PJC & Associates, Inc.
Consulting Engineers & Geologists

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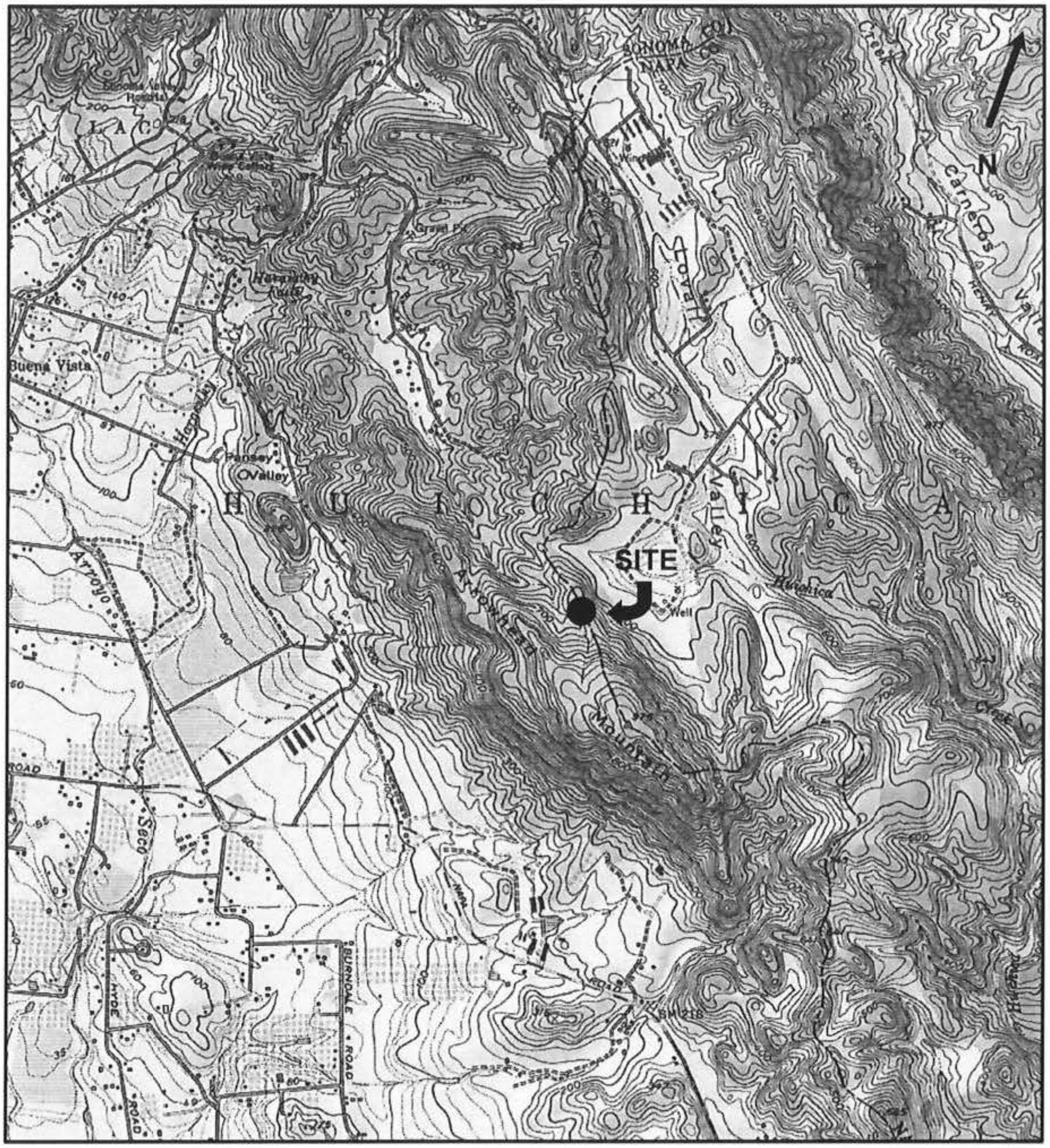
Subject: Geotechnical Investigation
Proposed McLaughlin Residence & Driveway
Lovall Valley Road
Napa APN: 050-036-013; Sonoma APN:127-132-005
Sonoma, California

PJC & Associates, Inc. (PJC) is pleased to submit this report which presents the results of our geotechnical investigation for the proposed McLaughlin residence and driveway located on Lovall Valley Road in Sonoma, California. The approximate location of the site is shown on the Site Location Map, Plate 1. Our services were completed in accordance with our proposal for geotechnical engineering services, dated December 3, 2018. This report presents our engineering opinions and recommendations regarding the geotechnical aspects of the design and construction of the proposed project. Based on the results of this study, it is our opinion that the 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 the information provided by you, and the preliminary project plans prepared by Hogan Land Services, it is our understanding that the project will consist of improving the site and constructing a new single-family residence with an attached garage, an accessory dwelling unit (ADU), and a barn. We anticipate that the buildings will consist of one or two story, wood frame structures with raised wood floors in living areas and concrete slab-on-grade floors in the garage and barn. Furthermore, the project will also include improving and realigning the existing driveway, and also may include the construction of new site retaining walls. The project will be serviced by underground municipal utilities, a private on-site septic sewer system and a private on-site domestic well.

Structural foundation loading information for the project 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



SCALE 1:24,000

REFERENCE: USGS SONOMA CALIFORNIA QUADRANGLE, DATED 1980.

PJC & Associates, Inc. Consulting Engineers & Geologists	SITE LOCATION MAP PROPOSED McLAUGHLIN RESIDENCE & DRIVEWAY LOVALL VALLEY ROAD SONOMA, CALIFORNIA	PLATE 1
	Proj. No: S1755.01 Date: 4/19 App'd by: AJD	

loads, we should be consulted to review the actual loading conditions and, if necessary, revise the recommendations of this report.

We anticipate that site grading for the structures will probably consist of cuts and fills on the order of five feet and less to achieve the desired pad grades and to provide adequate gradients for site drainage. However, based on the preliminary driveway plans, significant cuts and fills of 10 feet and greater will likely be required for the proposed driveway improvements.

2. SCOPE OF SERVICES

The purpose of this investigation was to evaluate the subsurface conditions at the site and to develop geotechnical criteria for design and construction of the project. Specifically, the scope of our services consisted of the following:

a. Excavate nine exploratory test pits to depths between one and one-half and five and one-half feet below the existing ground surface to observe the soil, bedrock and groundwater conditions. Our project engineer was on site to observe the excavation, log the materials encountered in the test pits and to obtain representative samples for visual classification and laboratory testing.

b. Perform laboratory tests on selected samples to evaluate their index and engineering properties.

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.).

d. Perform engineering analyses to develop geotechnical recommendations for site preparation and grading, foundation type(s) and design criteria, slab-on-grade recommendations, retaining wall design criteria, lateral earth pressures, site drainage, and construction considerations.

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

3. SITE CONDITIONS

a. <u>General</u>: The rectangular shaped parcels are located in a rural residential area that spans the Napa and Sonoma County line, approximately one-half mile southwest of the intersection of Lovall Valley Road and Lovall Valley Loop Road. The parcels are generally bounded by large, relatively undeveloped residential and agricultural lots to the north, south and east, and undeveloped land to the west. At the time of our investigation, the site was occupied by an existing, pre-manufactured auxiliary structure and an unimproved, earthen and gravel covered driveway, which extends through the property on the Napa County side, and into Sonoma County. The remaining portions of the site were generally undeveloped.

b. Topography and Drainage: The site traverses the low-lying areas of Lovall Valley on the eastern (Napa County) side of the parcel, and transitions up on to a northwest-southeast trending ridge on the western (Sonoma County) side of the site. The site consists of relatively level topography at the low lying, eastern portion of the site. Slope gradients increase to the west, as the site ascends the ridge, to steeply sloping topography with a maximum estimated gradient of two horizontal to one vertical (2H:1V). According to the United States Geological Survey (USGS) Sonoma, California, 7.5 Minute Quadrangle Map (Topographic), the lower portion of the eastern parcel in the vicinity of the ADU and barn is situated near an elevation of 590 feet above mean sea level (MSL). The upper portion of the western parcel in the vicinity of the main residence is situated near an elevation of 740 feet above mean sea level (MSL). The site drainage generally consists of sheet flow and surface infiltration. A small segment of a tributary of Huichica Creek extends into the eastern portion of the site, approximately 400 feet from the proposed ADU and barn. Regional drainage for the eastern half of the site is provided by the aforementioned creek. Site drainage for the western portion of the site is provided by Haraszathy Creek, which borders the western boundary of the site, approximately 250 feet west of the proposed main residence.

4. GEOLOGIC SETTING

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 late Jurassic time. This process involved eastward thrusting of oceanic crust beneath the continental crust (Klamath Mountains and Sierra Nevada) and the scraping off of materials that were 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 in mid-Cenozoic time, and has progressed northward to the vicinity of Cape Mendocino in Southern Humboldt County (Hart, Bryant and Smith, 1983). Thus, the principal structures south of Cape Mendocino are northwest-trending, nearly vertical faults of the San Andreas system.

According to published geologic literature, the site has been mapped to be underlain by bedrock deposits of the Sonoma Volcanic Series. The Sonoma Volcanic Series is divided into several subunits that range in age from about 7.9 to 5 million years old (Miocene age), and consist of mafic lava flows and tuffs, rhyolite to dacite ash flow tuff, lava flows, intrusions, and breccias. Specifically, the site is underlain by the Rhyolite of Arrowhead Mountain (T_{svra}), which primarily consists of silicic lava flows, domes and tuffs in the southwest portion of the quadrangle. Locally, the bedrock is masked by colluvial soil deposits. Additionally, the eastern portion of the site is mapped to be underlain by latest Pleistocene to Holocene alluvial deposits (Q_a), which occur as flat, relatively undissected fan, terrace, and basin deposits. However, based on our subsurface exploration, the alluvial deposits within the eastern portion of the site are relatively thin deposits, if present, and underlain by shallow bedrock of the T_{svra} subunit.

5. FAULTING

Geologic structures in the region are primarily controlled by northwest trending faults. No known active fault passes through the site. The site is not located in the Alquist-Priolo Earthquake Fault Studies Zone. Based on our research, the three closest potentially active faults to the site are the West Napa, Rodgers Creek and Green Valley faults. The West Napa fault is located five miles to the northeast, the Rodgers Creek fault is located six miles southwest and the Green Valley fault is located 13 miles east of the site. Table 1 outlines the closest known active faults and their associated maximum magnitude.

