.12	weight of water (lb):	1.57
dry weight of soil + can (g)	657.42	moisture content (%)	20.2
weight of can (g)	191.80		
weight of dry soil (g)	465.62		
weight of water (g)	91.70		
moisture content (%)	19.7		

THE GALLI GROUP
Geotechnical Consulting

MOISTURE-DENSITY WORK SHEET

Client: Parametrix
Project: Beach Loop Rd.
Job No: 02-6151-02
Date of Test: 9/12/22
Test Standard: ASTM D-698 A
Visual Classification: Light Brown Sand
Sample Date: 9/9/22
Sample Location: TP 4 4'-6'

Maximum Dry Density (PCF): **105.1** (without rock correction)
Optimum Moisture Content: **17.3%** (without rock correction)

volume of mold (cu. ft.)	0.0333				
wet weight of soil + mold (lb)	12.80	12.94	13.06	13.40	13.48
weight of mold (lb)	9.38	9.38	9.38	9.38	9.38
wet weight of compacted soil (lb)	3.42	3.56	3.68	4.02	4.10
wet density (lbs/cu. ft.)	102.6	106.8	110.4	120.6	123.0
dry density (lbs/cu. ft.)	100.9	102.1	100.2	104.5	103.3

can no.	PP	D22	D2 AB	AD2	555
wet weight of soil + can (g)	1672.3	1791.0	1849.0	2003.9	2034.5
dry weight of soil + can (g)	1646.6	1719.4	1694.3	1760.5	1737.9
weight of can (g)	113.5	173.6	179.1	179.6	179.5
weight of dry soil (g)	1533.1	1545.8	1515.2	1580.9	1558.4
weight of water (g)	25.7	71.6	154.7	243.4	296.6
moisture content (%)	1.7	4.6	10.2	15.4	19.0

Oversize Correction:

Total Weight of Sample (lb): 56.11

Weight Retained on No. 4 Sieve (lb):	0.00	Percent Retained on No. 4 Sieve:	0.0%
Weight Retained on 3/8" Sieve (lb):	0.00	Percent Retained on 3/8" Sieve:	0.0%
Weight Retained on 3/4" Sieve (lb):	0.00	Percent Retained on 3/4" Sieve:	0.0%

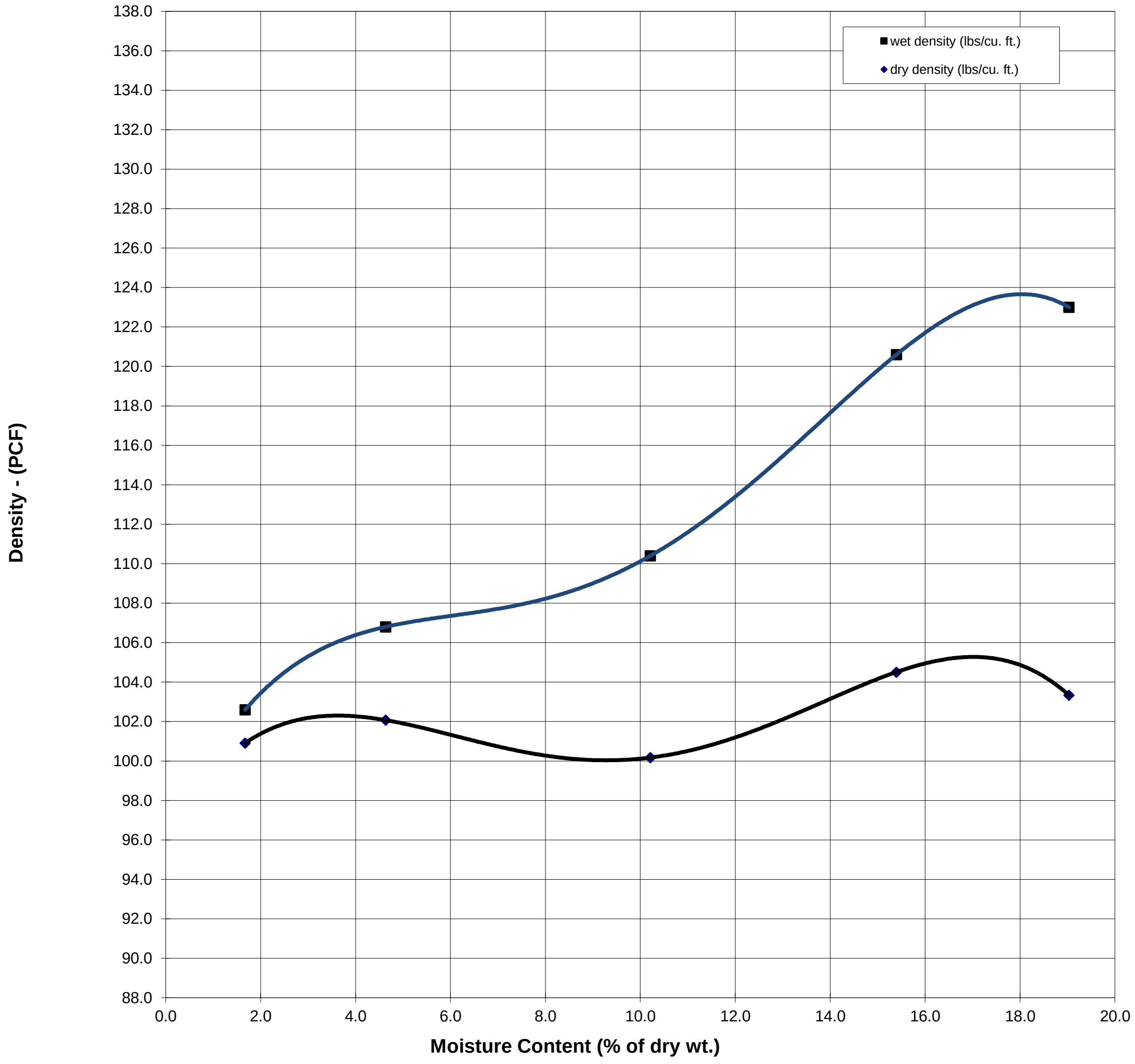
Corrected Maximum Dry Density (PCF): **N/A**

Corrected Moisture (%): **N/A**

By: Ken Perry

Client: Parametrix
Project: Beach Loop Rd.
Job No: 02-6151-02
Date of Test 9/12/2022
Method of Test: ASTM D-698 A
Visual Classification: Light Brown Sand
Sample Date: 9/9/2022
Sample Location: TP 4 4'-6'

Maximum Dry Density (PCF): 105.1
Optimum Moisture Content: 17.3%



THE GALLI GROUP
Geotechnical Consulting

Unit Weight

ASTM D7263

Client:	Bandon Beach Ventures, LLC
Project	Beach Loop Road
Job No:	02-6151-02
Date:	9/15/2022
Sample Location:	B-1, S-6
Sample Date:	9/9/2022
Description of Soil:	silty, clayey Sand

Sample Wet Weight (g):	158.96
Sample Length (in.):	1
Sample Diameter (in.):	2.5
Volume of sample (Ft^3):	0.00284
Sample Unit Wt. (PCF):	**123.3**
Sample Dry Unit Wt. (PCF):	**116.0**

can no.	26-Jan
wet weight of soil + can (g)	475.03
dry weight of soil + can (g)	451.2
weight of can (g)	71.94
weight of dry soil (g)	379.26
weight of water (g)	23.8
moisture content (% of dry weight	6.3

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Unit Weight
ASTM D7263

Client: Bandon Beach Ventures, LLC
Project Beach Loop Road
Job No: 02-6151-02
Date: 9/15/2022
Sample Location: TP-3 @ 5.2'
Sample Date: 9/9/2022
Description of Soil: silty, clayey Sand

Sample Wet Weight (g):	128.52
Sample Length (in.):	0.75
Sample Diameter (in.):	2.5
Volume of sample (Ft^3):	0.00213
Sample Unit Wt. (PCF):	**132.9**
Sample Dry Unit Wt. (PCF):	**111.9**

can no.	G2
wet weight of soil + can (g)	303.08
dry weight of soil + can (g)	285.35
weight of can (g)	190.82
weight of dry soil (g)	94.53
weight of water (g)	17.7
moisture content (% of dry weight)	18.8

