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GEOTECHNICAL STUDY REPORT

6204 DRY CREEK ROAD RESIDENTIAL PROJECT
6204 DRY CREEK ROAD
NAPA, CALIFORNIA

Project Number:

7362.01.04.2

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INTRODUCTION

This report presents the results of our geotechnical study for the residential project to be constructed at 6204 Dry Creek Road in Napa, California. The undeveloped parcel extends over relatively level to very steeply sloping terrain. We previously performed a preliminary geotechnical/geologic assessment of the property and presented the results in a letter dated October 25, 2019. The site location is shown on Plate 1, Appendix A.

We understand it is proposed to construct a residential project on the property. We assume the project will include a main residence, a garage, an accessory dwelling unit, a swimming pool and septic areas. Retaining walls may be needed to provide level breaks across the building areas. Actual foundation loads are not known at this time. We anticipate the loads will be typical for the light type of construction planned.

Grading plans are not available, but we anticipate that the planned grading will be the minimum amount needed to construct level building pads and provide the building site and driveway areas with positive drainage, and could include cuts and fills on the order of 5 to 10 feet. Utility plans are not available, but we have assumed for this study that the project utilities will extend no deeper than 5 feet below the existing ground surface. If project utilities extend deeper, supplemental exploration may be required to evaluate the soil and bedrock conditions within and below the utility excavations.

SCOPE

The purpose of our study, as outlined in our Professional Service Agreement dated October 10, 2018, was to generate geotechnical information for the design and construction of the project. Our scope of services included reviewing selected published geologic data pertinent to the site; evaluating the subsurface conditions with test pits and laboratory tests; analyzing the field and laboratory data; and presenting this report with the following geotechnical information:

1. A brief description of the soil, bedrock and groundwater conditions observed during our study;

2. A discussion of seismic hazards that may affect the proposed improvements; and

3. Conclusions and recommendations regarding:

    a. Primary geotechnical engineering concerns and mitigating measures, as applicable;

    b. Site preparation and grading including remedial grading of weak, porous, compressible and/or expansive, surface soil and the construction of hillside fills;

    c. Foundation types, design criteria, and estimated settlement behavior;

d. Lateral loads for retaining wall design;

e. Support of concrete slabs-on-grade;

f. Preliminary pavement thickness based on our experience with similar soil and projects;

g. Utility trench backfill;

h. Geotechnical engineering drainage improvements; and

i. Supplemental geotechnical engineering services.

STUDY

Site Exploration

We reviewed our preliminary study of the property and our other geotechnical studies in the vicinity and selected geologic references pertinent to the site. The geologic literature reviewed is listed in Appendix B. On October 17, 2019, we performed a geotechnical reconnaissance of the site and explored the subsurface conditions by excavating nine test pits to depths ranging from about 5 to 12 feet. The test pits were excavated with a track-mounted mini-excavator at the approximate locations shown on the Exploration Plan, Plate 2. The test pit locations were determined approximately by pacing their distance from features shown on the Exploration Plan and should be considered accurate only to the degree implied by the method used. Our engineering geologist located and logged the test pits and obtained samples of the materials encountered for visual examination, classification and laboratory testing.

The logs of the test pits showing the materials encountered, groundwater conditions, and sample depths are presented on Plates 3 through 6. The soil is described in accordance with the Unified Soil Classification System, outlined on Plate 7. Bedrock is described in accordance with Engineering Geology Rock Terms, shown on Plate 8.

The test pit logs show our interpretation of the subsurface soil, bedrock and groundwater conditions on the date and at the locations indicated. Subsurface conditions may vary at other locations and times. Our interpretation is based on visual inspection of soil and bedrock samples, laboratory test results, and interpretation of excavation and sampling resistance. The location of the soil and bedrock boundaries should be considered approximate. The transition between soil and bedrock types may be gradual.

Laboratory Testing

The samples obtained from the test pits were transported to our office and re-examined to verify soil classifications, evaluate characteristics, and assign tests pertinent to our analysis. Selected samples were laboratory tested to determine their classification (Atterberg Limits, percent of silt and clay) and expansion potential (Expansion Index - EI). Results of the classification and EI tests are presented on Plate 9.

SITE CONDITIONS

General

Napa County is located within the California Coast Range geomorphic province. This province is a geologically complex and seismically active region characterized by sub-parallel northwest-trending faults, mountain ranges and valleys. The oldest bedrock units are the Jurassic-Cretaceous Franciscan Complex and Great Valley sequence sediments originally deposited in a marine environment. Subsequently, younger rocks such as the Tertiary-age Sonoma Volcanics group, the Plio-Pleistocene-age Clear Lake Volcanics and sedimentary rocks such as the Guinda, Domengine, Petaluma, Wilson Grove, Cache, Huichica and Glen Ellen formations were deposited throughout the province. Extensive folding and thrust faulting during late Cretaceous through early Tertiary geologic time created complex geologic conditions that underlie the highly varied topography of today. In valleys, the bedrock is covered by thick alluvial soil. The site is located on a steep, north-facing slope descending towards Dry Creek, within the mountain range that forms the southwestern margin of Napa Valley.

Geology

Published geologic maps (Clahan et al., 2005) indicate the property is underlain by Cretaceous-Jurassic Great Valley Sequence sandstone, pebble conglomerate, siltstone, and shale. The slopes and ridgeline above the property are capped with the younger Tertiary Sonoma Volcanics. We did not observe surface outcrops within the property. Large angular cobbles and small boulders that are visible within the drainage ditches and along the road are interpreted to be imported and not “in-place”.

Landslides

Mapping by Clahan et al. (2005) as well as Dwyer (1976) indicate that large-scale landslide complexes blanket the bedrock, as shown on Plate 3 of our Preliminary Study report. Our observations of soil and bedrock within the test pits do indicate the presence of old landslide deposits. The extent and thickness of the landslide debris we observed is discussed in detail in the “Subsurface Conditions” section.

Surface

The undeveloped property extends over a grass-covered and wooded, north-sloping hillside. The property encompasses approximately 200 feet of elevation loss between the upper entrance on Dry Creek Road and the lower property line. Dry Creek flows generally southeast within the valley below. The upper part of the property, accessed from Dry Creek Road, includes a moderately steep descent from the entrance onto a wide and elongated, gently-sloping bench. The slopes descending from the bench toward Dry Creek are very steep. A prominent swale is located below approximately the central portion of the gentle bench.

The surface soil is disturbed by randomly arrayed shrinkage cracks generally associated with expansive soil. Locally, expansive soil shrinks and swells with the weather cycle. The cyclic shrinking and swelling tends to disturb the upper portion of the expansive clay. On sloping terrain, the weak, expansive surface materials undergo a gradual downhill movement known as creep. Soil creep is inherent to hillsides in the area and its force is directly proportional to slope inclination, the soil's plasticity, water content and expansion potential.

Our observations indicate that some chaparral, trees, and brush had been recently cleared from the bench area and the seasonal grasses mowed. Shallow V-ditches have been constructed approximately along contour within the bench, approximately at a break in slope between the bench and slightly steeper area below. A cut and fill dirt road was observed immediately inside the entrance leading to the west, skirting around the bench to descend the steep slopes below in a switchback fashion. Vertical cuts along this road expose sandy clay soils that are undergoing sloughing and erosion. We observed a v-ditch along the inboard side of the road draining off at the switchbacks. The moderately steep area receiving the drainage from this ditch was not experiencing accelerated erosion at the time of our reconnaissance (Plate 2). West of the bench area, alongside the graded road, we observed a shallow drainage channel with oversteepened, sloughing banks (Plate 2). We did not observe surface water, seeps or springs during our reconnaissance.

Subsurface

As observed within our test pits, the building area is blanketed with at least 5 to 10 feet of mottled orange, olive brown sandy clay with gravel. These soils are generally very stiff to hard, moist, and are interpreted to be landslide deposits. The uppermost ½ to 1 foot consists of weak and porous surface soil, with shrinkage cracks that extend at least ½ foot below the surface. The soils we tested in the upper 5 feet exhibit high plasticity (LL = 45.9 to 55.2; PI = 22.5 to 31.6) and medium expansion potential (EI = 59 to 79). The surface soil is locally covered by heterogeneous fill in the previously graded road areas. Heterogeneous fill is a material with varying density, strength, compressibility and shrink-swell characteristics that often has an unknown origin and placement history. As previously discussed, on hillsides 5:1 or steeper, the surface materials typically creep.

The thickness of the landslide deposits we encountered varies from 5 to more than 12 feet. Locally, these deposits include shattered layers of more competent sedimentary rock within a soft, plastic, sheared, disturbed matrix of sandy clay. In test pit TP-7, we observed gravels of the Sonoma volcanic bedrock, indicating transport of these materials from hundreds of feet upslope where the unit crops out. At test pit TP-6, the landslide deposits are more than 12 feet deep (limits of our exploration equipment). In test pits TP-2 and TP-3, we terminated the test pits in landslide deposits at 7 and 5½ feet below the surface, respectively. However, in test pits TP-4 ,TP-5, and TP-9, we reached the bottom of the landslide material and found shale bedrock at 10, 5, and 5 feet below the surface, respectively. The bedrock is generally thinly to medium bedded, with closely to very closely-spaced fractures, and is moderately hard, moderately strong, and moderately weathered.

We did not observe landslide deposits within test pits TP-1 and TP-8, located within two of the potential leachfield areas. In a third potential leachfield area, we terminated test pit TP-7 within landslide deposits at 5½ feet.

A detailed description of the subsurface conditions found in our test pits is given on Plates 3 through 6, Appendix A. Based on Table 20.3-1 of ASCE Standard 7-16, titled "Minimum Design Loads and Associated Criteria for Buildings and Other Structures" (2017), we have determined a Site Class of C should be used for the site.

Corrosion Potential

Mapping by the Natural Resources Conservation Service (2019) indicates that the corrosion potential of the near surface soil is moderate for uncoated steel and low for concrete. Performing corrosivity tests to verify these values was not part of our requested and/or proposed scope of work. Should the need arise, we would be pleased to provide a proposal to evaluate these characteristics.

Groundwater

Free groundwater was not observed in our test pits at the time of excavation. On hillsides, rainwater typically percolates through the porous surface materials and migrates downslope in the form of seepage at the interface of the surface materials and bedrock, and through fractures in the bedrock. Fluctuations in the seepage rates typically occur due to variations in rainfall intensity, duration and other factors such as periodic irrigation.

DISCUSSION AND CONCLUSIONS

Seismic Hazards

General

We did not observe subsurface conditions within the portion of the property we studied that would suggest the presence of materials that may be susceptible to seismically induced densification or liquefaction. Therefore, we judge the potential for the occurrence of these phenomena at the site to be low.

Faulting and Seismicity

We did not observe landforms within the area that would indicate the presence of active faults and the site is not within a current Alquist-Priolo Earthquake Fault Zone (Bryant and Hart, 2007). Therefore, we believe the risk of fault rupture at the site is low. However, the site is within an area affected by strong seismic activity and future seismic shaking should be anticipated at the site. It will be necessary to design and construct the proposed improvements in strict adherence with current standards for earthquake-resistant construction.

Lurching

Seismic slope failure or lurching is a phenomenon that occurs during earthquakes when slopes or man-made embankments yield and displace in the unsupported direction. Provided the foundations are installed as recommended herein, and proposed fills are adequately keyed into underlying bedrock material, as subsequently discussed, we judge the potential for impact to the proposed improvements from the occurrence of this phenomenon at the site is low. However, some of these secondary earthquake effects are unpredictable as to location and extent, as evidenced by the 1989 Loma Prieta Earthquake.

Geotechnical Issues

General

Based on our study, we judge the proposed improvements can be built as planned, provided the recommendations presented in this report are incorporated into their design and construction. The primary geotechnical concerns during design and construction of the project are:

1. The presence of approximately 2½ feet of expansive, weak, porous, compressible, clayey surface soil within the building area, and the potential presence of local heterogeneous fill;
2. The presence of ancient landslide deposits of varying depth at the site;
3. The presence of very steep slopes downhill of the proposed building site;
4. The detrimental effects of uncontrolled surface runoff and groundwater seepage on the long-term satisfactory performance of residences, especially those constructed on hillsides, given the erosion potential and porous nature of the surface soil; and
5. The strong ground shaking predicted to impact the site during the life of the project.

Heterogeneous Fill

Heterogeneous fills of unknown quality and unknown method of placement, such as those likely to be found locally at the site, can settle and/or heave erratically under the load of new fills, structures, slabs, and pavements. Footings, slabs, and pavements supported on heterogeneous fill could also crack as a result of such erratic movements. Thus, where not removed by planned grading, the heterogeneous fill must be excavated and replaced as an engineered fill if it is to be used for structural support.

