PJC & Associates, Inc.
Consulting Engineers & Geologists

April 11, 2023 Job No. 11223.01

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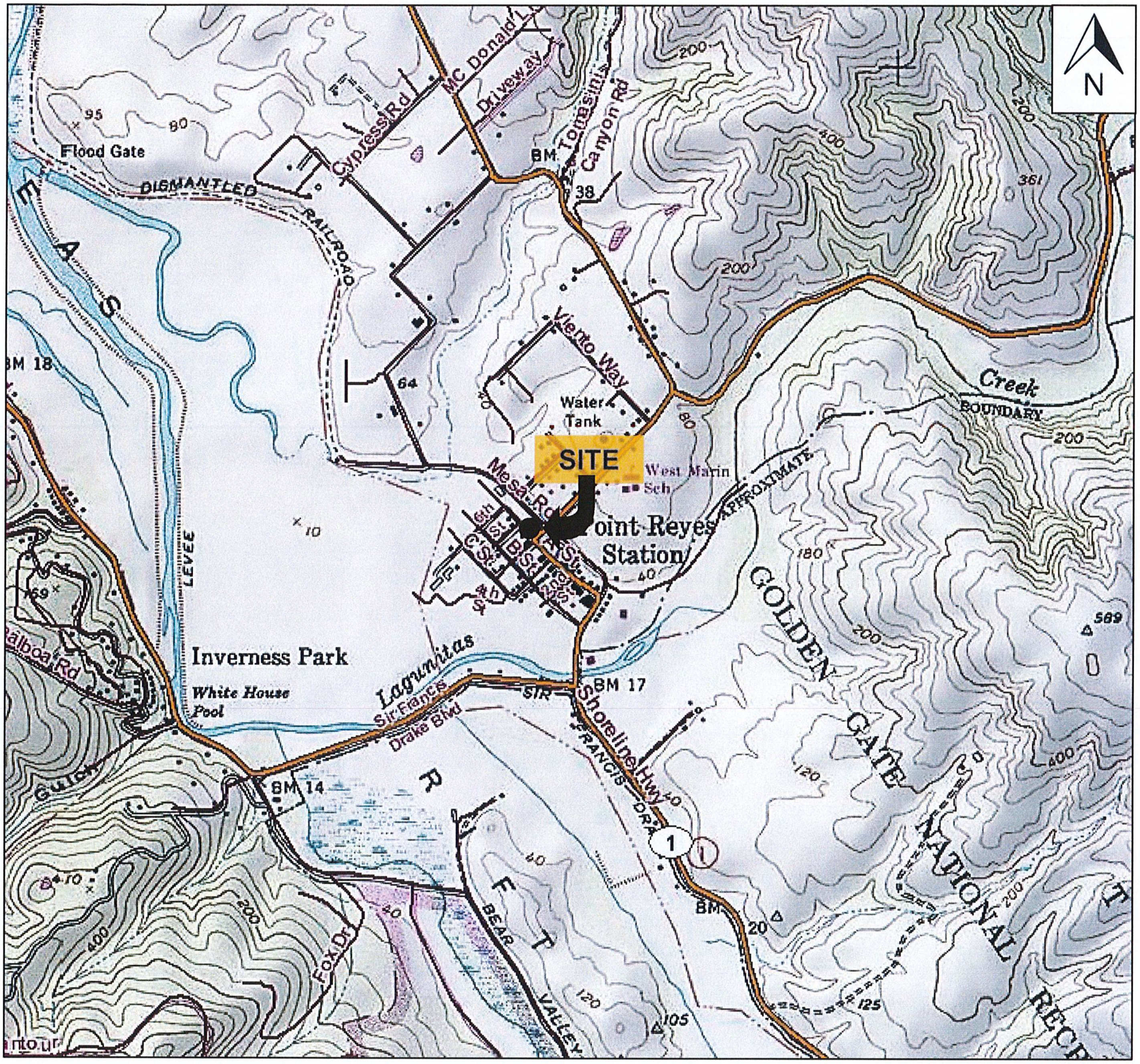
Subject: Geotechnical Investigation
Proposed Storage Warehouse Conversion
11401 Highway 1
Point Reyes Station, California

Dear Julie:

PJC and Associates, Inc. (PJC) is pleased to submit this report which presents the results of our geotechnical investigation for the proposed storage warehouse conversion located at 11401 Highway 1 in Point Reyes Station, California. The approximate location of the site is shown on the Site Location Map, Plate 1. The site corresponds to latitudinal and longitudinal coordinates of 38.0691° north and 122.8070° west, according to field GPS measurements. Our services were completed in accordance with our agreement for geotechnical engineering services, dated January 13, 2023 and your authorization to proceed with the work. This report presents our opinions and recommendations regarding the geotechnical engineering aspects of the design and construction of the proposed project. Based on the results of this study, it is our opinion that the project site can be developed from a geotechnical engineering standpoint provided the recommendations presented herein are incorporated in the design and carried out through construction.

1. PROJECT DESCRIPTION

Based on information provided and a site plan prepared by Trans Tech Consultants, latest revision dated December 30, 2023, it is our understanding that the project will consist of converting an existing storage warehouse into an apartment building and convenience store. The warehouse is located adjacent to an existing gas station at the property. Work for the project will include modifications to the interior elements of the building. It is planned to demolish the existing interior footings and concrete slab-on-grade of the structure and constructing a new slab-on-grade floor. The exterior wall footings of the building will remain as-is and supported on the existing foundation. The new apartment and store will be a one-story structure and of wood or steel frame construction. It is our understanding that the new concrete slab-on-grade floor will consist of a



SCALE: 1:24,000

REFERENCE: USGS INVERNESS, CALIFORNIA 7.5 MINUTE QUADRANGLE, DATED 1976.

PJC & Associates, Inc. Consulting Engineers & Geologists	SITE LOCATION MAP PROPOSED STORAGE WAREHOUSE CONVERSION 11401 HIGHWAY 1 POINT REYES STATION, CALIFORNIA	PLATE 1
	Proj. No: 11223.01 Date: 2/2023 App'd by: PJC	

structural slab-on-grade. There is an existing retaining wall at the west side of the building that is planned to be demolished and re-built. As part of the project, new landscaping and exterior flatwork, including four electric-vehicle charging stations, will be constructed. We anticipate that the structure will be serviced by the existing site utilities.

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

Grading plans and finish floor elevations were not available at the time of this report. We anticipate that construction will mainly be confined to the existing building envelope and site grading, if any, will be minimal with cuts and fills of two feet and less to achieve the finish pad grades and provided adequate gradients for site drainage. We anticipate that retaining walls may be required for the project.