THE GALLI GROUP
Geotechnical Consulting

Unit Weight
ASTM D7263

Client:	Bandon Beach Ventures, LLC
Project	Beach Loop Road
Job No:	02-6151-02
Date:	9/15/2022
Sample Location:	TP-3 @ 5.7'
Sample Date:	9/9/2022
Description of Soil:	silty, clayey Sand

Sample Wet Weight (g):	180.02
Sample Length (in.):	1.059
Sample Diameter (in.):	2.5
Volume of sample (Ft^3):	0.00301
Sample Unit Wt. (PCF):	**131.8**
Sample Dry Unit Wt. (PCF):	**113.7**

can no.	A3
wet weight of soil + can (g)	443.27
dry weight of soil + can (g)	410.2
weight of can (g)	202.01
weight of dry soil (g)	208.19
weight of water (g)	33.1
moisture content (% of dry weight	15.9

THE GALLI GROUP
Geotechnical Consulting

Unit Weight

ASTM D7263

Client:	Bandon Beach Ventures, LLC
Project	Beach Loop Road
Job No:	02-6151-02
Date:	9/16/2022
Sample Location:	TP-4
Sample Date:	9/9/2022
Description of Soil:	light brown, Sand

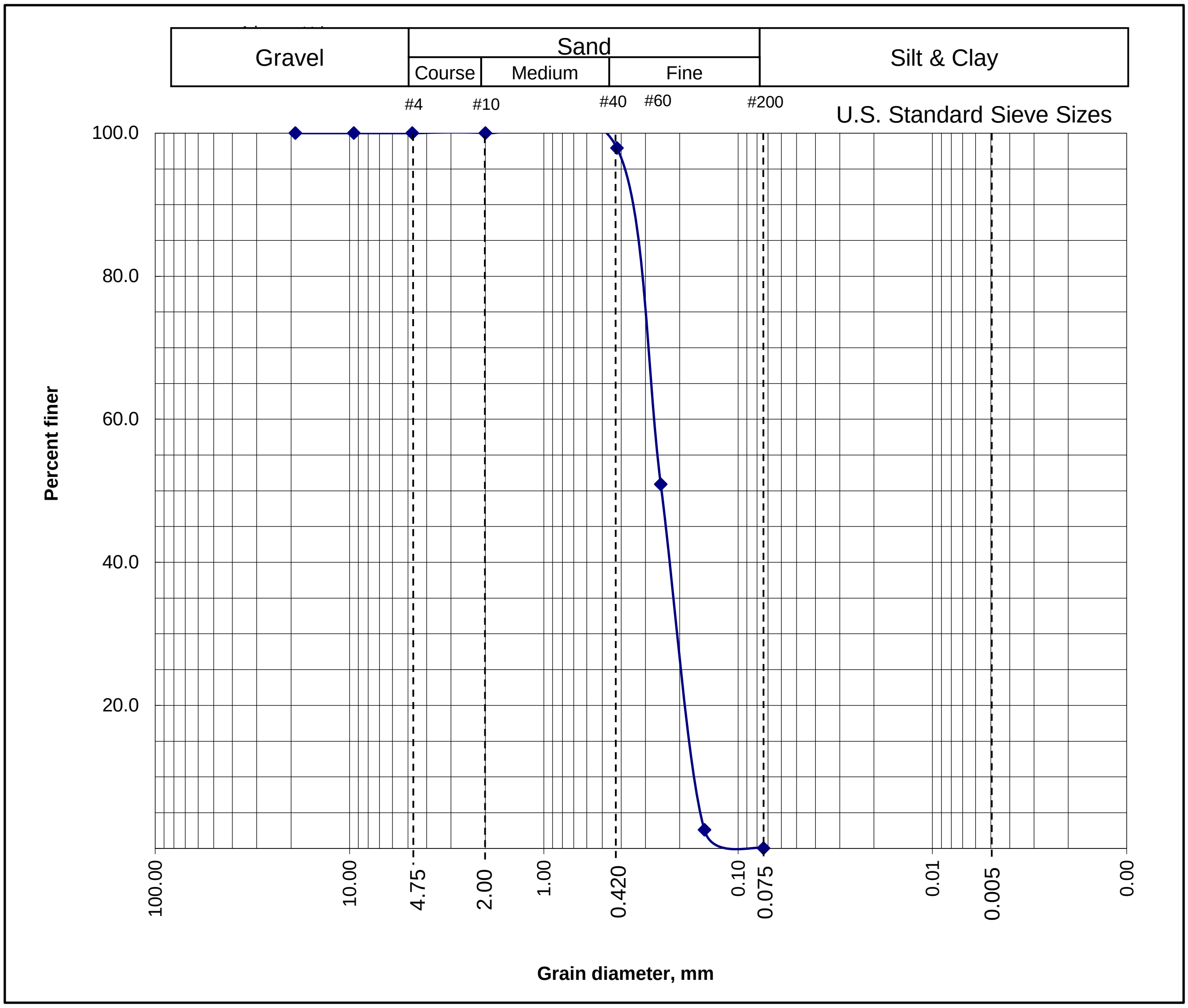
Sample Wet Weight (g):	159.9
Sample Length (in.):	1.05
Sample Diameter (in.):	2.5
Volume of sample (Ft^3):	0.00298
Sample Unit Wt. (PCF):	**118.1**
Sample Dry Unit Wt. (PCF):	**111.1**

can no.	2 4
wet weight of soil + can (g)	288.1
dry weight of soil + can (g)	276.22
weight of can (g)	87.9
weight of dry soil (g)	188.32
weight of water (g)	11.9
moisture content (% of dry weight)	6.3

THE GALLI GROUP
Geotechnical Consulting

Washed Sieve Analysis
(ASTM-D1140)

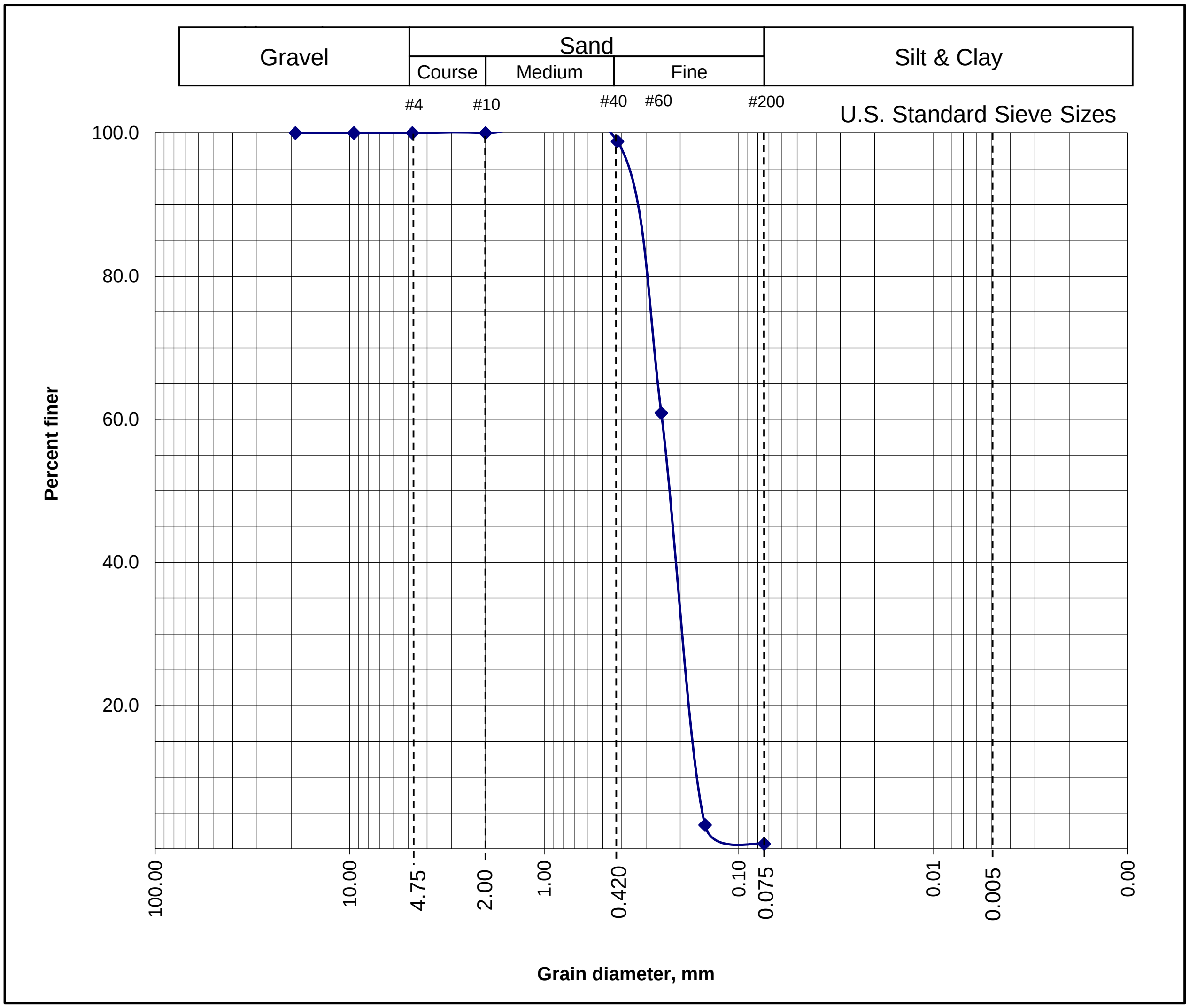
Client: Bandon Beach Ventures, LLC
Project: Beach Loop Rd.
Job No: 02-6151-02
Date Tested: 9/1/2022
Date Sampled: 8/29/2022
Description of Soil: Sand
Sample Location: B-1/S-2
Depth of Sample: 5.0' - 6.5'