TABLE 1
CLOSEST KNOWN ACTIVE FAULTS

Fault Name	Distance From Site (Miles)	Maximum Earthquakes (Moment Magnitude)
West Napa	5	6.5
Rodgers Creek	6	7.0
Green Valley	16	6.9

6. SEISMICITY

The site is located within a zone of high seismic activity related to the active faults that transverse through the surrounding region. Future damaging earthquakes could occur on any of these fault systems during the lifetime of the proposed project. In general, the intensity of ground shaking at the site will depend upon 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. Seismic considerations and hazards are discussed in the following subsections of this report.

7. SUBSURFACE CONDITIONS

a. <u>Soils and Bedrock</u>. The subsurface conditions of the site were investigated by excavating nine exploratory test pits (TP-1 through TP-9) near the proposed building envelopes, and along the alignment of the proposed driveway, to depths between one and one-half and five and one-half feet below the existing ground surface. The approximate test pit locations are shown on the Test Pit Location Plans, Plates 3 and 4. The test pits were used to observe the soil, bedrock and groundwater conditions. The excavation procedures and descriptive test pit logs are included in Appendix A of this report. The laboratory procedures are presented in Appendix B.

TP-1, TP-4 through TP-6, and TP-9 were excavated along the alignment of the proposed roadway. TP-2, TP-3, TP-7 and TP-8 were excavated in the vicinity of the proposed structures. The exploratory test pits generally encountered artificial fill, colluvial and residual soil deposits underlain by bedrock deposits of the Sonoma Volcanics Series that extended to the maximum explored depths. At the surface of TP-6 and TP-9, our exploration encountered deposits of artificial fill, likely from the previous grading of the existing driveway, consisting of sandy silts that extended to depths between one and two feet below the existing grounds surface. The artificial fill appeared moist, loosely placed and exhibited low plasticity characteristics. Underlying the artificial fill, and encountered at the surface of the other test pits, our exploration encountered colluvial soils consisting of sandy silts that extended to depths between one-half and three feet below the existing ground surface. The colluvial deposits appeared very moist, soft to medium stiff and exhibited low plasticity characteristics. The colluvial soils at TP-1 through TP-6 were underlain by residual soil deposits consisting of silty sands that extended to depths between two and four and one-half feet below the existing ground surface. The residual soils appeared very moist, medium dense to dense and fine to coarse grained. Underlying the artificial fill, colluvial and residual soil, our exploration encountered tuff bedrock deposits that extended to the maximum explored depths. The bedrock appeared slightly hard, friable to moderately strong and moderately to highly weathered.

b. Groundwater. Groundwater seepage was encountered in TP-1, TP-2 and TP-3, at depths between one and one-half and three feet below the existing ground surface during subsurface exploration on January 23, 2019. No groundwater or seepage was encountered in the other test pits excavated on January 23, 2019 or any of the test pits excavated on January 28, 2019. However, seepage within the upper soil layers and bedrock fractures should be anticipated in the winter and early spring, and may vary depending on the amount of rainfall.

8. GEOLOGIC HAZARDS & 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. Fault Rupture. Rupture of the ground surface is expected to occur along known active fault traces. No evidence of existing faults or previous ground displacement on the site due to fault movement is indicated in the geologic literature or field exploration. Therefore, the likelihood of ground rupture at the site due to faulting is considered to be low.

b. Ground Shaking. The site has been subjected in the past to ground shaking by earthquakes on the active fault systems that traverse the region. It is believed that earthquakes with significant ground shaking will occur in the region within the next several decades. Therefore, it must be assumed that the site will be subjected to strong ground shaking during the design life of the project.

c. Liquefaction. Our field exploration revealed no loose, saturated, granular soil stratums at the site. The site is underlain by shallow bedrock that likely extends to a great depth below the site. Therefore, it is judged that the risk of soil liquefaction at the site is low.

d. Lateral Spreading and Lurching. 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 a creek embankment adjacent to the proposed building sites. Haraszthy Creek is located approximately 250 feet west of the proposed main residence, and Huichica Creek is located approximately 400 feet east of the ADU and barn. We judge that the proposed structures are setback a sufficient distance from the creek banks to avoid negative effects from potential distress of the creek banks.

e. Expansive Soils. Based on our visual observations, laboratory testing (PI=8, 8 & 9), and our experience with similar soils at nearby sites, the surface and near surface soils at the site are judged to have a low expansion potential.

f. Slope Stability. According to published geologic literature, the site is not located within an existing landslide. Furthermore, no surface evidence of significant slope instability was observed near the site which could potentially affect the future project. However, soil creep of the surface soils should be expected on slopes greater than 5H:1V.

9. CONCLUSIONS

Based on our field and office studies, we judge that from a geotechnical engineering standpoint, the site is suitable for development provided the recommendations presented in this report are incorporated into the design and carried out through construction. The primary geotechnical considerations in design and construction is the presence of artificial fill, and weak and compressible surface and near surface soils.

Our exploration encountered deposits of artificial fill that extended to a depth between one and two feet below the existing ground surface. Although this material may have been present for some time, it appears to be of variable composition and density. These soils are not suitable for support of fills, foundations, concrete slabs or pavements. Therefore, the artificial fill soils should be completely removed from structural areas and replaced as compacted engineered fill.

The surface and near surface soils are weak and compressible, and are not suitable for support of fills, foundations, or slabs. These soils could experience significant differential settlement under loads generated by new construction. Below the weak and compressible colluvial soils are bedrock deposits of the Sonoma Volcanics Series, which are considered incompressible for the anticipated foundation loads. Therefore, the main residence and ADU may be supported by deepened spread footings extending through the weak and compressible soils and into the underlying bedrock.

It is our understanding that concrete slabs-on-grade will be used in the barn. Therefore, the upper weak and compressible soils should be upgraded by subexcavation and recompaction. Provided the weak and compressible soils are upgraded by subexcavation and recompaction, the barn may be adequately supported on shallow spread footings and conventional slabs-on-grade may be utilized.

As previously mentioned, conventional concrete slabs-on-grade placed on the weak soils will be prone to settlement and cracking. Grading for the garage may remove the weak and compressible soils and expose bedrock. Conventional concrete slabs on grade may be adequately supported on bedrock. If the grading for the garage does not remove the weak and compressible soils, the garage slab should be structurally designed, or the weak and compressible soils should be subexcavated and recompacted.

Detailed geotechnical engineering recommendations for use in design and construction of the project are presented in the subsequent sections of this report.

10. EARTHWORK AND GRADING

We anticipate that site grading for the structures will probably consist of cuts and fills on the order of five feet and less to achieve the desired pad grades and to provide adequate gradients for site drainage. However, based on the preliminary driveway plan, significant cuts and fills of 10 feet and greater will likely be required for the proposed driveway improvements.

a. Stripping. Structural areas should be stripped of the surface vegetation, old fills, debris, underground utilities, etc. These materials should be moved off site; some of them, if suitable could be stockpiled for later use in landscape areas. Septic tanks and leach fields, if encountered, should be abandoned according to regulations as set forth by the applicable county health department. Voids left from the removal of utilities or other obstructions should be replaced with compacted engineered fill under the observation of the project geotechnical engineer.

b. Excavation and Compaction. Following site stripping, areas to receive fill should be prepared by removing any artificial fill and weak soils and exposing bedrock as determined by the geotechnical engineer in the field during construction. The exposed surface should then be scarified to a minimum depth of eight inches, moisture conditioned to near optimum moisture content, and recompacted to at least 90 percent of the maximum dry density as determined by ASTM D-1557 test procedures.

Where fill is required on slopes steeper than 5H:1V, the soil mantle and any weak material should be removed and these areas should be positively benched horizontally into bedrock as determined by the geotechnical engineer in the field during construction in conjunction with fill placement.

The maximum height of benches should be reviewed by the geotechnical engineer. A key will be required at the toe of all fill embankments. Observation should be provided by the geotechnical engineer to determine where these keys should be constructed. All keys should be a minimum of eight feet in width and extend at least two feet into bedrock as measured on the downhill side. The materials excavated during keyway construction and benching may be used as engineered fill. Subdrains should be installed in all the keys as determined by the geotechnical engineer in the field during construction.

The subdrain should consist of a heavy walled, four inch diameter, perforated pipe sloped to drain to outlets by gravity, and of clean, free draining, three-quarter to one and one-half inch crushed rock or gravel. The depth of the subdrain should extend at least 12 inches below the bottom of the keyway. A drainage filter cloth should be placed between the soil and the drain rock or Class II permeable material be used in lieu of the filter fabric and drain rock.

The barn building pad should be prepared by removing the weak and compressible soils to their full depth and exposing bedrock. The actual depth of subexcavation should be determined by the geotechnical engineer in the field during construction. Based on our subsurface exploration, the subexcavation for the proposed barn will probably extend to an approximate depth of 30 inches below the existing ground surface. However, it may be necessary to subexcavate deeper to provide at least one foot of compacted engineered fill below the bottom of all footings. The lateral extent of the subexcavation should be a minimum of five feet beyond the foundations and three feet beyond exterior flatwork. Prior to placement of fill, the exposed surface should be scarified to a minimum depth of eight inches, moisture conditioned to at least two percent over optimum moisture content, and recompacted to at least 90 percent of the maximum dry density as determined by ASTM D-1557 test procedures

All fill material should be placed and compacted in accordance to the recommendations presented in Table 2. It is recommended that any import fill to be used on site be of a low to non-expansive nature and should meet the following criteria:

Plastic Index	less than 12

Liquid Limit	less than 35
Percent Soil Passing #200 Sieve	between 15% and 35%
Maximum Aggregate Size	4 inches

The existing on-site soils, free of organics and rocks larger than six inches in dimension, are suitable for use as compacted engineered fill. All fills should be placed in lifts no greater than eight inches in loose thickness and compacted to the general recommendations provided for engineered fill.

TABLE 2
SUMMARY OF COMPACTION RECOMMENDATIONS

Area	Compaction Recommendations*
General Engineered Fill (Import) SHALL BE COMPACTED	In lifts, a maximum of eight inches loose thickness, compact to a minimum of 90 percent relative compaction at or near optimum moisture content.
General Engineered Fill (Native) SHALL BE COMPACTED	In lifts, a maximum of eight inches loose thickness, compact to 90 percent relative compaction at least two percent over optimum moisture content.

*All compaction requirements stated in this report refer to dry density and moisture content relationships obtained through the laboratory standard described by ASTM D-1557-91

A representative of PJC should observe all site preparation and fill placement. It is important that during the stripping, grading and scarification processes, a representative of our firm be present to observe whether any undesirable material is encountered in the construction area.