2. SCOPE OF SERVICES

The purpose of this study is to provide geotechnical criteria for the design and construction of the proposed project as described above. Specifically, the scope of our services included the following:

a. Observing the drilling of four exploratory boreholes to depths of 10.0 and 31.5 feet below the existing ground surface to observe the soil and groundwater conditions underlying the site. The site is located within a low liquefaction zone. However, a deeper borehole was performed to evaluate the liquefaction potential at the site. Our project geologist was on site to log the materials encountered in the boreholes and to obtain representative samples for visual classification and laboratory testing.

b. Laboratory observation and testing of representative samples obtained during the course of our field investigation to evaluate the index and engineering properties of the subsurface soils at the site.

c. Review seismological and geologic literature on the site area, discuss site geology and seismicity, and evaluate potential geologic hazards and earthquake effects (i.e., liquefaction, ground rupture, settlement, lurching and lateral spreading, expansive soils, etc.). A liquefaction evaluation was performed for the project.

d. Perform engineering analyses to develop geotechnical recommendations for site preparation and earthwork, foundation type(s) and design criteria, lateral earth pressures, settlement, concrete slab-on-grade recommendations, surface and subsurface drainage control and construction considerations.

e. Preparation of this report summarizing our work on this project

3. SITE CONDITIONS

a. General. The project site is located in downtown Point Reyes Station in a fully developed commercial and residential area. At the time of our field investigation, the site was occupied by an existing warehouse and gas station. The site is bounded by a kayak business (on the undeveloped portion of site) to the northwest, Mesa Road to the northeast, Highway 1 to the southeast, and A Street to the southwest of the site.

b. Topography and Drainage. The site is located at the base of moderately to gently sloping foothills which descend towards Point Reyes Station. The site is situated on very gently sloping to nearly level terrain. According to the USGS Inverness, California 7.5 minute Quadrangle, the site lies at an approximate elevation of 37 feet above mean sea level.

Site drainage is provided by surface infiltration and sheet flow. Run-off from the site is channeled into city-maintained storm water systems. Local and regional drainage extends northwest towards Tomales Bay.

4. GEOLOGIC SETTING

a. General. The site is located in the Coast Ranges Geomorphic Province of California. This province is characterized by northwest trending topographic and geologic features, and includes many separate ranges, coalescing mountain masses and several major structural valleys. The province is bounded on the east by the Great Valley and on the west by the Pacific Ocean. It extends north into Oregon and south to the Transverse Ranges in Ventura County.

The structure of the northern Coast Ranges region is extremely complex due to continuous tectonic deformation imposed over a long period of time. The initial tectonic episode in the northern Coast Ranges was a result of plate convergence, which is believed to have begun during the late Jurassic period. This process involved eastward thrusting of oceanic crust beneath the continental crust (Klamath Mountains and Sierra Nevada) and the scraping off of materials that are now accreted to the continent (northern Coast Ranges). East-dipping thrust and reverse faults were believed to be the dominant structures formed.

Right lateral, strike slip deformation was superimposed on the earlier structures beginning mid-Cenozoic time, and has progressed northward to the vicinity of Cape Mendocino in Southern Humboldt County. Thus, the principal structures south of Cape Mendocino are northwest trending, nearly vertical faults of the San Andreas system.

b. Local Geology. According to a regional geologic map prepared by the USGS, the site is underlain by Quaternary older alluvium deposits (Qoal). Our subsurface exploration confirmed that the project site is underlain by alluvial deposits. These deposits likely extend to great depths below the project site.

5. FAULTING

Geologic structures in the region are primarily controlled by northwest trending faults. The site is not located in a State of California Alquist-Priolo Earthquake Fault Zone. However, the site is located in very close proximity (approx. 650 feet) to the active San Andreas fault and Alquist-Priolo Earthquake Fault Zone. According to the USGS National Seismic Hazard Map (2008), the three closest known active faults to the site are the San Andreas, Point Reyes, and Rodgers Creek faults. Table 1 outlines the nearest known active faults, their distance and direction from the project site, and their associated maximum moment magnitudes.

TABLE 1
CLOSEST KNOWN ACTIVE FAULTS

Fault Name	Direction	Distance (Miles)	Maximum Earthquake (Moment Magnitude)
San Andreas	southwest	0.52	8.05
Point Reyes	southwest	8.77	6.90
Rodgers Creek	northeast	18.15	7.33

Reference – USGS 2008 National Seismic Hazard Maps.

6. SEISMICITY

As mentioned in the previous section, the site is located approximately 650 feet from the San Andreas active fault zone. Most historic earthquakes originating on the San Andreas Fault in the northern California area have been relatively small. However, the earthquake occurring in 1906 had its epicenter near the town of Olema which is located southeast of the site. Movement along this segment resulted in up to 21 feet of reported right-lateral displacement.

During the lifetime of the proposed project, it is possible that future damaging earthquakes could occur on any one of the previously discussed faults, most notably the San Andreas Fault. In general, the intensity of ground shaking at the site will depend on the distance to the causative earthquake epicenter, the magnitude of the shock, the response characteristics of the underlying earth materials, and the quality of construction.

7. SUBSURFACE CONDITIONS

a. Exploration and Soils. The subsurface conditions were explored by drilling four exploratory boreholes (BH-1 through BH-4) at the subject property. The boreholes were drilled to depths of 10.0 and 31.5 feet below the existing ground surface. The approximate borehole locations are shown on the Borehole Location Plan, Plate 2. The boreholes were drilled to collect soil samples of the underlying strata for visual examination and laboratory testing and to evaluate liquefaction potential at the site. The borehole and sampling procedures and descriptive borehole logs are included in Appendix A. The laboratory procedures are included in Appendix B.

Our boreholes generally encountered artificial fill underlain by alluvial type soil deposits which extended to the maximum depths explored. The fill consisted of asphalt, aggregate base rock, clayey sands and sandy clays which extended 1.0 to 2.5 feet below existing grade. The clayey sands appeared slightly moist to moist, moderately compacted and fine to coarse-grained. The sandy clay appeared slightly moist to moist, moderately compacted and exhibited low to medium plasticity characteristics. Underlying the fill layer, alluvial type soil deposits consisting of silty clays and clayey sands were encountered which extended to the maximum depths explored. The silty clays appeared moist to saturated, soft to hard, and exhibited low to high plasticity characteristics. The clayey sand appeared moist to saturated, medium dense to dense, and fine to coarse-grained.

b. Groundwater. The phreatic groundwater table was observed at 5.5 feet below grade in BH-1 through BH-3, and 7.0 feet below grade in BH-4. Groundwater levels typically rise and fall by several feet due to variations in seasonal rainfall intensity, duration, and other factors. The groundwater may rise and fall by several feet throughout the year. Provided the project does not include significantly deep excavations, we do not anticipate the presence of groundwater will significantly impact the project.

9. GEOLOGIC HAZARDS AND SEISMIC CONSIDERATIONS

The site is located within a region subject to a high level of seismic activity. Therefore, the site could experience strong seismic ground shaking during the lifetime of the project. The following discussion reflects the possible earthquake effects which could result in damage to the proposed project.

a. <u>Fault Rupture</u>. Rupture of the ground surface is expected to occur along known active fault traces. No evidence of existing faults or previous ground displacement at the site due to fault movement is indicated in the geologic literature or field exploration. However, the site is in close proximity to active faults in the San Andreas fault zone. The current theory is that movement along a fault follows the trace of the most recent break. This is not always the case. Due to the close proximity to active faults, it should be considered that the risk of ground rupture at the site is moderate.

b. <u>Ground Shaking</u>. The site has been subjected in the past to ground shaking by earthquakes on the active fault systems that traverse the region. Based on this data and the anticipated life expectancy of the project, it is judged that there is a high potential that the site will be subjected to very strong seismic shaking. The severity of the shaking depends on many complex factors. Among these factors are the moment magnitude, focal depth, distance from the causative fault, source mechanism, duration of shaking, type of surficial deposits, and type and quality of building construction.

c. <u>Liquefaction</u>. Based on our review of the USGS Liquefaction Susceptibility Map, the site is underlain by soils which are considered to have low liquefaction potential. Liquefaction is a seismic hazard that occurs in saturated, low density, predominantly granular soils encountered below the phreatic groundwater table. In general, these loose materials experience a rapid, temporary loss in shear strength due to an increase in pore water pressure in response to strong earthquake ground shaking. Upon dissipation of pore water pressures following shaking, there is reduction in the void ratio of the impacted soil particles that can cause differential and erratic ground settlement. Low density, fine-grained sandy soils below the phreatic groundwater elevation are most susceptible to liquefaction. However, case studies have shown that soft silts, low plasticity clays and loose gravels with limited drainage paths are also susceptible to liquefaction. Bedrock materials and plastic clayey soils with a liquid limit (LL) greater than 32 are generally not known to be prone to liquefaction.