Tested by: Ken Perry

Washed Sieve Analysis
(ASTM-D1140)

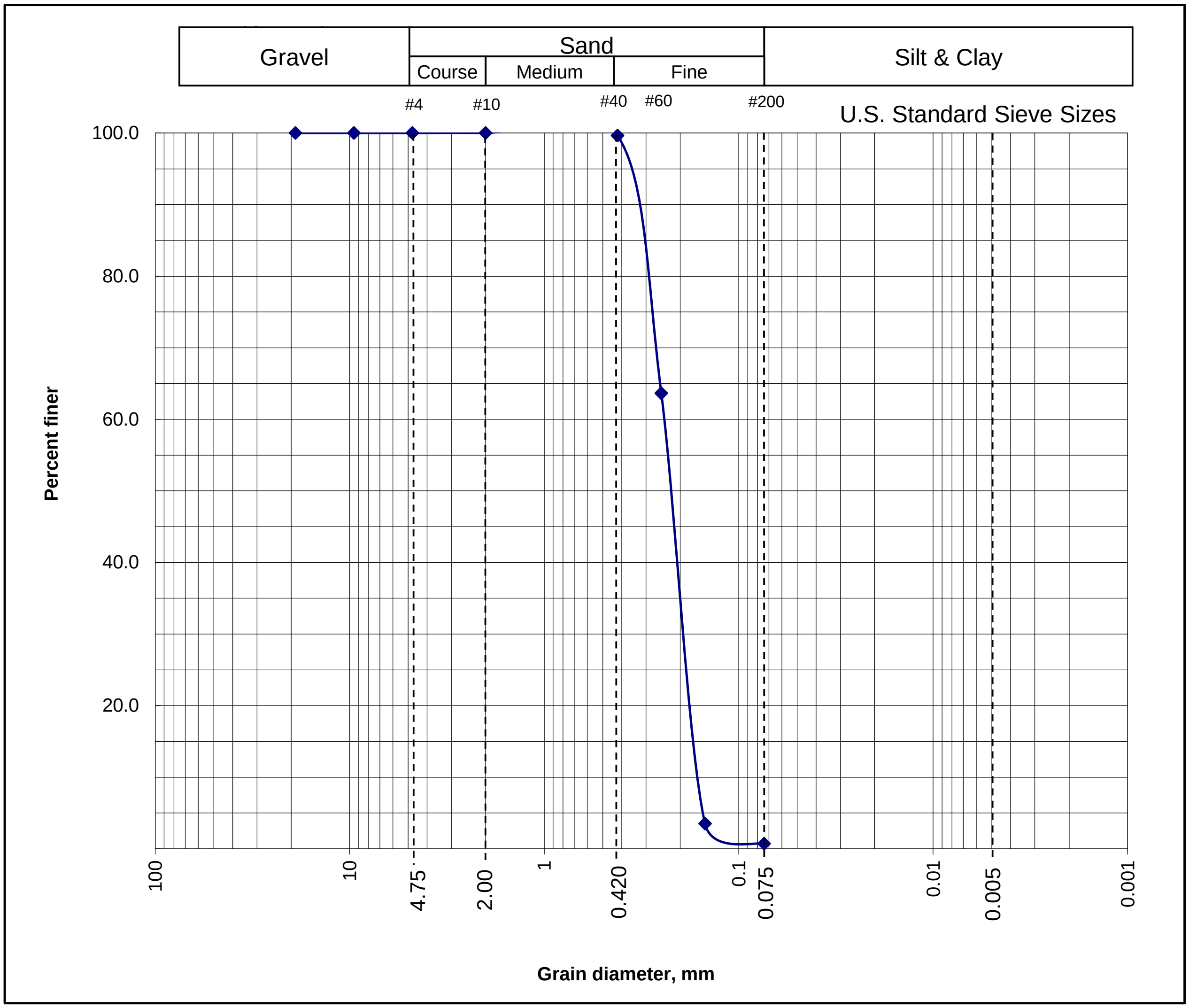
Client: Bandon Beach Ventures, LLC
Project: Beach Loop Rd.
Job No: 02-6151-02
Date Tested: 9/1/2022
Date Sampled: 8/29/2022
Description of Soil: Sand
Sample Location: B-2/S-4
Depth of Sample: 10.0' - 11.5'



Tested by: Ken Perry

Washed Sieve and Hydrometer Analysis
(ASTM D1140 and ASTM D7928)

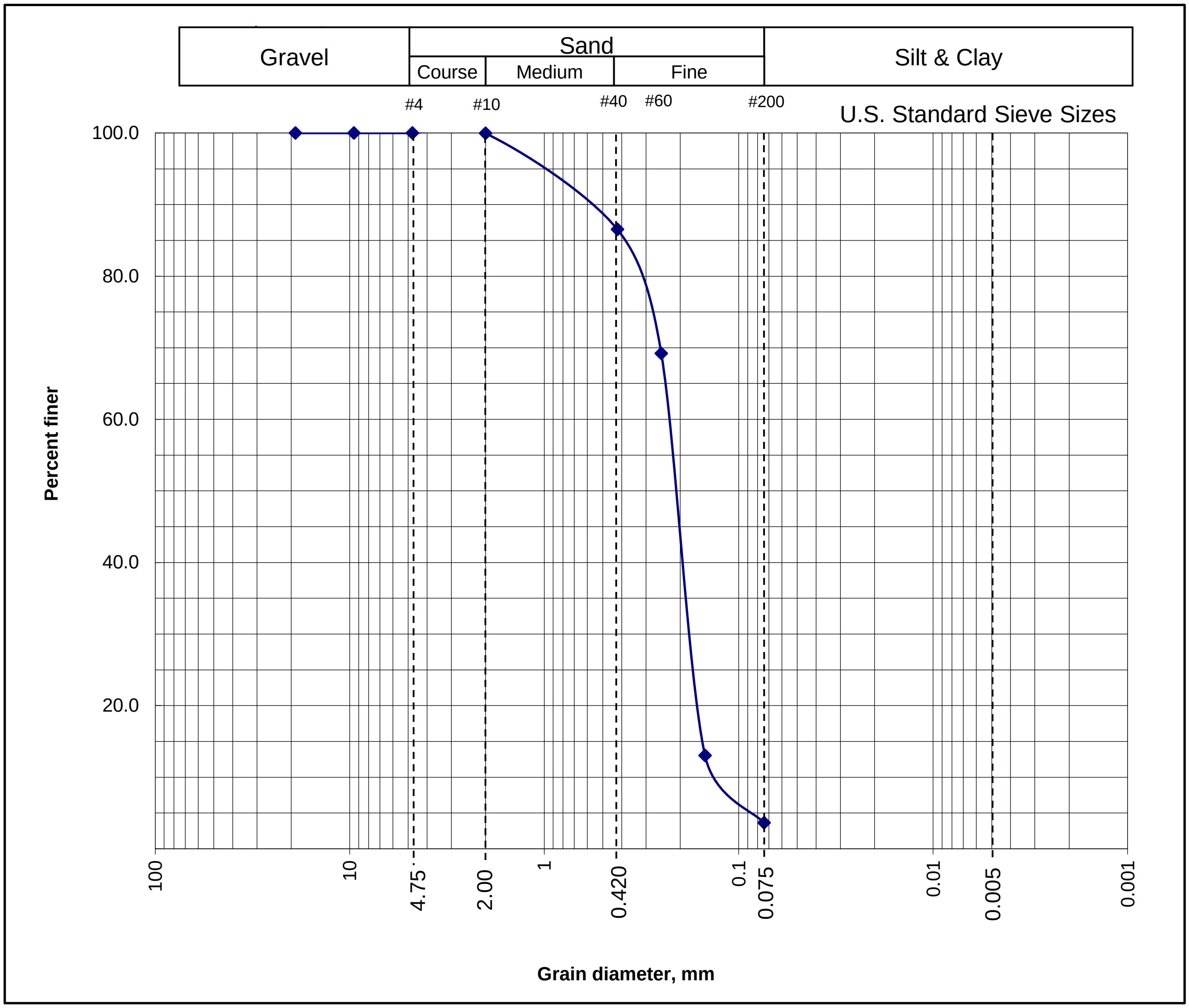
Client: Bandon Beach Ventures, LLC
Project: Beach Loop Rd.
Job No: 02-6151-02
Date Tested: 9/1/2022
Date Sampled: 8/29/2022
Description of Soil: Sand
Boring No / Sample No: B-2/S-6
Depth of Sample: 15.0' - 16.5'



