GRAY
GEOTECH

GEOTECHNICAL REPORT UPDATE

Proposed Development of Lots 59 and 60
Clayton Street and Ross Street Terrace, San Rafael
Project No. 20-018-01

April 1, 2020

Submitted to:

Coby Friedman

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1 Introduction and Project Description

At your request, we are providing this geotechnical report to assist with your planning and design for the development of two residential lots located near the intersection of Ross Street Terrace and Clayton Street in San Rafael, California.

We have been provided a site vicinity plan issued by Joseph Farrell Architecture (sheet A0.0 dated November 2019) which show each of the two subject lots is planned to be developed with a single-family dwelling having a footprint of about 1,400 to 1,500 square feet. Based on the specified finish floor elevations, it appears that the dwellings on both will be located on a cut-fill pad (where the back half of the building is planned to be terraced into the hillside and the front half will be on new fill. The proposed earthwork contours are not shown, but we anticipate the cut to be up to about 9 feet deep and the fill to be about 3 feet thick for Lot 59, and the cut to be up to 13 feet deep and the fill to be about 2 feet thick for Lot 60. Access to both lots is currently planned to be from Ross Street Terrace, which is currently an unimproved public right-of-way (dirt and gravel trail) located along the downslope side of the lots. Development of this right of way appears to require retaining walls on both the upslope and downslope side in order to widen the new road to about 20 feet wide. The heights of the walls are not specified, but we understand that they may be on the order of about 12 feet high in some locations.

We have also been provided a previous geotechnical report (issued by Geotechnical Engineering Inc in 2016) for this project, which included subsurface exploration with two borings in each of the two proposed building pads. The purpose of our report is to update the 2016 GEI report to address changes to the proposed access to the two lots, and to provide updated design parameters, primarily related to implementation of the new building code (2019 California Building Code). We have based this report on our site reconnaissance to visually observe the ground surface, a subsurface exploration consisting of three additional borings, and review of publicly available regional geologic maps or studies of the area.

This report is limited to the evaluation of the soil conditions near the proposed improvements. Evaluation of other areas of the property or for different future improvements is not within the scope of this report.

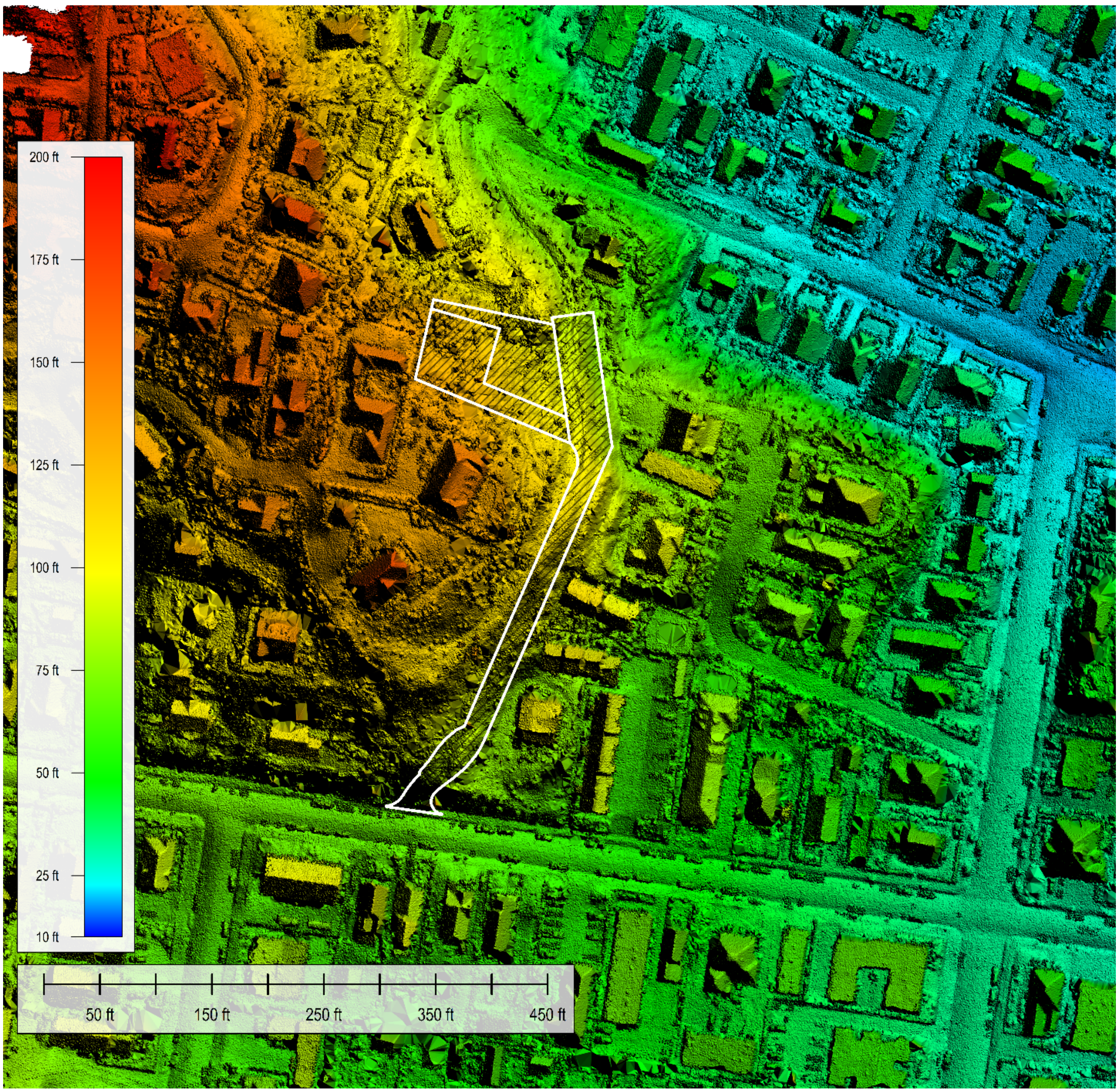
1.1 Site Description

The project area consists of two vacant lots (APN 012-141-59 and APN 012-141-60); Lot 59 having an area of about 5851 square feet and Lot 60 having an area of about 2,028 square feet. Access to the site is currently limited to foot traffic by a gravel road located east of both lots, which varies between about 6 to 10 feet wide and is overgrown with Oak, Acacia and other brush. We understand that this road is listed by the County Assessor as Ross Street Terrace and as a separate parcel (the assessors map appears to show it as a public road rather than an easement within the subject lots). Based on our review of historic aerial photographs, it appears that the initial grading to form this road may have occurred sometime between 1946 and 1952. The site location and the portion of Ross Street Terrace to be improved is shown in the Site Vicinity Map (Figure 1), below.