The occurrence of this phenomenon is dependent on many complex factors including the intensity and duration of ground shaking, groundwater elevation at time of shaking, particle size distribution, consistency/relative density of the soils, overburden stress, age of deposit, and many other factors.

In order to evaluate liquefaction potential at the site, a deeper borehole was drilled to a depth of 31.5 feet below the existing ground surface. The borehole generally encountered interbedded low to high plasticity clays and dense clayey sands which extended to the maximum depths explored. The granular strata encountered were saturated. However, they exhibited high relative densities and high fines clay content. These soils are not considered prone to liquefaction. Therefore, we judge the overall risk of liquefaction at the site to be low.

d. <u>Lateral Spreading and Lurching</u> Lateral spreading is normally induced by vibration of near-horizontal alluvial soil layers adjacent to an exposed face. Lurching is an action, which produces cracks or fissures parallel to streams or banks when the earthquake motion is at right angles to them. There are no exposed faces or creek embankments adjacent to the site. Therefore, we judge that the potential for lateral spreading and lurching at the site is low.

e. <u>Expansive Soils</u>. Based on our field observations and laboratory testing (PI = 11 & 12), the near surface site soils generally exhibit low plasticity characteristics and are judged to have low expansion potential. Based on our experience and observations of the site conditions, expansive soils are not a concern for the project.

10. CONCLUSIONS

Based on the results of our investigation, it is our professional opinion that the project is feasible from a geotechnical engineering standpoint provided the recommendations contained in this report are incorporated into the design and carried out through construction of the project. The primary geotechnical concerns in design and construction of the project are as follows:

1. The presence of artificial fill of unknown source and variable density.

2. Control of surface and subsurface drainage across the site.

Our field work encountered artificial fill extending up to two and one-half feet below the existing grade. The fill is of an unknown source and is of variable density. Although the fill has been present for some time, when exposed to loads from new foundations, slabs or new fills, this material could be prone to intolerable differential settlement. This can cause damage and cracking to structural and concrete elements if constructed on these materials in their existing state.

The boreholes generally encountered interbedded low to high plasticity clays and medium dense to dense clayey sands, which extended to the maximum depths explored. These soils exhibited relative high densities and are not considered to be prone to liquefaction.

It is crucial that all final grades be provided with positive gradients away from all foundations to provide rapid removal of surface water runoff to an adequate discharge point. No ponding of water should be allowed adjacent to building foundations or slabs. Care must be taken so that discharges from the roof gutter and downspout systems are not allowed to infiltrate the subsurface near structures.

11. RECOMMENDATIONS

If new interior foundations are required we recommend the new foundations bear on firm native soils. Due to the varying depths of firm native soils across the site, minimum footing depths of 24 to 36 inches should be anticipated.

New interior slabs-on-grade should be structurally designed. Exterior flatwork should be supported on a uniform layer of compacted engineered fill as recommended by the geotechnical engineer in the field during grading. The engineered fill should be placed in accordance with the recommendations of this report. The engineered fill should extend laterally five feet beyond the edges of foundations, if possible, and three feet beyond the edges of exterior flatwork.

The following sections present geotechnical recommendations and criteria for design and construction of the project.

12. EARTHWORK AND GRADING

a. <u>Demolition and Stripping</u>. Existing structures to be removed should be demolished and removed off site. For areas to be graded, we recommend that structural areas be stripped of surface vegetation, asphalt, old fill, debris, roots and the upper few inches of soil containing organic matter. These materials should be moved off site. Some of them, if suitable, could be stockpiled for later use in landscape areas. Where underground utilities pass through the site, we recommend that these utilities be removed in their entirety or rerouted where they exist outside an imaginary plane sloped two horizontal to one vertical (2H:1V) from the outside bottom edge of the nearest foundation element. Any existing wells, septic systems and leach fields should be abandoned and plugged according to regulations set forth by the Marin County Health Department. Voids left from the removal of utilities or other obstructions should be replaced with compacted engineered fill placed in conformance with the earthwork section of this report and should be observed by the geotechnical engineer in the field during grading. Loosely backfilled voids generated from demolition will settle excessively over time and potentially cause damage to structures constructed above them.

b. <u>Excavation and Compaction</u>. Following site stripping, excavation should be performed to achieve finished grade or prepare areas to receive fill.

The interior slab-on-grade should be structurally designed. However, the subgrade exposed following demolition should be scarified to at least eight inches deep, moisture-conditioned to within two percent of the optimum moisture content and compacted to a minimum of 90 percent of the material's maximum dry density. If loose and weak soils are encountered, it may be necessary to sub-excavate the material and recompact. This should be evaluated by and approved by the geotechnical engineer in the field during the grading operation.

Exterior flatwork should be underlain by a layer of compacted engineered fill. Final subexcavation depths in flatwork areas should be evaluated and approved by the Geotechnical Engineer in the field during grading. Subexcavations should extend at least five feet beyond the limits of perimeter foundation areas, where possible, and at least three feet beyond exterior flatwork edges. If obstructions are encountered within subexcavation areas, the geotechnical engineer should be consulted to provide specific recommendations for placement.

The exposed surface to receive fill or slabs-on-grade should be scarified to a depth of eight inches, moisture conditioned to within two percent of optimum moisture content and compacted to a minimum of 90 percent of the material's maximum dry density, as determined by the ASTM D 1557-12 laboratory compaction test procedures. The site soils are generally considered acceptable for use as engineered fill, if approved by the geotechnical engineer in the field during grading. Additional testing may be required during grading to further evaluate the suitability of the existing soil for use as engineered fill. The fill material should be spread in eight-inch-thick loose lifts, moisture conditioned to within two percent of optimum moisture content and compacted to at least 90 percent of the material's maximum dry density. The thickness of the fill should not vary by more than two feet across non-structural slabs-on-grade. Care in equipment selection must be implemented when compacting soil against existing walls and foundations. We recommend that hand held jumping jacks be utilized to compact fill within five feet of existing retaining walls and foundations.

Any imported fill should be evaluated and approved by the geotechnical engineer before importation. It is recommended that any import fill to be used on site should be of a low to non-expansive nature and should meet the following criteria:

Plasticity Index	less than 12
Liquid Limit	less than 38
Percent Soil Passing #200 Sieve	between 15% and 40%
Maximum Aggregate Size	4 inches

A representative of PJC should observe all site preparation and fill placement. It is important that during the stripping, excavation, grading, and scarification processes, a representative of our firm is present to observe whether any undesirable material is encountered in the construction area. If unforeseen soil conditions are encountered, deeper subexcavation depths may be necessary.