Tested by: Ken Perry

Washed Sieve Analysis
(ASTM D1140)

Client: Bandon Beach Ventures, LLC
Project: Beach Loop Rd.
Job No: 02-6151-02
Date Tested: 9/1/2022
Date Sampled: 8/29/2022
Description of Soil: Sand
Boring No / Sample No: B-2/S-9
Depth of Sample: 30.0' - 31.5'

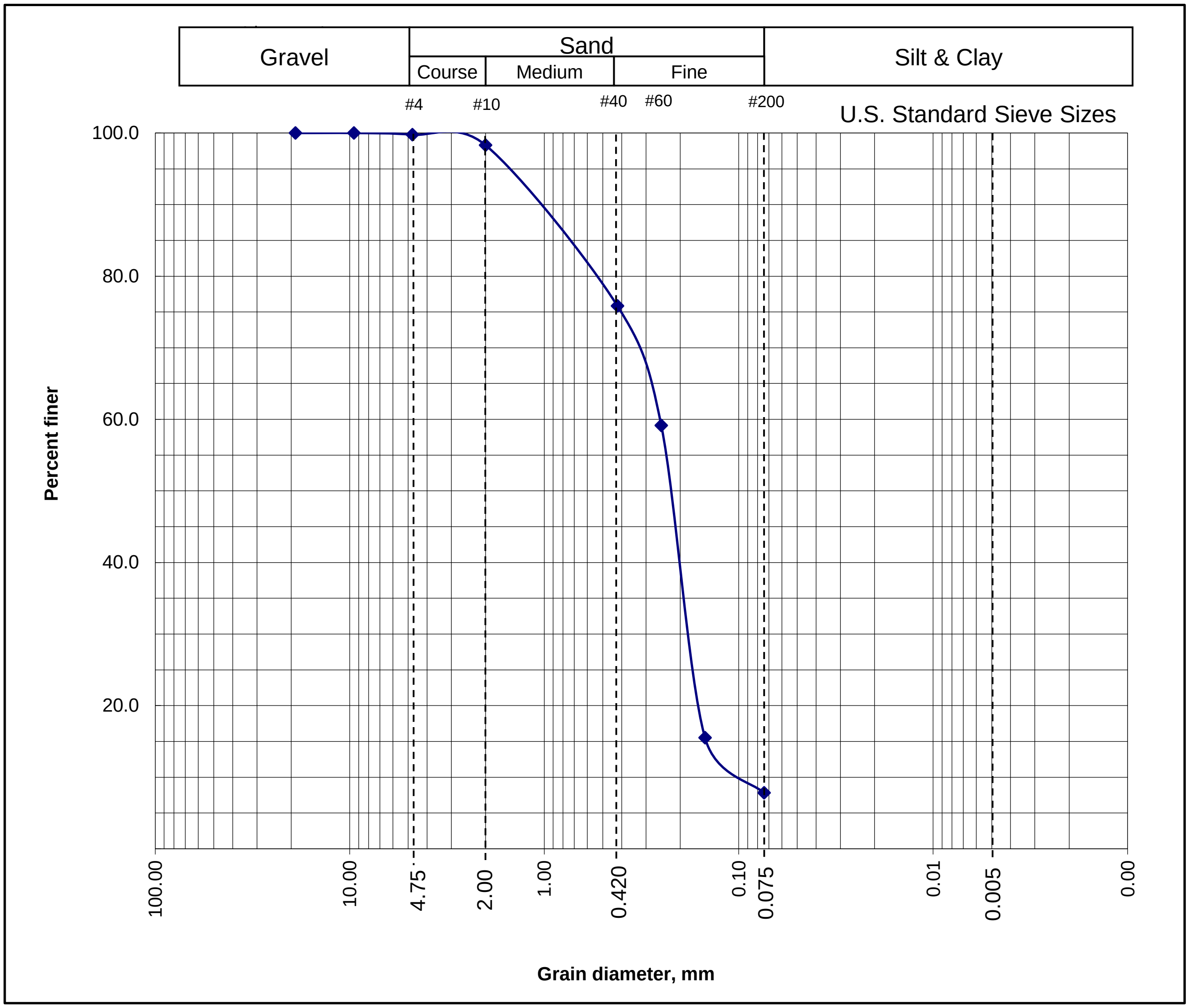


Tested by: Ken Perry

THE GALLI GROUP
Geotechnical Consulting

Washed Sieve Analysis
(ASTM-D1140)