Weak, Porous Surface Soil

Weak, porous surface soil, such as that found at the site, appears hard and strong when dry but will lose strength rapidly and settle under the load of fills, foundations, slabs, and pavements as its moisture content increases and approaches saturation. The moisture content of this soil can increase as the result of rainfall, periodic irrigation or when the natural upward migration of water vapor through the soil is impeded by, and condenses under fills, foundations, slabs, and pavements. The detrimental effects of such movements can be reduced by strengthening the soil during grading. This can be achieved by excavating the weak soil and replacing it as properly compacted (engineered) fill. Alternatively, satisfactory foundation support could be obtained below the weak surface soil.

Expansive Soil - In addition, the surface soil is expansive. Expansive surface soil shrinks and swells as they lose and gain moisture throughout the yearly weather cycle. Near the surface, the resulting movements can heave and crack lightly loaded shallow foundations (spread footings) and slabs and pavements. The zone of significant moisture variation (active layer) is dependent on the expansion potential of the soil and the extent of the dry season. In the project area, the active layer is generally considered to range in thickness from about 2 to 3 feet. The detrimental effects of the above-described movements can be reduced in exterior slab and pavement areas by pre-swelling the expansive soil and covering it with a moisture fixing and confining blanket of 12 inches of properly compacted select fill, as subsequently defined. The structures can be founded on a rigid slab system designed for use on expansive soils.

Ancient Landslide Deposits

Landslide deposits such as those encountered at the site can continue to move downslope along the landslide plane and/or new failures can occur within the landslide deposit. In the absence of movement, these deposits can behave erratically under new loads, such as from foundations and fills. Based on our observations of the landslide deposits, site bedrock, and subsurface moisture conditions, we judge that the landslide event responsible for the deposits we observed is ancient and no longer active. The soils interpreted as landslide deposits are very stiff and indurated. The moisture profile from the surface and through the landslide deposits is fairly uniform and not concentrated along the base of the debris, and potential slide plane material appears very old and "healed". The bedrock is highly fractured, which should allow continued dissipation of groundwater downward through the unit. In order to reduce potential impacts from the landslide deposits, a rigid slab system such as mat slab or post-tensioned slab designed for reduced soil bearing pressures needs to be used for foundation support of structures. In addition, structures need to be set back a minimum of 50 feet from breaks in slope of 2:1 (horizontal to vertical) or steeper.

Downslope Creep

Weak, creep-prone surface soil, such as that found on the steep slopes at the site, tends to naturally consolidate and settle on sloping terrain that is 5:1 (horizontal to vertical) or steeper. Fills and foundations deriving support from these materials will be susceptible and contribute to the downslope creep and settlement unless properly embedded in bedrock or buttressed (keyed, benched, drained and compacted). The settlement causes cracks in the slabs and structural distress in the form of cracked plaster and sticky doors and windows. Therefore, on hillsides, it will be necessary to obtain fill support below the creeping soil.

Fill Support - Hillside fills need to be constructed on level keyways and benches excavated entirely on rock. However, regardless of the care used during grading, buttressed fills of uneven thickness such as those typically built on hillsides, will settle differentially. Satisfactory performance of structural elements constructed on hillside fills, such as driveways, will require the use of specialized grading techniques discussed in the following sections of this report. These include excavating all creeping soil and replacing these materials as a buttressed fill of even thickness or constructing the improvements entirely on cut. For the purpose of this discussion, fills with a differential thickness of less than 5 feet can be assumed to have equal thickness. In order to provide the equal thicknesses, it may be necessary to overexcavate at least a few feet in cut areas. Where the total fill thickness is less than 3 feet, the fill can be placed at 95 percent relative compaction in lieu of overexcavation in cut areas.

Foundation Support -After remedial grading, satisfactory foundation support for planned structures can be obtained from rigid slab systems such as mat slab or post-tensioned slabs designed for reduced soil bearing pressures and for expansive soil conditions. Retaining walls and site walls that are not attached to a structure may be supported on drilled piers or deepened spread footings that gain support below the active layer.

Excavation Difficulty

In some areas, site excavation will encounter hard, resistant bedrock a few feet below the surface. Site excavations, including utility trenches will require heavy ripping and jack hammering. The contractors and subcontractors bidding this job should read this report and become familiar with site conditions as they pertain to their operation and the appropriate equipment needed to perform their tasks. If more detailed information regarding excavatability of the bedrock is required, a seismic refraction study should be performed or additional test pits should be excavated using the type and size of equipment planned for construction.

On-Site Soil Quality

All fill materials used in the upper 12 inches of exterior slab and pavement subgrade must be select, as subsequently described in “Recommendations.” We anticipate that, with the exception of organic matter and of rocks or lumps larger than 6 inches in diameter, the excavated material will be suitable for re-use as general fill, but will not be suitable for use as select fill unless stabilized with lime.

Select Fill

The select fill can consist of import materials with a low expansion potential or lime stabilized on-site clayey soil. Lime stabilized soil may prevent the growth of landscape vegetation due to the inherent elevated pH level of the soil. The geotechnical engineer must approve the use of on-site soil as select fill during grading.

Settlement

If remedial grading is performed and the rigid slabs are installed in accordance with the recommendations presented in this report, we estimate that post-construction differential settlements across each building will be about 1 inch.

Surface Drainage

Because of topography and location, the site will be impacted by surface runoff from the upgradient slopes. In addition, the site soil is susceptible to erosion and sloughing. Surface runoff typically sheet flows over the slopes but can be concentrated by the planned site grading, landscaping, and drainage. The ensuing erosion can create sloughing and promote slope instability or the surface runoff can pond against structures and cause deeper than normal soil heave and/or seep into the slab rock. Therefore, strict control of surface runoff is necessary to provide long-term satisfactory performance of projects constructed on or near hillsides. It will be necessary to divert surface runoff around slopes and improvements, provide positive drainage away from structures, and install energy dissipaters at discharge points of concentrated runoff. This can be achieved by constructing the building pad several inches above the surrounding area and conveying the runoff into man made drainage elements or natural swales that lead downgradient of the site.

Groundwater

We anticipate that rainwater will percolate through the active layer and migrate downslope at the interface of the surface soil and stiff subsoils or bedrock and through fractures in the bedrock and seep into the slab rock. Groundwater may also seep into excavations exposing the water migration zone or into hillside fills. Therefore, it will be necessary to intercept, collect and divert groundwater outside of the proposed improvements. This can be accomplished by installing slab underdrains as recommended herein.

RECOMMENDATIONS

Seismic Design

Seismic design parameters presented below are based on Section 1613 titled "Earthquake Loads" of both the 2019 California Building Code (CBC). The 2019 CBC becomes effective January 1, 2020. Based on Table 20.3-1 of ASCE Standard 7-16, titled "Minimum Design Loads and Associated Criteria for Buildings and Other Structures" (2017), we have determined a Site Class of C should be used for the site. Using a site latitude and longitude of 38.4100°N and 122.4580°W, respectively, and the OSHPD Seismic Design Maps website (https://seismicmaps.org), we recommend that the following seismic design criteria be used for structures at the site.

2019 CBC Seismic Criteria	
Spectral Response Parameter	**Acceleration (g)**
S_S (0.2 second period)	1.902
S_1 (1 second period)	0.689
S_{MS} (0.2 second period)	2.283
S_{M1} (1 second period)	0.965
S_{DS} (0.2 second period)	1.522
S_{D1} (1 second period)	0.643

Grading

Site Preparation

Areas to be developed should be cleared of vegetation and debris. Trees and shrubs that will not be part of the proposed development should be removed and their primary root systems grubbed. Cleared and grubbed material should be removed from the site and disposed of in accordance with County Health Department guidelines. We did not observe septic tanks, leach lines or underground fuel tanks during our study. Any such appurtenances found during grading should be capped and sealed and/or excavated and removed from the site, respectively, in accordance with established guidelines and requirements of the County Health Department. Voids created during clearing should be backfilled with engineered fill as recommended herein.

Stripping

Areas to be graded should be stripped of the upper few inches of soil containing organic matter. Soil containing more than two percent by weight of organic matter should be considered organic. Actual stripping depth should be determined by a representative of the geotechnical engineer in the field at the time of stripping. The strippings should be removed from the site, or if suitable, stockpiled for re-use as topsoil in landscaping.

Excavations

Following initial site preparation, excavation should be performed as recommended herein. Excavations extending below the proposed finished grade should be backfilled with suitable materials compacted to the requirements given below.

Within building and fill areas, the weak, porous, compressible surface soil should be excavated to within 6 inches of its entire depth (about 1 foot in our pits within the building area). If present in building areas, old fill should be removed for its full depth. The excavation of weak, compressible, soil and old fill should also extend at least 12 inches below exterior slab and pavement subgrade to allow space for the installation of the select fill blanket discussed in the conclusions section of this report. On sloping terrain 5:1 or steeper, fills should be constructed by excavating level keyways that expose undisturbed bedrock. The keyways should be at least 10 feet wide, extend at least 2 feet below the bedrock surface on the downhill side and should be sloped to drain to the rear. Keyway excavations should extend laterally to at least a 1:1 imaginary line extending down from the toe of the fill. Keyway subdrains are discussed hereinafter in “Subsurface Drainage.”

The excavation of weak, porous, compressible surface materials should extend at least 5 feet beyond the outside edge of the thickened edges of the rigid slabs and 3 feet beyond the edge of exterior slabs and pavements. The excavated materials should be stockpiled for later use as compacted fill, or removed from the site, as applicable.

At all times, temporary construction excavations should conform to the regulations of the State of California, Department of Industrial Relations, Division of Industrial Safety or other stricter governing regulations. The stability of temporary cut slopes, such as those constructed during the installation of underground utilities, should be the responsibility of the contractor. Depending on the time of year when grading is performed, and the surface conditions exposed, temporary cut slopes may need to be excavated to 1½:1, or flatter. The tops of the temporary cut slopes should be rounded back to 2:1 in weak soil zones.

Subsurface Drainage

A subdrain should be installed at the rear of the keyways and/or where evidence of seepage is observed. The subdrain should consist of a 4-inch diameter (minimum) perforated plastic pipe with SDR 35 or better embedded in Class 2 permeable material. The permeable material should be at least 12 inches thick and extend at least 48 inches above the bottom of the keyway (see Plate 10) and/or 12 inches above and below the seepage zone.

In addition, subdrains should be installed at a minimum slope of 1 percent and should have cleanouts located at their ends and at turning points. “Sweep” type elbows and wyes should be used at all turning points and cleanouts, respectively. Subdrain outlets and riser cleanouts should be fabricated of the same material as the subdrain pipe as specified herein. Outlet and riser pipe fittings should not be perforated. A licensed land surveyor or civil engineer should provide “record drawings” depicting the locations of subdrains and cleanouts.

Fill Quality

All fill materials should be free of perishable matter and rocks or lumps over 6 inches in diameter and must be approved by the geotechnical engineer prior to use. The upper 12 inches of fill beneath and within 3 feet of exterior slabs and pavement edges should be select fill. We judge the on-site soils are generally suitable for use as general fill but will not be suitable for use as select fill unless they are stabilized with lime. Lime stabilized soil may prevent the growth of landscape vegetation due to the inherent elevated pH level of the soil. The suitability of the on-site soil for use as select fill should be verified during grading.

Select Fill

Select fill should be free of organic matter, have a low expansion potential, and conform in general to the following requirements:

SIEVE SIZE	PERCENT PASSING (by dry weight)
6 inch	100
4 inch	90 – 100
No. 200	10 – 60

Liquid Limit – 40 Percent Maximum
Plasticity Index – 15 Percent Maximum
R-value – 20 Minimum (pavement areas only)

Expansive on-site soil may be used as select fill if it is stabilized with lime. In general, imported fill, if needed, should be select. Material not conforming to these requirements may be suitable for use as import fill; however, it shall be the contractor's responsibility to demonstrate that the proposed material will perform in an equivalent manner. The geotechnical engineer should approve imported materials prior to use as compacted fill. The grading contractor is responsible for submitting, at least 72 hours (3 days) in advance of its intended use, samples of the proposed import materials for laboratory testing and approval by the soils engineer.

Lime Stabilization

For preliminary planning purposes, we estimate that high calcium lime mixed at a minimum of 5½ percent (dry weight) will stabilize the expansive site soil. This percentage of lime needs to be verified prior to construction with engineering analysis and laboratory Atterberg Limits and/or pH testing using lime from the same source as that planned for use on the project and a sample of the soil to be treated. Laboratory test results and engineering analysis may indicate that a higher percentage of lime is required. The contractor should allow a minimum of 5 business days for the laboratory tests to be completed.

The lime stabilization should be performed in accordance with Section 24 of the Caltrans Standard Specifications except that a curing seal will not be required, provided the moisture content of the lime-stabilized material is maintained at or above optimum moisture content until it is permanently covered with subsequent construction. Lime stabilized materials are generally not suitable for reuse as general fill, select fill or backfill after compaction has taken place.

