March 6, 2023

Job No. S2192.01

Mary & Terry MacRae
2525 North Pearl Street
Dallas, TX 75201

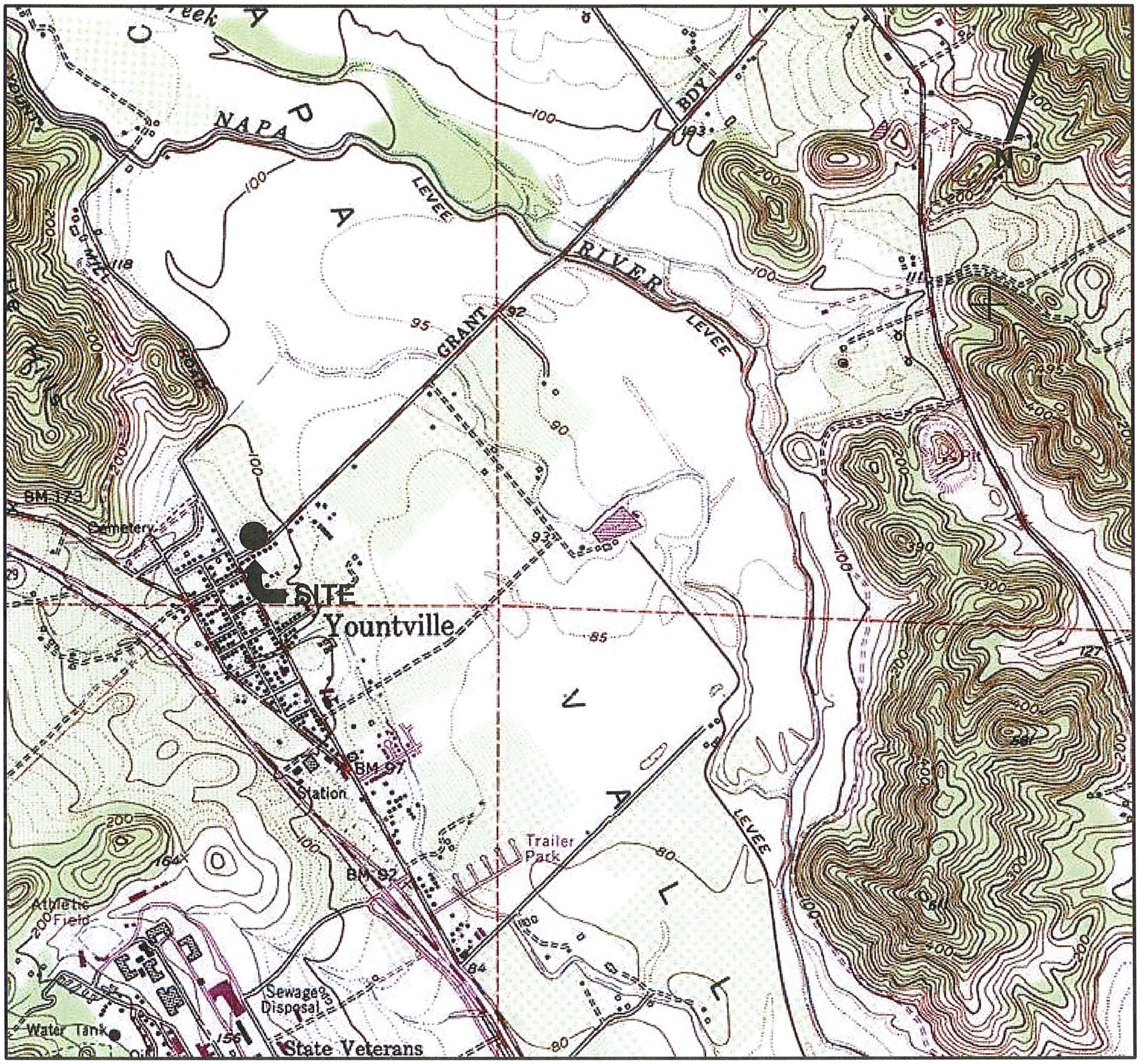
Subject: Geotechnical Investigation
Proposed Residential Development
1980 Yountville Cross Road
Yountville, California

PJC & Associates, Inc. (PJC) is pleased to submit this report which presents the results of our geotechnical investigation for the proposed residential development located at 1980 Yountville Cross Road in Yountville, 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 November 17, 2022. 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 and preliminary site plans provided by your representative, Ms. Kirsty L. Shelton, it is our understanding that the project will consist of subdividing the existing parcel into nine separate lots, relocating the existing residence, and constructing new single-family residences with an attached garage on each of the newly created lots. Additionally, accessory dwelling units (ADUs) may also be constructed on three of the lots. We anticipate that the buildings will consist of one or two story, wood frame structures with raised wood or concrete slab-on-grade floors.

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



SCALE 1:24,000

REFERENCE: USGS YOUNTVILLE CALIFORNIA QUADRANGLE, DATED 1978.

PJC & Associates, Inc. Consulting Engineers & Geologists	SITE LOCATION MAP PROPOSED RESIDENTIAL DEVELOPMENT 1980 YOUNTVILLE CROSS ROAD YOUNTVILLE, CALIFORNIA	PLATE 1
	Proj. No: S2192.01 Date: 3/23 App'd by: AJD	

Grading plans were not available at the time of this report. However, we anticipate that the structures will likely be constructed at or near existing grade. Therefore, site grading will probably consist of minor cuts and fills on the order of three feet and less to achieve the desired grades and to provide adequate site drainage.

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. Drill seven exploratory boreholes to depths between two and nine feet below the existing ground surface to observe the soil, bedrock and groundwater conditions. Our engineering geologist was on site to observe the drilling, log the materials encountered in the boreholes 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, lateral earth pressures, slab-on-grade recommendations, asphalt design criteria, site drainage, and construction considerations.

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

3. SITE CONDITIONS

a. General: The site is located in a rural residential and agricultural area of Yountville, approximately 300 feet northeast of the intersection of Yount Street and Yountville Cross Road. The site is bordered by single-family residences and vineyards to the east, high density housing to the north and west, and Yountville Cross Road to the south. At the time of our investigation, the site was occupied by an existing single-family residence, detached garage, barn structure, guest house and landscape areas. The remaining portions of the site was covered in perennial grasses and scattered trees.

b. Topography and Drainage: The site is located on nearly level to gently sloping topography. According to the United States Geological Survey (USGS) Yountville, California, 7.5 Minute Quadrangle Map (Topographic), the site is situated near an elevation of 120 feet above mean sea level (MSL). No creeks or seasonal drainage swales pass through the site. The site drainage generally consists of sheet flow and surface infiltration, and is provided by municipally maintained storm drain facilities. Regional drainage is provided by the Napa River which is located approximately one mile northeast of the site.