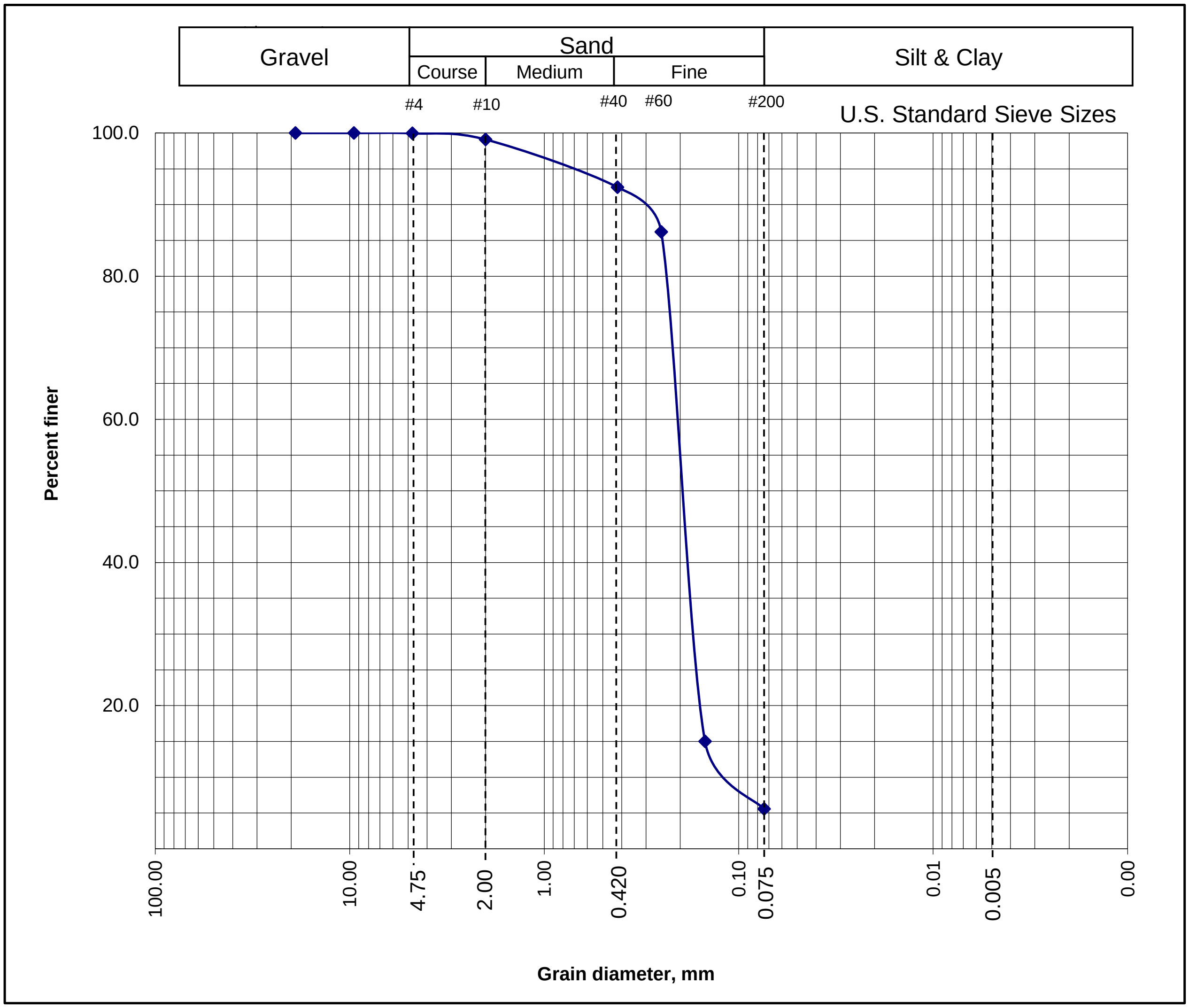
Client:	Bandon Beach Ventures, LLC
Project:	Beach Loop Rd.
Job No:	02-6151-02
Date Tested:	9/1/2022
Date Sampled:	8/29/2022
Description of Soil:	Sand, trace silt and clay
Sample Location:	B-4/S-2
Depth of Sample:	5.0' - 6.5'



Tested by: Ken Perry

Washed Sieve Analysis
(ASTM-D1140)

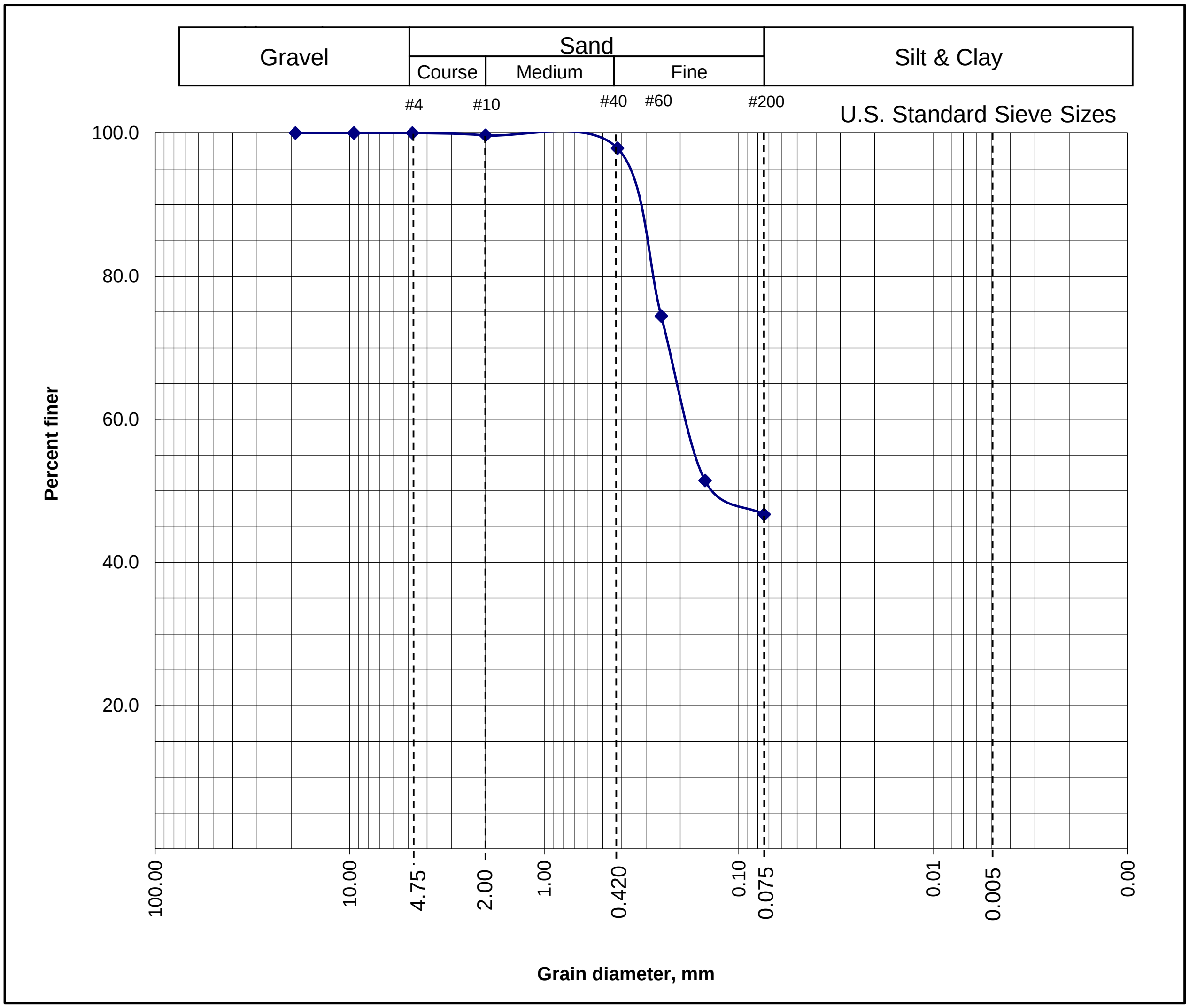
Client:	Bandon Beach Ventures, LLC
Project:	Beach Loop Rd.
Job No:	02-6151-02
Date Tested:	9/1/2022
Date Sampled:	8/29/2022
Description of Soil:	Sand
Sample Location:	B-4/S-4
Depth of Sample:	10.0' - 11.5'



Tested by: Ken Perry

Washed Sieve Analysis
(ASTM-D1140)