Figure 1: Site Vicinity Map

We obtained data from a 2017 LiDAR survey of the area, which was performed under direction of the USGS. This LiDAR survey consists of aerial laser measurements generating point elevations of the structures, vegetation and the ground surface. With this data we were able to generate a digital elevation map of the site, which shows the site is located on the north-eastern face of a small spur ridge. The ground surface descends from an elevation of 134 feet at the southwestern corner of Lot 59 to about 94 feet at the eastern boundary of Lot 60. The alignment of Ross Street Terrace generally follows the contour along the end of the spur ridge, and appears to have been formed with shallow cuts 2 to 4 feet high on the upslope side with the spoils side cast as fill on the downhill side. The elevation of the centreline appears to gently slope up from around 88 feet at the subject lots, to a high point of about 108 feet at the midpoint (south of the lots), and then descending to about 70 feet at the “intersection” with Ross Street. The southern terminus of Ross Street Terrace is a steep cut bank, sloping at about 1:1.7 (H:V) over a span of about 25 feet. The elevations referenced here are based on the NAVD88 datum.

Figure 2: Site Topography

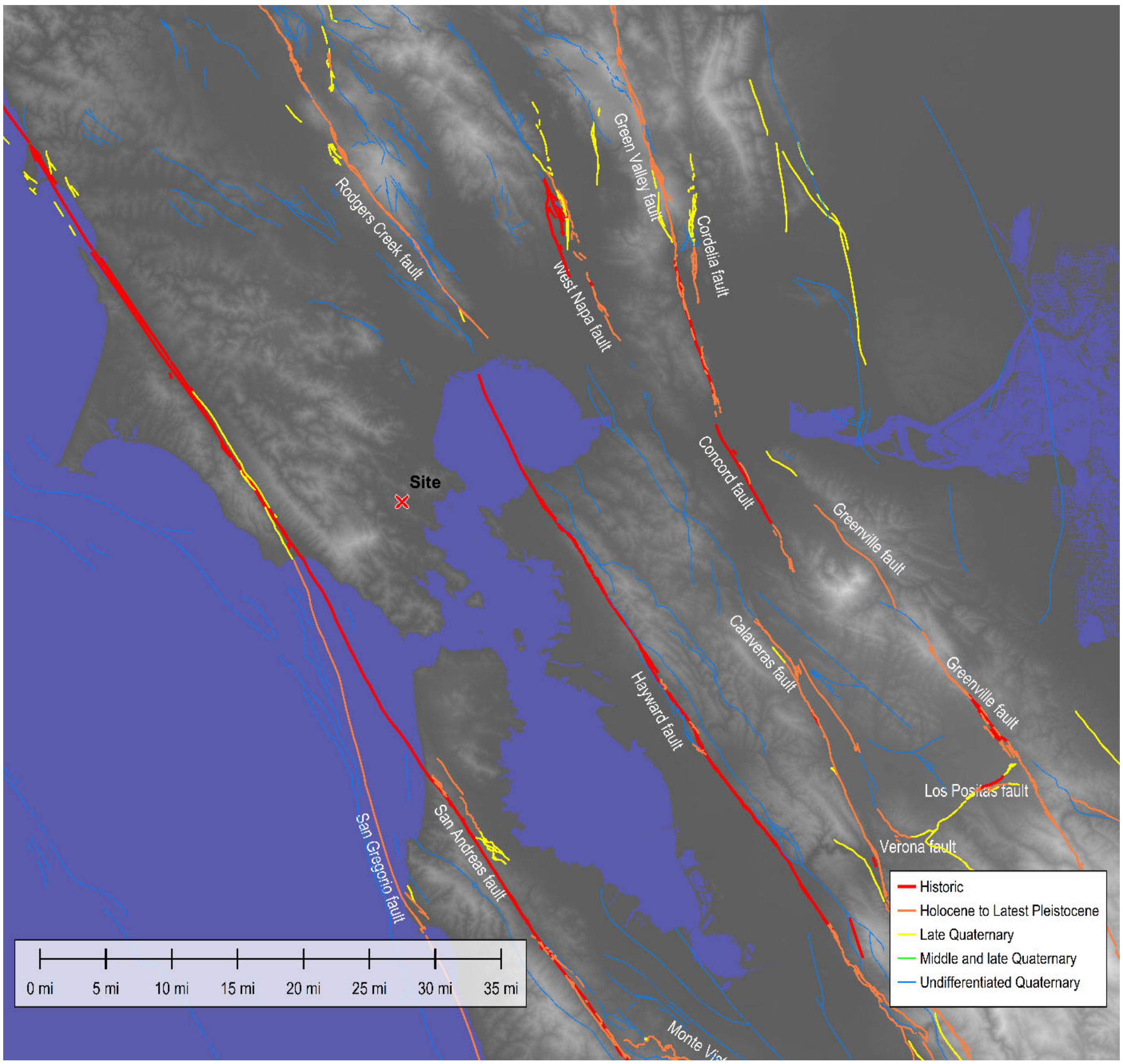


2 Findings

2.1 Faults and Seismicity

The San Francisco Bay Area is a seismically active area; at least 20 strong earthquakes measuring M6 or greater have occurred in the Bay Area in the last 200 years (Ellsworth, 1990), many of these would have likely resulted in moderate to severe ground shaking at the site. The map below shows the location of faults that have been historic and ancient earthquake sources in the San Francisco Bay Area, classed based on the age of last known movement.

Figure 3: USGS Quaternary Fault Map



It is likely the site will experience one or more episodes of strong ground shaking during the design life of the proposed improvements. The United States Geologic Survey and State of California have developed an earthquake rupture forecast; an estimate on “when and where” a future earthquake might occur amongst the State’s many faults. This model (referred to as UCERF3) provides a 63 percent probability that M6.7 or greater earthquake will occur in the Bay Area by 2044, and classes the Hayward fault and the Calaveras fault as two of the area’s “particularly ready faults”.

The table below summarizes significant active faults located within 50 km of the project site, including estimated slip rates and Maximum Moment Magnitude. The maximum moment magnitude earthquake (Mmax) is defined as the largest earthquake that a given fault is considered capable of generating.