Fill Placement

The surface exposed by stripping and removal of heterogeneous fill and weak, compressible, surface soil should be scarified to a depth of at least 6 inches, uniformly moisture-conditioned to at least 4 percent above optimum and compacted to at least 90 percent of the maximum dry density of the materials as determined by ASTM Test Method D-1557. In expansive soil areas, moisture conditioning should be sufficient to completely close all shrinkage cracks for their full depth within pavement, exterior slab and building areas. If grading is performed during the dry season, the shrinkage cracks may extend to a few feet below the surface. Therefore, it may be necessary to excavate a portion of the cracked soil to obtain the proper moisture condition and degree of compaction. Approved fill material should then be spread in thin lifts, uniformly moisture-conditioned to near optimum and properly compacted. All structural fills, including those placed to establish site surface drainage, should be compacted to at least 90 percent relative compaction. Expansive soil used as fill should be moisture-conditioned to at least 4 percent above optimum. Only approved select materials should be used for fill within the upper 12 inches of exterior slabs and pavement subgrades. Fills placed on terrain sloping at 5:1 or steeper should be continually keyed and benched into firm, undisturbed bedrock. The benches should allow space for the placement of engineered fill of even thickness under settlement-sensitive structural elements supported directly on the fill. An illustration of this grading technique is shown on Plate 10.

SUMMARY OF COMPACTION RECOMMENDATIONS	
Area	Compaction Recommendation (ASTM D-1557)
Preparation for areas to receive fill	After preparation in accordance with this report, compact upper 6 inches to a minimum of 90 percent relative compaction.
General fill (native or import)	Compact to a minimum of 90 percent relative compaction.
Structural fill beneath buildings, extending outward to 5' beyond building perimeter	Compact to a minimum of 90 percent relative compaction.
Trenches	Compact to a minimum of 90 percent relative compaction. Compact the top 6 inches below vehicle pavement subgrade to a minimum of 95 percent relative compaction.
Retaining wall backfill	Compact to a minimum of 90 percent relative compaction, but not more than 95 percent.
Pavements, extending outward to 3' beyond edge of pavement	Compact upper 6 inches of subgrade to a minimum of 95 percent relative compaction.
Concrete flatwork and exterior slabs, extending outward to 3' beyond edge of slab	Compact subgrade to a minimum of 90 percent relative compaction. Where subject to vehicle traffic, compact upper 6 inches of subgrade to at least 95 percent relative compaction.
Aggregate Base	Compact aggregate base to at least 95 percent relative compaction.

Permanent Cut and Fill Slopes

In general, cut and fill slopes should be designed and constructed at slope gradients of 3:1 (horizontal to vertical) or flatter, unless otherwise approved by the geotechnical engineer in specified areas. Where steeper slopes are required, retaining walls should be used. Fill slopes should be constructed by overfilling and cutting the slope to final grade. “Track walking” of a slope to achieve slope compaction is not an acceptable procedure for slope construction. Permanent cut slopes should be observed in the field by the geotechnical engineer to verify that the exposed soil conditions are as anticipated. The geotechnical engineer is not responsible for measuring the angles of these slopes. Denuded slopes should be planted with fast-growing, deep-rooted groundcover to reduce sloughing or erosion. The cut and fill slope inclinations recommended herein address only the stability of the slopes. It should not be inferred that they address the feasibility of landscaping and weed control. Where these are concerns, the slopes should be flattened accordingly.

Wet Weather Grading

Generally, grading is performed more economically during the summer months when the on-site soil is 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 soil. Special and relatively expensive construction procedures, including dewatering of excavations and importing granular soil, should be anticipated if grading must be completed during the winter and early spring or if localized areas of soft saturated soil are found during grading in the summer and fall.

Open excavations also tend to be more unstable during wet weather as groundwater seeps towards the exposed cut slope. Severe sloughing and occasional slope failures should be anticipated. The occurrence of these events will require extensive clean up and the installation of slope protection measures, thus delaying projects. The general contractor is responsible for the performance, maintenance and repair of temporary cut slopes.

Foundation Support

Because of the presence of expansive surface soil and heterogeneous, old landslide materials, the residence and other structures should be supported on a rigid slab system such as mat slabs or post-tensioned slabs. In addition, structures should be set back 50 feet from breaks in slope of 2:1 (horizontal to vertical) or steeper.

Mat Slabs

Mat slabs of the size required for this project are typically a double mat reinforced slab with thickened areas at the edges and where heavier loads are anticipated, such as at columns. The bottoms of all excavations for thickened areas should be thoroughly cleaned out or wetted and compacted using hand-operated tamping equipment prior to placing steel and concrete. This will remove the soils disturbed during excavations, restore their adequate bearing capacity, and reduce post-construction settlements.

A mat slab installed in accordance with the recommendations presented herein may be designed using allowable bearing pressures of 1,600, 2,400 and 3,200 pounds per square foot (psf), for dead loads, dead plus code live loads, and total loads (including wind and seismic), respectively. In addition, a modulus of subgrade reaction (k) of 70 pounds per cubic inch (pci) may be used for design. The portion of the foundation extending into engineered fill or bedrock may impose a passive equivalent fluid pressure and a friction factor of 300 pounds per cubic foot (pcf) and 0.30, respectively, to resist sliding. Passive pressure should be neglected within the upper 6 inches, unless the soils are confined by concrete slabs or pavements.

The mat slab should be designed for 1-inch post-construction differential settlement across the building. Due to the presence of expansive soil and landslide debris, the slab should be a designed to span 10 feet of non-support and cantilever 5 feet at the edges.

Post-Tension Slabs

A post tension (PT) slab should be a designed to accommodate edge moisture variation distances of 4.9 and 8.0 feet for edge and center lift conditions, respectively, a differential edge swell of 0.6 inch and a center swell of 0.8 inch. These parameters were developed using the Post-Tensioning Institute manual "Design and Construction of Post-Tensioned Slabs-On-Ground, Third Edition" (2004). A PT slab installed in accordance with the foregoing recommendations may be designed using allowable bearing pressures of 1,600, 2,400 and 3,200 pounds per square foot (psf) for dead loads, dead plus code live loads, and total loads, including wind and seismic, respectively. We recommend a minimum slab thickness of 10 inches and a 12-inch-wide (minimum) perimeter thickened edge. Concentrated loads in the slab interior should also be supported by thickened beams within the slab. The portion of the PT slab extending into engineered fill or bedrock may impose a passive equivalent fluid pressure of 300 pcf and 0.30, respectively, to resist sliding. Passive pressure should be neglected within the upper 6 inches, unless the soils are confined by concrete slabs or pavements.

General

The PT slab or mat slab should be underlain with a capillary moisture break consisting of at least 4 inches of clean, free-draining crushed rock or gravel (excluding pea gravel) at least ¼-inch and no larger than ¾-inch in size. The subgrade soil within and for a distance of 5 feet beyond the footprint of the building(s) should be kept pre-swelled until the capillary moisture break is placed. The moisture content of the subgrade soil should be approved by the geotechnical engineer within 24 hours prior to placing the capillary moisture break. Where migration of moisture vapor through slabs would be detrimental, a moisture vapor barrier should be provided.

Because rigid slabs are designed to move with the expansive soil as it shrinks and swells, structural elements that are attached to the structure, but have their own foundation should not be used or should be founded on the rigid slab. Exterior flatwork and concrete walkway subgrades should be underlain by at least 12 inches of select fill and be pre-swelled by soaking prior to installation of the walkway. In addition, concrete walkways should be:

1. Cast separate from the rigid slabs to allow differential settlement to occur without distressing the walkway;

2. Reinforced to reduce cracks; and

3. Grooved to induce cracking in a non-obtrusive manner.

The Post-Tensioning Institute states "Consideration should be given to 'artificial' effects, such as planter units adjacent to structural bearing areas. Tree roots can be a serious problem and cause volume reduction in limited areas, thus causing distress to the slab foundation. Trees that are planted closer to the foundation than half their ultimate height can be expected to cause significant differential movement."

Retaining Walls

Retaining walls constructed at the site must be designed to resist lateral earth pressures plus additional lateral pressures that may be caused by surcharge loads applied at the ground surface behind the walls. Retaining walls free to rotate (yielding greater than 0.1 percent of the wall height at the top of the backfill) should be designed for active lateral earth pressures. If walls are restrained by rigid elements to prevent rotation, they should be designed for "at rest" lateral earth pressures.

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

EARTH EQUIVALENT FLUID PRESSURES

Loading Condition	Pressure (pcf)	Additional Seismic Pressure (pcf)*
Active - Level Backfill	42	18
Active - Sloping Backfill 3:1 or Flatter	53	56
At Rest - Level Backfill	63	47

* If required

These pressures do not consider additional loads resulting from adjacent foundations or other loads. If these additional surcharge loadings are anticipated, we can assist in evaluating their effects. Where retaining wall backfill is subject to vehicular traffic, the walls should be designed to resist an additional surcharge pressure equivalent to two feet of additional backfill.

Retaining walls will yield slightly during backfilling. Therefore, walls should be backfilled prior to building on, or adjacent to, the walls. Backfill against retaining walls should be compacted to at least 90 and not more than 95 percent relative compaction. Over-compaction or the use of large compaction equipment should be avoided because increased compactive effort can result in lateral pressures higher than those recommended above.

Foundation Support

Retaining walls should be supported on deepened spread footings or on drilled piers designed in accordance with the recommendations presented in this report. Retaining wall foundations should be designed by the project civil or structural engineer to resist the lateral forces set forth in this section.

Spread Footings - Spread footings should be at least 12 inches wide and should bottom on firm, natural soil or undisturbed bedrock at least 36 inches below lowest adjacent exterior grade. Additional embedment or width may be needed to satisfy code and/or structural requirements. On ungraded sloping terrain, the footings should be stepped as necessary to produce level tops and bottoms. Footings should be deepened as necessary to provide at least 7 feet of horizontal confinement between the footing bottoms and the face of the nearest slope.

The bottoms of all footing excavations should be thoroughly cleaned out or wetted and compacted using hand-operated tamping equipment prior to placing steel and concrete. This will remove the soil disturbed during footing excavations, or restore their adequate bearing capacity, and reduce post-construction settlements. Footing excavations should not be allowed to dry before placing concrete. If shrinkage cracks appear in soil exposed in the footing excavations, the soil should be thoroughly moistened to close all cracks prior to concrete placement. The moisture condition of the foundation excavations should be checked by the geotechnical engineer no more than 24 hours prior to placing concrete.

Footings installed in accordance with these recommendations may be designed using allowable bearing pressures of 1,200, 1,800 and 2,400 psf, for dead loads, dead plus code live loads, and total loads (including wind and seismic), respectively. The portion of spread footing foundations extending into firm soils or undisturbed bedrock may impose a passive equivalent fluid pressure and a friction factor of 300 pcf and 0.30, respectively, to resist sliding. Passive pressure on ungraded weak surface soil should be reduced to 150 pcf. Passive pressure should be neglected within the upper 12 inches, unless the soil is confined by concrete slabs or pavements.

Drilled Piers

Drilled piers should be at least 12 inches in diameter and should develop support below the active layer (3 feet below the ground surface). Larger piers and deeper embedment may be needed to resist the lateral forces imposed by earthquakes per the 2019 California Building Code. Piers should be spaced no closer than 3 pier diameters, center to center.

Skin Friction - The portion of the piers extending below 3 feet may be designed using an allowable skin friction of 450 psf for dead load plus long term live loads. This value can be increased by ⅓ for total loads, including downward vertical wind or seismic forces. A skin friction value of 300 psf should be used to resist uplift forces. End bearing should be neglected because of the difficulty of cleaning out small diameter pier holes, and the uncertainty of mobilizing end bearing and skin friction simultaneously.

Lateral Forces - On terrain sloping 5:1 or steeper, the piers should be designed and reinforced, by the project structural engineer, to resist creep forces equivalent to a 3-foot thick zone exerting an equivalent fluid pressure of 65 pcf acting on two pier diameters. Lateral loads on piers will be resisted by passive pressure on the soil or bedrock. An equivalent fluid pressure of 300 pcf acting on two pier diameters should be used. Confinement for passive pressure may be assumed from 3 feet below the lowest adjacent finished ground surface.

The piers should be interconnected with grade beams to support the wall loads and to redistribute stresses imposed by wind or earthquakes and the creeping surface soils. The grade beams should be designed to span between the piers in accordance with structural requirements. The steel from the piers should extend sufficient distance into the grade beams to develop its full bond strength.

The piers and grade beams should be designed to resist uplift pressures imposed by expansive soils. The uplift pressure should be assumed to be 2,000 psf of grade beam surface contact.