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

Cut and fill slopes should be no steeper than two horizontal to one vertical (2H:1V). However, cut slopes in competent bedrock may be steepened as approved by the geotechnical engineer in the field during construction. However, steepened slopes will be prone to sloughing and may require a catchment wall at the base of the slope. Regardless, cut slopes steeper than 2H:1V should be evaluated by the geotechnical engineer in the field during construction to determine the feasibility and provide additional recommendations for construction, as necessary. Disturbed slopes should be planted with deep rooted groundcover to reduce and control erosion.

11. FOUNDATIONS-DEEPENED SPREAD FOOTINGS (RESIDENCE & ADU)

a. Vertical Loads. The main residence and ADU may be adequately supported on a deepened spread footing foundation extending through the upper unsuitable surface and near surface soils and at least 12 inches into the underlying bedrock. Based on our subsurface exploration, we anticipate that the footing depths could extend to depths of three and one-half feet and greater below the existing ground surface. All footings should be reinforced. The recommended soil bearing pressures, depth of minimum embedment, and minimum widths of spread footings are presented in Table 3. The bearing values provided have been calculated assuming that all footings extend at least 12 inches into the underlying bedrock, as determined by the geotechnical engineer in the field during construction.

TABLE 3
FOUNDATION DESIGN CRITERIA

Footing Type	Bearing Pressure (psf)*	Minimum Embedment (in)**	Minimum Width (in)
Continuous Wall	3000	12	12
Isolated Column	3500	12	18

*Dead plus live load
** Depth into bedrock

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

b. Lateral Loads. Resistance to lateral forces may be computed using friction or passive pressure. A friction factor of 0.40 is considered appropriate between the bottom of concrete structures and the bearing soils. A passive pressure equivalent to that exerted by a fluid weighing 400 pounds per square foot per foot of depth (psf/ft) may be used. The upper six inches of bedrock should be neglected for passive resistance. Furthermore, there should be at least seven feet of horizontal confinement between the bottom of the footing and the face of the nearest slope.

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 bearing values provided. Maximum settlements of shallow foundations designed and constructed in accordance with the preceding recommendations are estimated to be less than one inch. Differential settlement between similarly loaded, adjacent footings are expected to be less than one-half of one inch. The majority of the settlement is expected to occur during construction and placement of dead loads.

Footing concrete should be placed neat against competent bedrock. 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 to close all cracks prior to concrete placement.

The geotechnical engineer should observe the bearing surfaces of the spread footings after the cleaning and prior to placement of concrete and steel to assess the conditions of the foundation bearing materials.

12. FOUNDATIONS-CONVENTIONAL SPREAD FOOTINGS (BARN)

a. Vertical Loads. Provided the weak soils are subexcavated and recompacted in accordance with the earthwork and grading section of this report, the barn may be adequately supported by spread footings extending at least 12 inches into compacted, engineered fill. All footings should be reinforced. The recommended soil bearing pressures, depths of embedment and minimum width of spread footings are presented in Table 4. The bearing values provided have been calculated assuming that all footings uniformly bear on at least 12 inches of compacted engineered fill.

TABLE 4
FOUNDATION DESIGN CRITERIA

Footing Type	Bearing Pressure (psf)*	Minimum Embedment (in)**	Minimum Width (in)
Continuous Wall	2000	12	12
Isolated Column	2500	12	18

*Dead plus live load
** Depth into compacted engineered fill

The allowable soil bearing pressures are net values. The weight of the foundation and backfill over the foundation may be neglected when computing dead loads. Allowable soil 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 or passive pressure. A friction factor of 0.35 is considered appropriate between the bottom of the concrete structures and the engineered fill. A passive pressure equivalent to that exerted by a fluid weighing 350 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.

Furthermore, there should be at least seven feet of horizontal confinement between the bottom of the footing and the face of the nearest slope.

Footing concrete should be placed neat against engineered fill. 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 to close all cracks 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 bearing values provided. Maximum settlements of shallow foundations designed and constructed in accordance with the preceding recommendations are estimated to be less than one inch. Differential settlement between similarly loaded, adjacent footings are expected to be less than one-half of one inch. The majority of the settlement is expected to occur during construction and placement of dead loads.

The geotechnical engineer should observe the bearing surfaces of the spread footings after the cleaning and prior to placement of concrete and steel to assess the conditions of the foundation bearing materials.

13. SLAB-ON-GRADE

Conventional slabs-on-grade should be supported entirely on compacted engineered fill. However, provided the excavation for the garage removes the weak surface and near surface soils and exposes bedrock, the garage slab may be supported on the underlying bedrock. All slabs should be supported on at least four inches of clean gravel or crushed rock to provide a capillary moisture 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. Furthermore, the slabs-on-grade should be provided with underslab drains to prevent hydrostatic uplift and control seepage, as shown on Plate 2.

We recommend that the gravel be placed as soon as possible after compaction of the subgrade to prevent drying of the subgrade soils. If the subgrade is allowed to dry out prior to slab-on-grade construction, the subgrade soils should be moisture conditioned by sprinkling prior to concrete placement.

We recommend that slabs be at least five inches thick and designed and reinforced as determined by the project structural engineer. Slabs should be provided with control joints at regular intervals to induce and control cracking. Special care should be taken to insure that reinforcement is placed at the slab mid-height.

For slabs-on-grade with moisture sensitive surfacing, we recommend that an impermeable membrane be placed over the rock to prevent migration of moisture vapor through the concrete slab.

14. MECHANICALLY STABILIZED EARTH RETAINING WALLS

The mechanically stabilized earth retaining walls may be used for the proposed driveway. The first course of the wall should be founded at least 12 inches into the underlying bedrock or compacted engineered fill, and have at least seven feet of horizontal confinement between the bottom of the first course and the face of the nearest slope. The walls should be designed using the following parameters as determined by the vendor.

Soil (Compacted Fill)

Total Unit Weight =120 pcf
Friction Angle (Φ)= 30°
Cohesion (c)= 150 psf

Provisions to allow for the release and prevention of hydrostatic pressure build up, such as backdrain or weep holes, should be incorporated in the design or the walls should be designed for full hydrostatic pressure.

15. RETAINING WALLS

Retaining walls free to rotate on the top and supporting a level to gently sloping backfill may be designed to resist an active equivalent fluid pressure of 40 pcf acting in a triangular pressure distribution. Retaining supporting a steeply sloping backfill should be designed to resist an active equivalent fluid pressure of 60 pcf acting in a triangular pressure distribution. These pressures do not consider surcharge loads resulting from adjacent foundations, traffic loads or earthquake loads. If additional surcharge loading is anticipated, we can assist in evaluating their effects.

We recommend that a backdrain be provided behind all retaining walls or that the walls be designed for full hydrostatic pressures. The backdrain should consist of a heavy walled, four inch diameter, perforated pipe sloped to drain to outlets by gravity, and of clean, free-draining, three-quarter to one-inch crushed rock or gravel. The crushed rock or gravel should extend to within one foot of the surface. The upper foot should be backfilled with compacted, fine grained soil to exclude surface water intrusion. A drainage filter cloth should be placed between the soil and the drain rock or Class II permeable material may be used in lieu of the filter fabric and drain rock.

We recommend that the ground surface behind the retaining walls be sloped to drain. Under no circumstances should the surface water be diverted into back drains. Where migration of moisture through walls would be detrimental, the walls should be waterproofed.

16. RETAINING WALLS-SEISMIC LOADING

PJC has performed analysis to estimate the anticipated dynamic load due to seismic shaking on retaining walls at the site. Based on our pseudostatic analysis, the walls should be designed for a dynamic lateral force equivalent to a uniform point load, P_e, as determined by the following equation:

$$P_e=7.8*H^2$$

Where:

H = height of retaining wall in feet

P_e = pseudostatic seismic loading in lbs/ft

The pseudostatic force, P_e should be applied at a distance of (2/3)H above the base of the retaining wall.

17. ASPHALTIC CONCRETE PAVEMENTS

Based on our investigation, we believe that the native soils have a moderate supporting capacity (after properly compacted) when used as a pavement subgrade. Based on our laboratory testing, an R-value of 31 was determined and used in asphaltic concrete pavement design calculations.

Pavement thicknesses were computed from Chapter 600 of the Caltrans Highway Design Manual and are based on a pavement life of 20 years. The Traffic Indexes (TI) used are judged representative of the anticipated traffic but are not based on actual vehicle counts. The actual traffic indexes should be determined and provided by the project civil engineer.

Prior to placement of the aggregate base material, the upper eight inches of the pavement subgrade should be scarified to at least eight inches deep, moisture conditioned to between two and four percent over optimum moisture content, and compacted to at least 95 percent relative compaction. Aggregate base material should be spread in thin layers and compacted to at least 95 percent relative compaction to form a firm and unyielding base.

The materials and methods used should conform to the requirements of the County of Sonoma specifications or the current edition of the Caltrans Standard Specifications, except that compaction requirements for the soil subgrade and aggregate baserock should be based on ASTM D-1557-91. Aggregate used for the base course should comply with the minimum requirements specified in Caltrans Standard Specifications, Section 26, for Class II Aggregate Base.

In general, the pavements should be constructed during the dry season to avoid the saturation of subgrade and base materials which often occurs during the wet winter months. If pavements are constructed during the winter and early spring, a cost increase relative to drier weather construction should be anticipated. Unstable areas may require subexcavation to remove soft soils. The excavations will probably require geotextile fabric and backfilling with imported crushed rock. The soils engineer should be contacted for recommendations at the time of construction.

Where pavements will abut landscaped areas, water can seep below the concrete curb and into the base rock within the pavement section. Continued saturation of the base rock leads to permanent wetness towards the lower elevation of the pavement where water ponds. Soft subgrade conditions and pavement damage can occur as a result. Furthermore, differential settlement from cut and fill areas can lead to pavement cracking at the transition. The owner should understand and accept this risk. In order to minimize the risk, the owner may elect to overexcavate the bedrock and replace as engineered fill. Although, this can be cost prohibitive, particularly when hard bedrock conditions are encountered.

Where trees are located adjacent to pavement areas, we recommend that a suitable impervious root barrier be included to minimize water mitigation into the pavement layer.

TABLE 5
PAVEMENT DESIGN FOR PAVEMENT AREAS
(Subgrade R-Value=31)

Traffic Index	Asphaltic Concrete (in)	Class II Aggregate Base (in)
4.0	2.0	6.0
5.0	2.5	6.5
6.0	3.0	8.0
7.0	3.5	10.0

18. SEISMIC DESIGN

Geologic structures in the region are primarily controlled by northwest trending faults. No known active fault passes through the site. The site is not located in the Alquist-Priolo Earthquake Fault Studies Zone. Based on the data reviewed, it is concluded that the project site could be subjected to seismic shaking resulting from earthquakes on the active faults primarily in the Coast Ranges. For design, a site class type C, spectral accelerations of S_S of 1.50 g and S_1 of 0.60 g are recommended.