Table 1: Distance to known active faults

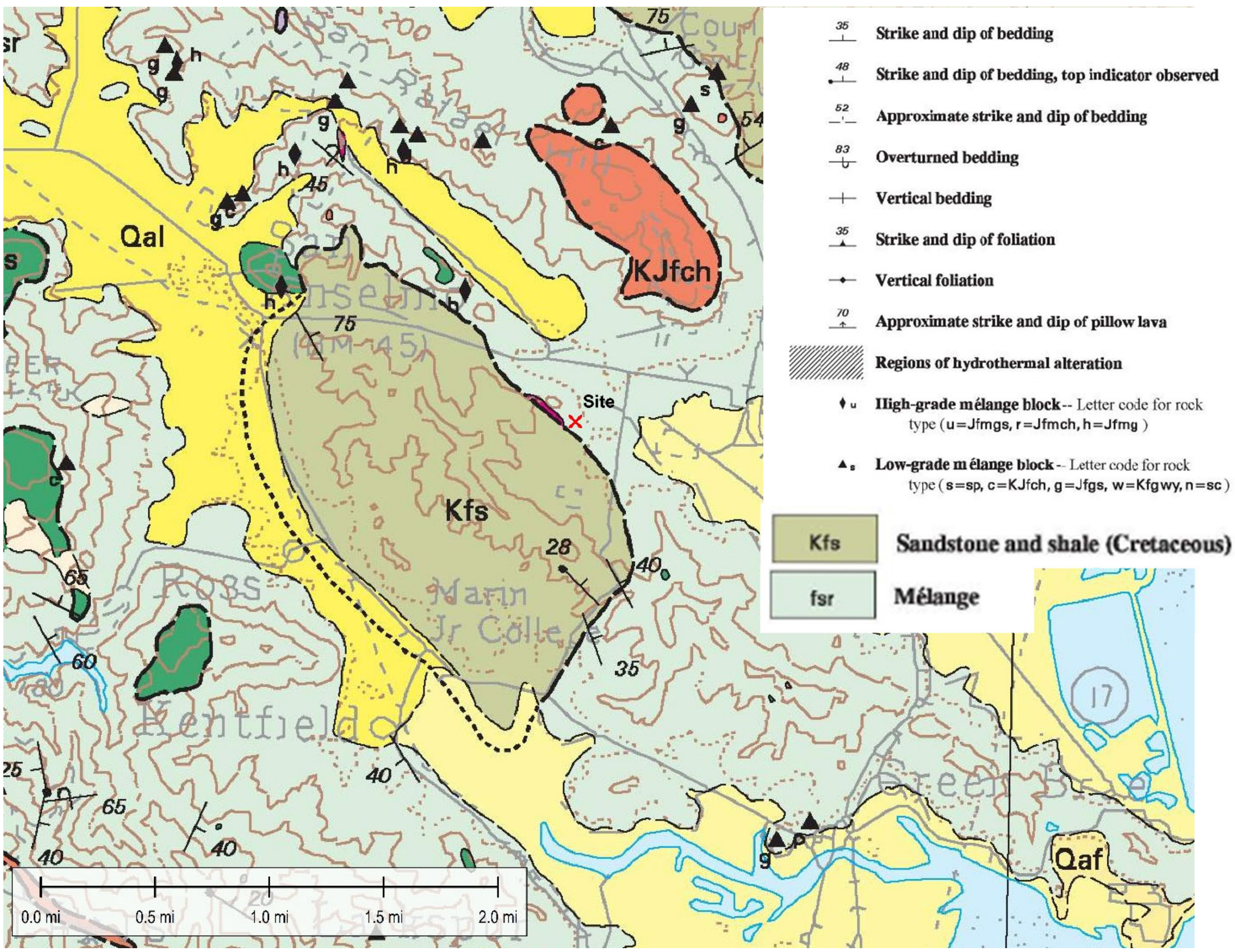
Fault Name	Distance to Site	Maximum Moment Magnitude	Slip Rate (mm/year)
Calaveras Fault	45.4 km	7.0	6 ±2
Concord - Green Valley Fault	39.5 km	6.8	4 ±2
Greenville Fault	50 km	6.2	2 ±1
Hayward Fault	14.5 km	7.0	9 ±2
Rodgers Creek Fault	23.7 km	7.1	9 ±2
San Andreas Fault	13.8 km	7.9	24 ±3
West Napa Fault	33.2 km	6.7	1 ±1

In addition to ground shaking, other seismic hazards include fault rupture or displacement. The State of California has prepared a series of maps known as Seismic Hazard Zone Maps (SHZM), which delineate regulatory hazard zones in accordance with the Alquist-Priolo Act. The initial hazard zone shown on the SHZM is along active earthquake faults. In addition, the SHZM have been periodically updated since 1972 to include other risks such as earthquake induced landslides and liquefaction. Projects within these zones require special studies (fault investigation reports) to attempt to identify the location of fault trace(s) and to confirm the age of the last fault activity or to determine the risk of property damage from liquefaction or landslide movement. The site is not located within an AP fault rupture zone, liquefaction zone or earthquake trigged landslide hazard zone.

2.2 Regional Geology

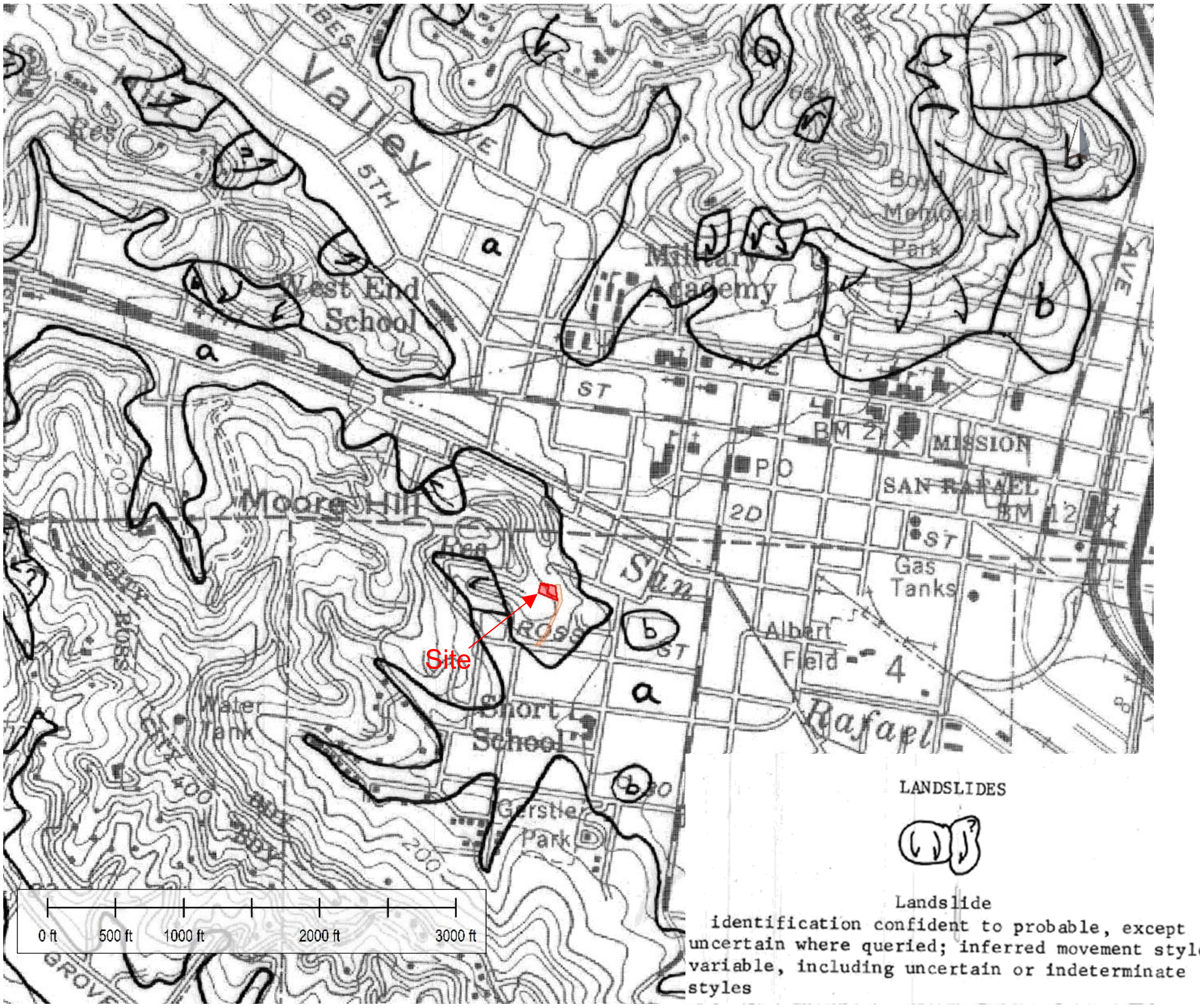
The site is located in the Coastal Range geomorphic province of California. The Coastal Range province is characterized by a series of nearly parallel northwest-trending mountain ranges and alluviated valleys that were formed from tectonic activity between the Pacific and North American Plates. Considerable faulting, deformation, and erosion have resulted in irregular topography and contacts between the various geologic units. A regional geologic map (below) shows the site is underlain by Melange (fsr) of the Franciscan complex; a Jurassic-aged bedrock unit that consists of variably sheared shale and sandstone, and containing hard inclusions (from tectonic mixing), typically of greenstone, chert, and graywacke. An outcrop of Cretaceous-aged Sandstone and Shale (Kfs) is shown southwest of the site with a geologic contact near the intersection of Ross Street and Ross Street Terrace. This outcrop is characterized to have similar properties to the fsr-unit, but typically with thicker bedding and less tectonic inclusions of hard rock.