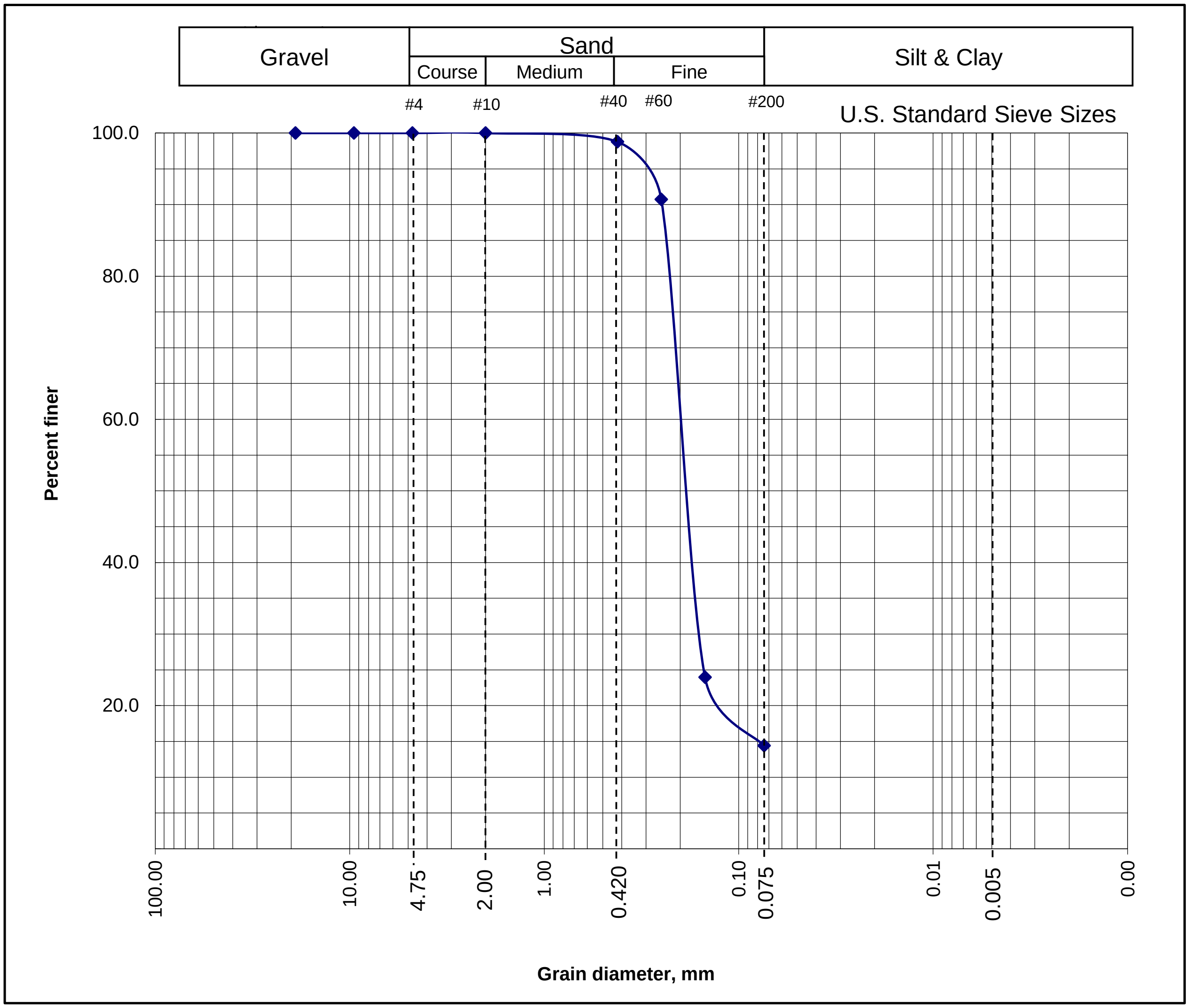
Client: Bandon Beach Ventures, LLC
Project: Beach Loop Rd.
Job No: 02-6151-02
Date Tested: 9/12/2022
Date Sampled: 9/9/2022
Description of Soil: silty, clayey Sand
Sample Location: TP-5/S-1
Depth of Sample: 2.0'-3.0'



Tested by: Ken Perry

Washed Sieve Analysis
(ASTM-D1140)

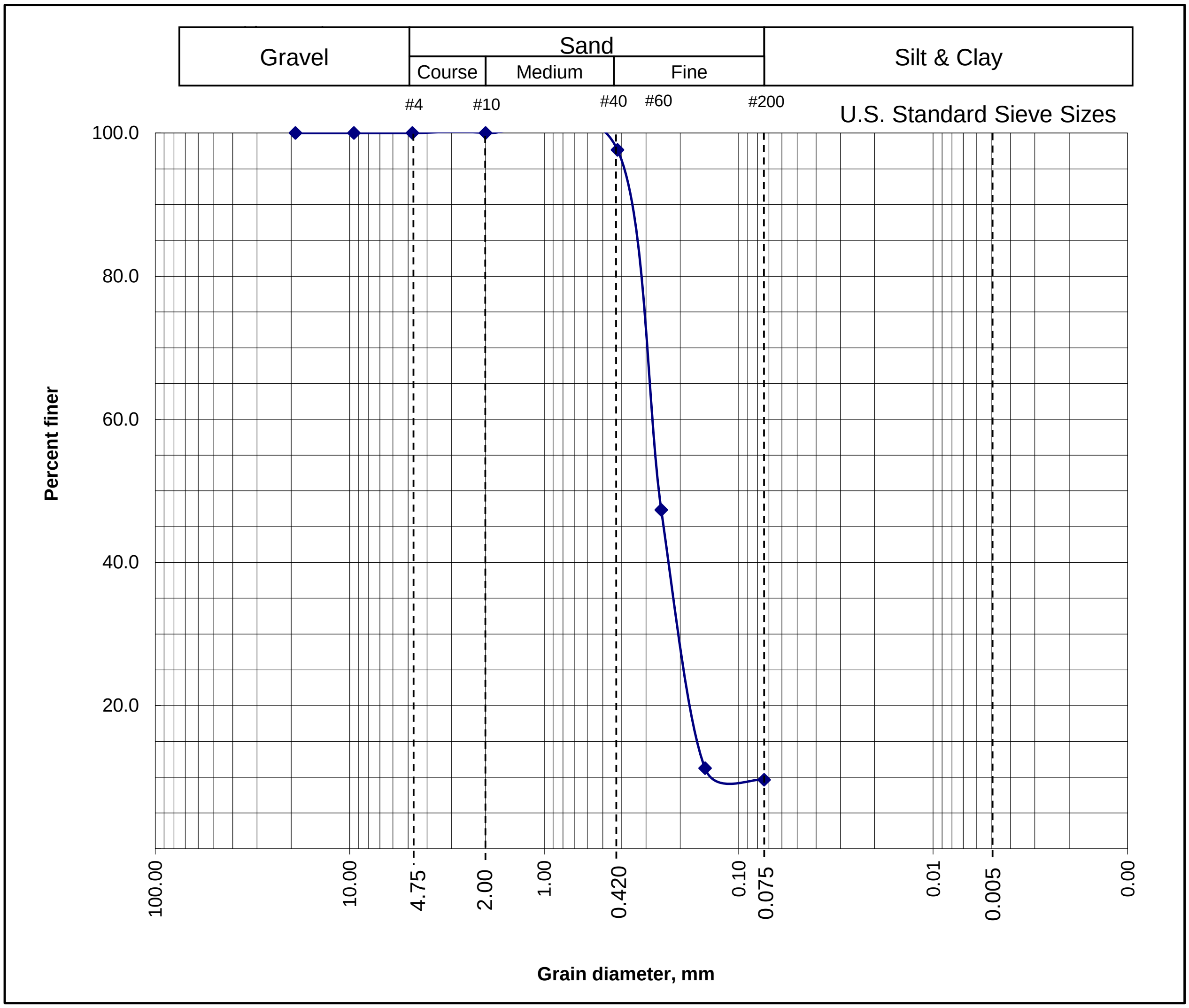
Client: Bandon Beach Ventures, LLC
Project: Beach Loop Rd.
Job No: 02-6151-02
Date Tested: 9/12/2022
Date Sampled: 9/9/2022
Description of Soil: Sand, with silt and clay
Sample Location: TP-5/S-2
Depth of Sample: 5.5' - 6.0'



Tested by: Ken Perry

Washed Sieve Analysis
(ASTM-D1140)