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 soils underlying the site comprise Pleistocene fan deposits (Q_{pf}). These deposits are described as consisting of sand, gravel, silt and clay that is moderately to poorly sorted and bedded. However, during our exploration, we encountered bedrock deposits indicative of the Sonoma Volcanics 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, we encountered agglomerate and lithic tuff bedrock indicative of the T_{svt} subunit. The T_{svt} subunit is described as light colored ash flow tuff, and lithic rich in places. Furthermore, according to the geologic mapping, deposits of the T_{svt} subunit are mapped approximately one-quarter mile northwest of the site. Therefore, based on our subsurface exploration, and the close proximity to the mapped bedrock deposits, we judge that the site is likely underlain by bedrock deposits of the T_{svt} subunit of the Sonoma Volcanics Series.

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, Hunting Creek and Green Valley faults. The West Napa fault is located less than one mile to the southwest, the Hunting Creek fault is located nine miles northeast and the Green Valley fault is located ten miles southeast 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	<1	6.7
Hunting Creek	9	7.1
Green Valley	10	6.8

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. Soils & Bedrock. The subsurface conditions of the site were investigated by drilling seven exploratory boreholes (BH-1 through BH-7) near the proposed structures to depths between two and nine feet below the existing ground surface. The approximate borehole locations are shown on the Borehole Location Plan, Plate 3. The test pits were used to perform standard penetration tests (SPT), observe the soil, bedrock and groundwater conditions, and obtain samples for visual examination and laboratory testing. The drilling and sampling procedures, and descriptive borehole logs are included in Appendix A of this report. The laboratory procedures are presented in Appendix B.

The exploratory boreholes generally encountered alluvial and residual soil deposits, underlain by bedrock deposits of the Sonoma Volcanics Series, that extended to the maximum depths explored. At the surface of BH-7, our exploration encountered aggregate base rock extending to a depth of one foot below the existing ground surface. The aggregate base rock appeared moist, moderately compacted and fine to coarse grained. At the surface of other borings, our exploration encountered young alluvial deposits consisting of sandy silts that extended to depths between one-half and two feet below the existing ground surface. The surface soils appeared very moist to wet, soft to very stiff and exhibited low plasticity characteristics. The surface soils at BH-2, BH-3, BH-4, and BH-7 were underlain by older alluvial deposits consisting of sandy silts that extended to depths between one and one-half and three feet below the existing ground surface. The older alluvial silts appeared very moist, medium stiff to hard and exhibited high plasticity characteristics. Underlying the alluvial soils at BH-1 through BH-5 and BH-7, our exploration encountered residual soils consisting of sandy silts that extended to depths between two and five and one-half feet below the existing ground surface. The residual soils appeared very moist to wet, stiff to hard, and exhibited medium plasticity characteristics. The alluvial and residual soils were underlain by agglomerate and tuff bedrock deposits of the Sonoma Volcanics Series that extended to the maximum depths explored. The bedrock appeared soft to slightly hard, friable to weak and highly weathered.

b. Groundwater. Groundwater was encountered in BH-1 at a depth of three feet below the existing ground surface during our subsurface exploration on January 19, 2023. Groundwater was not encountered in the other boreholes. 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. Furthermore, based on our experience at the adjacent parcel, the groundwater flow can be significant in the winter and early spring.

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. According to recent geologic mapping, the West Napa fault is located approximately one-quarter mile southwest of the site. The current theory is that movement along a fault follows the trace of the most recent break. This is not always the case. The recent seismic event of the South Napa Earthquake along the West Napa fault has resulted in significant surface ruptures and a northward shift of the West Napa fault by two inches. Furthermore, the surface ruptures were not limited to known fault traces of the West Napa fault. According to recently released data from the United States Geological Survey (USGS) and the California Geological Survey (CGS), fault ruptures occurred on previously unknown and unmapped faults in the west Napa area, approximately one-half mile east of the West Napa fault trace. The project site is located within a seismically active, tectonic area where new, unknown, and previously dormant fault traces can illicit surface fault rupture during high magnitude seismic events. We did not observe signs of fault rupture at the site. However, due to the close proximity to the fault, and recent surface ruptures along previously unknown fault traces near the West Napa fault, it should be considered that the risk of future ground rupture at the site is moderate.

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 within four and one-half feet of the ground surface at the site. Furthermore, the site is underlain by relatively shallow bedrock. Therefore, it is judged that liquefaction is not likely to occur at the site.

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 site. Therefore, we judge that the potential for lateral spreading and lurching at the site is low.

e. Expansive Soils. Based on our visual observations and Atterberg Limits testing (PI=12), the surface soils at the site are judged to generally have a low expansion potential. However, near surface clay soils at the site are judged to have a moderate (PI=18 & 19) to high (PI=29) expansion potential.

9. CORROSIVE SOILS

PJC performed laboratory testing to determine the corrosive potential of the native soils. Specifically, the pH, resistivity and chloride and sulfate contents were determined from a composite sample obtained during our field investigation. The results of the testing are summarized in Table 2 below. Based on the results of our laboratory testing, the native soils at the site are considered strongly acidic. Lime or mild cement treatment of native soils for use as engineered fill could be of some significant benefit to steel longevity due to the highly acidic nature of the native soils. Otherwise, to increase steel longevity in the soil would require upgrading and/or other actions. This should be accounted for in design of the project. The resistivity is fair (>1,328 ohm-cm), the chloride and the sulfate contents are low and should not have a corrosion impact on concrete steel reinforcement, concrete, grout, mortar or cement.

TABLE 2
RESULTS OF CORROSION TESTING

Type of Testing	Results	Required*
pH Determination	4.9	>5.5
Minimum Resistivity, ohm-cm	1,328	>1000
Chloride Content, ppm	200	<500
Sulfate Content, ppm	54	<2000

***Based on applicable Caltrans standards**

10. CONCLUSIONS

Based on our field and office studies, we judge that from a geotechnical engineering standpoint, the project is feasible provided the recommendations presented in this report are incorporated into the design and carried out through construction. The primary geotechnical concerns in design and construction of the project is the presence of weak and compressible alluvial soils, and weak, compressible, and expansive near surface soils.

The surface and near surface alluvial 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. Furthermore, based on our visual observations and laboratory testing (PI=18, 19 & 29), the near surface soils have a moderate to high expansion potential. Shrinking and/or swelling of these materials due to loss or increase of moisture content can cause irregular and excessive ground movement and distress and damage to foundations. Underlying the unsuitable surface and near surface soils are bedrock deposits suitable for foundation support. Therefore, if raised wood floors are desired in living areas, it will be necessary to extend the foundations through the unsuitable surface and near surface soils, through the zone of significant moisture variation and into the underlying bedrock. This can be accomplished with a drilled pier and grade beam foundation system.

It is our understanding that slabs-on-grade may also be utilized in living areas for some structures. Furthermore, in order to minimize disturbance to the existing trees, it is desired to avoid significant grading. Therefore, the structures utilizing concrete slab-on-grade floors may be adequately supported on a post-tensioned or conventional mat slab. The mat slabs should be designed to span areas of non-uniform support and resist the effects of the expansive soils. Furthermore, the slabs-on-grade should be provided with underslab drains to prevent hydrostatic uplift and control seepage, as shown on Plate 2.