Figure 4: Regional Geology (USGS MF-2337)



A map prepared by the USGS (Wentworth 1975) identifies areas of hillside deposits that may be at risk to ground movement (based on interpretation of aerial photographs) to redflag sites that may require further site-specific investigation prior to development. Based on this map (shown below), there are a few landslide deposits within hillside areas west of the site, but no known slides or areas of potential slide deposits were mapped within the site, adjacent parcels or the Ross Street Terrace. The map indicates that young sedimentary deposits (a) may be located along Ross Street.

Figure 5: Distribution of Landslides and Earthflows (USGS OFR 75-281)



Another USGS study issued in 1976 also shows regional slope stability, based on the mapped location and properties of geologic units and the steepness of slopes. This map is not site-specific, but intended to broadly characterize relative slope stability of areas into four categories (which shows the subject lots are mapped in Zone 2/3, and the Ross Street Terrace improvement area is mapped in Zone 2).

Figure 6: Relative Stability of Slopes (OFR 76-02)

Zone 1 - The most stable category. This zone includes resistant rock that is either exposed or is covered only by shallow colluvium or soil. Also included in this zone are broad, relatively level areas along the tops of ridges or in valley bottoms that may be underlain by material that is quite weak (such as Franciscan melange matrix and alluvium) but occupies a relatively stable position. Some landslide deposits that have moved to relatively stable positions at or beyond the base of the slopes from which they were derived are also included in zone 1.

Zone 2 - Includes narrow ridge and spur crests that are underlain by relatively competent bedrock, but are flanked by steep, potentially unstable slopes.

Zone 3 - Areas where the steepness of the slopes approaches the stability limits of the underlying geological materials. Some landslide deposits that appear to have relatively more stable positions than those classified within zone 4 are also shown here.

Zone 4 - The least stable category. This includes most landslide deposits in upslope areas, whether presently active or not, and slopes on which there is substantial evidence of downslope creep of the surface materials. These areas should be considered naturally unstable, subject to potential failure even in the absence of man's activities and influences. Banks along deeply incised streams are also included in zone 4.

These judgments are interpretive, and apply generally to large areas. Within each area conditions may range locally in detail through all stability categories. Hence, an area designated 1 may locally contain unmapped landslides, and an area designated 4 may locally contain relatively stable sites.

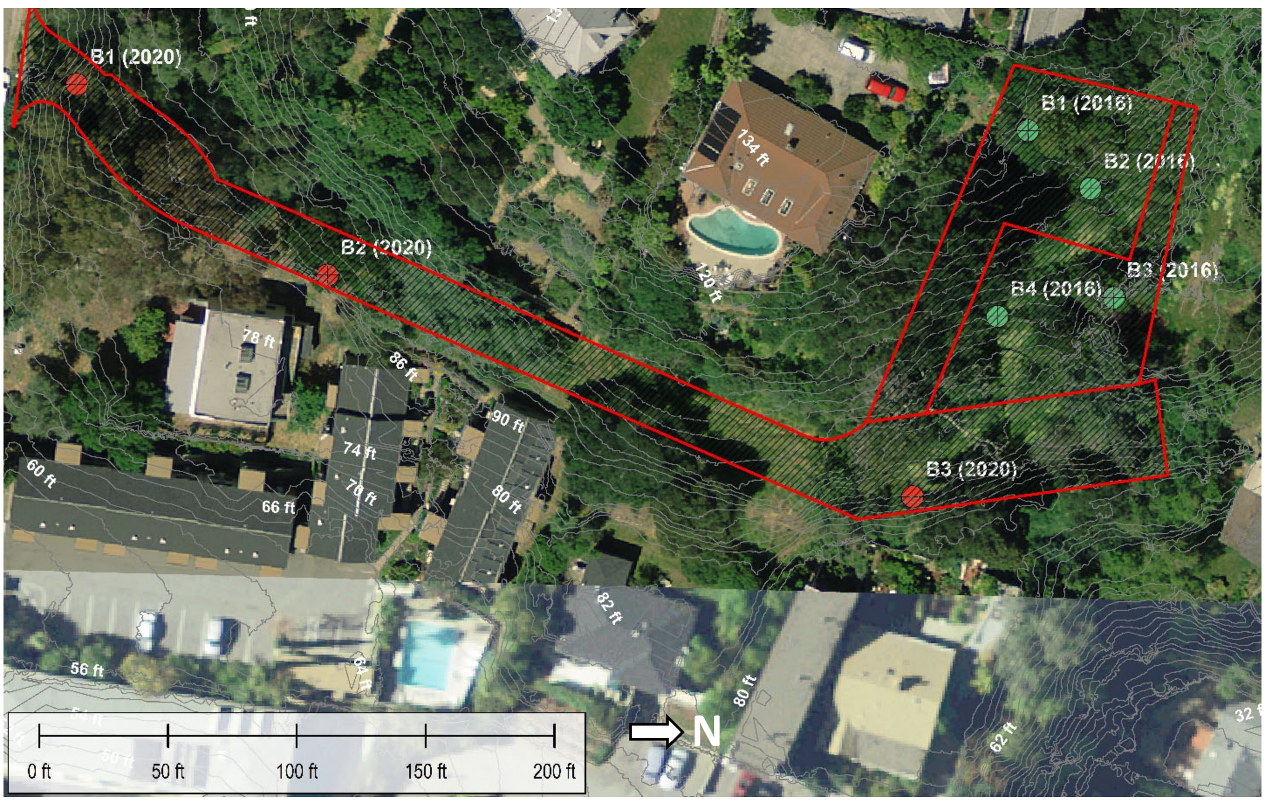
2.3 Subsurface Investigations

We carried out a subsurface exploration consisting of three borings to a depth up to 7.4 feet below the current ground surface. We visually classified the soil samples in accordance with ASTM D-2488-93, Standard Practice for Description and Identification of Soils (Visual-Manual Procedure), and have attached the bore logs as Appendix 1.