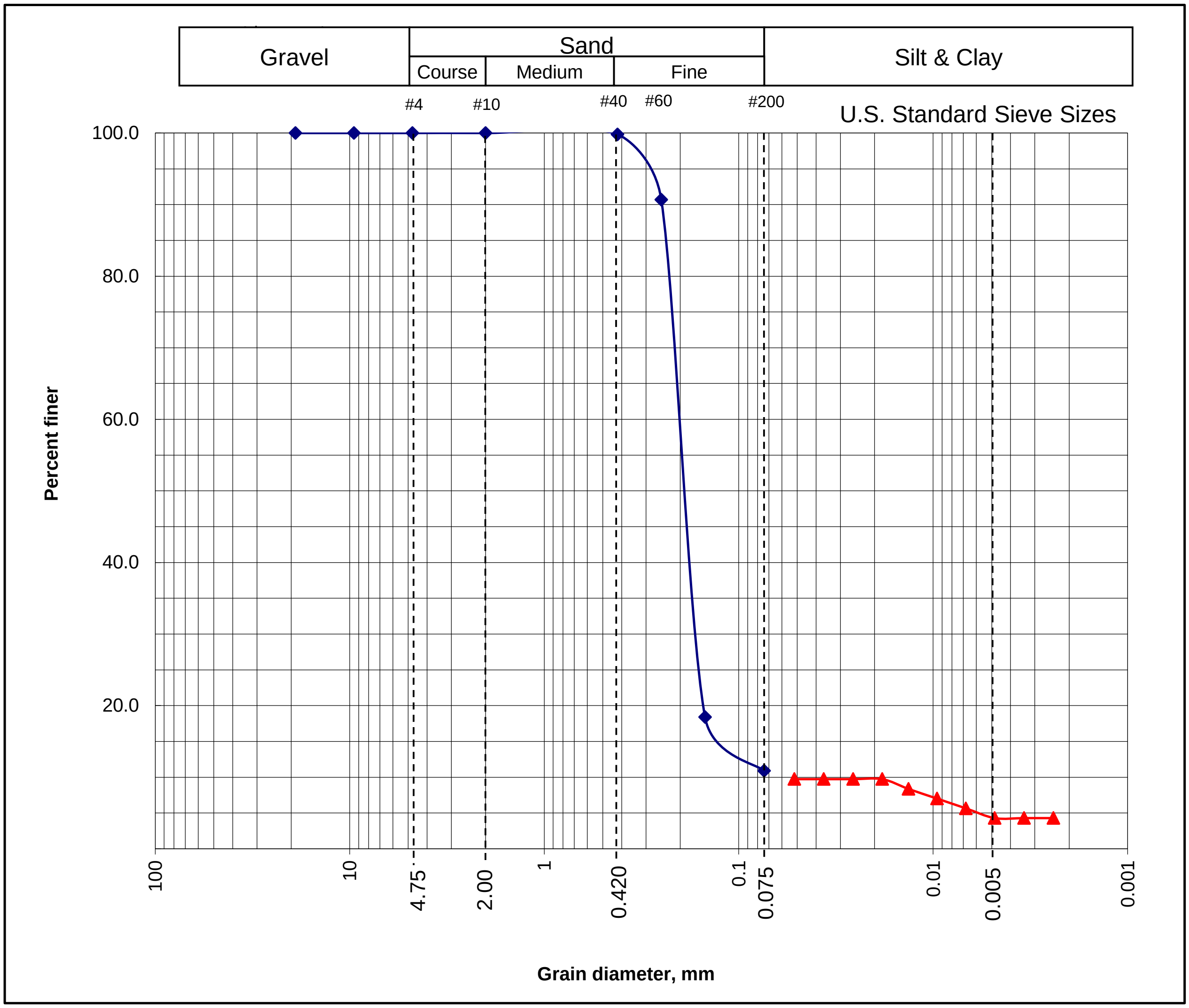
Client:	Bandon Beach Ventures, LLC
Project:	Beach Loop Rd.
Job No:	02-6151-02
Date Tested:	9/12/2022
Date Sampled:	9/9/2022
Description of Soil:	Sand, trace silt and clay
Sample Location:	TP-6/S-1
Depth of Sample:	2.0' - 3.5'



Tested by: Ken Perry

Washed Sieve and Hydrometer Analysis
(ASTM D1140 and ASTM D7928)

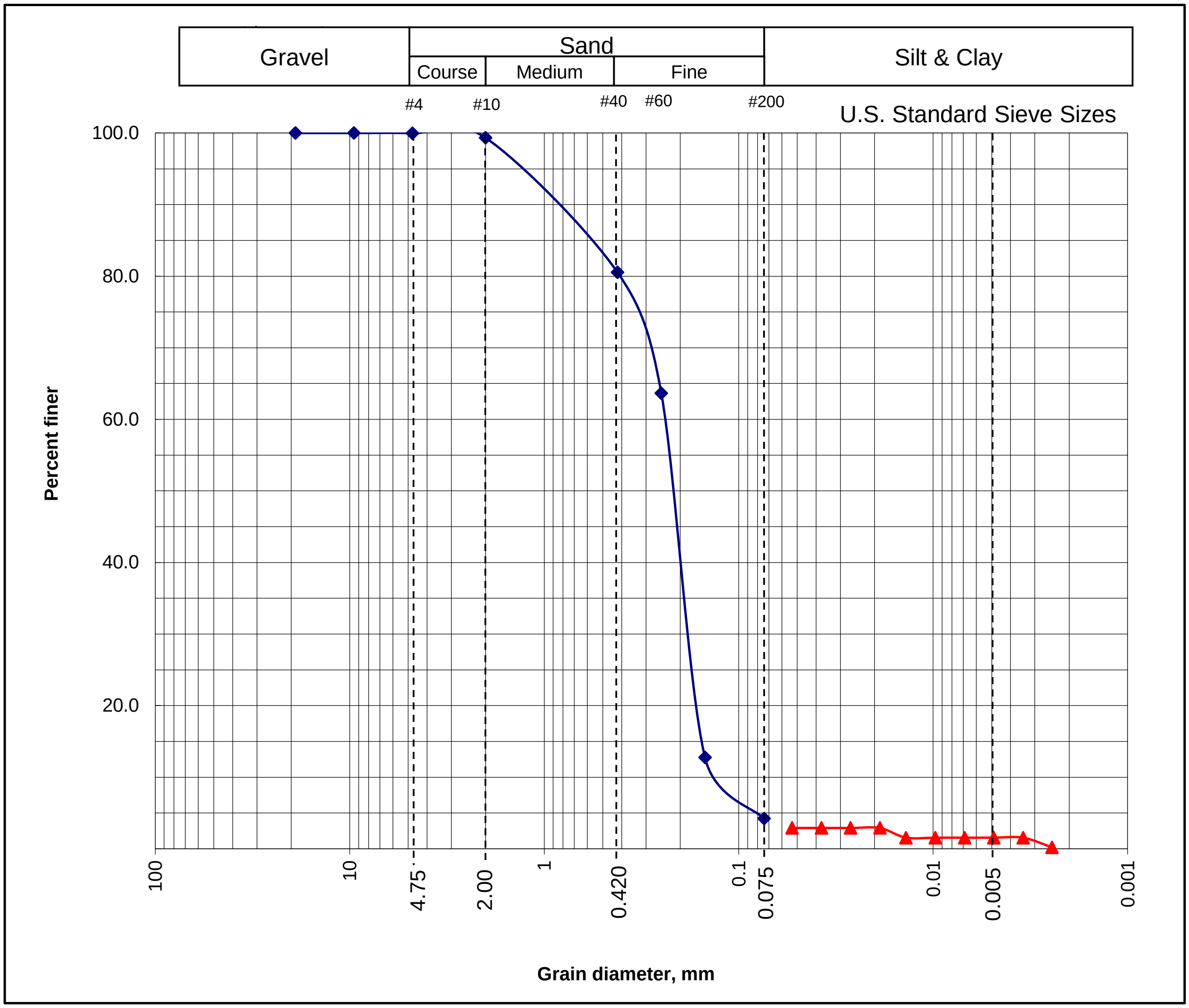
Client: Bandon Beach Ventures, LLC
Project: Beach Loop Rd.
Job No: 02-6151-02
Date Tested: 9/1/2022
Date Sampled: 8/29/2022
Description of Soil: Sand, trace silt and clay
Boring No / Sample No: B-1/S-4
Depth of Sample: 10.0' - 11.5'



Tested by: Ken Perry

Washed Sieve and Hydrometer Analysis
(ASTM D1140 and ASTM D7928)