We did not encounter groundwater and/or caving-prone soil within the planned pier depth during our study. If groundwater is encountered during drilling, it may be necessary to de-water the holes and/or place the concrete by the tremie method. If caving soil is encountered, it may be necessary to case the holes. Difficult drilling may be required to achieve the required penetration. The drilling subcontractor should review this report, become familiar with site conditions as they pertain to his operation and draw his own conclusions regarding drilling difficulty, suitable drill rigs and the need for casing and dewatering prior to bidding.

Concrete mix design and placement should be done in accordance with the current ADSC and/or ACI specifications. Concrete should not be allowed to mushroom at the top of the piers or below the bottom of grade beams.

Wall Drainage and Backfill

Retaining walls should be backdrained as shown on Plate 11, Appendix A. The backdrains should consist of 4-inch diameter, rigid perforated pipe embedded in Class 2 permeable material. The pipe should be PVC Schedule 40 or ABS with SDR 35 or better, and the pipe should be sloped to drain to outlets by gravity. The top of the pipe should be at least 8 inches below lowest adjacent grade. The Class 2 permeable material should extend to within 1½ feet of the surface. The upper 1½ feet should be backfilled with compacted soil to exclude surface water. Expansive soil should not be used for wall backfill. Where expansive soil is present in the excavation made to install the retaining wall, the excavation should be sloped back 1:1 from the back of the footing or grade beam. The ground surface behind retaining walls should be sloped to drain. Where migration of moisture through retaining walls would be detrimental, retaining walls should be waterproofed.

Slab-On-Grade

Because of expansive soil, conventional slabs-on-grade should not be used in interior areas. Slab floor systems should be provided by the rigid slabs. However, exterior slabs may be used provided they are underlain by 12 inches of select engineered fill (not counting the aggregate base course) placed in accordance with the recommendations presented herein.

Exterior slab subgrade should be rolled to produce a dense, uniform surface. The future expansion potential of the subgrade soil should be reduced by thoroughly presoaking the slab subgrade prior to concrete placement. The moisture condition of the subgrade soil should be checked by the geotechnical engineer no more than 24 hours prior to placing the aggregate base. Class 2 aggregate base can be used for slab rock under exterior slabs. Slabs should be designed by the project civil or structural engineer to support the anticipated loads and reduce cracking.

Utility Trenches

The shoring and safety of trench excavations is solely the responsibility of the contractor. Attention is drawn to the State of California Safety Orders dealing with “Excavations and Trenches.”

Unless otherwise specified by the County of Napa, on-site, inorganic soil may be used as utility trench backfill. Where utility trenches support pavements, slabs and foundations, trench backfill should consist of aggregate baserock. The baserock should comply with the minimum requirements in Caltrans Standard Specifications, Section 26 for Class 2 Aggregate Base. Trench backfill should be moisture-conditioned as necessary, and placed in horizontal layers not exceeding 8 inches in thickness, before compaction. Each layer should be compacted to at least 90 percent relative compaction as determined by ASTM Test Method D-1557. The top 6 inches of trench backfill below vehicle pavement subgrades should be moisture-conditioned as necessary and compacted to at least 95 percent relative compaction. Jetting or ponding of trench backfill to aid in achieving the recommended degree of compaction should not be attempted.

Pavements

Provided the site grading is performed to remediate expansive soil heave, as recommended herein, the uppermost 12-inches of pavement subgrade soil will be either imported select fill with a minimum R-value of 20 or lime stabilized site soil that generally has an R-value of at least 50. Based on those R-values we recommend the pavement sections listed in the tables below be used.

PAVEMENT SECTIONS WITH IMPORTED SELECT FILL SUBGRADE

TI	ASPHALT CONCRETE (feet)	CLASS 2 AGGREGATE BASE (feet)	IMPORTED SELECT FILL* (feet)
7.0	0.30	1.15	1.0
6.0	0.25	1.05	1.0
5.0	0.20	0.90	1.0

* R-value ≥ 20

PAVEMENT SECTIONS WITH LIME STABILIZED SELECT FILL SUBGRADE

TI	ASPHALT CONCRETE (feet)	CLASS 2 AGGREGATE BASE (feet)	LIME STABILIZED SELECT FILL* (feet)
7.0	0.35	0.50	1.0
6.0	0.30	0.50	1.0
5.0	0.20	0.50	1.0

* R-value ≥ 50

Pavement thicknesses were computed using Caltrans design methods and are based on a pavement life of 20 years. These recommendations are intended to provide support for traffic represented by the indicated

Traffic Indices. They are not intended to provide pavement sections for heavy concentrated construction storage or wheel loads such as forklifts, parked truck-trailers and concrete trucks.

In areas where heavy construction storage and wheel loads are anticipated, the pavements should be designed to support these loads. Support could be provided by increasing pavement sections or by providing reinforced concrete slabs. Alternatively, paving can be deferred until heavy construction storage and wheel loads are no longer present.

Because of the expansion potential of the soil and bedrock at the site and the difficulty in controlling seasonal moisture variation beneath and adjacent to the driveway, significant cracking may develop in the pavement even if 12-inches of select fill is installed. Increasing the thickness of select fill or installing moisture cutoffs may reduce but not eliminate the potential for cracks to develop. It should be understood that pavements will likely require regular maintenance including crack sealing and the aesthetics may not be desirable.

Prior to placement of aggregate base, the upper 6 inches of the pavement subgrade soil (excluding lime stabilized soil) should be scarified, uniformly moisture-conditioned to near optimum, and compacted to at least 95 percent relative compaction to form a firm, non-yielding surface. Lime stabilized select fill subgrade soil should be compacted as specified in Section 24 of the Caltrans Standard Specifications.

Aggregate base materials should be spread in thin layers, uniformly moisture-conditioned, and compacted to at least 95 percent relative compaction to form a firm, non-yielding surface. The materials and methods used should conform to the requirements of the County of Napa and the current edition of the Caltrans Standard Specifications, except that compaction requirements should be based on ASTM Test Method D-1557. Aggregate used for the base course should comply with the minimum requirements specified in Caltrans Standard Specifications, Section 26 for Class 2 Aggregate Base.

Wet Weather Paving

In general, the pavements should be constructed during the dry season to avoid the saturation of the subgrade and base materials, which often occurs during the wet winter months. If pavements are constructed during the winter, a cost increase relative to drier weather construction should be anticipated. Unstable areas may have to be overexcavated to remove soft soil. The excavations will probably require backfilling with imported crushed (ballast) rock. The geotechnical engineer should be consulted for recommendations at the time of construction.

Geotechnical Drainage

This section presents recommendations for surface and subsurface drainage. For the discussion of subsurface drainage related to grading, especially on hillsides, refer to the “Subsurface Drainage” section.

Surface

Surface water should be diverted away from slopes, foundations and edges of pavements. Surface drainage gradients should slope away from building foundations in accordance with the requirements of the CBC or local governing agency. Where a gradient flatter than 2 percent for paved areas and 4 percent for unpaved areas is required to satisfy design constraints, area drains should be installed with a spacing no greater than about 20 feet. Roofs should be provided with gutters and the downspouts should be connected to closed (glued Schedule 40 PVC or ABS with SDR of 35 or better) conduits discharging well away from foundations, into erosion resistant natural drainages or into the site's surface drainage system. Roof downspouts and surface drains must be maintained entirely separate from the slab underdrains recommended hereinafter.

Water seepage or the spread of extensive root systems into the soil subgrade of footings, slabs or pavements could cause differential movements and consequent distress in these structural elements. Landscaping should be planned with consideration for these potential problems.

Slab Underdrains

Where interior slab subgrades are less than 6 inches above adjacent exterior grade and where migration of moisture through the slab would be detrimental, slab underdrains should be installed to dispose of surface and/or groundwater that may seep and collect in the slab rock. Slab underdrains should consist of 6-inch wide trenches that extend at least 6 inches below the bottom of the slab rock and slope to drain by gravity. The slab underdrain trenches should be spaced no further than 15 feet, both ways. Additional drain trenches should be installed, as necessary, to drain all isolated under slab areas. Four-inch diameter perforated pipe (SDR 35 or better) sloped to drain to outlets by gravity should be placed in the bottom of the trenches. Slab underdrain trenches should be backfilled to subgrade level with clean, free draining slab rock. An illustration of this system is shown on Plate 12. If slab underdrains are not used, it should be anticipated that water will enter the slab rock, permeate through the concrete slab and ruin floor coverings.

Additional Site Subdrainage

Based on our observations of the landslide deposits, site bedrock, and subsurface moisture conditions, we judge that the landslide event responsible for the deposits we observed is ancient and no longer active. The soils interpreted as landslide deposits are very stiff and indurated. The moisture profile from the surface and through the landslide deposits is fairly uniform and not concentrated along the base of the debris, and potential slide plane material appears very old and "healed". The bedrock is highly fractured, which should allow continued dissipation of groundwater downward through the unit.

However, if desired, additional measures could be evaluated to reduce the potential for reactivated movement within the landslide, such as construction of a deep subsurface moisture cutoff drain on the uphill side of the building area, or finger drains within the slope. Evaluation and design of such a system would need to be preceded by additional subsurface exploration consisting of drilling vertical borings in the area of the proposed subdrain or finger drains, to determine the depth of the landslide plane in those areas. This exploration and evaluation can be performed under separate scope and fee, if desired.

Leachfields

Because of the introduction of water underground, the installation of leachfields tends to lower the stability of hillsides. Therefore, leachfields should be installed in level to gently sloping relatively stable areas. Leachfields should not be located in areas of steep to very steep slopes, active landslides or creeping soils. In addition, leachfield areas should be setback from building sites to reduce the risk of hillside instability impacting the residence. Leachfield areas with a low risk of instability are shown on Plate 2. A subdrain should be installed upslope of leachfields. Refer to the “Subsurface Drainage” section for subdrain construction recommendations.

Maintenance

Periodic land maintenance, especially on hillsides, will be required. Surface and subsurface drainage facilities should be checked frequently, and cleaned and maintained as necessary or at least annually. A dense growth of deep-rooted ground cover must be maintained on all slopes to reduce sloughing and erosion. Sloughing and erosion that occurs must be repaired promptly before it can enlarge.

Supplemental Services

Pre-Bid Meeting

It has been our experience that contractors bidding on the project often contact us to discuss the geotechnical aspects. Informal contacts between RGH Consultants (RGH) and an individual contractor could result in incomplete or misinterpreted information being provided to the contractor. Therefore, we recommend a pre-bid meeting be held to answer any questions about the report prior to submittal of bids. If this is not possible, questions or clarifications regarding this report should be directed to the project owner or their designated representative. After consultation with RGH, the project owner or their representative should provide clarifications or additional information to all contractors bidding the job.

Plan and Specifications Review

Coordination between the design team and the geotechnical engineer is recommended to assure that the design is compatible with the soil, geologic and groundwater conditions encountered during our study. RGH recommends that we be retained to review the project plans and specifications to determine if they are consistent with our recommendations. In the event we are not retained to perform this recommended review, we will assume no responsibility for misinterpretation of our recommendations.

Construction Observation and Testing

Prior to construction, a meeting should be held at the site that includes, but is not limited to, the owner or owner’s representative, the general contractor, the grading contractor, the foundation contractor, the underground contractor, any specialty contractors, the project civil engineer, other members of the project design team and RGH. This meeting should serve as a time to discuss and answer questions regarding the recommendations presented herein and to establish the coordination procedure between the contractors and RGH.

In addition, we should be retained to monitor all soil related work during construction, including:

- Site stripping, over-excavation, grading, and compaction of near surface soil;
- Placement of all engineered fill and trench backfill with verification field and laboratory testing;
- Observation of all foundation excavations; and
- Observation of foundation and subdrain installations.

If, during construction, we observe subsurface conditions different from those encountered during the explorations, we should be allowed to amend our recommendations accordingly. If different conditions are observed by others, or appear to be present beneath excavations, RGH should be advised at once so that these conditions may be evaluated and our recommendations reviewed and updated, if warranted. The validity of recommendations made in this report is contingent upon our being notified and retained to review the changed conditions.

If more than 18 months have elapsed between the submission of this report and the start of work at the site, or if conditions have changed because of natural causes or construction operations at, or adjacent to, the site, the recommendations made in this report may no longer be valid or appropriate. In such case, we recommend that we be retained to review this report and verify the applicability of the conclusions and recommendations or modify the same considering the time lapsed or changed conditions. The validity of recommendations made in this report is contingent upon such review.

These supplemental services are performed on an as-requested basis and are in addition to this geotechnical study. We cannot accept responsibility for items that we are not notified to observe or for changed conditions we are not allowed to review.

LIMITATIONS

This report has been prepared by RGH for the exclusive use of the property owner and their consultants as an aid in the design and construction of the proposed improvements described in this report.