We generally encountered 1.7 to 3.5 feet of native sandy silt soil overlaying sandstone bedrock, with the exception of B3 where we encountered 4 feet of fill with a buried topsoil layer at a depth between 4 to 6.8 feet, and a probable depth to bedrock at 7.4 feet. This is consistent with the previous exploration data from 2016, which generally encountered 1.5 to 3.0 feet of "silty clay with many sand" overlaying sandstone bedrock.

Groundwater was not encountered at the time of either the 2016 or 2020 exploration, however we did encounter very wet soil in B3. We anticipate that variations in the groundwater level at the site are likely to occur due to changes in precipitation and irrigation practices. It is our opinion that the subsurface conditions are consistent with the regionally mapped geology.

Figure 7: Subsurface Exploration Location Map



3 Assessment of Findings

3.1 General

It is our opinion that the site is suitable for the proposed project from a geotechnical point of view, provided the recommendations presented in this report and standard development practices are incorporated in the design and construction of the project.

The primary geotechnical considerations for the project are the potential of unsuitable fill, buried objects or tree roots from past land use, slope stability and retaining wall design, ground motions in a future seismic event, and managing storm water. These considerations are discussed further in the following sections.

3.2 Earthwork

3.2.1 Site Preparation

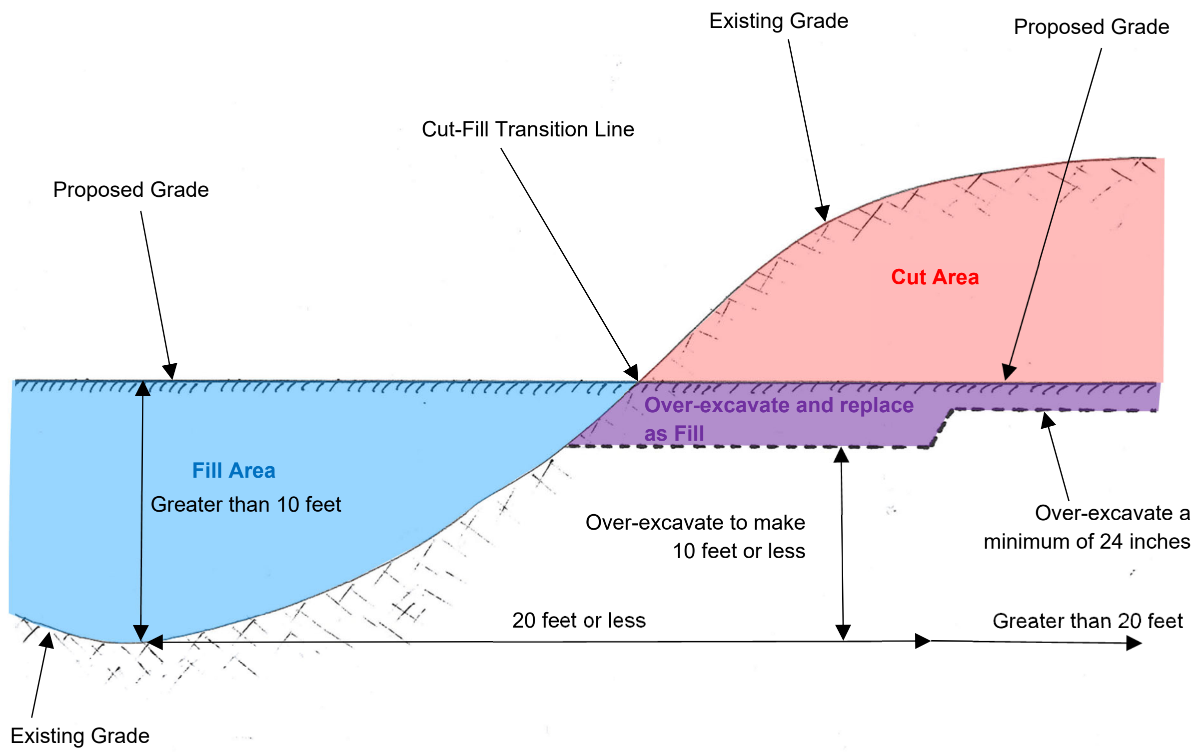
The building site and areas to receive fill should be stripped of all topsoil, existing (undocumented) fill, soil with heavy roots or an organics content greater than 6 percent, desiccated material and debris. In areas of tree or shrub removal, where planned structures or roadways are located, the full root ball and/or main mass of roots at the base of the plant should be removed.

Existing underground utilities, tanks or structures, if affected by construction activities, should be removed or relocated prior to site development. Debris generated from the demolition of underground facilities, including abandoned pipes, should be removed from the site as construction proceeds. If pipes are abandoned in place, they should be capped or filled to mitigate the potential of water seepage, loss of soil into the pipe, or risk of pipe collapse. In general, this may be accomplished with filling pipes greater than 4-inches in diameter with a plug of lean cement that has a length at least 4 times the pipe diameter or to the extent of the property boundary, and smaller pipes may be sealed with an end cap.

3.2.2 Cut-Fill Transitions

Improvements constructed across a cut-fill transition may experience differential settlement and cracking due to the difference in stiffness between exposed or shallow bedrock in the cut areas and compacted fill. We recommend that cut areas are over-excavated by a minimum of 24-inches (below the proposed civil grade levels) and recompacted with fill to minimize the cut-fill transition. We recommend a greater over-excavation of the cut areas for the case where there may be deep fills adjacent to cuts; in this case we recommend that the over-excavation depth is extended to keep the maximum fill thickness to be no greater than 10 feet thick within a 20-foot distance of the cut area (as shown in the figure below).

Figure 8: Typical Detail for Cut-Fill and Differential Fill Treatment



3.2.3 Excavation Stability

The contractor is the sole party responsible for excavation stability and compliance with OSHA work site safety regulations. Trenches or narrow excavations greater than 4 feet deep may require shoring for worker safety. Temporary cut slopes should be benched at a gradient no steeper than 1:1 (H:V) or retained. As a preliminary value, we recommend that an undrained shear strength of 500 psf, a soil unit weight of 120 pcf and an active pressure coefficient of k_a=0.35 be used to determine the required shoring type.