We do not anticipate the use of conventional concrete slabs-on-grade in living areas. However, conventional concrete slabs-on-grade will be used for the garages. Conventional concrete slabs-on-grade placed on the existing native soils may be subjected to settlement, heave and cracking. If the owner understands and accepts this risk, convention concrete slabs-on-grade may be adequately supported on properly moisture conditioned and compacted native soils in garage areas. If this is not acceptable, the slabs should be structurally designed, or the unsuitable surface and near surface soils should be removed and replaced with at least 24 inches of non-expansive compacted engineered fill. Regardless, any artificial fill soils encountered in slab-on-grade areas 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.

11. EARTHWORK AND GRADING

We anticipate that site grading will consist of minor cuts and fills to achieve pad and roadway grades, and to provide adequate gradients for site drainage. We do not anticipate the placement of significant fill at the site.

a. Stripping. Structural areas should be stripped of surface vegetation, debris, old fills, topsoil containing a significant amount of organic matter (more than three percent by volume), underground utilities, etc. These materials should be moved off site; some of them, if suitable, could be stockpiled for later use in landscape areas. If underground utilities pass through the site, we recommend that these utilities be removed in their entirety or rerouted where they exist outside an imagery plane sloped one horizontal to one vertical (1H:1V) from the outside bottom edge of the nearest foundation. Septic tanks, leach fields, and wells, should be abandoned according to regulations as set forth by the County of Napa Health Department. Voids left by 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 the weak surface soils. The thickness of the weak soils varies across the site and should be determined by the geotechnical engineer in the field during construction. The exposed surface should be scarified to a depth of eight inches, moisture conditioned to at least two percent over optimum moisture content, and compacted to at least 90 percent of the materials relative maximum dry density as determined by ASTM D-1557 test procedures. The excavated on-site material, free of organics and rocks larger than four inches in diameter, may be reused as compacted engineered fill. All fill material should be placed and compacted in accordance with the recommendations presented in Table 3. 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:

Plastic Index	less than 12
Liquid Limit	less than 35
Percent Soil Passing #200 Sieve	between 15% and 35%
Maximum Aggregate Size	4 inches

All fill should be placed in lifts no greater than eight inches in loose thickness and compacted to the general recommendations provided for engineered fill.

TABLE 3
SUMMARY OF COMPACTION RECOMMENDATIONS

Area	Compaction Recommendations*
General Engineered Fill (Import)	In lifts, a maximum of eight inches loose thickness, compact to a minimum of 90 percent relative compaction near optimum moisture content.
General Engineered Fill (Native)	In lifts, a maximum of eight inches loose thickness, compact to 90 percent relative compaction at least two over optimum moisture content.
Trenches** (Import)	Compact to at least 90 percent relative compaction near optimum moisture content.
Parking and Access Driveways (Native)	Compact the top eight inches of subgrade to 95 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
**Depths below finished subgrade elevations

All site preparation and fill placement should be observed by a representative of PJC. It is important that during the stripping, subexcavation and grading/scarifying 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 the 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). Steeper slopes should be retained. Disturbed slopes should be planted with deep rooted groundcover to reduce and control erosion.

12. FOUNDATIONS: DRILLED CAST-IN-PLACE PIERS

a. Vertical Loads. The structures may be supported on a drilled pier and grade beam foundation system. The drilled piers should have a minimum diameter of 12 inches and be spaced at least three pier diameters center to center. The piers will derive their support through peripheral friction. Perimeter and interior piers should extend at least eight feet below the existing ground surface and at least five feet into the underlying bedrock. The piers should be reinforced and designed by the project structural engineer. All piers should be tied together with grade or tie beams.

That portion of the piers extending into the underlying bedrock may be designed using an allowable dead plus live skin friction of 650 pounds per square foot (psf). This value may be increased by one-third for short duration wind and seismic loads. A value equal to one-half the downward capacity of the pier may be used to resist uplift forces. An uplift swelling pressure of 1,500 psf should be used for the design of the grade beams. End bearing should be neglected because of difficulty in cleaning out small diameter pier holes and the uncertainty of mobilizing skin friction and end bearing simultaneously.

b. Lateral Loads. Lateral loads resulting from wind or earthquake can be resisted by the pier through a combination of cantilever action and passive resistance of the soils surrounding the pier. A passive equivalent fluid pressure of 350 psf/ft acting on two pier diameters should be used. The soils overlying the bedrock should be neglected for passive resistance.

c. Settlement. The maximum and differential settlements of the piers is estimated to be small and within tolerable limits.

If groundwater is encountered, it may be necessary to de-water the holes and/or place concrete by the tremie method. Hard drilling may be required to achieve the required penetration depths of the piers. If caving soils are encountered, it may be necessary to case the holes.

13. MAT FOUNDATIONS

It is our understanding that slabs-on-grade may also be utilized in living areas for some structures. Furthermore, in order to minimize disturbance to the existing trees, it is desired to avoid significant grading. Therefore, the structures utilizing concrete slab-on-grade floors may be adequately supported on a post-tensioned or conventional mat slab. The mat slabs should be designed to span areas of non-uniform support and resist the effects of the expansive soils.

a. Vertical Loads. The mat slab should be designed to be rigid and capable of resisting both positive and negative moments in areas of potential heave and non-uniform support due to soil shrinkage or consolidation. The slab subgrade should be moisture conditioned to at least three percent over optimum. To aid in providing uniform support, the mat should be supported on a six-inch layer of compacted gravel at least ¾ to 1½ inches in size. We recommend that the gravel be placed as soon as possible after preparation of the subgrade soils to prevent drying of the subgrade soils. The slabs should be designed according to the following criteria

Edge Moisture Variation Distance (center lift) = 7.5 feet
Edge Moisture Variation Distance (edge lift) = 5 feet
Estimated Differential Swell (center lift) = -1.02 inches

Estimated Differential Swell (edge lift) = 0.78 inches
Allowable Bearing Capacity = 1500 psf

b. Lateral Loads. Lateral loads resulting from wind or seismic forces may be resisted in the form of base friction between the base of the mat and the soil which it is supported. A friction factor of 0.25 is considered appropriate between the bottom of the concrete structures and the subgrade soils.

We recommend a minimum slab thickness of 10 inches. To minimize moisture propagation through the slab, the rock should be covered by a layer of a 10 mil thick impermeable membrane. Furthermore, the slabs-on-grade should be provided with underslab drains to prevent hydrostatic uplift and control seepage, as shown on Plate 2.

14. CONVENTIONAL SLAB-ON-GRADE

Conventional slabs-on-grade will not be used in the living areas, but will be used for the garages. The garage slabs may be supported on the weak and expansive native soils provided the owner understands that the slab could be prone to settlement, heave and cracking. If optimum slab performance is desired, the slab should be structurally designed, or the unsuitable surface and near surface soils should be removed and replaced with at least 24 inches of compacted non-expansive engineered fill. Regardless, slab subgrade should be moisture conditioned to over optimum moisture content and rolled to provide a firm and unyielding subgrade. Furthermore, any artificial fill soils encountered in slab-on-grade areas should be subexcavated and recompacted.