The validity of the recommendations contained in this report depends upon an adequate testing and monitoring program during the construction phase. Unless the construction monitoring and testing program is provided by our firm, we will not be held responsible for compliance with design recommendations presented in this report and other addendum submitted as part of this report.

Our services consist of professional opinions and conclusions developed in accordance with generally accepted geotechnical engineering principles and practices. We provide no warranty, either expressed or implied. Our conclusions and recommendations are based on the information provided to us regarding the proposed construction, the results of our field exploration, laboratory testing program, and professional judgment. Verification of our conclusions and recommendations is subject to our review of the project plans and specifications, and our observation of construction.

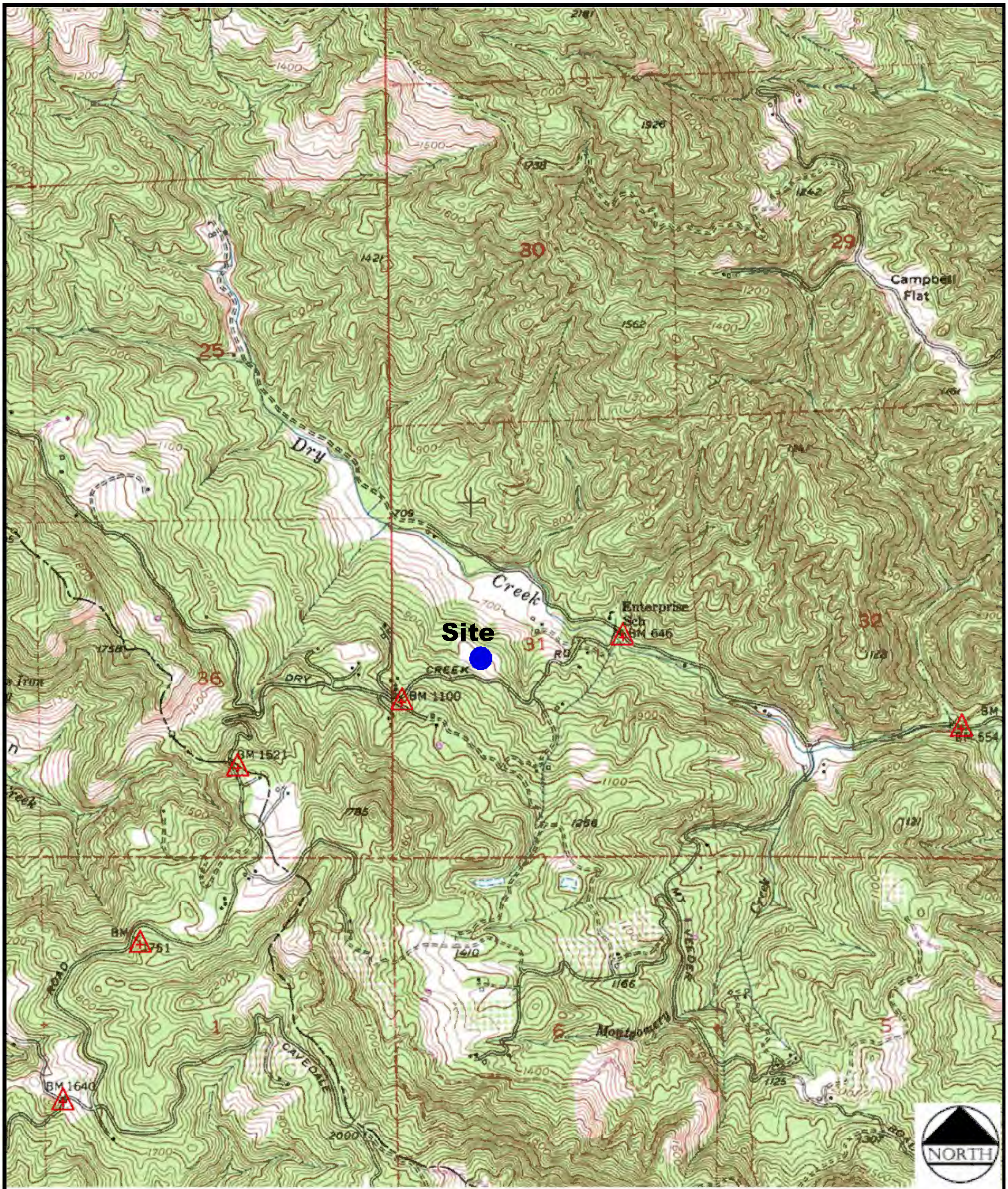
The test pits represent the subsurface conditions at the locations and on the date indicated. It is not warranted that they are representative of such conditions elsewhere or at other times. Site conditions and cultural features described in the text of this report are those existing at the time of our field exploration and may not necessarily be the same or comparable at other times.

It should be understood that slope failures including landslides, debris flows and erosion are on-going natural processes which gradually wear away the landscape. Residual soil and weathered bedrock can be susceptible to downslope movement, even on apparently stable sites. Such inherent hillside and slope risks are generally more prevalent during periods of intense and prolonged rainfall, which occasionally occur, in northern California and/or during earthquakes. Therefore, it must be accepted that occasional, unpredictable slope failure and erosion and deposition of the residual soil and weathered bedrock materials are irreducible risks and hazards of building upon or near the base of any hillside or any steeper slope area throughout northern California. By accepting this report, the client and other recipients acknowledge their understanding and acceptance of these risks and hazards, and the terms and conditions herein.

The scope of our services did not include an environmental assessment or a study of the presence or absence of toxic mold and/or hazardous, toxic or corrosive materials in the soil, surface water, groundwater or air (on, below or around this site), nor did it include an evaluation or study for the presence or absence of wetlands. These studies should be conducted under separate cover, scope and fee and should be provided by a qualified expert in those fields.

APPENDIX A - PLATES

LIST OF PLATES



Reference: Maptech Topoquad, Rutherford, California Quadrangle

Scale: 1" = 2000'

RGH CONSULTANTS

Job No: 7362.01.04.2 | Date: DEC 2019

SITE LOCATION MAP
6204 Dry Creek Road Residential Project
6204 Dry Creek Road
Napa, California

PLATE
1

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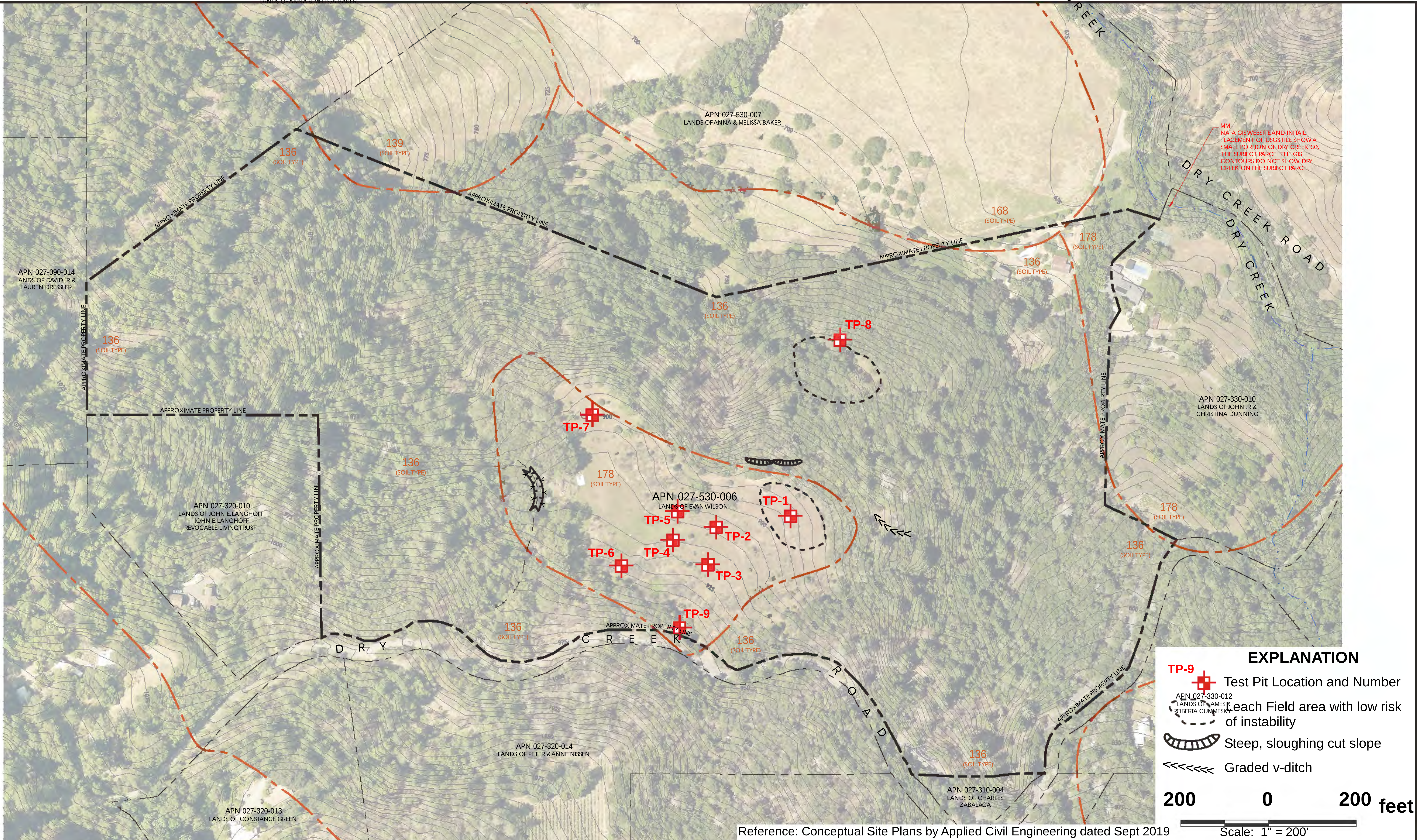
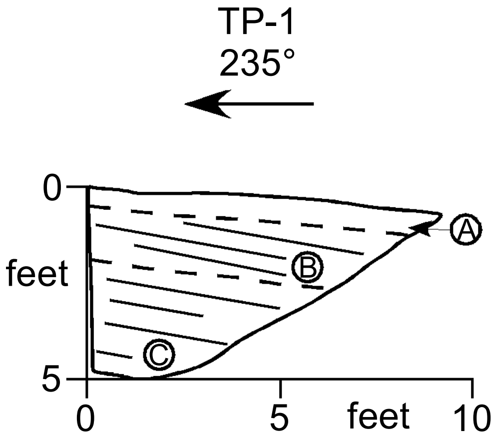


Reference: Conceptual Site Plans by Applied Civil Engineering dated Sept 2019

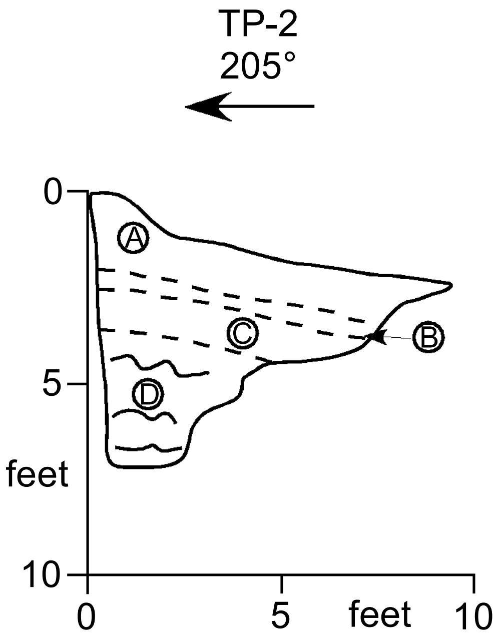
RGH CONSULTANTS	EXPLORATION PLAN 6204 Dry Creek Road 6204 Dry Creek Road Napa, California	PLATE 2
Job No: 7362.01.01.2 / Date: DEC 2019		



Ⓐ LIGHT YELLOW-BROWN SANDY CLAY (CL), medium stiff, dry, weak and porous

Ⓑ BROWN SHALE, very thinly bedded, extremely closely spaced fractures, moderately hard, weak to moderately strong, moderately to highly weathered, with concretions

Ⓒ BROWN SANDSTONE, thin to medium bedded, very close to closely spaced fractures, moderately hard, moderately strong, moderately weathered, with concretions, becomes moist at 2 feet, with shale interbeds