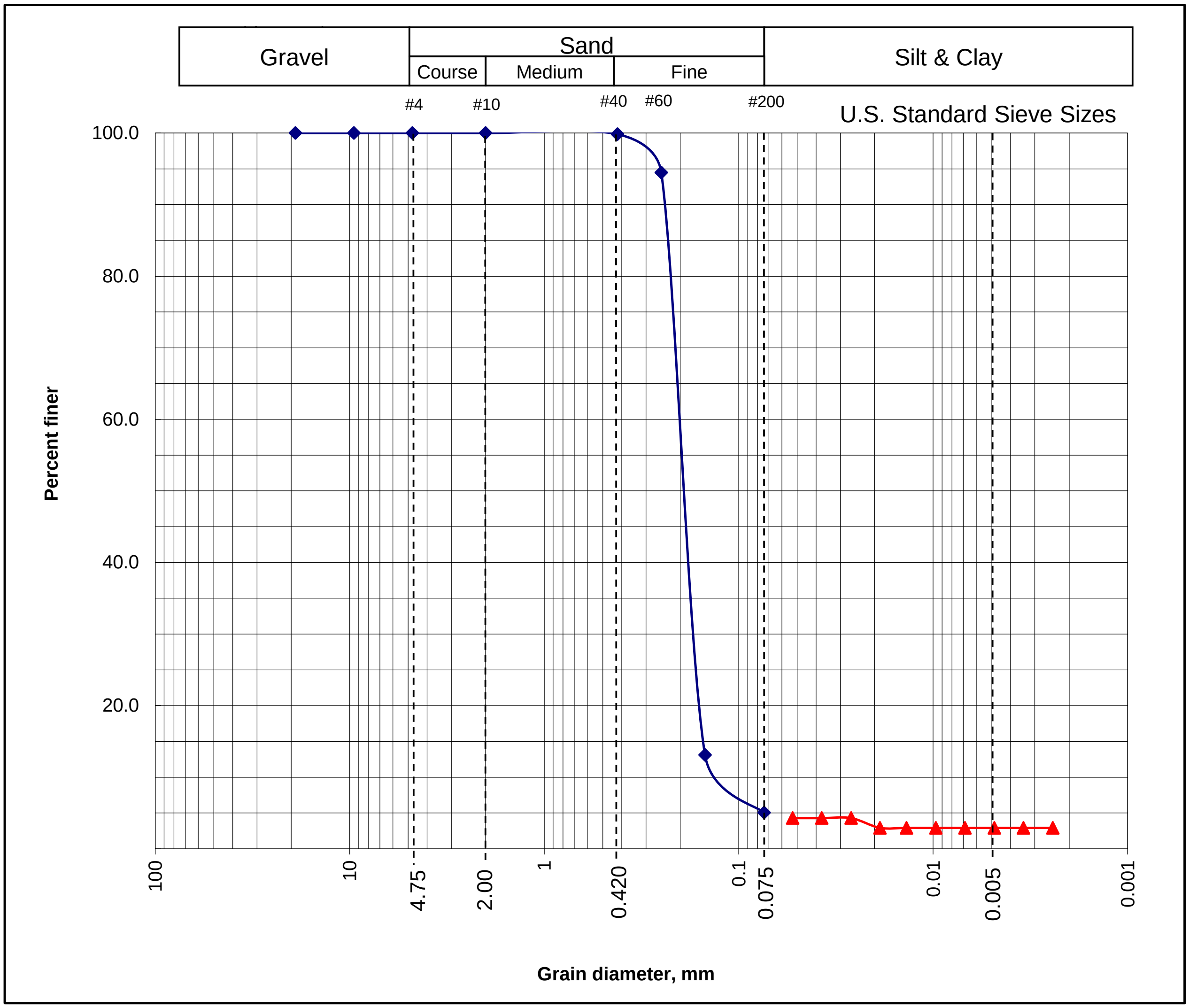
Client: Bandon Beach Ventures, LLC
Project: Beach Loop Rd.
Job No: 02-6151-02
Date Tested: 9/1/2022
Date Sampled: 8/29/2022
Description of Soil: Sand
Boring No / Sample No: B-3/S-2
Depth of Sample: 5.0' - 6.5'



Tested by: Ken Perry

Washed Sieve and Hydrometer Analysis
(ASTM D1140 and ASTM D7928)

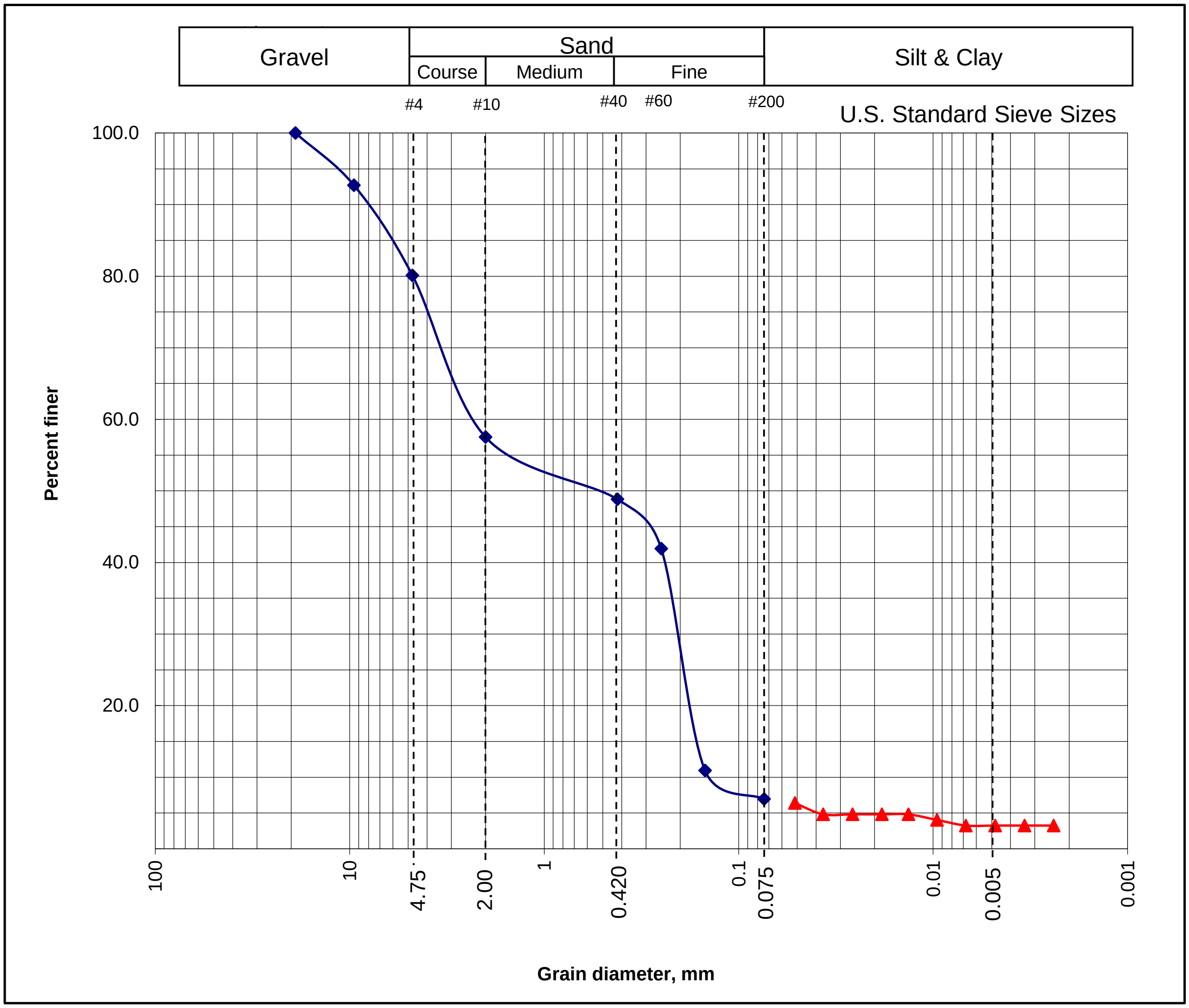
Client:	Bandon Beach Ventures, LLC
Project:	Beach Loop Rd.
Job No:	6151-02
Date Tested:	9/1/2022
Date Sampled:	8/29/2022
Description of Soil:	Sand
Boring No / Sample No:	B-3/S-4
Depth of Sample:	10.0' - 11.5'



Tested by: Ken Perry

Washed Sieve and Hydrometer Analysis
(ASTM D1140 and ASTM D7928)

Client: Bandon Beach Ventures, LLC
Project: Beach Loop Rd.
Job No: 02-6151-02
Date Tested: 9/1/2022
Date Sampled: 8/29/2022
Description of Soil: gravelly Sand
Boring No / Sample No: B-3/S-6
Depth of Sample: 15.0' - 16.5'



Tested by: Ken Perry