19. DRAINAGE

We recommend that the roofs be provided with gutters and that the downspouts be connected to closed conduits discharging to a designated area away from foundations and slopes. Surface water should be channeled away from slopes and foundations.

We recommend that foundation subdrains be placed adjacent to all foundations, except the downhill foundation. The foundation subdrains should extend at least 12 inches below the interior subgrade. The bottom of the trench should be sloped to drain by gravity and lined with a few inches of three quarter to one and a half inch-drain rock. The subdrain should consist of a heavy walled, four inch diameter, perforated pipe sloped to drain to outlets by gravity. The trench should then be backfilled to within six inches of finished surface with drain rock. The upper few inches should consist of compacted soil to reduce surface water inclusion. We recommend that a drainage filter cloth be placed between the soil and the drain rock or Class II permeable material be used in lieu of the filter fabric and drain rock. Furthermore, the interior slabs-on-grade should be provided with underslab drains to prevent hydrostatic uplift and control seepage, as shown on Plate 2.

Roof downspouts and surface drains must be maintained entirely separate from the foundation subdrains. The outlets should discharge onto erosion resistant areas.

20. LIMITATIONS

The data, information, interpretations and recommendations in this report are presented solely as bases and guides for the geotechnical design of the proposed McLaughlin residence and driveway located on Lovall Valley Road in Sonoma, California. The conclusions and professional opinions presented herein were developed in accordance with generally accepted geotechnical engineering principles and practices. As with all geotechnical reports, the opinions expressed here are subject to revisions in light of new information, which may be developed in the future, and no warranties are either expressed or implied.

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 purpose 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 and approved in writing. This report and the drawings contained herein are intended only for the design of the proposed project. 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 the 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 PJC does not and cannot have complete knowledge of the subsurface conditions underlying the subject site. The criteria presented are based upon the findings at the points of exploration and upon 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. Observation and testing services should be provided by PJC to verify that the intent of the plans and specifications is carried out during construction; these services should include observing the foundation excavations, field density testing of fill, and installation of the subsurface 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 us if you have any questions regarding the results of this investigation, or if we can be of further assistance.

Sincerely,

PJC & Associates, Inc.

Donald A. Whyte
Project Geologist
PG 9109, California

Anthony J. DeMartini
Geotechnical Engineer
GE 2750, California

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REGISTERED PROFESSIONAL ENGINEER
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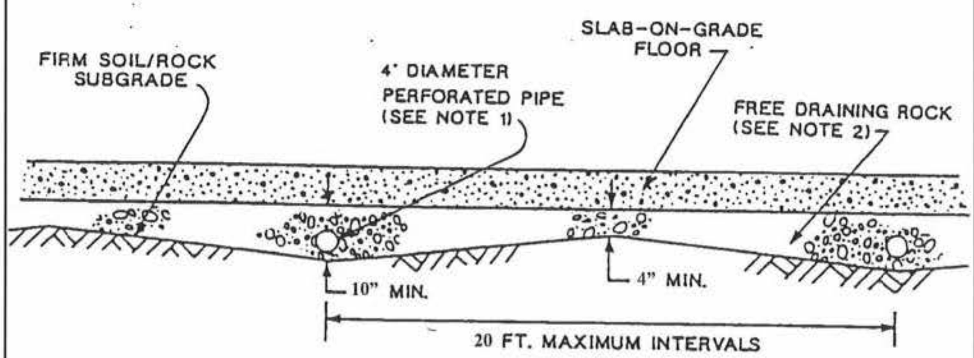
GEOTECHNICAL
STATE OF CALIFORNIA



Notes:

1. PERFORATED PIPE (PVC OR EQUIVALENT) SHOULD BE PLACED WITH PERFORATIONS DOWN. THE PIPE SHOULD BE SLOPED FOR GRAVITY FLOW AND OUTLET THROUGH SOLID PIPE TO DAYLIGHT.

2. DRAIN ROCK SHOULD BE AT LEAST 4" THICK AND A MINIMUM OF 10" WHERE PIPES ARE LOCATED. THE DRAIN ROCK SHOULD BE ½ OR ¾ INCH DRAIN ROCK ON FILTER FABRIC OR CONSIST OF CLASS II PERMEABLE MATERIAL.

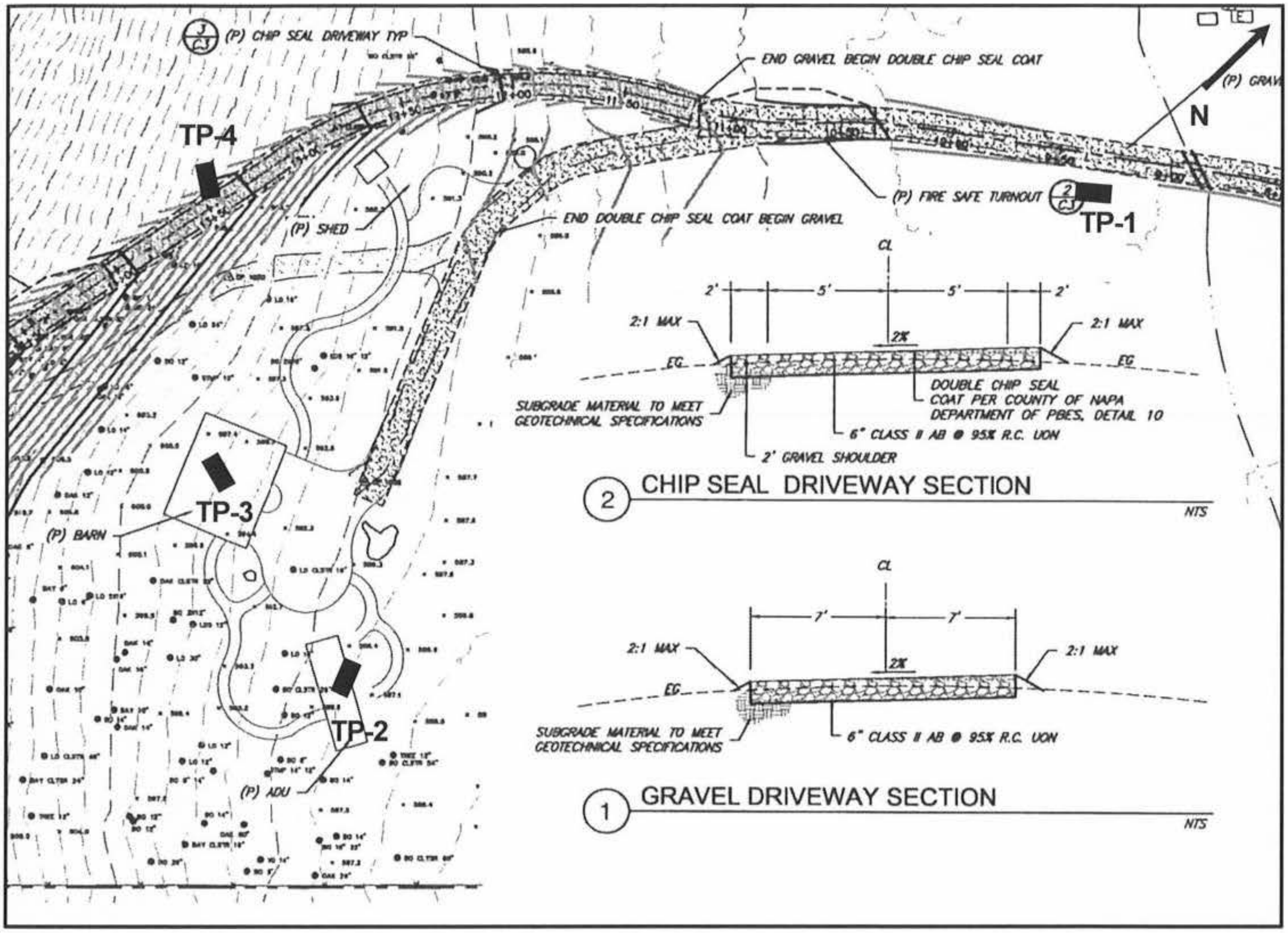
APPENDIX A
FIELD INVESTIGATION

1. INTRODUCTION

The field program performed for this study consisted of excavating nine exploratory test pits (TP-1 through TP-9) in the vicinity of the proposed structures and along the existing and proposed driveway alignment. The test pits were excavated on January 23, 2019 and January 28, 2019. The test pit locations are shown on the Test Pit Location Plans, Plates 3 and 4. Descriptive logs of the test pits are presented in this appendix as Plates 5 through 13.

2. TEST PITS

The test pits were excavated using a track-mounted excavator with a 30-inch bucket. Disturbed samples were obtained for visual classification and laboratory testing. The soils were classified in accordance with the Unified Soil Classification System, as explained in Plate 14. The bedrock was classified according to Plate 15.