Slabs-on-grade should be at least five inches thick and underlain by a six inch layer of compacted clean gravel or crushed rock. The rock will serve as a capillary break; however moisture may accumulate in the base course. Therefore, a plastic vapor barrier of at least ten mil thickness should be provided over the rock where moisture protection is desired. To control cracking, the slabs should be reinforced as determined by the project structural engineer. The garage slabs should be carefully separated from foundations with felt paper or other positive and low friction material. Furthermore, the slabs-on-grade should be provided with underslab drains to prevent hydrostatic uplift and control seepage, as shown on Plate 2.

15. ASPHALTIC CONCRETE PAVEMENTS

Based on our investigation, the surface and near surface soils have a low supporting capacity (after properly compacted) when used as a pavement subgrade. Based on our laboratory testing, an R-value of 11 was determined and used in asphaltic concrete pavement design calculations. Pavement design sections are presented in Table 4.

Based on our visual observations, previous experience with similar soils on nearby projects, and Atterberg limits testing (PI=12, 18, 19, 23 & 29), the surface and near surface soils at the site are judged to have a low to high expansion potential. Asphaltic concrete pavements may be supported directly on properly moisture conditioned and compacted expansive native soils, if the owner understands and accepts that periodic maintenance, including repair of edge cracking, will probably be required. Future maintenance of pavement areas could be significantly reduced by placing imported select fill under the aggregate base. The thickness of the select import would generally be at least 18 inches. The lateral extant of import should extend at least three feet beyond the edges of asphaltic concrete pavements.

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 at least two 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 Napa County 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 D1557-91. Aggregate used for the base coarse 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 geotechnical 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.

Several precautionary measures can be taken to minimize the intrusion of water into the base rock; however, the cost to install the protective measures should be balanced against the cost of repairing damaged pavement sections. The following are alternatives which can be taken to extend the life of the pavement.

a. Extend the concrete curb at least six inches below the bottom of the base rock layer.

b. Construct a concrete cutoff wall along the perimeter edge of the pavement. The wall should consist of a lean concrete mix. The trench should be four inches wide and extend at least 36 inches deep.

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 4
PAVEMENT DESIGN FOR PAVEMENT AREAS
(Subgrade R-Value=11)

Traffic Index	Asphaltic Concrete (in)	Class 2 Aggregate Base (in)
4.0	3.0	6.0
5.0	3.0	8.5
6.0	3.0	12.5
7.0	3.5	15.0

16. 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 soil surcharges are located near the top of the excavation. Where trenches extend deeper than five feet, the excavation may become unstable. 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.

We recommend trenches be backfilled with granular import fill and compacted to at least 90 percent of maximum dry density. The moisture content of compacted backfill soils should be within two percent of optimum moisture content. Jetting should not be used.

Special care should be taken in the control of utility trench backfilling in pavement areas and slab-on-grade areas. Poor compaction may cause excessive settlements resulting in damage to the pavements and concrete slabs-on-grade. In pavement areas, the top eight inches of trench backfill should be compacted to at least 95 percent relative compaction.

17. 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. Due to the presence of shallow bedrock, we recommend using a site class type C, spectral accelerations of S_S of 1.858 g and S_1 of 0.656 g are recommended.

18. 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 subdrain should consist of a heavy walled four-inch diameter perforated pipe. 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 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 slabs-on-grade should be provided with underslab drains to prevent hydrostatic uplift and control seepage, as shown on Plate 2.

Should the foundation subdrains prove to be problematic in the effort to protect the sensitive trees near proposed new structures, PJC may be consulted to provide alternative subsurface drainage recommendations, on a case by case basis.

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

19. LIMITATIONS

The data, information, interpretations and recommendations in this report are presented solely as bases and guides for the geotechnical design of the proposed residential development located at 1980 Yountville Cross Road in Yountville, 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.

20. ADDITIONAL SERVICES

Upon completion of the project plans, they should be reviewed by our firm to verify that the design is consistent with the recommendations of this report. During the course of this investigation, several assumptions were made regarding building loads and 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 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 the responsibility for items that they 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.

CERTIFIED ENGINEERING GEOLOGIST
DONALD A. WHYTE
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STATE OF CALIFORNIA

Donald A. Whyte
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REGISTERED PROFESSIONAL ENGINEER
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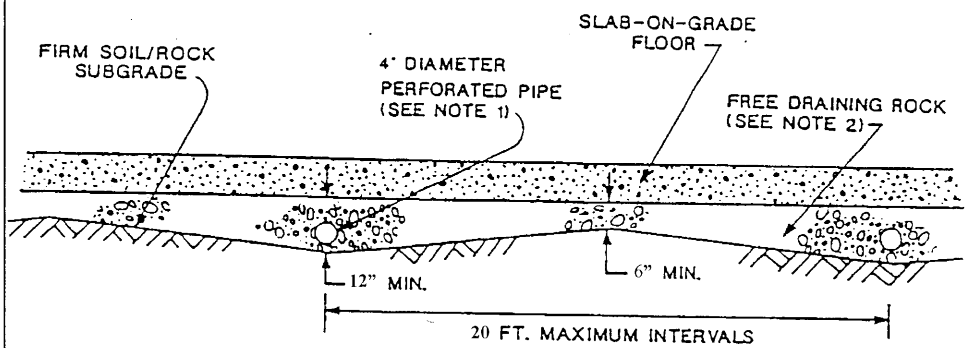


Notes:

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 6" THICK AND A MINIMUM OF 12" 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