Generally, grading is most economically performed during the summer months when the onsite soils are usually dry of their optimum moisture content. Delays should be anticipated in site grading performed during the rainy season or early spring due to excessive moisture in onsite soils. Special and relatively expensive construction procedures should be anticipated if grading must be completed during the winter and early spring.

13. FOUNDATIONS: SPREAD FOOTINGS

We recommend the new foundations consist of spread footings that extend into compacted engineered fill or firm native soils.

a. Vertical Loads. The anticipated structural loads may be adequately supported by spread footings extending a minimum of 12 inches into firm native soils or compacted engineered fill. Footings should be approved by the geotechnical engineer before reinforcing steel is placed. All footings should be reinforced as determined by the project structural engineer. The recommended soil bearing pressures, depth of embedment and minimum widths of spread footings are presented in Table 2. The bearing values provided have been calculated assuming that all footings uniformly bear on firm native soils or compacted engineered fill, as determined by the geotechnical engineer on site during construction.

TABLE 2
FOUNDATION DESIGN CRITERIA

Footing Type	Bearing Pressure (psf)*	Minimum Depth (in)**	Minimum Width (in)
Continuous wall	2,000	12	12
Isolated Column	2,500	12	18

* Dead plus live load.
** Into firm native soils or compacted engineered fill. Footing depths of three feet maybe necessary to achieve the recommended depths.

The allowable bearing pressures are net values. The weight of the foundation and backfill over the foundation may be neglected when computing dead loads. Allowable bearing pressures may be increased by one-third for transient applications such as wind and seismic loads.

b. Lateral Loads. Resistance to lateral forces may be computed by using friction and passive pressure. A friction factor of 0.30 is considered appropriate between the bottom of the concrete structures and the bearing soils. A passive pressure of 300 pounds per square foot per foot of depth (psf/ft) is recommended. Unless restrained at the surface, the upper six inches should be neglected for passive resistance due to soil disturbance and desiccation. Footing concrete should be placed neat against undisturbed native soils. Footing excavations should not be allowed to dry before placing concrete. If shrinkage cracks appear in the footing excavations, the soil should be thoroughly moistened prior to concrete placement.

c. Settlement. Total settlement of individual foundations will vary depending on the width of the foundation and the actual load supported. Foundation settlements have been estimated based on the foundation loads and bearing values provided. Maximum settlements of shallow foundations designed and constructed in accordance with the preceding recommendations are estimated to be one inch or less. Differential settlement between similarly loaded, adjacent footings is expected to be one-half inch or less. The majority of the settlement is expected to occur during construction and placement of dead loads, and occur within a few weeks upon application of the loads.

14. STRUCTURAL CONCRETE SLABS-ON-GRADE

Due to the presence of fill soils, the interior slab-on-grade should be structurally designed. The structural slab should be designed for a center span at least six feet and an edge span of at least three feet and be capable of supporting full structural loading. Structural slabs should be at least eight inches thick and reinforced as determined by the project structural engineer. The slab subgrade should be firm and unyielding and maintained within two percent of optimum moisture content at all times. The slab subgrade should not be allowed to dry.

All slabs should be supported on at least four inches of clean gravel or crushed rock to provide a capillary break and provide uniform support for the slab. The rock should be graded so that 100 percent passes the one inch sieve and no more than five percent passes the No. 4 sieve.

A 15 mil vapor barrier should be placed below the slab. We recommend that slabs be designed and reinforced as determined by the project structural engineer. Special care should be taken to insure that reinforcement is placed in the slab mid-height. The gravel should be moistened prior to placing concrete.

Special precautions must be taken during the placement and curing of concrete slabs-on-grade. Excessive slump (high water-cement ratio) of the concrete and/or improper curing procedures and ad mixtures used during either hot or cold weather conditions will lead to excessive shrinkage, cracking or curling of the slabs. High water-cement ratios and/or improper curing also greatly increases water vapor transmission through the concrete. Concrete placement and curing operations should be performed in accordance with the American Concrete Institute (ACI) manual.

15. EXTERIOR NON-STRUCTURAL CONCRETE SLABS-ON-GRADE

New non-structural exterior slabs-on-grade may be used provided they are underlain by a uniform layer of compacted engineered fill. The actual thickness of the fill should be determined by the geotechnical engineer in the field during grading. The engineered fill should extend at least five feet beyond foundations, where possible, and three feet beyond exterior flatwork edges.

All slab subgrades should be moisture conditioned and compacted to produce a firm and unyielding subgrade. The slab subgrade should not be allowed to dry. Non-structural slabs should be at least five inches thick and underlain with a capillary moisture break consisting of at least four inches of clean, free-draining crushed rock or gravel. The rock should be graded so that 100 percent passes the one-inch sieve and no more than five percent passes the No. 4 sieve.

For slabs-on-grade with moisture sensitive surfacing, we recommend that a vapor barrier at least 15 mils in thickness be placed over the rock to prevent migration of moisture vapor through the concrete slabs.

We recommend that slabs be designed and reinforced as determined by the project structural engineer. Special care should be taken to insure that reinforcement is placed at the slab mid-height. The gravel should be moistened prior to placing concrete. Exterior slabs should not be attached to foundations. Control joints should be provided to induce and control cracking.

Special precautions must be taken during the placement and curing of concrete slabs-on-grade. Excessive slump (high water-cement ratio) of the concrete and/or improper curing procedures and ad mixtures used during either hot or cold weather conditions will lead to excessive shrinkage, cracking or curling of the slabs. High water-cement ratios and/or improper curing also greatly increases water vapor transmission through the concrete. Concrete placement and curing operations should be performed in accordance with the American Concrete Institute (ACI) manual.

16. DRAINAGE

a. Surface Drainage. Drainage control design should include provisions for positive surface gradients so that surface runoff is not permitted to pond, particularly adjacent to slabs and foundations. Surface runoff should be directed away from foundations. If drainage facilities discharge onto the natural ground, adequate means should be provided to control erosion and to create sheet flow. The structure should be provided with gutters and downspouts. The downspouts should be connected to closed conduits and discharged away from the structure.

17. UTILITY TRENCHES

Shallow excavations for utility trenches can be readily made with either a backhoe or trencher. Larger earth moving equipment should be used for deeper excavations. We expect the walls of trenches less than five feet deep, excavated into engineered fill or native soils, to remain in a near-vertical configuration during construction provided no equipment or excavated spoil surcharges are located near the top of the excavation. Where trenches extend deeper than five feet, the excavation may become unstable. Furthermore, groundwater maybe encountered. All trenches, regardless of depth, should be evaluated to monitor stability prior to personnel entering the trenches. Shoring or sloping of any deep trench wall may be necessary to protect personnel and to provide stability. All trenches should conform to the current CAL-OSHA requirements for worker safety.

Utility trenches may be backfilled with native or imported soils placed, moisture conditioned, and compacted in conformance with Table 3. Jetting of soils should not be allowed.