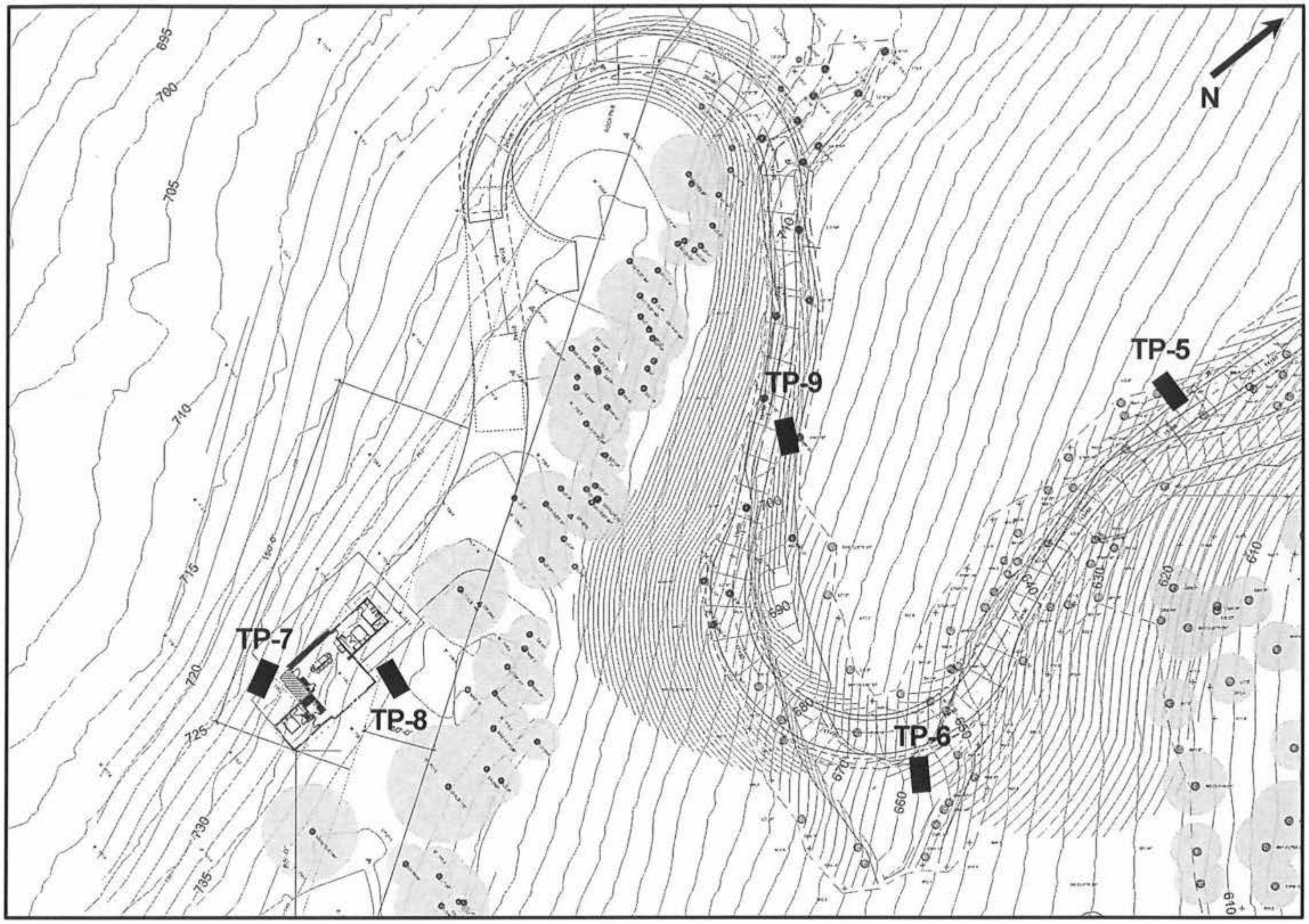
EXPLANATION

TEST PIT LOCATION AND DESIGNATION

NO SCALE

REFERENCE: PRELIMINARY DRIVEWAY DESIGN, PREPARED BY HOGAN LAND SERVICES, DATED NOVEMBER 29, 2018.

PJC & Associates, Inc. Consulting Engineers & Geologists	TEST PIT LOCATION PLAN PROPOSED McLAUGHLIN RESIDENCE & DRIVEWAY LOVALL VALLEY ROAD SONOMA, CALIFORNIA	PLATE 3
	Proj. No: S1755.01 Date: 4/19 App'd by: AJD	



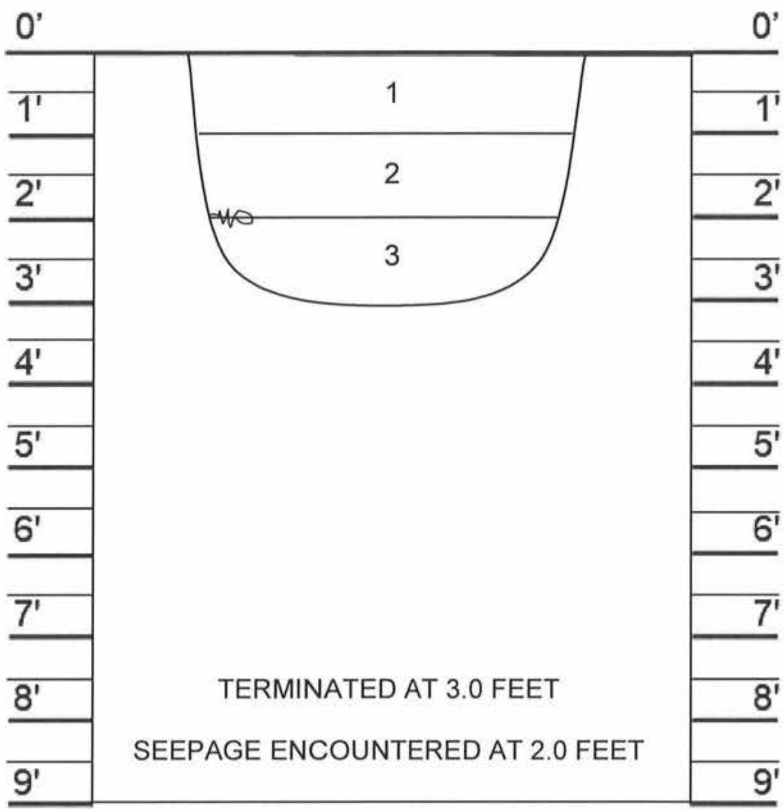
EXPLANATION

TEST PIT LOCATION AND DESIGNATION

NO SCALE

REFERENCE: CONCEPTUAL MASTER PLAN R1, PROVIDED BY HOGAN LAND SERVICES, DATED FEBRUARY 28, 2019.

PJC & Associates, Inc. Consulting Engineers & Geologists	TEST PIT LOCATION PLAN PROPOSED McLAUGHLIN RESIDENCE & DRIVEWAY LOVALL VALLEY ROAD SONOMA, CALIFORNIA	PLATE 4
	Proj. No: S1755.01 Date: 4/19 App'd by: AJD	



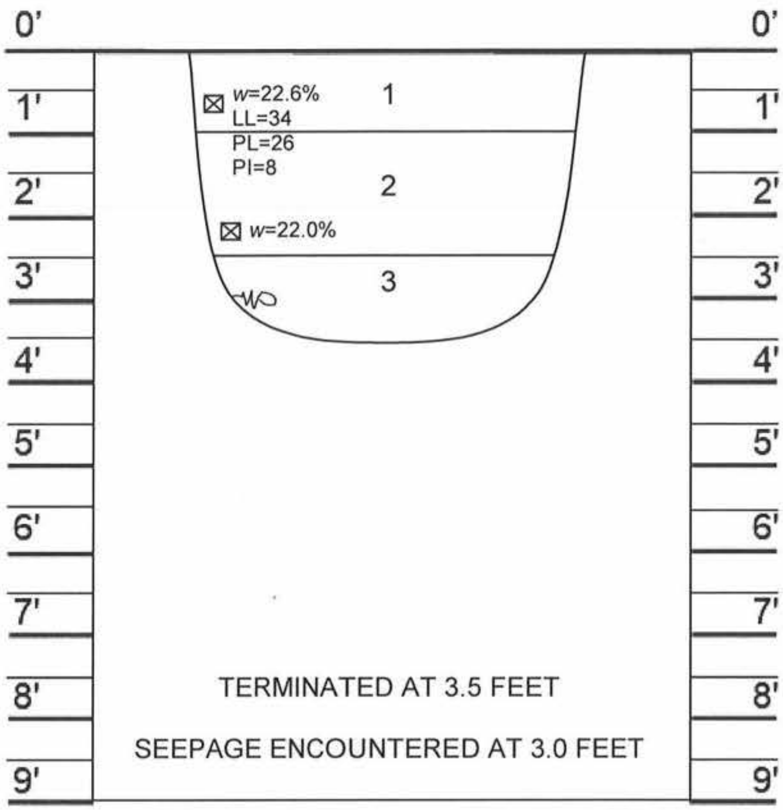
LITHOLOGY

1) 0.0-1.0'; SANDY SILT (ML); dark brown, very moist, soft, low plasticity. (COLLUVIUM)

2) 1.0-2.0'; SILTY SAND (SM); light grayish brown, very moist to wet, medium dense, fine to coarse grained. (COLLUVIUM)

3) 2.0-3.0'; TUFF; mottled light brown and orange, slightly hard, friable, highly weathered. (SONOMA VOLCANICS)

PJC & Associates, Inc. Consulting Engineers & Geologists	LOG OF TEST PIT 1 PROPOSED McLAUGHLIN RESIDENCE & DRIVEWAY LOVALL VALLEY ROAD SONOMA, CALIFORNIA	PLATE 5
	Proj. No: S1755.01 Date: 4/19 App'd by: AJD	



<u>LITHOLOGY</u>

1) 0.0-1.0'; SANDY SILT (ML); brown, very moist, soft, low plasticity. (COLLUVIUM)

2) 1.0-2.5'; SILTY SAND (SM); light grayish brown, very moist, medium dense to dense, fine to coarse grained. (COLLUVIUM)

3) 2.5-3.5'; TUFF; light yellow, slightly hard, friable to weak, highly weathered. (SONOMA VOLCANICS)

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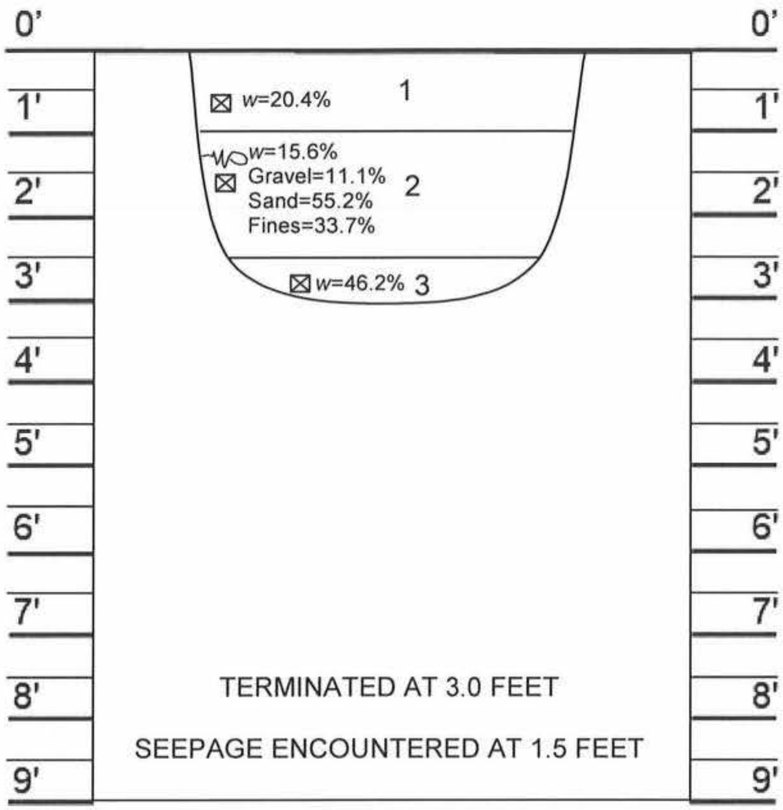
Consulting Engineers & Geologists