Temporary shoring or underpinning may be required if excavations to construct new foundations or underground utilities extend below a 2:1 (H:V) plane projected downward from the bottom of existing foundations or adjacent property boundaries.

3.2.4 Excavatability of Rock

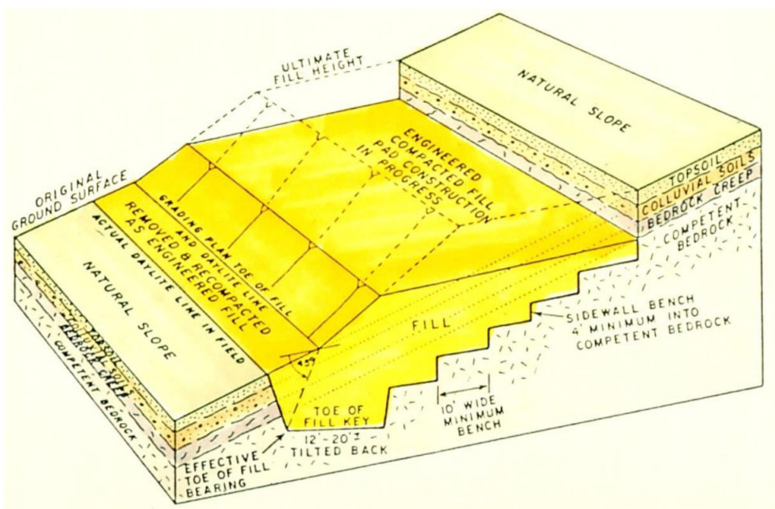
It is our opinion that excavations may be made with conventional heavy construction equipment. We anticipate excavation of the bedrock will be achievable by ripping along the weak planes or along preexisting joints and fractures in the bedrock. Use of a penetrating tip digging tooth (i.e. tiger teeth) or a single-tine ripper may reduce the excavation effort required if harder sandstone beds or hard tectonic inclusions are encountered. Percussion or diamond core drilling may be required if hard inclusions are encountered in pier holes or drilled shafts for ground anchors. The selection of equipment and mean/methods of excavation are ultimately the responsibility of the excavation contractor. They should consider a test program to confirm the suitability of their equipment and whether other methods may be more productive.

3.2.5 Placement of Fill

Following site preparation (as outlined in Section 3.2.1) the subgrade in fill areas should be scarified and moisture conditioned. Depending on the time of year, fill material should be blended and allowed to temper following addition of water and prior to compaction.

Fill should be benched and keyed if the existing ground surface is steeper than 4:1 (H:V). We recommend that the benching remove topsoil or other poor quality soil with sidewalls cut 4 feet into competent material. Keyways should have a minimum embedment of 2 feet into intact rock or approved native soil, have a minimum width of 10 feet and the bottom of the keyway should be graded at 2% into the slope to a 4-inch perforated drain pipe.

Figure 9: Typical Detail for Earthwork Keyway and Benching (Orange County Grading Manual, 1965)



Fill should be placed in lifts not exceeding approximately 8-inches in loose thickness and compacted using mechanical compaction equipment. Unless otherwise specified or approved, material to be used as structural fill and backfill should be non-expansive with the following properties:

- predominantly granular material should be well-graded with crushed or angular particles (typically with 75% having at least two fracture faces), and;
- particles should be less than 4 inches in any dimension, and;

- isolated cobbles up to 12 inches in diameter may remain in the fill, provided the oversized material is not nesting or stacked together to form voids or prevent compaction of the smaller soil particles, and;
- free of organic and inorganic debris, and;
- contain less than 30 percent of mostly non-plastic fines passing the No. 200 sieve, and;
- have a liquid limit less than 35 and plasticity index less than 12.

Non-expansive aggregate fill should be compacted to at least 95 percent of the maximum dry density and at or above the optimum moisture point as determined per ASTM D1557. Onsite soil should be evaluated by the geotechnical engineer prior to use as engineered fill and approved low or moderate plasticity soils should be compacted to between 88 to 92 percent of the maximum dry density and at least 4 points over the optimum moisture content. Test results of imported fill and backfill materials should be submitted to the geotechnical engineer prior to delivery to site (or stockpiling of onsite material for reuse), to confirm that they meet the above criteria.

3.3 Slopes

We recommend that final slopes that are formed or altered by earthworks have a gradient no steeper that 3:1 (H:V). We recommend that a concrete lined drainage channel is formed at the bottom of the slope, above retaining walls or the above the structures to intercept surface runoff and convey the flow to an approved outlet. We recommend that erosion control measures are placed on disturbed slopes until the final vegetation/landscaping is established to be able to reduce sediment runoff.

3.4 Seismic Design and Ground Motion Parameters

Based on the regional geology, subsurface conditions encountered mapped seismic ground motions determined using ASCE 7-16 procedures and 2019 California Building Code, we present site coefficients for seismic design on Table 2, below.

Table 2: Seismic Design Criteria

	Factor	Value
Site Class		C
Mapped Short-Period MCE_R, g	S_{s*}	1.5
Mapped MCE_R at 1 second, g	S_{1*}	0.6
Short Period Site Coefficient	F_a	1.2
Long Period Site Coefficient	F_v	1.4
Site Short-Period MCE_R, g	S_{MS}	1.8
Site MCE_R at 1 second, g	S_{M1}	0.84
Short Period Design Spectral Response Acceleration Parameter, g	S_{DS}	1.2
1 second Design Spectral Response Acceleration Parameter, g	S_{D1}	0.56
Peak Ground Acceleration	PGA	0.644

Seismic design provisions of current building codes generally prescribe minimum lateral forces, applied statically to the structure, combined with the gravity forces of dead-and-live loads. The prescribed lateral forces are generally considered to be substantially smaller than the actual peak forces that would be associated with a major earthquake. Consequently, structures should be able to: (1) resist minor earthquakes without damage, (2) resist moderate earthquakes without structural damage but with some nonstructural damage, and (3) resist major earthquakes without collapse but with some structural as well as nonstructural damage. Conformance to the current building code recommendations does not constitute any kind of guarantee that significant structural damage would not occur in the event of a maximum magnitude earthquake; however, it is reasonable to expect that a well-designed and well-constructed structure will not collapse or cause loss of life in a major earthquake (SEAOC, 1996).

3.5 Drilled Pier Foundations

We recommend that the dwelling and associated retaining walls are supported with cast-in-place straight-shaft concrete piers. Temporary casing (such as with a cardboard form or sonotube) may be required to maintain a straight shaft and constant diameter at the top of the pier. Dewatering methods or other methods to address groundwater (such as tremie placement of the concrete) should be coordinated with the structural engineer and implemented as needed during construction.

The axial capacity of the piers should be determined using an allowable skin friction value of 800 psf, neglecting the capacity from the upper 3-feet of embedment and all capacity from the tip (end bearing). We recommend a minimum pier diameter of 14-inches and a minimum length of 10 feet below grade (either existing or final grade, whichever is greatest).