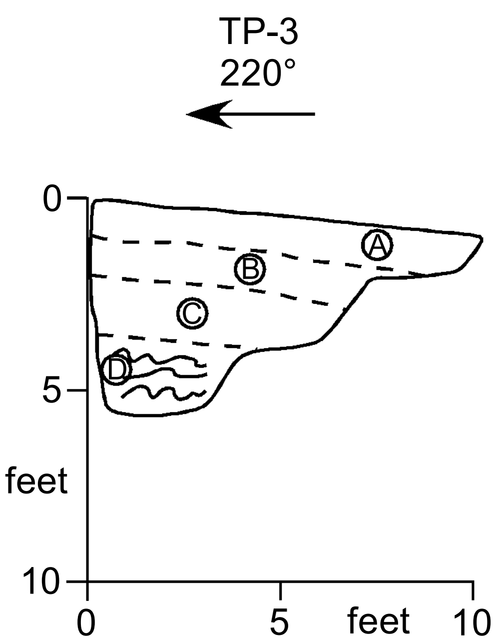


Ⓐ LIGHT YELLOW-BROWN SANDY CLAY (CL), medium stiff, dry, weak and porous to 1/2'

Ⓑ RED-BROWN CLAY (CH), very stiff, moist (Qls)

Ⓒ GRAY SHALE, shattered/crushed, firm, friable, highly weathered to red-brown, sandstone interbeds at base (Qls)

Ⓓ MOTTLED ORANGE-OLIVE GRAY-BROWN SHALE, sheared/crushed, firm, moderately hard, highly weathered to clay (CH) [hard, moist], with slicks, shearing, roots, local black clay (Qls)



Ⓐ LIGHT YELLOW-BROWN SANDY CLAY (CL), medium stiff, dry, weak and porous

Ⓑ OLIVE BROWN SANDY CLAY (CH), hard, moist, with remnant concretions, with vpf's and vertical cracks (Qls)

Ⓒ GRAY SHALE, shattered/crushed, firm, friable, highly weathered to red-brown, sandstone interbeds at base (Qls)

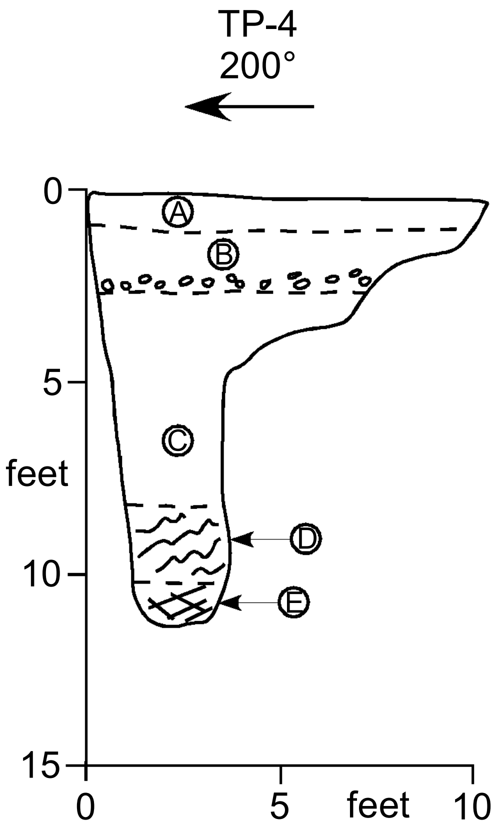
Ⓓ MOTTLED ORANGE, OLIVE BROWN SHALE, sheared, firm, plastic, completely weathered to clay, tectonically sheared (Qls)

RGH CONSULTANTS

Job No: 7362.01.04.2 | Date: DEC 2019

LOG OF TEST PITS TP-1, TP-2 & TP-3
6204 Dry Creek Road Residential Project
6204 Dry Creek Road
Napa, California

PLATE
3



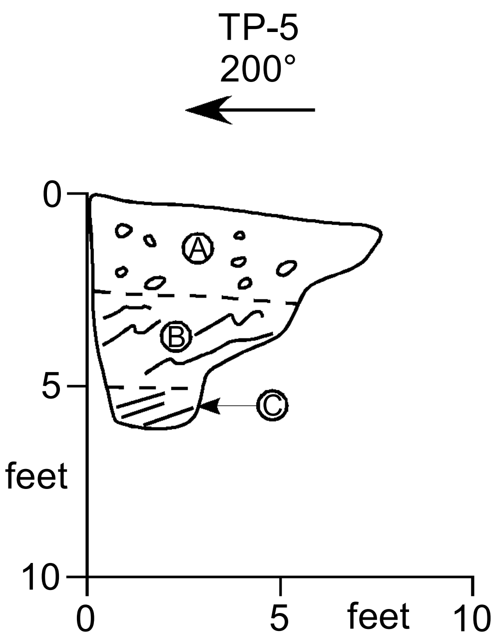
(A) LIGHT YELLOW-BROWN SANDY CLAY (CL), medium stiff, dry, weak and porous

(B) RED BROWN SANDY CLAY WITH GRAVEL (CH), hard, dry to moist, with poorly sorted, angular sandstone gravels to small cobbles, sparse roots (Qls)

(C) MOTTLED ORANGE, OLIVE BROWN SHALE, sheared, firm, plastic, completely weathered to clay, tectonically sheared, with remnant concretations, digs like hard clay, dry to moist (Qls)

(D) OLIVE GREEN CLAYSTONE, sheared, with slicks, firm, plastic, moderately weathered (Qls)

(E) OLIVE-GRAY BROWN SHALE, thinly bedded, moderately hard, weak to moderately strong, moderately weathered

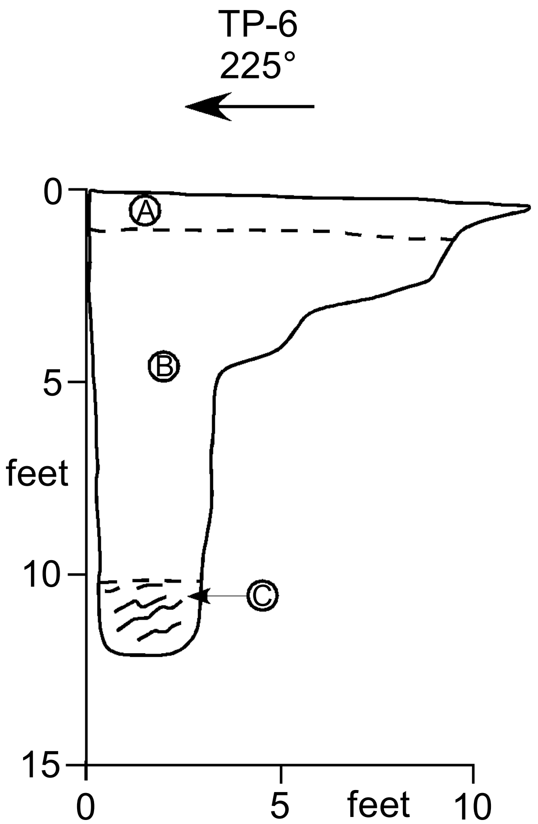


(A) LIGHT YELLOW BROWN TO OLIVE BROWN CLAY WITH SAND (CH), hard, dry to moist, with angular sandstone gravels, weak and porous to 1 foot, shrinkage cracks at surface (Qls)

(B) MOTTLED ORANGE, OLIVE BROWN SHALE, sheared, firm, plastic, completely weathered to clay, tectonically sheared (Qls)

(C) OLIVE-GRAY SHALE, thinly bedded, closely spaced fractures, moderately hard, moderately strong, moderately weathered, with concretions

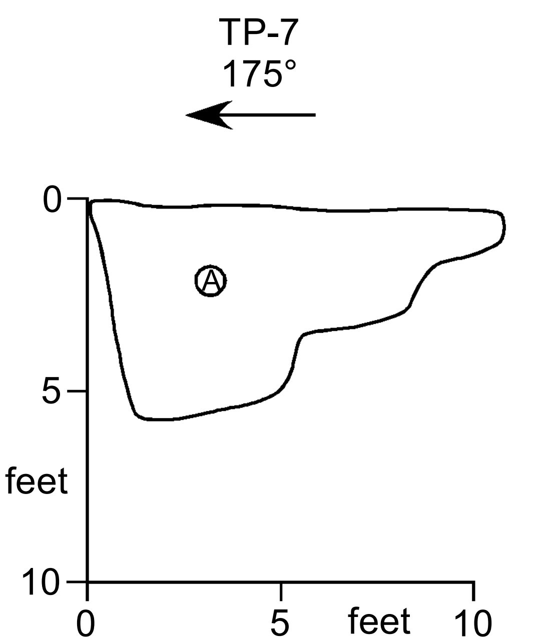
RGH CONSULTANTS	**LOG OF TEST PITS TP-4 & TP-5** 6204 Dry Creek Road Residential Project 6204 Dry Creek Road Napa, California	PLATE **4**
Job No: 7362.01.04.2 — Date: DEC 2019		



Ⓐ LIGHT OLIVE GRAY-BROWN SANDY CLAY (CH), medium stiff, dry, weak and porous

Ⓑ MOTTLED RED-OLIVE GRAY-BROWN SANDY CLAY (CL), hard, dry to moist, sparse gravels, sheared, very uniform look (Qls)

Ⓒ MOTTLED RED AND OLIVE CLAYSTONE, sheared, soft, plastic, highly weathered (Qls)



Ⓐ MOTTLED RED-BROWN OLIVE SANDY CLAY (CH), hard, dry to moist, angular shale, sandstone and volcanic (Tsv) gravels; weak and porous to 1' (Qls)

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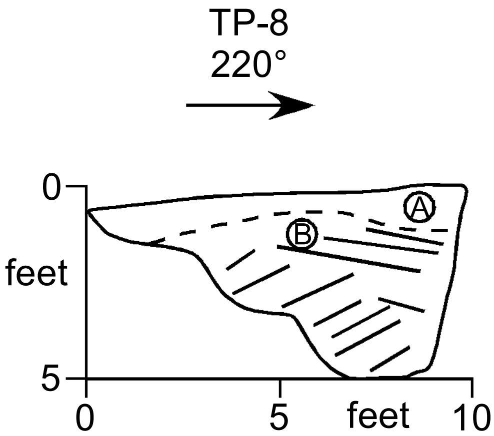
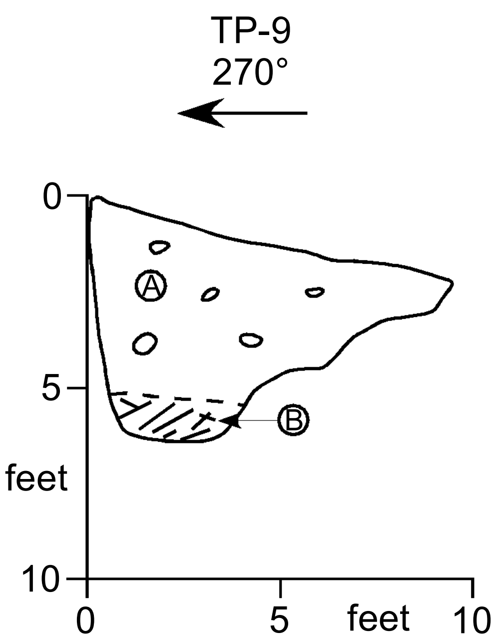
LOG OF TEST PITS TP-6 & TP-7
6204 Dry Creek Road Residential Project
6204 Dry Creek Road
Napa, California

PLATE
5



Ⓐ LIGHT YELLOW BROWN SANDY CLAY (CL-CH), medium stiff, dry, weak and porous

Ⓑ GRAY TO BROWN SHALE, very closely spaced fractures, moderately hard, moderately strong, moderately weathered, faulted, variable bedding, with concretions



Ⓐ MOTTLED ORANGE-OLIVE GRAY-BROWN SHALE, sheared/crushed, firm, moderately hard, highly weathered to clay (CH) [hard, moist], with slicks, shearing, roots, locally black clay, weak and porous to 1 foot (Qls)

Ⓑ OLIVE-GRAY SHALE, thinly bedded, closely spaced fractures, moderately hard, moderately strong, moderataely weathered, with concretions

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LOG OF TEST PITS TP-8 & TP-9
6204 Dry Creek Road Residential Project
6204 Dry Creek Road
Napa, California

PLATE
6

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVEL AND GRAVELLY SOILS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVEL (LITTLE OR FINES)		GW	WELL-GRADED GRAVEL, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
				GP	POORLY-GRADED GRAVEL, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
		GRAVEL WITH FINES (OVER 12% OF FINES)		GM	WELL-GRADED GRAVEL, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
				GC	CLAYEY GRAVEL, POORLY GRADED GRAVEL-SAND-CLAY MIXTURES
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	CLEAN SANDS (LITTLE OR NO FINES)		SW	WELL-GRADED SAND, GRAVELLY SAND, LITTLE OR NO FINES
				SP	POORLY-GRADED SAND, GRAVELLY SAND, LITTLE OR NO FINES
		SANDS WITH FINES (OVER 12% OF FINES)		SM	SILTY SANDS, POORLY GRADED SAND-SILT MIXTURES
				SC	CLAYEY SANDS, POORLY GRADED SAND-CLAY MIXTURES
FINE GRAINED SOILS MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS LIQUID LIMIT LESS THAN 50			ML	INORGANICS SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS, OR CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC CLAYS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS LIQUID LIMIT GREATER THAN 50			MH	ORGANIC 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, HUMUS, SWAMP SOILS AND OTHER SOILS WITH HIGH ORGANIC-CONTENTS

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

KEY TO TEST DATA

Consol - Consolidation
Gs - Specific Gravity
SA - Sieve Analysis
■ - "Undisturbed" Sample
☒ - Bulk or Disturbed Sample
◪ - Standard Penetration Test
⧄ - Sample Attempt With No Recovery
◫ - Sample Recovered But Not Retained

	Shear Strength, psf	Confining Pressure, psf	
Tx	320	(2600)	- Unconsolidated Undrained Traixial
TxCU	320	(2600)	- Consolidated Undrained Triaxial
DS	2750	(2600)	- Consolidated Drained Direct Shear
UC	2000		- Unconfined Compression
FVS	470		- Field Vane Shear
LVS	700		- Laboratory Vane Shear
SS			- Shrink Swell
EXP			- Expansion
P			- Permeability

Note: All strength tests on 2.8-in. or 2.4-in. diameter sample, unless otherwise indicated.