LOG OF TEST PIT 2
PROPOSED McLAUGHLIN RESIDENCE & DRIVEWAY
LOVALL VALLEY ROAD
SONOMA, CALIFORNIA

PLATE
6

Proj. No: S1755.01 Date: 4/19 App'd by: AJD



LITHOLOGY

1) 0.0-1.0'; SANDY SILT (ML); brown, very moist, soft, low plasticity. (COLLUVIUM)

2) 1.0-2.5'; CLAYEY SAND (SC); grayish brown, very moist to wet, dense, fine to coarse grained. (COLLUVIUM)

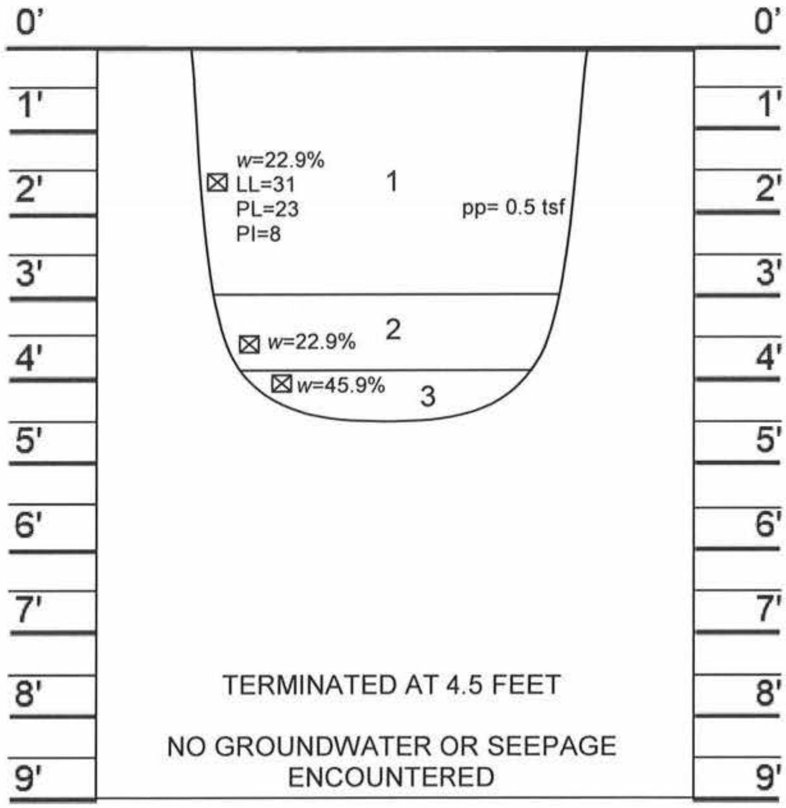
3) 2.5-3.0'; TUFF; light yellow, slightly hard, weak, highly weathered. (SONOMA VOLCANICS)

PJC & Associates, Inc.
Consulting Engineers & Geologists

LOG OF TEST PIT 3
PROPOSED McLAUGHLIN RESIDENCE & DRIVEWAY
LOVALL VALLEY ROAD
SONOMA, CALIFORNIA

PLATE
7

Proj. No: S1755.01 Date: 4/19 App'd by: AJD



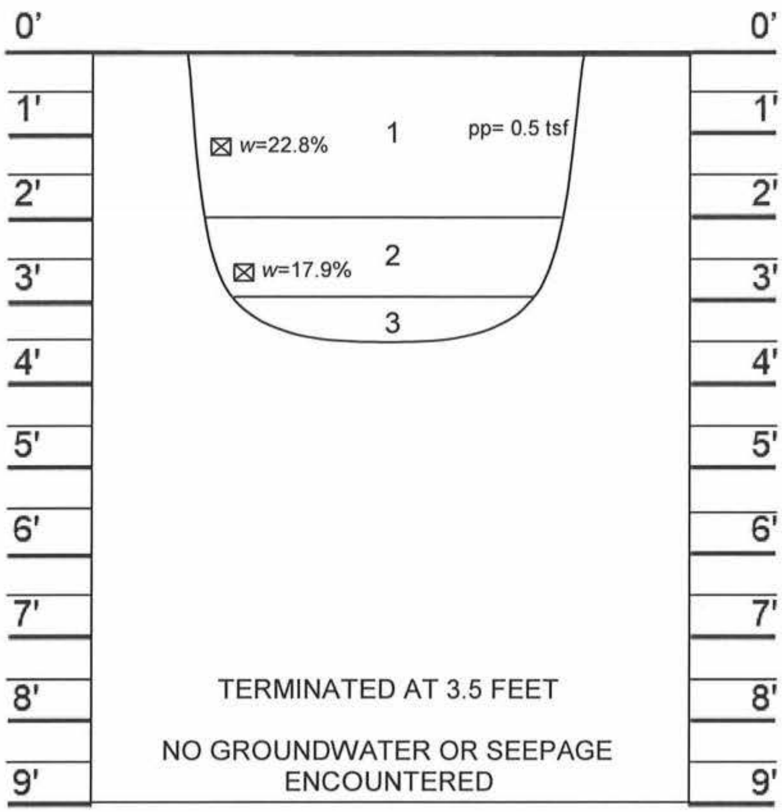
LITHOLOGY

1) 0.0-3.0'; SANDY SILT (ML); brown, very moist, soft, low plasticity. (COLLUVIUM)

2) 3.0-4.0'; SILTY SAND (SM); light yellowish brown, very moist, dense, fine to coarse grained. (COLLUVIUM)

3) 4.0-4.5'; TUFF; light yellow, slightly hard, weak, highly weathered. (BEDROCK)

PJC & Associates, Inc. Consulting Engineers & Geologists	LOG OF TEST PIT 4 PROPOSED McLAUGHLIN RESIDENCE & DRIVEWAY LOVALL VALLEY ROAD SONOMA, CALIFORNIA	PLATE **8**
	Proj. No: S1755.01 Date: 4/19 App'd by: AJD	



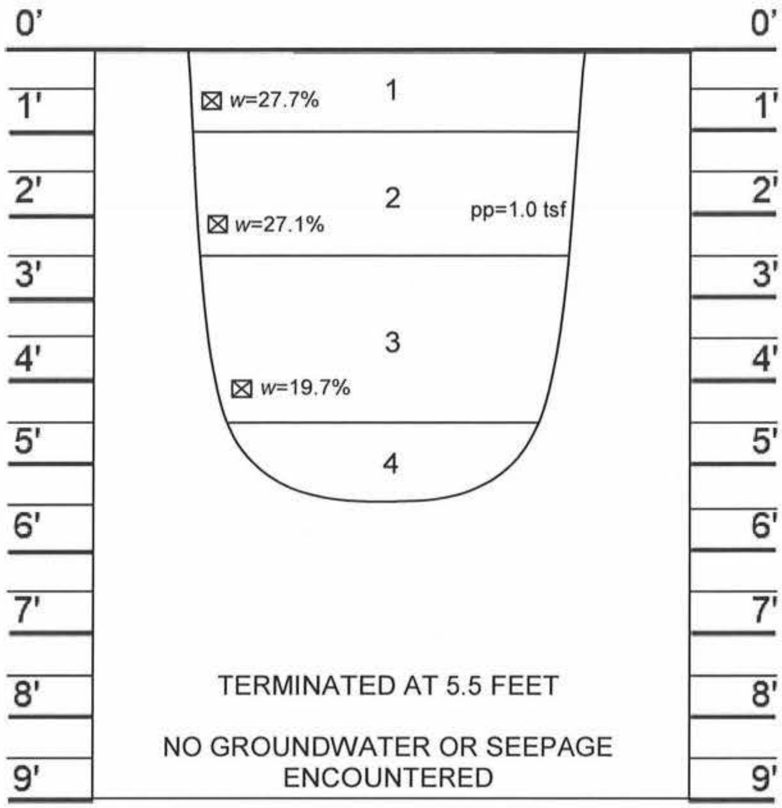
<u>LITHOLOGY</u>

1) 0.0-2.0'; SANDY SILT (ML); brown, very moist, soft, low plasticity. (COLLUVIUM)

2) 2.0-3.0'; SILTY SAND (SM); light grayish brown, very moist, dense, fine to coarse grained. (COLLUVIUM)

3) 3.0-3.5'; TUFF; light yellow, slightly hard, weak, highly weathered. (BEDROCK)

PJC & Associates, Inc. Consulting Engineers & Geologists	LOG OF TEST PIT 5 PROPOSED McLAUGHLIN RESIDENCE & DRIVEWAY LOVALL VALLEY ROAD SONOMA, CALIFORNIA	PLATE 9
	Proj. No: S1755.01 Date: 4/19 App'd by: AJD	



LITHOLOGY

1) 0.0-1.0'; SANDY SILT (ML); brown, very moist, loosely placed, low plasticity. (FILL)

2) 1.0-2.5'; SANDY SILT (ML); dark gray, very moist, medium stiff, low plasticity. (COLLUVIUM)

3) 2.5-4.5'; SILTY SAND (SM); light grayish brown, very moist, dense, fine to coarse grained. (COLLUVIUM)

4) 4.5-5.5'; TUFF; mottled light yellow and orange, slightly hard, friable to weak, highly weathered. (BEDROCK)

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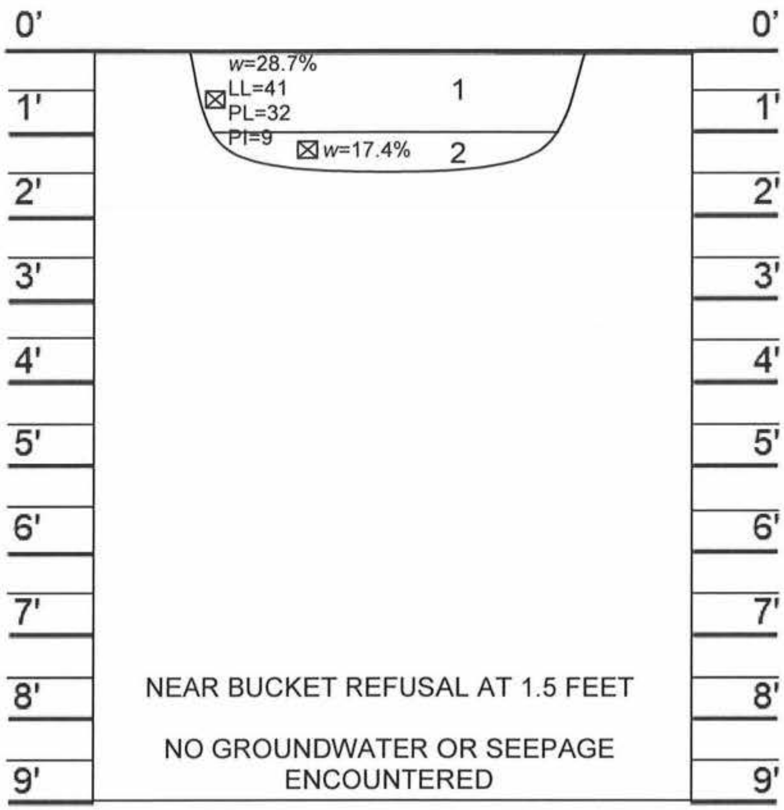
Consulting Engineers & Geologists