TABLE 3
SUMMARY OF TRENCH BACKFILL RECOMMENDATIONS

Area	Compaction Recommendations*
Trench Backfill** (Onsite Native Material)	Placed in loose lifts, moisture conditioned to within two percent of the optimum moisture content, and compacted to a minimum of 90 percent relative compaction.
Trench Backfill** (Low to Non-Expansive Import)	Placed in loose lifts, moisture conditioned to within two percent of the optimum moisture content, and compacted to a minimum of 90 percent relative compaction.
Loose Lift Thickness	Jumping Jack – Eight inches Excavator with Wheel – Eight to ten inches

* All compaction requirements stated in this report refer to dry density and moisture content relationships obtained through the laboratory standard described by ASTM D 1557-12.

** Depths below finished subgrade elevations

18. SEISMIC DESIGN

Based on criteria presented in the 2022 edition of the California Building Code (CBC) and ASCE (American Society of Civil Engineers) STANDARD ASCE/SEI 7-16, the following minimum criteria should be used in seismic design:

a.	Site Class:	D
b.	Mapped Acceleration Parameters:	S_S = 2.73 g S_1 = 1.09 g
c.	Spectral Response Acceleration Parameters:	S_{MS} = 2.61 g S_{M1} = 2.92 g
d.	Design Spectral Acceleration Parameters:	S_{DS} = 1.74 g S_{D1} = 1.95 g

19. RETAINING WALLS

Retaining walls may be supported on spread footings, per the recommendations presented in Section 13 of this report.

a. Static Lateral Earth Pressures. Retaining walls free to rotate on the top should be designed to resist active lateral earth pressures. If walls are restrained by rigid elements to prevent rotation or supporting compacted engineered fill, they should be designed for "at rest" lateral earth pressures.

Retaining walls should be designed to resist the following earth pressures (triangular distribution):

Active Pressure (level backfill) (15% or less)....................40 psf/ft
At Rest Pressure (level backfill) (15% or less)..................55 psf/ft
Active Pressure (sloping backfill).......................................55 psf/ft
At Rest Pressure (sloping backfill).....................................70 psf/ft

These pressures to not include external surcharge loads. If surcharge loads are anticipated, we should be consulted to provide recommendations for design.

b. Pseudostatic Force. For retaining walls taller than six feet, the horizontal pseudostatic force acting upon the retaining wall during a seismic event should be calculated from the following equation:

$P_E = 17.5 H^2$

P_E = Pseudostatic Force (lbs)
H = retained height (ft)

The location of the pseudostatic force is assumed to act at a distance of 0.33H above the base of the wall.

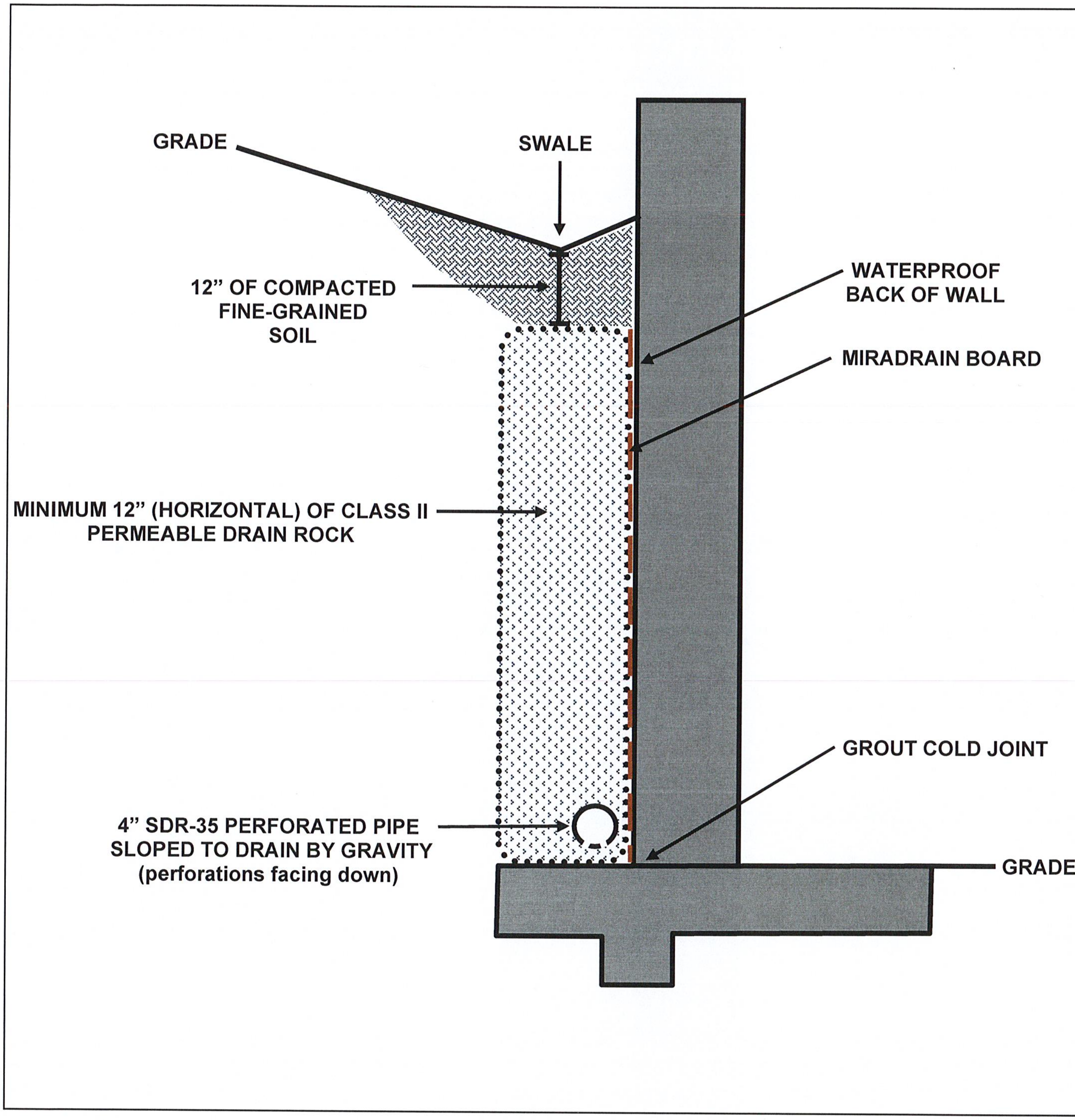
c. Drainage. We recommend that a back drain be provided behind all retaining walls or that the walls be designed for full hydrostatic pressures. The back drains should consist of four-inch diameter SDR 35 perforated pipe sloped to drain to outlets by gravity, and of clean, free-draining Class II permeable drain rock. The Class II permeable drain rock should extend 12 inches horizontally from the back face of the wall and extend from the bottom of the wall to one foot below the finished ground surface. The upper 12 inches should be backfilled with compacted fine-grained soil to exclude surface water. We recommend that the ground surface behind retaining walls be sloped to drain. Under no circumstances should surface water be diverted into retaining wall back drains. Where migration of moisture through walls would be detrimental, the walls should be waterproofed. Retaining wall backdrain detail is presented on Plate 1A.