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SOIL CLASSIFICATION AND KEY TO TEST DATA
6204 Dry Creek Road Residential Project
6204 Dry Creek Road
Napa, California

PLATE 7

Job No: 7362.01.04.2 | Date: DEC 2019

LAYERING	
MASSIVE	Greater than 6 feet
THICKLY BEDDED	2 to 6 feet
MEDIUM BEDDED	8 to 24 inches
THINLY BEDDED	2½ to 8 inches
VERY THINLY BEDDED	¾ to 2½ inches
CLOSELY LAMINATED	¼ to ¾ inches
VERY CLOSELY LAMINATED	Less than ¼ inch

JOINT, FRACTURE, OR SHEAR SPACING	
VERY WIDELY SPACED	Greater than 6 feet
WIDELY SPACED	2 to 6 feet
MODERATELY SPACED	8 to 24 inches
CLOSELY SPACED	2½ to 8 inches
VERY CLOSELY SPACED	¾ to 2½ inches
EXTREMELY CLOSELY SPACED	Less than ¼ inch

HARDNESS

Soft - pliable; can be dug by hand
Firm - 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 produces little powder and is often faintly visible
Very Hard - cannot be scratched with pocket knife, leaves a metallic streak

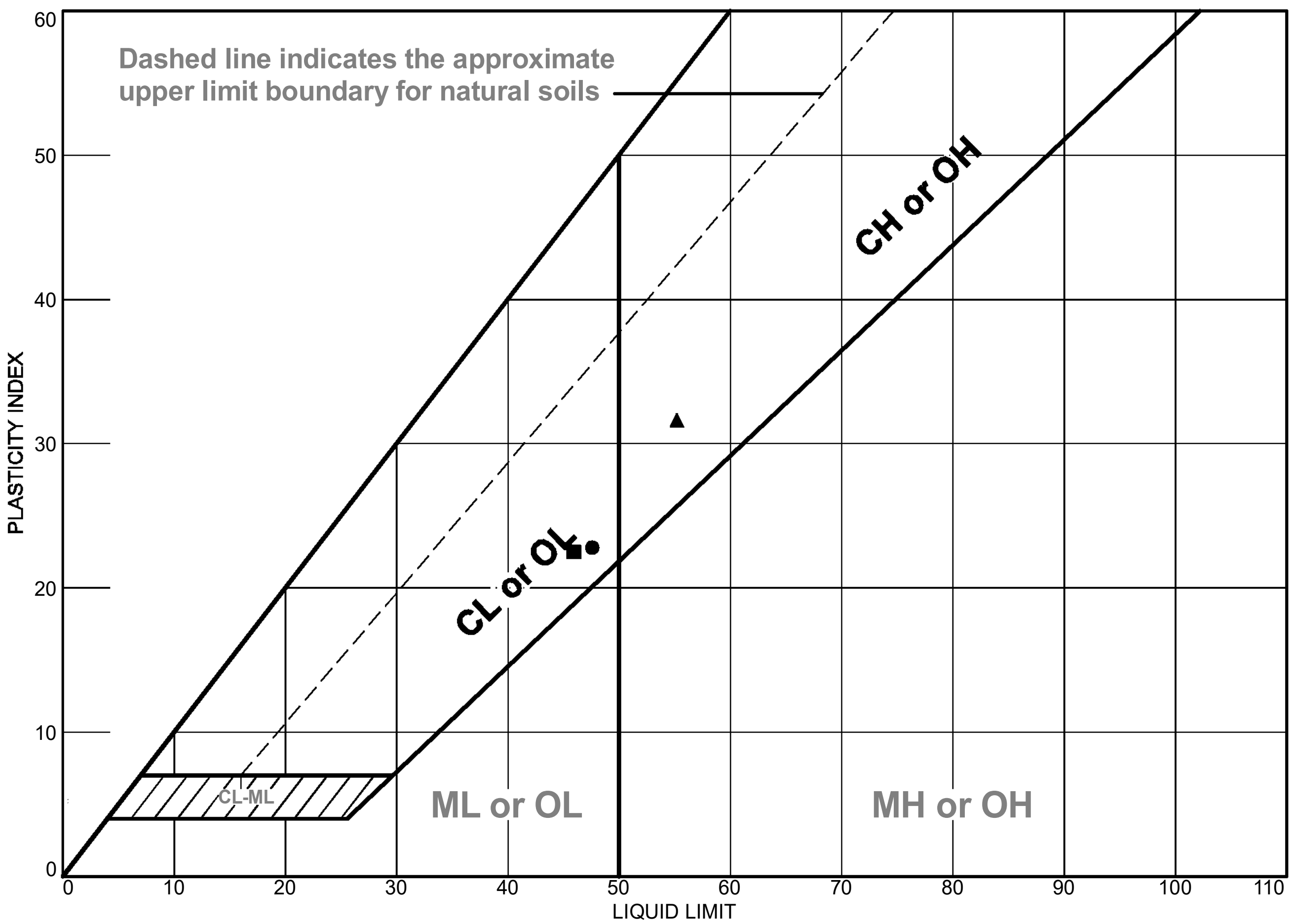
STRENGTH

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 heavy 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., thorough 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 or no effect on cementation, no mineral composition
Fresh - unaffected by weathering agents; no appreciable change with depth

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Brown Sandy Clay (CL)	47.6	24.8	22.8		67.5	CL
■	Gray/Brown Sandy Clay (CL)	45.9	23.4	22.5		69.1	CL
▲	Brown Clay W/ Sand (CH)	55.2	23.6	31.6		75.3	CH

Project No. 7362.01.04.2 **Client:** RGH Consultants

Project: 6204 Dry Creek Road Residential Project

● **Source of Sample:** TP-2 **Depth:** 0.0'-2.5'
■ **Source of Sample:** TP-6 **Depth:** 1.0'-5.0'
▲ **Source of Sample:** TP-5 **Depth:** 0-2.5

Tested By: SCW

Checked By: SEF

Remarks:
● Expansion Index = 59 (Medium)
■ Expansion Index = 59 (Medium)
Reported 12/13/19
▲ Expansion Index = 79 (Medium)
Reported: 12/19/2019

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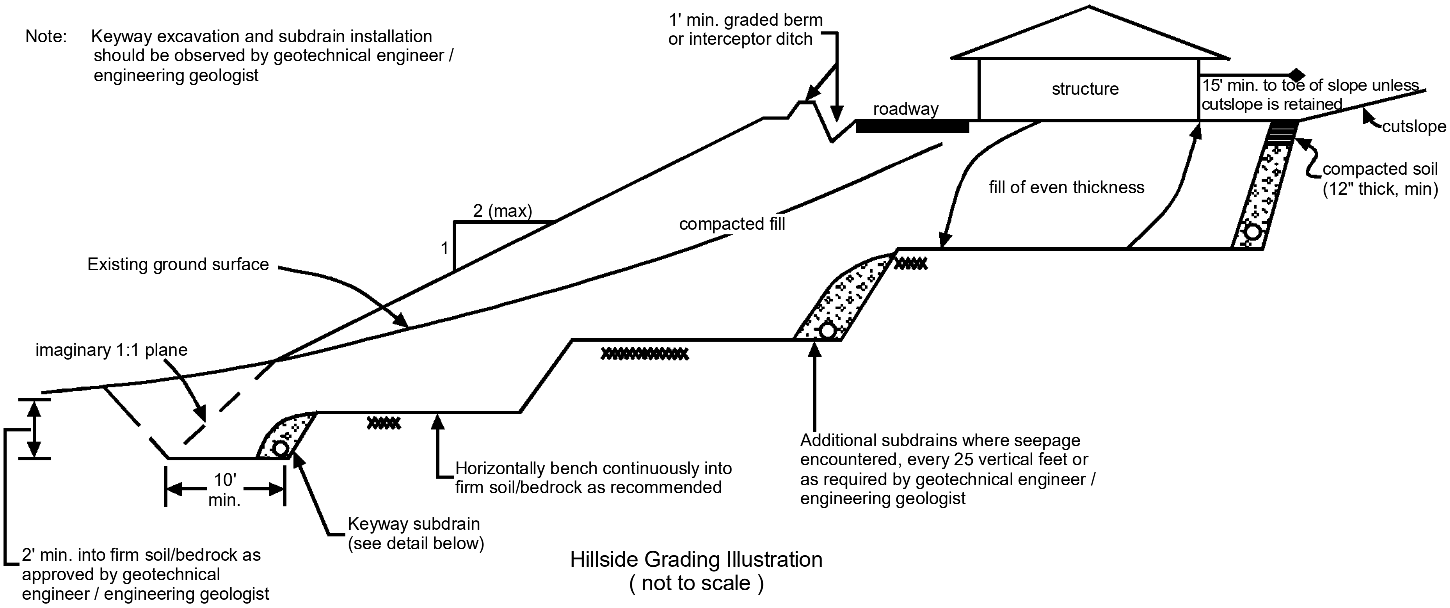
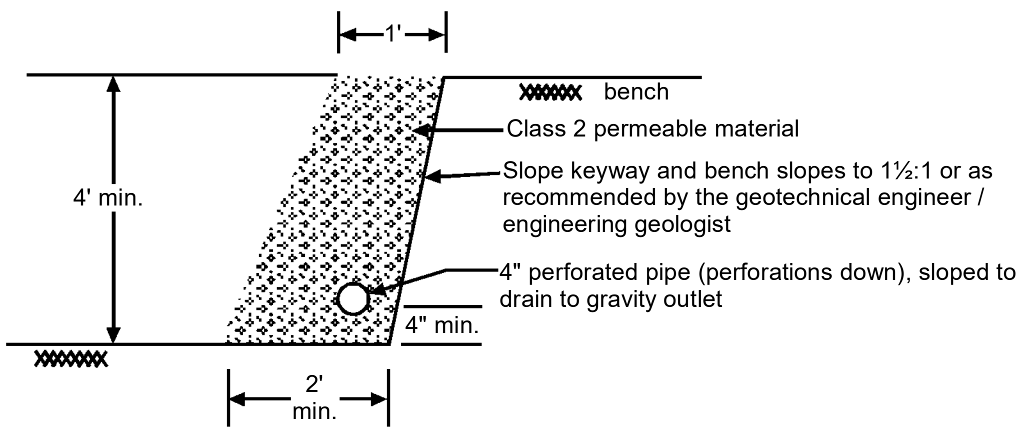
CLASSIFICATION TEST DATA
6204 Dry Creek Road Residential Project
6204 Dry Creek Road
Napa, California

PLATE 9

Job No: 7362.01.04.2 Date: DEC 2019



Hillside Grading Illustration
(not to scale)



Keyway Subdrain
(not to scale)