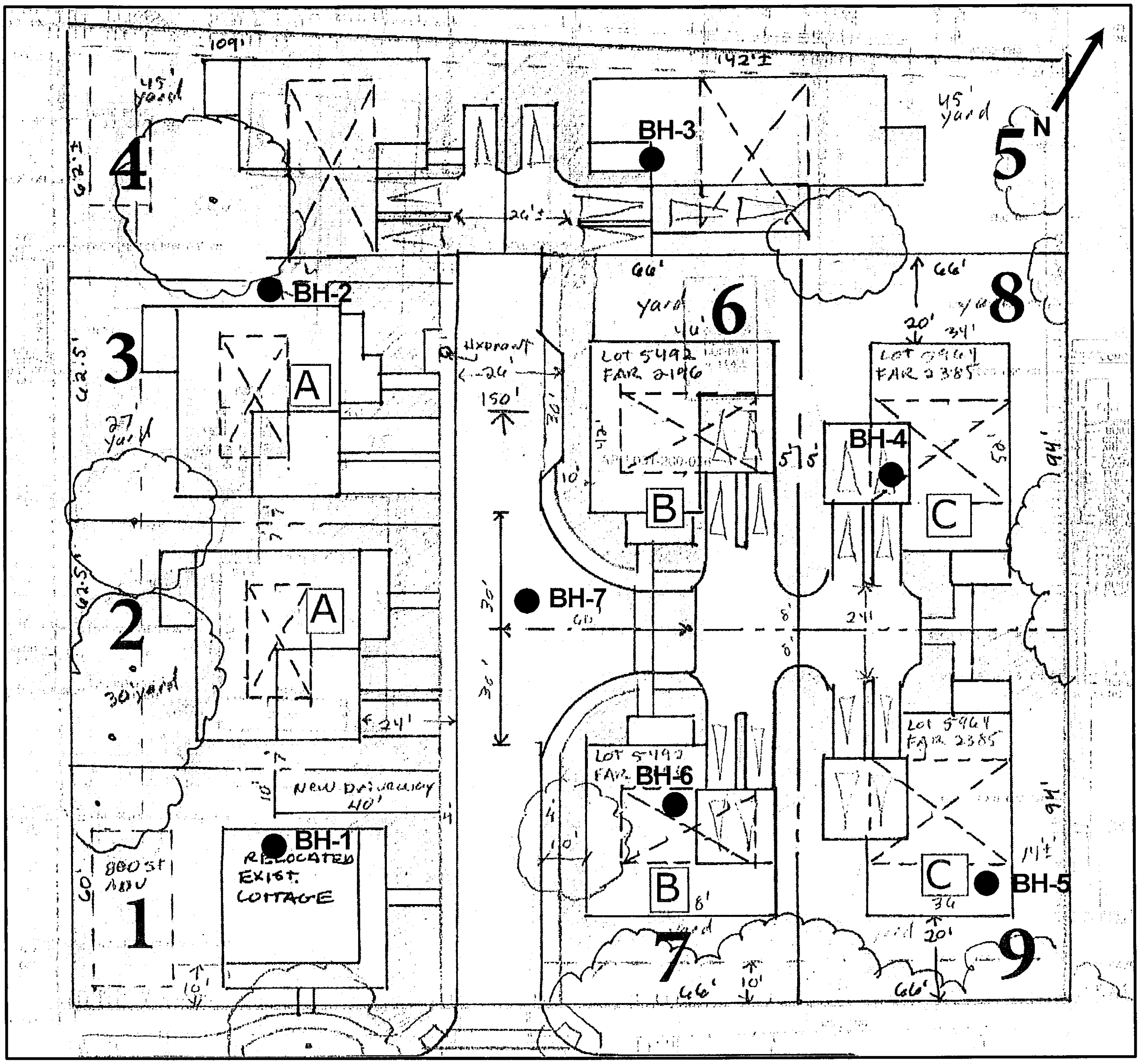
1. INTRODUCTION

The field program performed for this study consisted of drilling seven exploratory boreholes (BH-1 through BH-7) in the vicinity of the proposed structures. The exploration was completed on January 19, 2023. The approximate borehole locations are shown on the Borehole Location Plan, Plate 3. Descriptive logs of the boreholes are presented in this appendix as Plates 4 through 10.

2. BOREHOLES

The boreholes were advanced using a track-mounted drill rig with solid stem flight augers. The drilling was performed under the observation of a engineering geologist of PJC who maintained a continuous log of soil conditions and obtained samples suitable for laboratory testing. The soils were classified in accordance with the Unified Soil Classification System, as explained in Plate 11. The bedrock was classified according to Plate 12.

Relatively undisturbed and disturbed samples were obtained from the exploratory boreholes. A 2.43 in I.D. California Modified Sampler, or a 1.5 in I.D. Standard Sampler, was driven into the underlying soil using a 140-pound hammer falling 30 inches to obtain an indication of the density of the materials and to allow visual examination of at least a portion of the soil column. Samples obtained with the split-spoon sampler were retained for further observation and testing. The number of blows required to drive the sampler at six-inch increments was recorded on each borehole log. All samples collected were labeled and transported to PJC's office for examination and laboratory testing.



EXPLANATION

● BOREHOLE LOCATION AND DESIGNATION

NO SCALE

REFERENCE: SITE PLAN PREPARED BY K. AUSTIN, AIA ARCHITECT, DATED NOVEMBER 15, 2022.

PJC & Associates, Inc.
Consulting Engineers & Geologists

BOREHOLE LOCATION PLAN
PROPOSED RESIDENTIAL DEVELOPMENT
1980 YOUNTVILLE CROSS ROAD
YOUNTVILLE, CALIFORNIA

PLATE 3

Proj. No:S2192.01 Date:3/23 App'd by: AJD

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BORING NUMBER BH-1; PLATE 4

CLIENT MARY & TERRY MACRAE
PROJECT NAME PROPOSED RESIDENTIAL DEVELOPMENT
PROJECT NUMBER S2192.01
PROJECT LOCATION 1980 YOUNTVILLE CROSS ROAD, YOUNTVILLE, CA
DATE STARTED 1/19/23 COMPLETED 1/19/23
GROUND ELEVATION HOLE SIZE 6"
DRILLING CONTRACTOR Pearson Drilling
GROUND WATER LEVELS:
DRILLING METHOD TRACK MOUNTED DRILL RIG
AT TIME OF DRILLING 3.00 ft
LOGGED BY D.W. CHECKED BY
AT END OF DRILLING ---
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		0.0-2.0'; SANDY SILT (ML); light brown, very moist, soft to medium stiff, low plasticity. (ALLUVIUM)										
			MC		5-12 (17)	1.0	88	28				
2		2.0-3.0'; SANDY SILT (MH); yellow brown and orange brown mottling, very moist to wet, stiff, medium plasticity. (RESIDUAL SOIL)				2.0	81	33				
3		3.0-5.5'; AGGLOMERATE; yellow brown and orange brown mottling, slightly hard, friable to weak, highly weathered. (SONOMA VOLCANICS)										
4			MC		40-50/4"		64	28				
5			SPT		54/6"			19				

REFUSAL AT 5.5'
Bottom of borehole at 5.5 feet.

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BORING NUMBER BH-2; PLATE 5

CLIENT MARY & TERRY MACRAE
PROJECT NAME PROPOSED RESIDENTIAL DEVELOPMENT
PROJECT NUMBER S2192.01
PROJECT LOCATION 1980 YOUNTVILLE CROSS ROAD, YOUNTVILLE, CA
DATE STARTED 1/19/23 COMPLETED 1/19/23
GROUND ELEVATION HOLE SIZE 6"
DRILLING CONTRACTOR Pearson Drilling
GROUND WATER LEVELS:
DRILLING METHOD TRACK MOUNTED DRILL RIG
AT TIME OF DRILLING ---
LOGGED BY D.W. CHECKED BY
AT END OF DRILLING ---
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		0.0-1.5'; SANDY SILT (ML); brown, very moist to wet, soft to medium stiff, low plasticity. (ALLUVIUM)										
			MC		6-6 (12)	1.0	86	31	38	26	12	
		1.5-3.0'; SANDY SILT (MH); yellow brown, very moist, medium stiff to stiff, high plasticity, with few gravels. (OLDER ALLUVIUM)				1.0	96	29				
2.5												
		3.0-5.5'; SANDY SILT (ML); dark yellow brown and orange brown mottling, very moist, hard, medium plasticity. (RESIDUAL SOIL)	MC		23-28 (51)	3.5	91	29				
5.0												
		5.5-9.0'; TUFF; orange brown, soft, friable, highly weathered. (SONOMA VOLCANICS)	SPT		34-25/3"			20				
7.5												
			SPT		48			21				

TERMINATED AT 9.0'

Bottom of borehole at 9.0 feet.