20. LIMITATIONS

The data, information, interpretations and recommendations contained in this report are presented solely as bases and guides to the geotechnical design of the proposed storage warehouse conversion located at 11401 Highway 1 in Point Reyes Station, California. The conclusions and professional opinions presented herein were developed by PJC in accordance with generally accepted geotechnical engineering principles and practices. No warranty, either expressed or implied, is intended.

This report has not been prepared for use by parties other than the designers of the project. It may not contain sufficient information for the purposes of other parties or other uses. If any changes are made in the project as described in this report, the conclusions and recommendations contained herein should not be considered valid, unless the changes are reviewed by PJC and the conclusions and recommendations are modified or approved in writing. This report and the figures contained herein are intended for design purposes only. They are not intended to act by themselves as construction drawings or specifications.

Soil deposits may vary in type, strength, and many other important properties between points of observation and exploration. Additionally, changes can occur in groundwater and soil moisture conditions due to seasonal variations or for other reasons. Therefore, it must be recognized that we do not and cannot have complete knowledge of the subsurface conditions underlying the subject site. The criteria presented herein are based on the findings at the points of exploration



PJC & Associates, Inc.
Consulting Engineers & Geologists

RETAINING WALL BACKDRAIN DETAIL
PROPOSED STORAGE WAREHOUSE CONVERSION
11401 HIGHWAY 1
POINT REYES STATION, CALIFORNIA

PLATE
1A

Proj. No: 11223.01 Date: 3/2023 App'd by: PJC and on interpretative data, including interpolation and extrapolation of information obtained at points of observation.

21. ADDITIONAL SERVICES

Upon completion of the project plans, they should be reviewed by our firm to determine that the design is consistent with the recommendations of this report. During the course of this investigation, several assumptions were made regarding development concepts. Should our assumptions differ significantly from the final intent of the project designers, our office should be notified of the changes to assess any potential need for revised recommendations. Observation and testing services should also be provided by PJC to verify that the intent of the plans and specifications are carried out during construction; these services should include observing and testing during grading and earthwork, observing the foundation excavations, approving slab subgrades, and observing the installation of drainage facilities. These services will be performed only if PJC is provided with sufficient notice to perform the work. PJC does not accept responsibility for items we are not notified to observe.

It has been a pleasure working with you on this project. Please call if you have any questions regarding this report or if we can be of further assistance.

Sincerely,

PJC & ASSOCIATES, INC.

Patrick J. Conway
Geotechnical Engineer
GE 2303, California

REGISTERED PROFESSIONAL ENGINEER
PATRICK J. CONWAY
No. 2303
Exp. 3-31-24
GEOTECHNICAL
STATE OF CALIFORNIA

PJC/tjc

APPENDIX A
FIELD INVESTIGATION

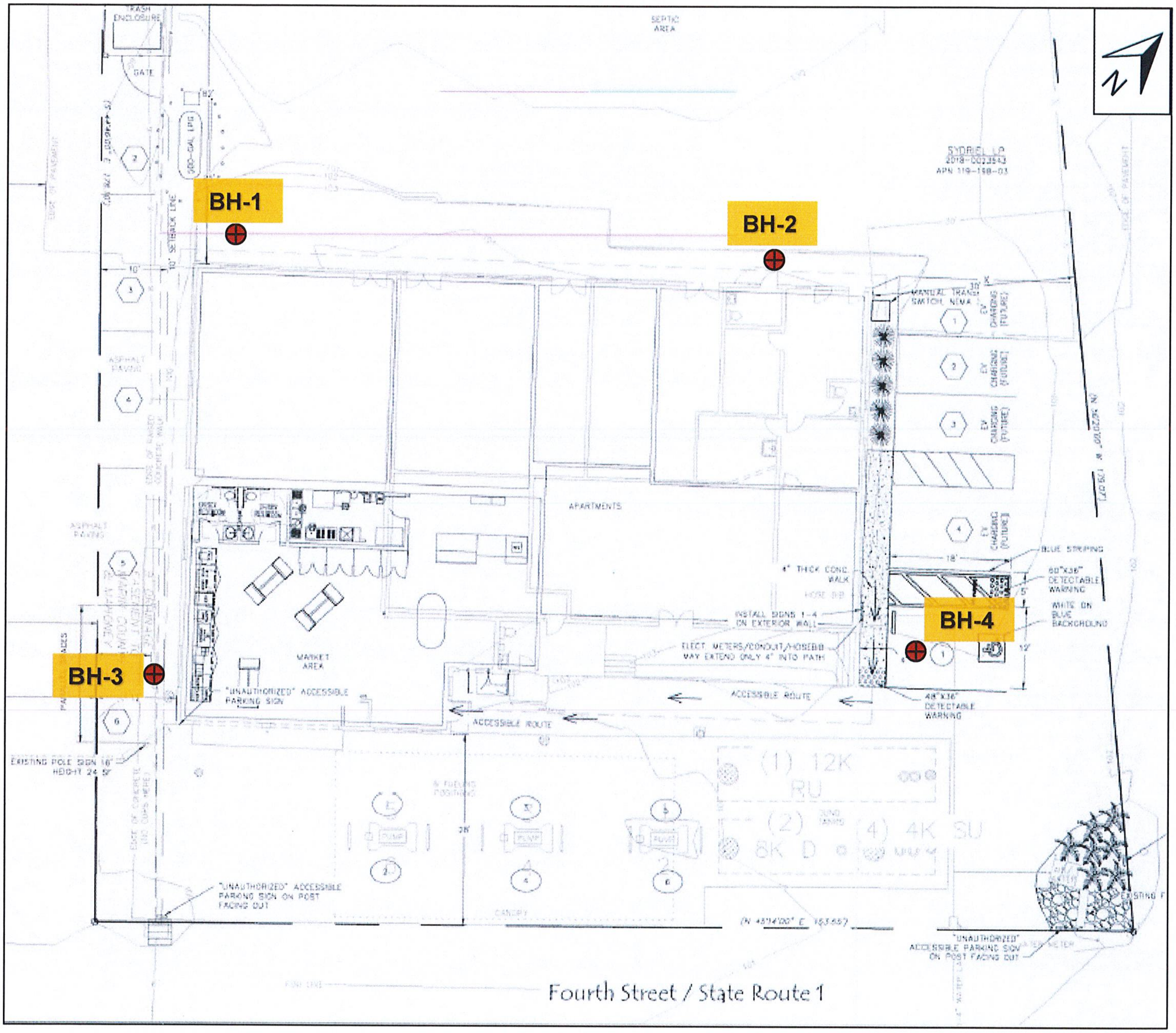
1. INTRODUCTION

The field program performed for this study consisted of drilling four exploratory boreholes at the subject site. The approximate borehole locations are shown on the Borehole Location Plan, Plate 2. Descriptive logs of the boreholes are presented in this appendix on Plates 3 through 6.

2. BOREHOLES

The boreholes were advanced using a truck-mounted B-53 drill rig equipped with 8-inch diameter hollow stem and 6-inch solid stem flight augers. The drilling was performed under the observation of our project geologist who maintained a continuous log of the soil conditions and obtained samples suitable for laboratory testing.