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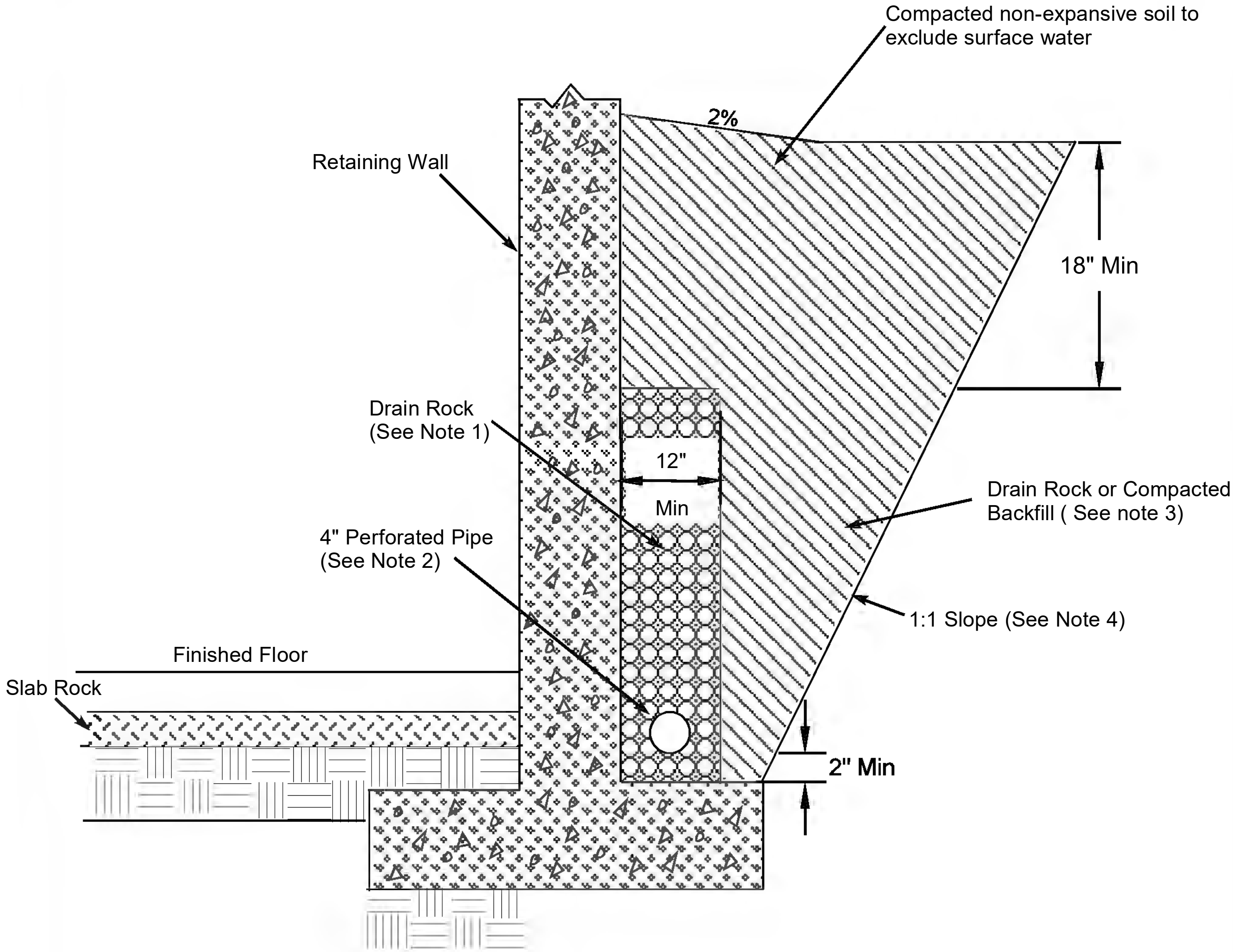
Job No: 7362.01.04.2 | Date: DEC 2019

HILLSIDE GRADING ILLUSTRATION
6204 Dry Creek Road Residential Project
6204 Dry Creek Road
Napa, California

PLATE
10



Notes:

1. Drain rock should meet the requirements for Class 2 Permeable Material, Section 68, State of California "Caltrans" Standard Specification, latest edition. Drain rock should be placed to approximately three-quarters the height of the retaining wall.

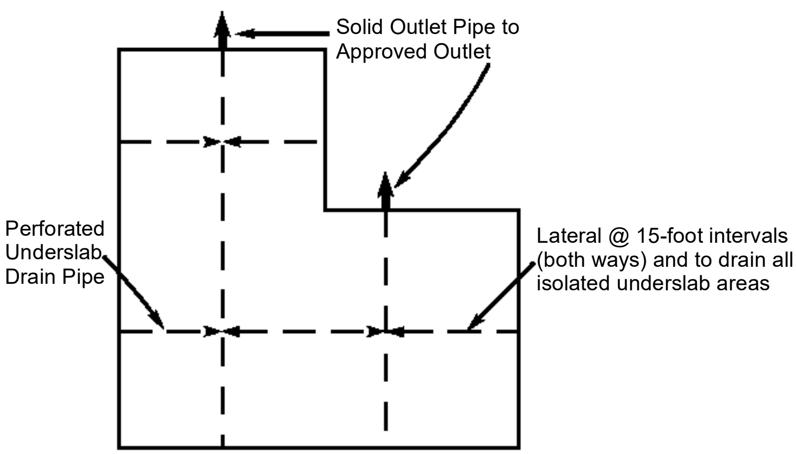
2. Pipe should conform to the requirements of Section 68 of State of California "Caltrans" Standards, perforations placed down, sloped at 1% for gravity flow to outlet or sump with automatic pump. The pipe invert should be located at least 8 inches below the lowest adjacent finished surface.

3. During construction the contractor should use appropriate methods such as temporary bracing and/or light compaction equipment to avoid overstressing the walls. Non-expansive soils to be used as backfill.

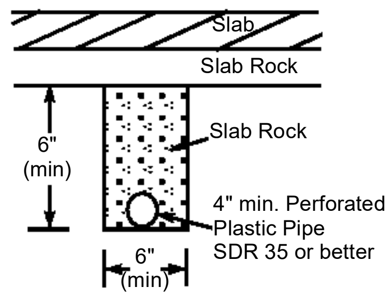
4. Slope excavation back at a 1:1 gradient from the back of footing where expansive materials are exposed.

Not to Scale

RGH CONSULTANTS Job No: 7362.01.04.2 | Date: DEC 2019	**RETAINING WALL BACKDRAIN ILLUSTRATION** 6204 Dry Creek Road Residential Project 6204 Dry Creek Road Napa, California	PLATE **11**



TYPICAL UNDERSLAB DRAIN PLAN



SLAB UNDERDRAIN

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Job No: 7362.01.04.2 | Date: DEC 2019

TYPICAL SUBDRAIN DETAILS

6204 Dry Creek Road Residential Project
6204 Dry Creek Road
Napa, California

PLATE 12

APPENDIX B - REFERENCES

American Society of Civil Engineers, 2017, Minimum Design Loads for Buildings and Other Structures, ASCE Standard ASCE/SEI 7-16.

Bryant, W.A., and Hart, E.W., Interim Revision 2007, Fault-Rupture Zones in California; California Geological Survey, Special Publication 42, p. 21 with Appendices A through F.

California Building Code, 2019, California Building Standard Commission.

Clahan, K.B., Wagner, D.L., Bezore, S.P., Sowers, J.M., and Witter, R.C., 2005, Geologic Map of the Rutherford 7.5' Quadrangle, Sonoma and Napa Counties, California: A Digital Database.

Dwyer, M.J., Noguchi, N., and O'Rourke, J., 1976, Reconnaissance Photo-Interpretation Map of Landslides in 24 Selected 7.5-Minute Quadrangles in Lake, Napa, Solano, and Sonoma Counties, California: U.S. Geological Survey OFR 76-74, 25 Plates, Scale 1:24,000.

Natural Resources Conservation Service, United States Department of Agriculture, accessed December 2019, Web Soil Survey, available online at http://websoilsurvey.nrcs.usda.gov/.

APPENDIX C - DISTRIBUTION

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BasilEnan@gmail.com

SCL:EGC:sl:nvd

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Important Information About Your
Geotechnical Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes

The following information is provided to help you manage your risks.

Geotechnical Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical engineering study conducted for a civil engineer may not fulfill the needs of a construction contractor or even another civil engineer. Because each geotechnical engineering study is unique, each geotechnical engineering report is unique, prepared *solely* for the client. No one except you should rely on your geotechnical engineering report without first conferring with the geotechnical engineer who prepared it. *And no one - not even you* - should apply the report for any purpose or project except the one originally contemplated.

Read the Full Report

Serious problems have occurred because those relying on a geotechnical engineering report did not read it all. Do not rely on an executive summary. Do not read selected elements only.

A Geotechnical Engineering Report Is Based on A Unique Set of Project-Specific Factors

Geotechnical engineers consider a number of unique, project-specific factors when establishing the scope of a study. Typical factors include: the client's goals, objectives, and risk management preferences; the general nature of the structure involved, its size, and configuration; the location of the structure on the site; and other planned or existing site improvements, such as access roads, parking lots, and underground utilities. Unless the geotechnical engineer who conducted the study specifically indicates otherwise, do not rely on a geotechnical engineering report that was:

- not prepared for you,
- not prepared for your project,
- not prepared for the specific site explored, or
- completed before important project changes were made.

Typical changes that can erode the reliability of an existing geotechnical engineering report include those that affect:

- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from alight industrial plant to a refrigerated warehouse,
- elevation, configuration, location, orientation, or weight of the proposed structure,
- composition of the design team, or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes - even minor ones - and request an assessment of their impact. *Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.*

Subsurface Conditions Can Change

A geotechnical engineering report is based on conditions that existed at the time the study was performed. *Do not rely on a geotechnical engineering report* whose adequacy may have been affected by: the passage of time; by man-made events, such as construction on or adjacent to the site; or by natural events, such as floods, earthquakes, or groundwater fluctuations. *Always* contact the geotechnical engineer before applying the report to determine if it is still reliable. A minor amount of additional testing or analysis could prevent major problems.

Most Geotechnical Findings Are Professional Opinions

Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. Geotechnical engineers review field and laboratory data and then apply their professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ-sometimes significantly from those indicated in your report. Retaining the geotechnical engineer who developed your report to provide construction observation is the most effective method of managing the risks associated with unanticipated conditions.

A Report's Recommendations Are *Not* Final

Do not overrely on the construction recommendations included in your report. *Those recommendations are not final*, because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations only by observing actual subsurface conditions revealed during construction. The geotechnical engineer who developed your report cannot assume responsibility or liability for the report's recommendations if that engineer does not perform construction observation.

A Geotechnical Engineering Report Is Subject to Misinterpretation

Other design team members' misinterpretation of geotechnical engineering reports has resulted in costly problems. Lower that risk by having your geotechnical engineer confer with appropriate members of the design team after submitting the report. Also retain your geotechnical engineer to review pertinent elements of the design team's plans and specifications. Contractors can also misinterpret a geotechnical engineering report. Reduce that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing construction observation.

Do Not Redraw the Engineer's Logs

Geotechnical engineers prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical engineering report should *never* be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, *but recognize that separating logs from the report can elevate risk.*

Give Contractors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can make contractors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give contractors the complete geotechnical engineering report, *but* preface it with a clearly written letter of transmittal. In that letter, advise contractors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with the geotechnical engineer who prepared the report (a modest fee may be required) and/or to conduct additional study to obtain the specific types of information they need or prefer. A prebid conference can also be valuable. *Be sure contractors have sufficient time* to perform additional study. Only then might you be in a position to give contractors the best information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions.

Read Responsibility Provisions Closely

Some clients, design professionals, and contractors do not recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that have led to disappointments, claims, and disputes. To help reduce the risk of such outcomes, geotechnical engineers commonly include a variety of explanatory provisions in their reports. Sometimes labeled "limitations" many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The equipment, techniques, and personnel used to perform a *geoenvironmental* study differ significantly from those used to perform a *geotechnical* study. For that reason, a geotechnical engineering report does not usually relate any geoenvironmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated environmental problems have led to numerous project failures.* If you have not yet obtained your own geoenvironmental information, ask your geotechnical consultant for risk management guidance. *Do not rely on an environmental report prepared for someone else.*

Obtain Professional Assistance To Deal with Mold

Diverse strategies can be applied during building design, construction, operation, and maintenance to prevent significant amounts of mold from growing on indoor surfaces. To be effective, all such strategies should be devised for the express purpose of mold prevention, integrated into a comprehensive plan, and executed with diligent oversight by a professional mold prevention consultant. Because just a small amount of water or moisture can lead to the development of severe mold infestations, a number of mold prevention strategies focus on keeping building surfaces dry. While groundwater, water infiltration, and similar issues may have been addressed as part of the geotechnical engineering study whose findings are conveyed in-this report, the geotechnical engineer in charge of this project is not a mold prevention consultant; ***none of the services performed in connection with the geotechnical engineer's study were designed or conducted for the purpose of mold prevention. Proper implementation of the recommendations conveyed in this report will not of itself be sufficient to prevent mold from growing in or on the structure involved.***

Rely on Your ASFE-Member Geotechnical Engineer For Additional Assistance

Membership in ASFE/The Best People on Earth exposes geotechnical engineers to a wide array of risk management techniques that can be of genuine benefit for everyone involved with a construction project. Confer with your ASFE-member geotechnical engineer for more information.

ASFE
The Best People on Earth

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