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BORING NUMBER BH-3; PLATE 6

CLIENT MARY & TERRY MACRAE
PROJECT NAME PROPOSED RESIDENTIAL DEVELOPMENT
PROJECT NUMBER S2192.01
PROJECT LOCATION 1980 YOUNTVILLE CROSS ROAD, YOUNTVILLE, CA
DATE STARTED 1/19/23 **COMPLETED** 1/19/23
GROUND ELEVATION **HOLE SIZE** 6"
DRILLING CONTRACTOR Pearson Drilling
GROUND WATER LEVELS:
DRILLING METHOD TRACK MOUNTED DRILL RIG
AT TIME OF DRILLING ---
LOGGED BY D.W. **CHECKED BY**
AT END OF DRILLING ---
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		0.0-1.5'; SANDY SILT (ML); brown, very moist, soft to medium stiff, low plasticity. (ALLUVIUM)										
1												
		1.5-3.0'; SANDY SILT (MH); yellow brown, very moist, very stiff to hard, high plasticity. (OLDER ALLUVIUM)										
2												
			MC		29-25/2"	4.5+	88	32				
3		3.0-4.0'; SANDY SILT (ML); orange brown and olive gray mottling, very moist, very stiff to hard, medium plasticity. (RESIDUAL SOIL)				4.5+	89	32	48	30	18	
4		4.0-5.0'; TUFF; medium brown, soft, friable, highly weathered. (SONOMA VOLCANICS)										
5			SPT		48			23				

TERMINATED AT 5.0'

Bottom of borehole at 5.0 feet.

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BORING NUMBER BH-4; PLATE 7

CLIENT MARY & TERRY MACRAE
PROJECT NAME PROPOSED RESIDENTIAL DEVELOPMENT
PROJECT NUMBER S2192.01
PROJECT LOCATION 1980 YOUNTVILLE CROSS ROAD, YOUNTVILLE, CA
DATE STARTED 1/19/23 **COMPLETED** 1/19/23
GROUND ELEVATION **HOLE SIZE** 6"
DRILLING CONTRACTOR Pearson Drilling
GROUND WATER LEVELS:
DRILLING METHOD TRACK MOUNTED DRILL RIG
AT TIME OF DRILLING ---
LOGGED BY D.W. **CHECKED BY**
AT END OF DRILLING ---
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		0.0-0.5'; SANDY SILT (ML); brown, very moist, soft, low plasticity. (COLLUVIUM)										
		0.5-1.5'; SANDY CLAY (CH); yellow brown, very moist, stiff to very stiff, high plasticity. (OLDER ALLUVIUM)										
1			AU					31	60	31	29	
		1.5-2.5'; SANDY SILT (ML); medium brown and orange brown mottling, very moist, hard, medium plasticity. (RESIUDAL SOIL)										
2			MC		25/2"	4.5+	93	28				
		2.5-4.5'; TUFF; yellow brown, soft, friable, highly weathered. (SONOMA VOLCANICS)										
3												
4			SPT		50/4"			19				

REFUSAL AT 4.5'

Bottom of borehole at 4.5 feet.

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BORING NUMBER BH-5; PLATE 8

CLIENT MARY & TERRY MACRAE
PROJECT NAME PROPOSED RESIDENTIAL DEVELOPMENT
PROJECT NUMBER S2192.01
PROJECT LOCATION 1980 YOUNTVILLE CROSS ROAD, YOUNTVILLE, CA
DATE STARTED 1/19/23 **COMPLETED** 1/19/23
GROUND ELEVATION **HOLE SIZE** 6"
DRILLING CONTRACTOR Pearson Drilling
GROUND WATER LEVELS:
DRILLING METHOD TRACK MOUNTED DRILL RIG
AT TIME OF DRILLING ---
LOGGED BY D.W. **CHECKED BY**
AT END OF DRILLING ---
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		0.0-1.0'; SANDY SILT (ML); dark brown, very moist, soft, low plasticity. (ALLUVIUM)										
			AU					26				
1		1.0-2.0'; SANDY SILT (MH); light brown with yellow brown, very moist, stiff to hard, medium plasticity, with tuff fragments. (RESIDUAL SOIL)	MC		18-45	3.0	88	29	66	47	19	
2		2.0-3.5'; TUFF; yellow brown and orange brown mottling, soft to slightly hard, friable to weak, highly weathered. (BEDROCK)										
3			SPT		32			33				

TERMINATED AT 3.5'
Bottom of borehole at 3.5 feet.

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BORING NUMBER BH-6; PLATE 9

CLIENT MARY & TERRY MACRAE
PROJECT NAME PROPOSED RESIDENTIAL DEVELOPMENT
PROJECT NUMBER S2192.01
PROJECT LOCATION 1980 YOUNTVILLE CROSS ROAD, YOUNTVILLE, CA
DATE STARTED 1/19/23 **COMPLETED** 1/19/23
GROUND ELEVATION **HOLE SIZE** 6"
DRILLING CONTRACTOR Pearson Drilling
GROUND WATER LEVELS:
DRILLING METHOD TRACK MOUNTED DRILL RIG
AT TIME OF DRILLING ---
LOGGED BY D.W. **CHECKED BY**
AT END OF DRILLING ---
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		0.0-1.0'; SANDY SILT (ML); brown, very moist to wet, medium stiff to very stiff, low plasticity. (COLLUVIUM)										
			MC		24-50/4"	2.5	96	24				
1		1.0-2.0'; TUFF; medium brown and gray, slightly hard, friable to weak, highly weathered. (SONOMA VOLCANICS)						15				
			SPT		50/4"			18				
2												

REFUSAL AT 2.0'
Bottom of borehole at 2.0 feet.

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BORING NUMBER BH-7; PLATE 10

CLIENT MARY & TERRY MACRAE

PROJECT NAME PROPOSED RESIDENTIAL DEVELOPMENT

PROJECT NUMBER S2192.01

PROJECT LOCATION 1980 YOUNTVILLE CROSS ROAD, YOUNTVILLE, CA

DATE STARTED 1/19/23 COMPLETED 1/19/23

GROUND ELEVATION

HOLE SIZE 6"

DRILLING CONTRACTOR Pearson Drilling

GROUND WATER LEVELS:

DRILLING METHOD TRACK MOUNTED DRILL RIG

AT TIME OF DRILLING ---

LOGGED BY D.W. CHECKED BY

AT END OF DRILLING ---

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		0.0-1.0'; SANDY GRAVEL (GW); grayish brown, moist, moderately compacted, fine to coarse grained. (FILL)										
1		1.0-2.5'; SANDY SILT (MH); dark yellow brown with light brown, very moist, stiff, high plasticity. (OLDER ALLUVIUM)										
2												
		2.5-3.5'; SANDY SILT (MH); yellow brown, very moist, very stiff to hard, medium plasticity, with gravels. (RESIDUAL SOIL)										
3												
		3.5-4.0'; TUFF; orange brown and yellow brown, soft to slightly hard, friable, highly weathered. (SONOMA VOLCANICS)										
4												

TERMINATED AT 4.0'
Bottom of borehole at 4.0 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

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 and "undisturbed" 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 borehole 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 and dry density, pocket penetrometer, Atterberg Limits and grain-size distribution testing.

a. <u>Natural Water Content and Dry Density</u>. Natural water content and dry density of the samples were determined on selected undisturbed samples. The samples were extruded, visually classified, trimmed to obtain a smooth flat face, and accurately measured to obtain volume and 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 and dry density calculated. The water content and dry density results are summarized on the borehole logs, Plate 4 through 10.

b. <u>Pocket Penetrometer</u>. Pocket Penetrometer tests were performed on 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 test are indicated on the borehole logs.

c. Atterberg Limits. Liquid and plastic limits were determined on selected samples in accordance with ASTM D4318-83. The results of the limits are summarized on Plate 13.

d. 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 14.