Relatively undisturbed and disturbed samples were obtained from the exploratory boreholes. A 2.43 in I.D. California Modified Sampler containing liners was driven into the underlying soil using a 140 pound hammer falling 30 inches. A 1.375-inch inside diameter Standard Penetration Test (SPT), without liners, was also driven into the soils. The samplers were driven to obtain an indication of the consistency and relative density of the soil and to allow visual examination of at least a portion of the soil column. Soil samples obtained with the split-spoon samplers were retained for further observation and testing. The number of blows required to drive the samplers at 6-inch increments was recorded on each borehole log, and converted to equivalent SPT blow counts for correlation with empirical data. All samples collected were labeled and transported to PJC's office for laboratory examination and testing.



NO SCALE

EXPLANATION

BOREHOLE LOCATION AND DESIGNATION

REFERENCE: SITE PLAN TITLED "INTERIOR ALTERATIONS FOR FOOD FACILITY AND RESIDENCES- REDWOOD OIL COMPANY", PREPARED BY TRANS TECH CONSULTANTS, DATED DECEMBER 30, 2022.

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BOREHOLE LOCATION PLAN
PROPOSED STORAGE WAREHOUSE CONVERSION
11401 HIGHWAY 1
POINT REYES STATION, CALIFORNIA

PLATE
2

Proj. No: 11223.01 | Date: 2/2023 | App'd by: PJC

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BORING NUMBER BH-1

CLIENT Julie Van Alyea, Redwood Oil Company **PROJECT NAME** Proposed Storage Warehouse Conversion

JOB NUMBER 11223.01 **LOCATION** 11401 Highway 1, Point Reyes Station, California

DATE STARTED 2/10/23 **COMPLETED** 2/10/23 **GROUND ELEVATION** **HOLE SIZE** 8"

DRILLING CONTRACTOR Pearson Exploration **GROUND WATER LEVELS:**

DRILLING METHOD B-53 Hollow Stem Auger with 140lb hammer ▽ **AT TIME OF DRILLING** 7.50 ft

LOGGED BY TC **CHECKED BY** PJC ▼ **AT END OF DRILLING** 5.50 ft

NOTES Caving encountered at approx 14.0 feet. **AFTER DRILLING** ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
0		0.0' - 2.5'; CLAYEY SAND (SC); moderate brown, moist, moderately compacted, fine to coarse-grained, with gravels (FILL).										
		2.5' - 4.5'; SILTY CLAY (CL); orangish brown, moist, very stiff, low plasticity, with gravels (ALLUVIUM).	MC		19	3.0	119	10				
5		4.5' - 14.5'; CLAYEY SAND (SC); tan & orangish brown, moist to saturated, medium dense to dense, fine to coarse-grained, with gravels (ALLUVIUM). ▼	MC		46		114	13				
		▽										
10			MC		42		123	13				35
15		14.5' - 31.5'; SILTY CLAY (CH); bluish gray, saturated, stiff to hard, high plasticity (ALLUVIUM).	SPT		6			30				

(Continued Next Page)

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BORING NUMBER BH-1

CLIENT Julie Van Alyea, Redwood Oil Company **PROJECT NAME** Proposed Storage Warehouse Conversion

JOB NUMBER 11223.01 **LOCATION** 11401 Highway 1, Point Reyes Station, California

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
		14.5' - 31.5'; SILTY CLAY (CH); bluish gray, saturated, stiff to hard, high plasticity (ALLUVIUM). *(continued)*										
20			MC		20	2.75	89	31				
25			MC		28	4.0	97	25				
30			MC		26	3.5	89	31				

Bottom of borehole at 31.5 feet.

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BORING NUMBER BH-2

CLIENT Julie Van Alyea, Redwood Oil Company
PROJECT NAME Proposed Storage Warehouse Conversion
JOB NUMBER 11223.01
LOCATION 11401 Highway 1, Point Reyes Station, California
DATE STARTED 2/10/23 **COMPLETED** 2/10/23
GROUND ELEVATION ______ **HOLE SIZE** 6"
DRILLING CONTRACTOR Pearson Exploration
GROUND WATER LEVELS:
DRILLING METHOD B-53 Solid Stem Auger with 140lb Hammer
AT TIME OF DRILLING ---
LOGGED BY TC **CHECKED BY** PJC
▼ AT END OF DRILLING 5.50 ft
NOTES Caving encountered at approx 10.0 feet.
AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
0		0.0' - 0.2'; GRAVEL (GP).										
		0.2' - 2.0'; CLAYEY SAND (SC); slightly moist, moderately compacted, fine to coarse-grained, with subround and subangular gravels (FILL).										
			MC		16	3.0T/ 3.5B	107	17	28	16	12	
		2.0' - 4.5'; SILTY CLAY (CL); orangish brown, moist, very stiff, low plasticity, with gravels (ALLUVIUM).										
			MC		22	3.0T/ 4.5+B	113	16				
5		4.5' - 10.0'; CLAYEY SAND (SC); tan & orangish brown, moist to saturated, medium dense, fine to coarse-grained, with fine to ▼ medium gravels (ALLUVIUM).										
			SPT		22			13				
10		10.0' - 15.0'; SILTY CLAY (CH); bluish gray, saturated, stiff, high plasticity (ALLUVIUM).										
			GB					27				
			MC		22							
15			SPT		10							

Bottom of borehole at 15.0 feet.

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BORING NUMBER BH-3

CLIENT Julie Van Alyea, Redwood Oil Company
PROJECT NAME Proposed Storage Warehouse Conversion
JOB NUMBER 11223.01
LOCATION 11401 Highway 1, Point Reyes Station, California
DATE STARTED 2/10/23 **COMPLETED** 2/10/23
GROUND ELEVATION
HOLE SIZE 6"
DRILLING CONTRACTOR Pearson Exploration
GROUND WATER LEVELS:
DRILLING METHOD B-53 Solid Stem Auger with 140lb Hammer
AT TIME OF DRILLING ---
LOGGED BY TC **CHECKED BY** PJC
AT END OF DRILLING 5.50 ft
NOTES
AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
0		0.0' - 0.3'; ASPHALTIC CONCRETE (AC).										
		0.3' - 1.0'; SANDY CLAY (CL); grayish brown, moist, moderately compacted, medium plasticity, with gravels (FILL).										
		1.0' - 4.0'; SILTY CLAY (CL); orangish brown, moist, very stiff, low plasticity, with gravels (ALLUVIUM).	MC		13	3.0-3.5	107	16	26	15	11	
		4.0' - 9.0'; CLAYEY SAND (SC); tan & orangish brown, moist to saturated, medium dense, fine to coarse-grained, with fine to coarse-gravels (ALLUVIUM).	MC		32		107	19				
5												
			MC		20		123	12				
10		9.0' - 10.0'; SILTY CLAY (CH); orangish brown to bluish gray, saturated, ~~medium~~ stiff, high plasticity (ALLUVIUM).	SPT		8			33				

Bottom of borehole at 10.0 feet.

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PJC & Associates, Inc.