LOG OF TEST PIT 6
PROPOSED McLAUGHLIN RESIDENCE & DRIVEWAY
LOVALL VALLEY ROAD
SONOMA, CALIFORNIA

PLATE **10**

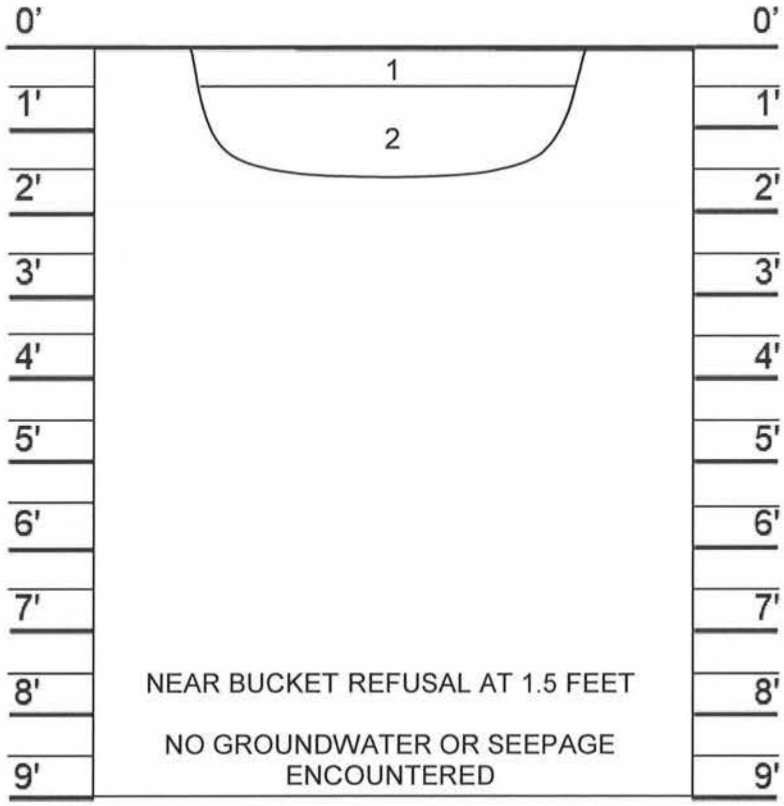
Proj. No: S1755.01 Date: 4/19 App'd by: AJD



LITHOLOGY

1) 0.0-1.0'; SANDY SILT (ML); brown, very moist, soft, low plasticity. (COLLUVIUM)

2) 1.0-1.5'; TUFF; light pink, slightly hard, weak, moderately weathered. (BEDROCK)

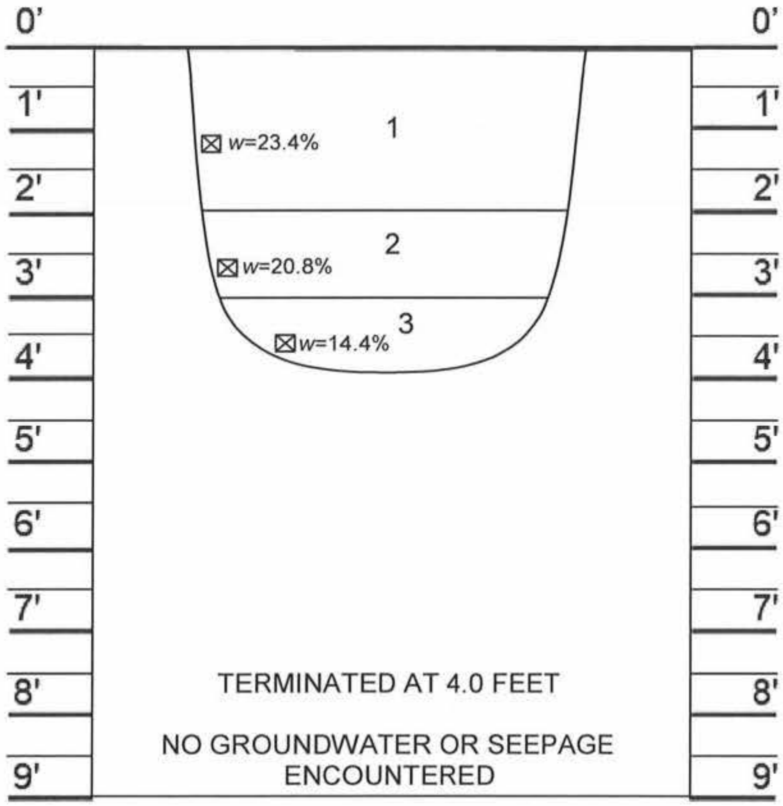


LITHOLOGY

1) 0.0-0.5'; SANDY SILT (ML); brown, very moist, soft, low plasticity. (COLLUVIUM)

2) 0.5-1.5'; TUFF; mottled light gray and orange, slightly hard, moderately strong, highly weathered. (BEDROCK)

PJC & Associates, Inc. Consulting Engineers & Geologists	LOG OF TEST PIT 8 PROPOSED McLAUGHLIN RESIDENCE & DRIVEWAY LOVALL VALLEY ROAD SONOMA, CALIFORNIA	PLATE 12
	Proj. No: S1755.01 Date: 4/19 App'd by: AJD	



LITHOLOGY

1) 0.0-2.0'; SANDY SILT (ML); brown, moist, loosely placed, low plasticity, with cobbles. (FILL)

2) 2.0-3.0'; SANDY SILT (ML); light brown, very moist, soft, low plasticity. (COLLUVIUM)

3) 3.0-4.0'; TUFF; mottled light gray and orange, slightly hard, weak, highly weathered. (BEDROCK)

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

ROCK TYPES

Conglomerate	Shale	Metamorphic Rocks Hydrothermally Altered Rocks
Sandstone	Sheared Shale Melange	Igneous Rocks
Meta-Sandstone	Chert	

Bedding Thickness		Joint, Fracture or Shear Spacing	
Massive	Greater than 6 feet	Very Widely Spaced	Greater than 6 feet
Thickly Bedded	2 to 6 feet	Widely Spaced	2 to 6 feet
Medium Bedded	8 to 24 inches	Moderately Widely Spaced	8 to 24 inches
Thinly Bedded	2-1/2 to 8 inches	Closely Spaced	2-1/2 inches
Very Thinly Bedded	3/4 to 2-1/2 inches	Very Closely Spaced	3/4 to 2-1/2 inches
Closely Laminated	1/4 to 3/4 inches	Extremely Closely Spaced	Less than 3/4 Inch
Very Closely Laminated	Less than 1/4 inch		

HARDNESS

Soft - Pliable, can be dug by hand

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

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

Hard - Can be scratched with difficulty; scratch produced little powder and is faintly visible

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

STRENGTH

Plastic- Capable of being molded by hand

Friable - Crumbles by rubbing with fingers

Weak - an unfractured specimen of such material will crumble under light hammer blows

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

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

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

DEGREE OF WEATHERING

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

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

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

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

APPENDIX B
LABORATORY INVESTIGATION

1. INTRODUCTION

This appendix includes a discussion of test procedures and results of the laboratory investigation performed for the proposed project. The investigation program was carried out by employing currently accepted test procedures of the American Society of Testing and Materials (ASTM).

Disturbed samples used in the laboratory investigation were obtained during the course of the field investigation as described in Appendix A of this report. Identification of each sample is by pit number and depth.

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 described by ASTM D-2487-83. 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, Atterberg limits tests and Grain-size distribution.

a. Natural Water Content. Natural water content was determined on selected disturbed samples. The samples were extruded, visually classified, and accurately weighed to obtain wet weight. The samples were then dried, in accordance with ASTM D-2216, 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. The water content results are summarized on the test pit logs, Plates 5 through 13.

b. Atterberg Limits Determination. The liquid and plastic limits of a selected fine-grained soil sample were determined by air drying and breaking down the sample. The results of the limits are shown on Plate 16.

c. Grain-Size Distribution. The gradation characteristics of a selected sample were determined in accordance with ASTM D422-63. The sample was soaked in water until individual soil particles were separated and then washed on the No. 200 mesh sieve. That portion of the material retained on the No. 200 mesh sieve was oven-dried and then mechanically sieved. The grain-size distribution test is presented on Plate 17.

3. ENGINEERING PROPERTIES

The engineering properties testing consisted of R-value testing.

a. R-Value. An R-value test was performed on a representative sample of the surface soils to develop criteria for the design of pavement sections. The test was conducted in accordance with the California Division of Highways Test Method No. 310; the test results are shown on Plate 18.