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 15.

4. CORROSION TESTING

The corrosion properties testing consisted of PH, minimum resistivity, sulfate content and chloride content determination.

a. PH. The PH of a composite sample of the upper five feet of the on-site soils was determined in accordance with ASTM G 51. The results are presented on Table 2 in the body of this report.

b. Minimum Resistivity. The minimum resistivity of a composite sample of the upper five feet of the on-site soils was determined in accordance with ASTM G 57. The results are presented on Table 2 in the body of this report.

c. Sulfate Content. The sulfate content of a composite sample of the upper five feet of the on-site soils was determined in accordance with ASTM D 516. The results are presented on Table 2 in the body of this report.

d. Chloride Content. The chloride content of a composite sample of the upper five feet of the on-site soils was determined in accordance with ASTM D 512. The results are presented on Table 2 in the body of this report.

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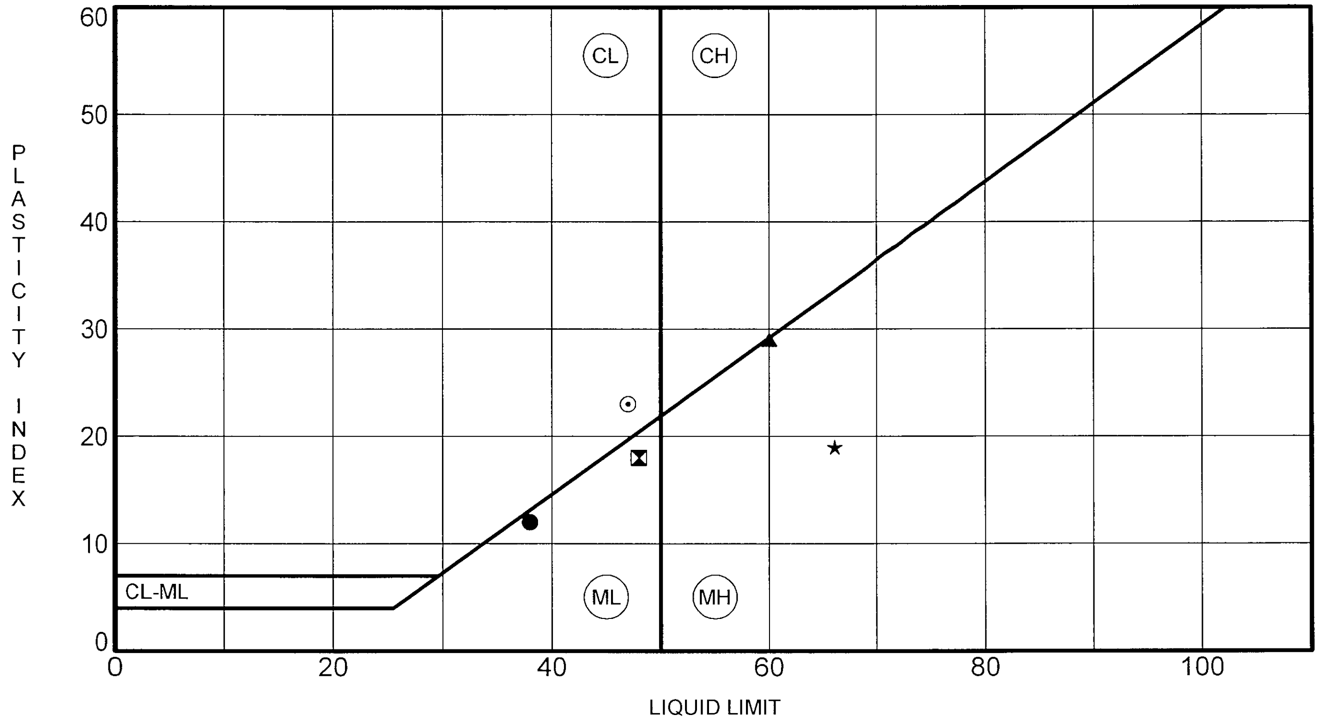
ATTERBERG LIMITS' RESULTS

PLATE 13

CLIENT MARY & TERRY MACRAE **PROJECT NAME** PROPOSED RESIDENTIAL DEVELOPMENT

PROJECT NUMBER S2192.01 **PROJECT LOCATION** 1980 YOUNTVILLE CROSS ROAD, YOUNTVILLE, CA



	Specimen Identification		LL	PL	PI	Fines	Classification
●	BH-2	1.0	38	26	12		BROWN SANDY SILT (ML)
⊠	BH-3	3.0	48	30	18		ORANGE BROWN & GRAY SANDY SILT (ML)
▲	BH-4	1.0	60	31	29		YELLOW BROWN SANDY SILT (MH)
★	BH-5	1.5	66	47	19		LIGHT BROWN SANDY SILT (MH)
⊙	BULK 1	0.0	47	24	23	56	BROWN SANDY CLAY (CL)