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BORING NUMBER BH-4

CLIENT Julie Van Alyea, Redwood Oil Company **PROJECT NAME** Proposed Storage Warehouse Conversion

JOB NUMBER 11223.01 **LOCATION** 11401 Highway 1, Point Reyes Station, California

DATE STARTED 2/10/23 **COMPLETED** 2/10/23 **GROUND ELEVATION** **HOLE SIZE** 6"

DRILLING CONTRACTOR Pearson Exploration **GROUND WATER LEVELS:**

DRILLING METHOD B-53 Solid Stem Auger with 140lb Hammer **AT TIME OF DRILLING** 7.00 ft

LOGGED BY TC **CHECKED BY** PJC **AT END OF DRILLING** 7.00 ft

NOTES **AFTER DRILLING** ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	FINES CONTENT (%)
0		0.0' - 0.2'; ASPHALTIC CONCRETE (AC).										
		0.2' - 0.5'; AGGREGATE BASE (AB).										
		0.5' - 2.25'; SANDY CLAY (CL); grayish & moderate brown, slightly moist, moderated compacted, low to medium plasticity, with gravels (FILL).	GB					9				
			MC		9	2.5T/ 3.0B	105	18				
		2.25' - 4.5'; SILTY CLAY (CL); orangish brown, moist, very stiff, low plasticity, with gravels (ALLUVIUM).										
5		4.5' - 10.0'; CLAYEY SAND (SC); orangish brown & tan, moist to saturated, medium dense, fine to coarse-grained, with fine to coarse gravels (ALLUVIUM).	MC		33			12				
			MC		42		121	11				
10			SPT		26			15				

Bottom of borehole at 10.0 feet.

MAJOR DIVISIONS				TYPICAL NAMES
COARSE GRAINED SOILS More than half is larger than #200 sieve	GRAVELS more than half coarse fraction is larger than no. 4 sieve size	CLEAN GRAVELS WITH LITTLE OR NO FINES	GW	WELL GRADED GRAVELS, GRAVEL-SAND MIXTURES
			GP	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES
		GRAVELS WITH OVER 12% FINES	GM	SILTY GRAVELS, POORLY GRADED GRAVEL-SAND MIXTURES
			GC	CLAYEY GRAVELS, POORLY GRADED GRAVEL-SAND MIXTURES
	SANDS more than half coarse fraction is smaller than no. 4 sieve size	CLEAN SANDS WITH LITTLE OR NO FINES	SW	WELL GRADED SANDS, GRAVELLY SANDS
			SP	POORLY GRADED SANDS, GRAVEL-SAND MIXTURES
		SANDS WITH OVER 12% FINES	SM	SILTY SANDS, POORLY GRADED SAND-SILT MIXTURES
			SC	CLAYEY SANDS, POORLY GRADED SAND-CLAY MIXTURES
FINE GRAINED SOILS More than half is smaller than #200 sieve	SILTS AND CLAYS LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS, SILTY OR CLAYEY FINE SANDS, VERY FINE SANDS, ROCK FLOUR, CLAYEY SILTS WITH SLIGHT PLASTICITY
			CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS OR LEAN CLAYS
			OL	ORGANIC CLAYS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SANDY OR SILTY SOILS, ELASTIC SILTS
			CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
			OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS			Pt	PEAT AND OTHER HIGHLY ORGANIC SOILS

KEY TO TEST DATA

LL — Liquid Limit (in %)
PL — Plastic Limit (in %)
G — Specific Gravity
SA — Sieve Analysis
Consol — Consolidation
"Undisturbed" Sample
Bulk or Disturbed Sample
No Sample Recovery

	Shear Strength, psf	Confining Pressure, psf	
*Tx	320	(2600)	Unconsolidated Undrained Triaxial
Tx CU	320	(2600)	Consolidated Undrained Triaxial
DS	2750	(2000)	Consolidated Drained Direct Shear
FVS	470		Field Vane Shear
*UC	2000		Unconfined Compression
LVS	700		Laboratory Vane Shear

Notes: (1) All strength tests on 2.8" or 2.4" diameter sample unless otherwise indicated
(2) * Indicates 1.4" diameter sample

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USCS SOIL CLASSIFICATION KEY
PROPOSED STORAGE WAREHOUSE CONVERSION
11401 HIGHWAY 1
POINT REYES STATION, CALIFORNIA

PLATE 7

Proj. No: 11223.01 Date: 2/2023 App'd by: PJC

APPENDIX B
LABORATORY INVESTIGATION

1. INTRODUCTION

This appendix includes a discussion of the test procedures of the laboratory tests performed by PJC for use in the geotechnical study. The testing was carried out employing, whenever practical, currently accepted test procedures of the American Society for Testing and Materials (ASTM).

Undisturbed and disturbed samples used in the laboratory investigation were obtained from various locations during the course of the field investigation, as discussed in Appendix A of this report. Identification of each sample is by borehole number and depth. All of the various laboratory tests performed during the course of the investigation are described below.

2. INDEX PROPERTY TESTING

In the field of soil mechanics and geotechnical engineering design, it is advantageous to have a standard method of identifying soils and classifying them into categories or groups that have similar distinct engineering properties. The most commonly used method of identifying and classifying soils according to their engineering properties is the Unified Soil Classification System as described by ASTM D-2487. The USCS is based on a recognition of the various types and significant distribution of soil characteristics and plasticity of materials. The index properties tests discussed in this report include the determination of natural water content and dry density and Atterberg Limits.

a. Dry Density and Natural Water Content. Dry Density and natural water content was determined on selected undisturbed and disturbed samples. The samples were visually classified and accurately measured to obtain volume and weighed to obtain wet weight. The samples were then dried, in accordance with ASTM D-2216-80, for a period of 24 hours in an oven maintained at a temperature of 100 degrees C. After drying, the weight of each sample was determined and the moisture content calculated. Dry density and natural water content of the soils is presented on the borehole logs.

b. Atterberg Limits. Liquid and plastic limits were determined on selected samples in accordance with ASTM D4318. The test results are presented on the borehole logs.

3. ENGINEERING PROPERTIES TESTING

The engineering properties testing consisted of pocket penetrometer testing.

a. Pocket Penetrometer. Pocket Penetrometer tests were performed on all cohesive samples. The test estimates the unconfined compressive strength of a cohesive material by measuring the materials resistance to penetration by a calibrated, spring-loaded cylinder. The maximum capacity of the cylinder is 4.5 tons per square foot (tsf). The results of these tests are indicated on the borehole logs.

APPENDIX C
REFERENCES

1. "Foundations and Earth Structures" Department of the Navy Design Manual 7.2 (NAVFAC DM-7.2), dated May 1982.

2. "Soil Dynamics, Deep Stabilization, and Special Geotechnical Construction" Department of the Navy Design Manual 7.3 (NAVFAC DM-7.3), dated April 1983.

3. USGS Geologic map and Map Database of Parts of Marin, San Francisco, Alameda, Contra Costa and Sonoma Counties, Scale: 1:75,000, compiled by M.C. Blake Jr., R.W. Graymer & D.L. Jones, 2000.

4. "Soil Mechanics" Department of the Navy Design Manual 7.1 (NAVFAC DM-7.1), dated May 1982.

5. USGS Inverness 7.5-Minute Topographic Map, dated 1976.

6. USGS Liquefaction Susceptibility Map, 2023.

7. USGS National Seismic Hazard Maps, 2008.

8. McCarthy, David. Essential of Soil Mechanics and Foundations. 5th Edition, 1998.

9. Bowels, Joseph, Engineering Properties of Soils and Their Measurement. 4th Edition, 1992.

10. California Building Code (CBC), 2022 edition.

11. ASCE STANDARD ASCE/SEI 7-22, prepared by the American Society of Civil Engineers.

12. Site Plan, prepared by Trans Tech Consultants, dated December 30, 2022.