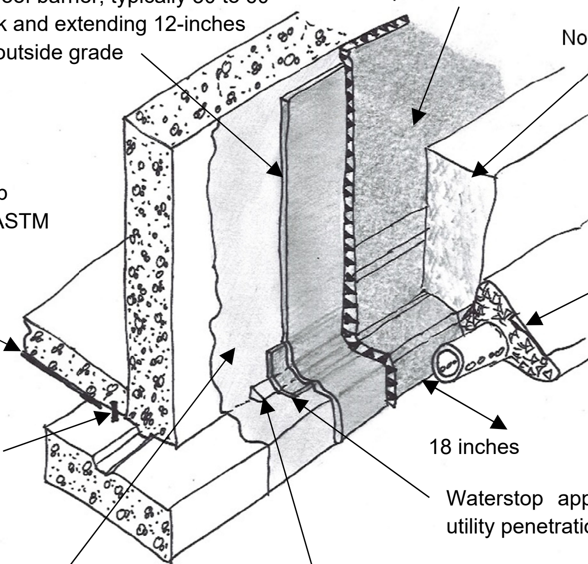
We recommend that the lateral capacity of the piers is calculated assuming passive pressure with an equivalent fluid pressure of Kp=3.0 and a unit weight of 110 pcf, acting over two pier diameters. The lateral capacity of the upper portion of embedment should be neglected where a descending slope or lower retaining wall is located within 10 feet (ignore the capacity until the embedded portion has at least a 10-foot horizontal distance to daylight). Where piers are spaced at least 5 diameters (5D), we recommend use a p-multiplier of 1.0. Piers within the same row and spaced 3D should use a p-multiplier of 0.8. Piers within the same row and spaced 2D should use a p-multiplier of 0.5. If there is a case where there are multiple rows (such as where there is another row within 4D in front/downslope), the second-row piers should use a p-multiplier of 0.4 and third-row piers should use a p-multiplier of 0.3.

The structural engineer should specify details of steel reinforcement, and if a greater pier diameter or embedment is required depending on the load capacity required. Special design and detailing requirements will be required to account for lateral soil deformation. We recommend a series of grade beams or foundation ties to interconnect the piers or pile caps in both directions. The grade beams should have a minimum embedment of 12-inches to reduce seepage into the crawlspace. The foundation ties should be designed to have a tensile and compressive strength capability of preventing up to 2.5-inch of lateral deformation between piers, or with a tensile/compressive capacity at least equal to a force of 10% SDS (1.222) multiplied by the pile or column load (dead+live). The pier and connection to the cap or superstructure should be designed to remain ductile assuming a lateral spread of 6-inches and a differential settlement 2-inches across the building platform. The residual pile lateral strength must remain at least 67% of the nominal capacity. Concrete piers should be designed to comply with Section 18.7.5.2 through 18.7.5.4 of ACI 318 from the top of the pile to a depth of 20 feet.

3.6 Dampproofing and Waterproofing

Slab-on-grade floors and basement walls or partially below grade walls that are part of the dwelling (or other areas treated as conditioned space with moisture sensitive finishes) shall be coated with an approved waterproofing material on the exterior or pressure side. Slab-on-grade floor slabs should be underlain by a vapor barrier meeting ASTM 1745 Class A specifications. Ensure that this barrier is strong enough to resist puncture during slab construction. Joints and penetrations should be sealed with a waterproof material. There are several types of waterproofing products available; the application of each system varies depending on the specific system. We recommend referring to the manufacture's specifications for detailed design requirements and determining other products that may be required (such as primers, adhesives, sealants, etc.), and installation should be carried out by a contractor familiar with the selected system. We provide the following sketch for general concepts and best-practices for basement waterproofing systems.

Figure 10: Typical Basement Waterproofing Concepts



3.1 Retaining Wall Lateral Load Design

Retaining walls should be designed to retain the backfill and any additional lateral forces due to traffic or surface loads. Where no movement is tolerable at the top of the wall, at-rest earth pressures need to be resisted. If the wall is allowed to deflect outward at the top at least 0.02 H, where H is the wall height, the wall may be designed to resist active pressures. We recommend the use of the following parameters:

- Active Pressure Coefficient, K_a=0.5 (level backslope), K_a=0.7 (2:1 backslope)
- At-rest Pressure Coefficient, K_o=0.7 (level backslope), K_o=0.9 (2:1 backslope)
- Soil Unit Weight, γ=110 pounds per cubic foot

The wall design should consider horizontal accelerations during seismic events for individual walls higher than 6 feet, or tiered walls with a cumulative height higher than 6 feet. We recommend that lateral design consider the following dynamic pressures in addition to the static lateral pressure:

- Displacing Retaining Walls, ∆Kae=0.25*PGA
- Non-Displacing Retaining Walls, ∆Kae=0.75*PGA

The internal stability and wall stress should consider a seismic lateral earth pressure with a triangular pressure distribution where the resultant seismic force would act at 0.3H above the base of the wall. Sliding and overturning resistance of retaining walls shall be designed with a minimum safety factor of 1.5 in the static case and 1.1 in the seismic case.

The above values do not include lateral forces due to hydrostatic pressures. Therefore, we recommend that the wall backfill be free draining with provisions to collect and dispose of excess water that may accumulate, as discussed in section 3.3 below.

For tiered wall systems, where an upper wall footing is located within a 1:1 projection line extending from the heel of the retaining wall, the lower retaining wall(s) should consider surcharge loads from walls and soil backfill located above. The surcharge load of buildings or other structures should be included if they are located within a 1:1 projection line extending from the bottom of the retaining wall.

3.2 Secondary Slabs, Pavement and Exterior Flatwork

We recommend that slabs and other exterior hardscape areas, including concrete patios, are supported directly on a layer of compacted non-expansive fill at least 12 inches thick and structurally independent from the perimeter foundations and “free-floating”. Slab-on-grade should be expected to crack. Control joints should be at a maximum spacing of 10 feet in both directions. The slabs should be designed as a rigid slab capable of resisting shrink/swell movement of expansive soil without significant deformation or cracking. Secondary slabs-on-grade should be designed specifically for their intended use and loading requirements. As a minimum requirement and in reference to the WRI design manual for slab on grade foundations with expansive soils, we recommend that the slabs are designed to support a cantilever span of 9 feet and reinforced with a minimum Asfy of 5,200 lbs.

3.3 Surface Drainage and Storm Water Management

Ponding of storm water should not be permitted near or under the building or footings during prolonged periods of inclement weather.

We recommend that the building pad is positively graded at all times to provide for rapid removal of surface water runoff from around the foundation elements and to prevent ponding of water or seepage toward the foundation systems at any time during or after construction. As a minimum requirement, finished grades in landscaped areas should have downslopes of at least 5 percent within 10 feet from the exterior walls to allow surface water to drain positively away from the structures. For hardscape areas, the slope gradient can be reduced to 2 percent. Storm water from roof downspouts should be directed to a solid pipe. We recommend that drains are sloped at a minimum of 5 percent towards an engineered drain system.