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ATTERBERG LIMITS' RESULTS

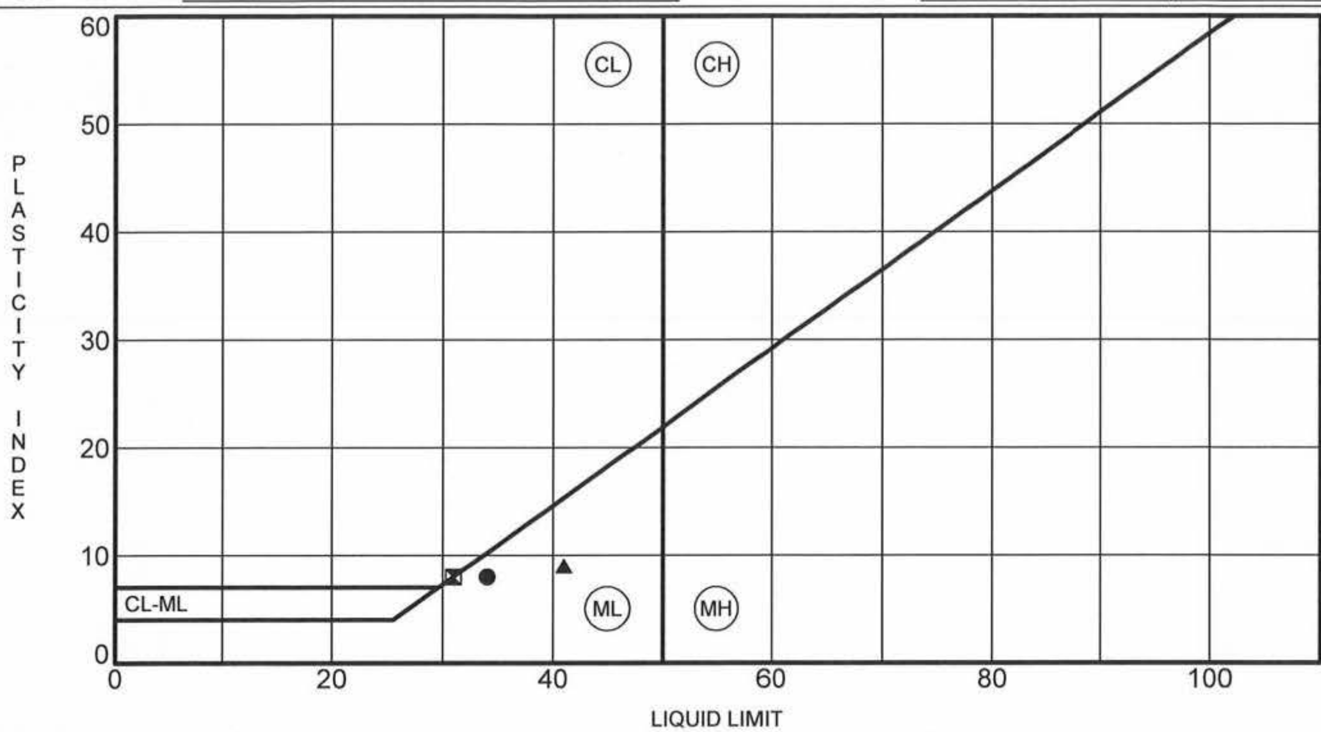
PLATE 16

CLIENT JANET MCLAUGHLIN

PROJECT NAME PROPOSED MCLAUGHLIN RESIDENCE & DRIVEWAY

PROJECT NUMBER S1755.01

PROJECT LOCATION LOVALL VALLEY ROAD; SONOMA CA



	Specimen Identification		LL	PL	PI	Fines	Classification
●	**TP-2**	**0.5**	**34**	**26**	**8**		**BROWN SANDY SILT (ML)**
⊠	**TP-4**	**1.5**	**31**	**23**	**8**		**BROWN SANDY SILT (ML)**
▲	**TP-7**	**0.5**	**41**	**32**	**9**		**BROWN SANDY SILT (ML)**

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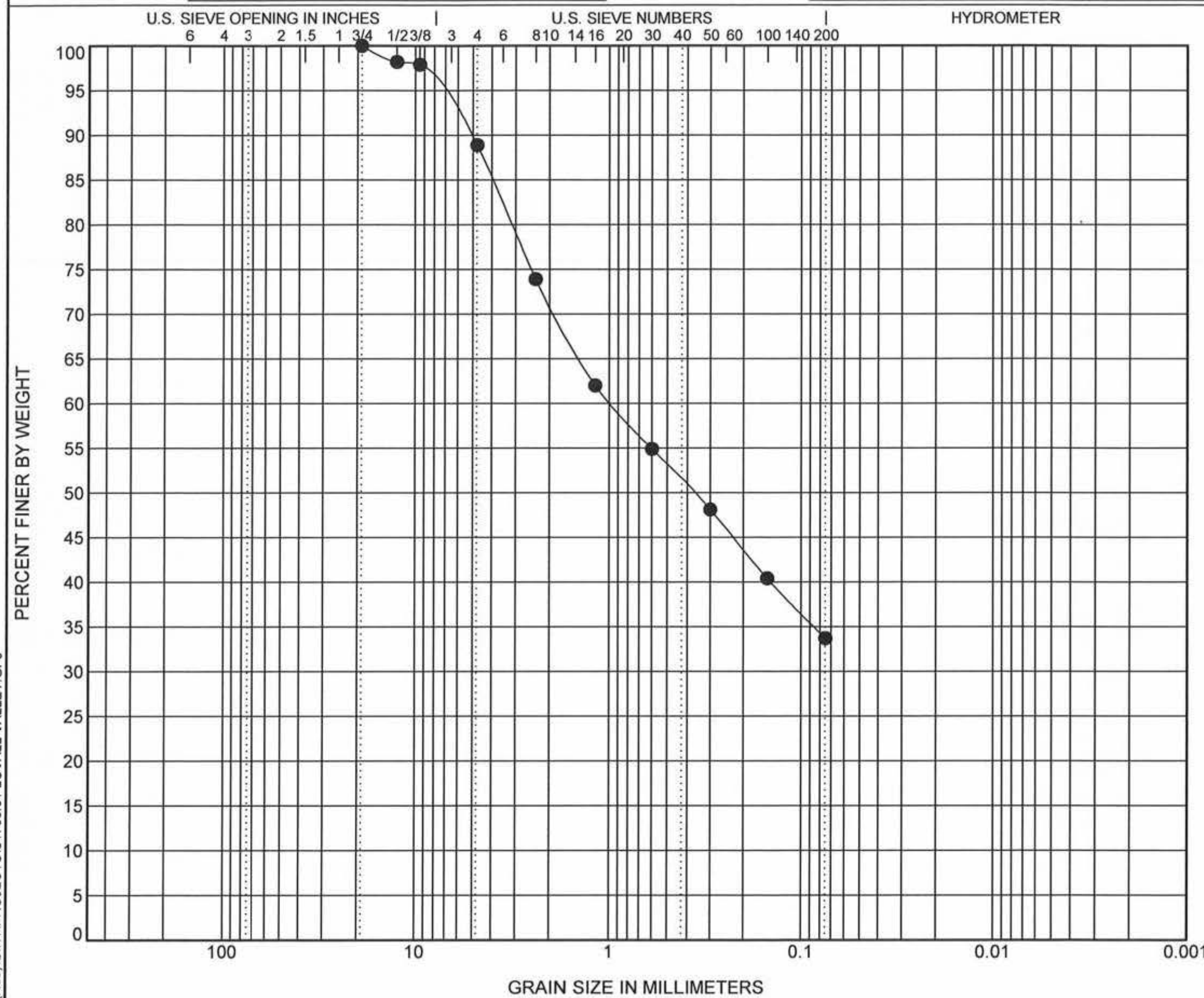
GRAIN SIZE DISTRIBUTION

PLATE 17

CLIENT JANET MCLAUGHLIN **PROJECT NAME** PROPOSED MCLAUGHLIN RESIDENCE & DRIVEWAY

PROJECT NUMBER S1755.01 **PROJECT LOCATION** LOVALL VALLEY ROAD; SONOMA CA



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

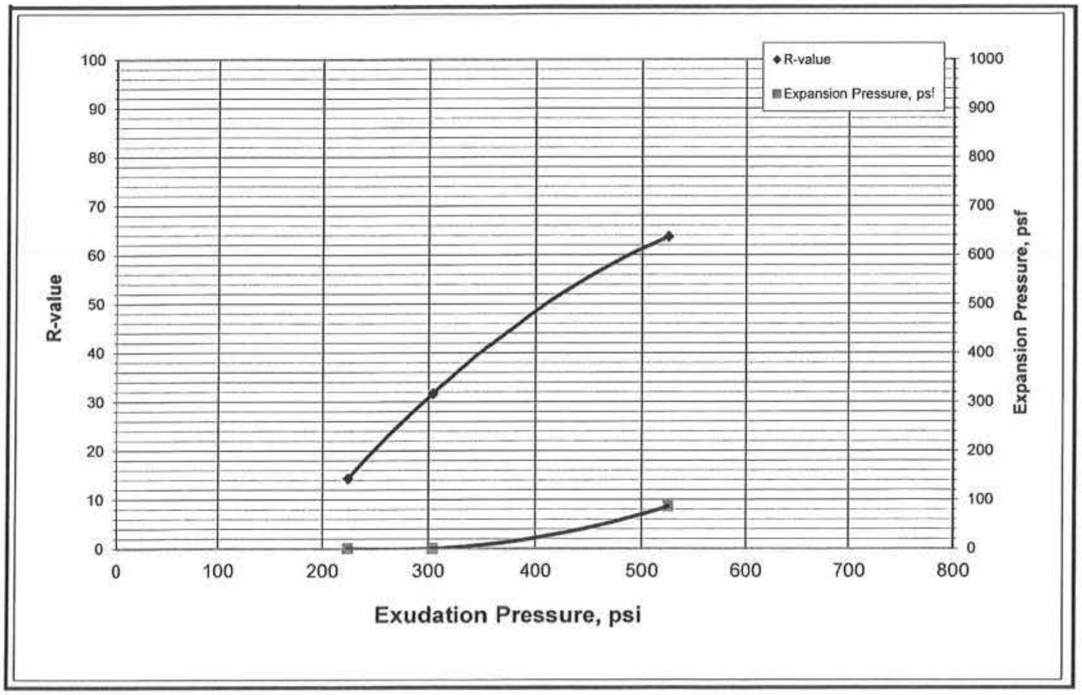
Specimen Identification		Classification	LL	PL	PI	Cc	Cu
● TP-3	1.5	**GRAYISH BROWN CLAYEY SAND (SC)**					

Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
● TP-3	1.5	19	0.975			11.1	55.2	33.7	

GRAIN SIZE - GINT STD US LAB.GDT - 4/24/19 13:42 - C:\PROGRAM FILES (X86)\GINT\PROJECTS\S1755.01 LOVALL VALLEY.GPJ

RESISTANCE VALUE TEST RESULTS

SAMPLE NO. 1



SAMPLE DESCRIPTION :	BULK – COMPOSITE SAMPLE GRAYISH BROWN SANDY SILT (ML)		
Specimen	**A**	**B**	**C**
Exudation Pressure, psi	224	304	525
Expansion Dial (0.0001")	0	0	0
Expansion Pressure, psf	0	0	86
Resistance Value, "R"	14	32	64
% Moisture at Test	16.7	15.3	14.0
Dry Density at Test, pcf	106.6	108.3	111.7
"R" Value at 300 psi, Exudation Pressure	31		
Expansion Pressure at 300 psi, Exudation Pressure	0		

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 Sonoma, California Quadrangle 7.5-Minute Topographic Map, dated 1980.

4. Geologic Map of the Sonoma 7.5-Minute Quadrangle, Sonoma and Napa Counties, California, by David L. Wagner, Kevin B. Clahan, Carolyn E. Randolph-Loar, and Janet M. Snowers, 2007.

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