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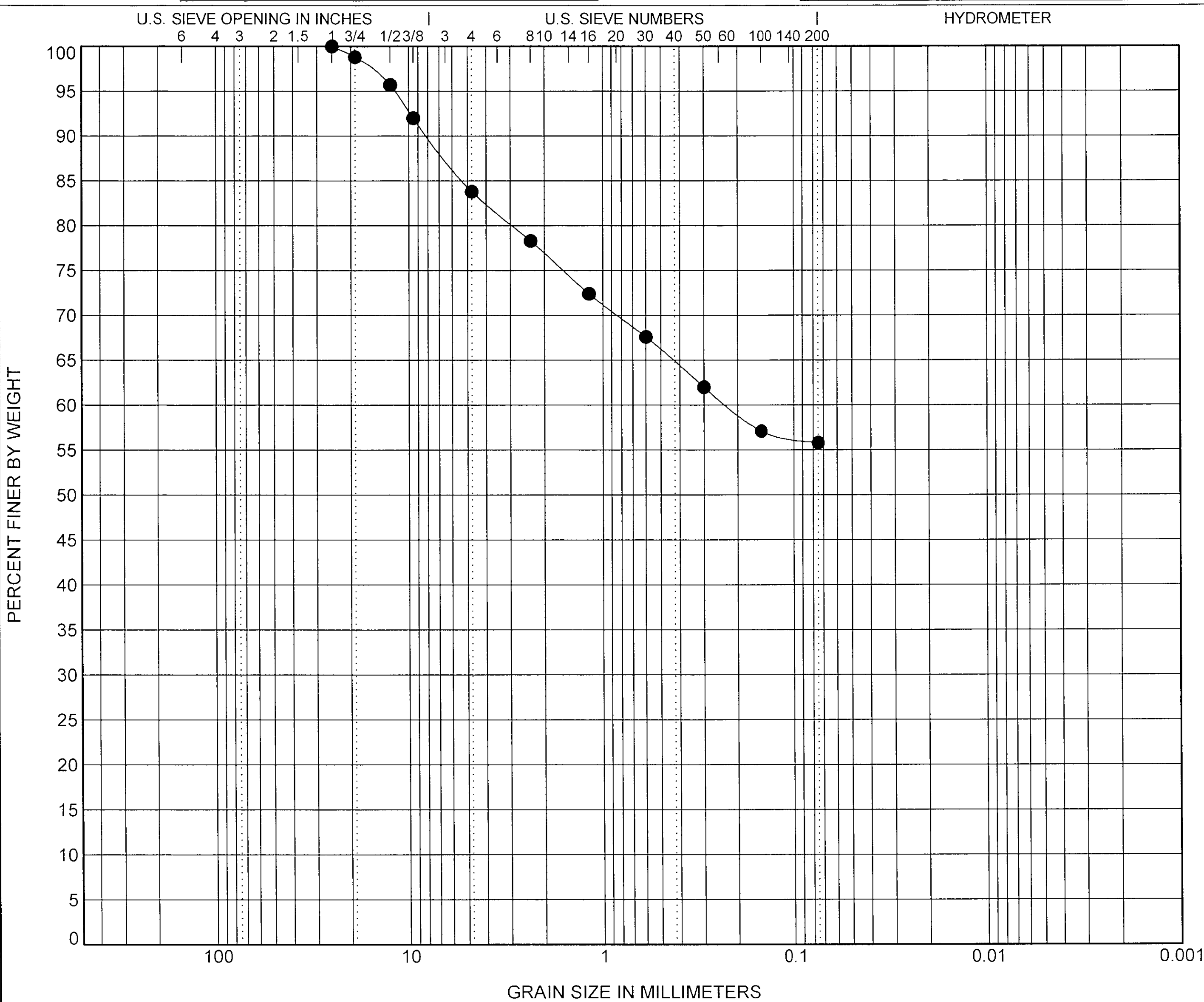
GRAIN SIZE DISTRIBUTION

PLATE 14

CLIENT MARY & TERRY MACRAE **PROJECT NAME** PROPOSED RESIDENTIAL DEVELOPMENT

PROJECT NUMBER S2192.01 **PROJECT LOCATION** 1980 YOUNTVILLE CROSS ROAD, YOUNTVILLE, CA



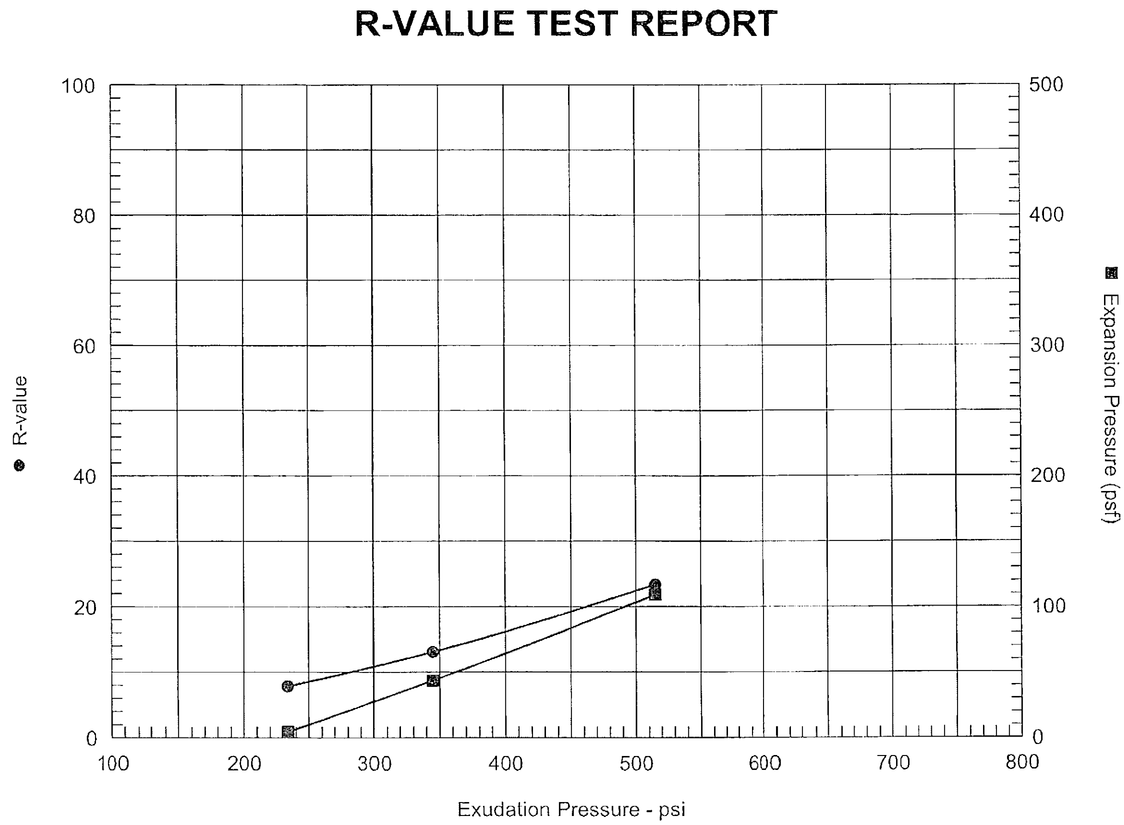
COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Specimen Identification		Classification	LL	PL	PI	Cc	Cu
●	**BULK 1**	**0.0**	**BROWN SANDY CLAY (CL)**	**47**	**24**	**23**		

	Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**BULK 1**	**0.0**	**25**	**0.226**			**16.2**	**28.0**	**55.8**	

RESISTANCE VALUE TEST RESULTS

SAMPLE NO. 1



SAMPLE DESCRIPTION:	BULK – BROWN SANDY CLAY (CL)		
Specimen	**A**	**B**	**C**
Exudation Pressure, psi	234	345	515
Expansion Dial (0.0001")	1	10	25
Expansion Pressure, psf	4	44	109
Resistance Value, "R"	8	13	23
% Moisture at Test	21.3	20.4	18.3
Dry Density at Test, pcf	104.1	105.6	110.3
"R" Value at 300 psi, Exudation Pressure	11		
Expansion Pressure at 300 psi, Exudation Pressure (psf)	28		

APPENDIX C
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