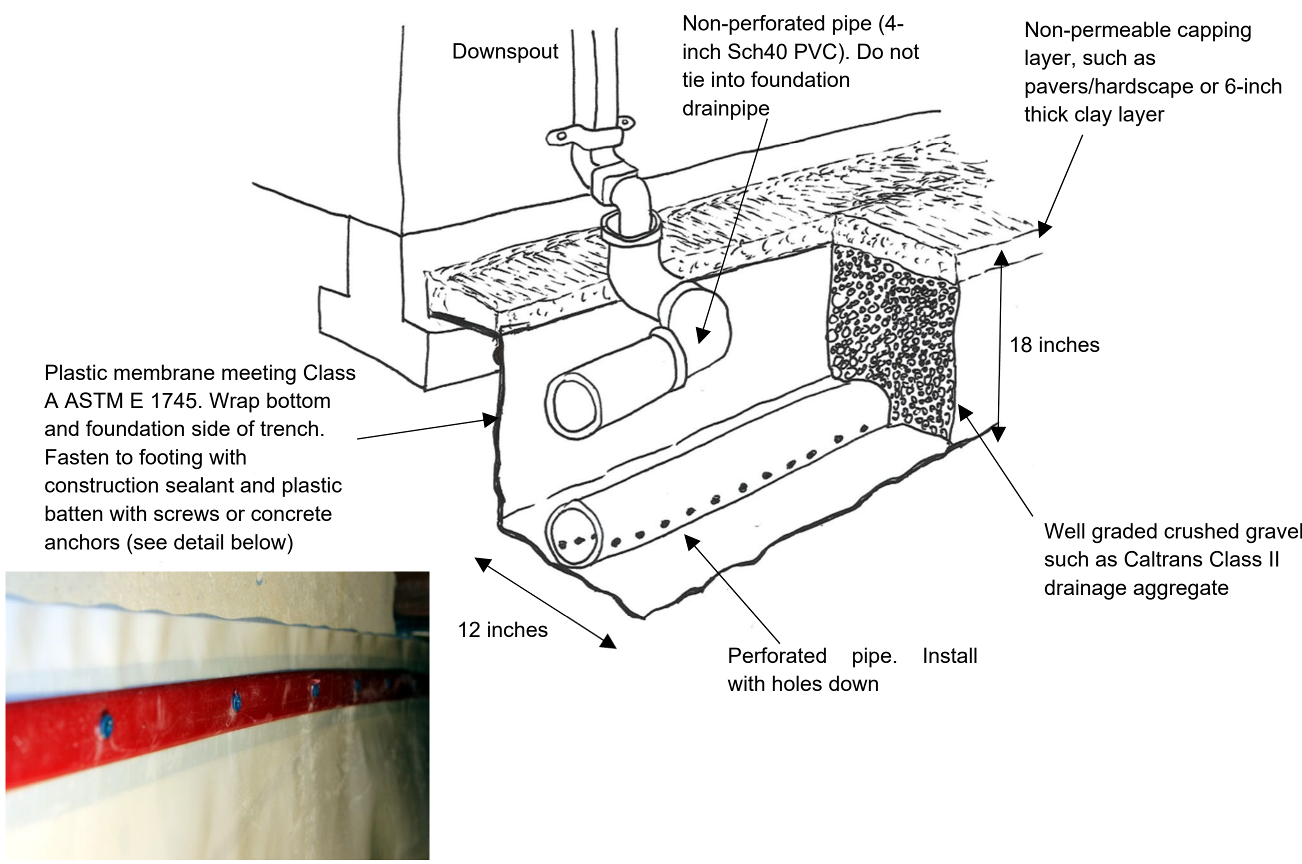
Based on the shallow depth to bedrock, it is our opinion that storm water management practices that rely on percolation into the native soil will not be effective. In addition, infiltration stormwater into the hillside may decrease slope stability or cause saturated soil loading onto retaining walls. As such, we recommended pavement areas are constructed using concrete or asphalt, and interlocking pavers or other pervious pavements which allow a higher amount of water to infiltrate the soil are not used.

In addition to the above, we recommend the following measures to minimize soil moisture problems with the foundation:

- Maintenance of drains and downspouts to prevent water from saturating the soil around the building.
- Eliminating or reducing irrigation of garden areas around the building.
- As practical, do not install landscaping where the root zone of nearby plants results in wicking/drying of the soils under the building. This is typically the case where a tree's dripline is adjacent or over the building footprint, or where the roots extend under the foundation.
- Construction of a foundation subdrain along the outside building perimeter to stabilize the soil moisture variation under the building. The exact location of these drains may vary, but should be placed relative to the pattern of surface water flow and upstream of the dwelling.

We recommend that the foundation drain pipes are separate from the pipes that carry water from downspouts and surface drain inlets. The downspouts from roof gutters should be connected to the drain system in a way that prevents overtopping or spilling of the captured runoff or backflow into the subdrain pipes. The figure below shows typical drain details.

Figure 11: Typical Foundation Subdrain Detail



Subdrains should be provided with adequately spaced cleanouts so that their effectiveness can be monitored in the future. Use 4-inch diameter or larger, rigid-walled, PVC drainage pipe (Class SDR35 or stronger) with glued joints. Exposed PVC drain inlets and downspout adapters may be protected with opaque covering or painted with acrylic-based latex paint to increase UV protection.

Excavation of perimeter drains parallel to existing footings should remain outside the foundation influence zone (above a 1:1 plane extending down from the bottom of the footing), make use of 'hit-and-miss' excavation staging, or use underpinning to prevent undermining the existing footing.

4 Corrosion

The American Concrete Institute guideline ACI318 outlines exposure categories regarding the attack on concrete (due to water-soluble sulfate) or reinforcing steel (due to water-soluble chloride ion). The durability design depends on the exposure class, typically specifying a maximum water-cement ratio, a minimum compressive strength or use a specific cement type. We did not perform site-specific soil corrosivity tests, and recommend consulting with a corrosion engineer or your structural engineer regarding measures that may be appropriate for the protection of buried steel or concrete in contact with soil and bedrock.

5 Construction Considerations

The geotechnical engineer should review project plans and specifications prior to construction to check that the geotechnical aspects of the project are consistent with the intent of the recommendations presented herein. This is to confirm that geotechnical conditions have been interpreted with the intent of the recommendations of this report. In addition, the local Building Official may require we issue a letter documenting our review of the final plans, to be included with the permit submittal. We recommend that the designers discuss their designs with us as their work progresses to avoid surprises at submittal time, which could delay the job and commencement of construction.

Although the information in this report is primarily intended for the design engineers, it may also be useful to the contractors. However, it is the responsibility of the bidders and contractors to evaluate soil and groundwater conditions independently and to develop their own conclusions and designs regarding excavation, grading, foundation construction, and other construction or safety aspects.

We recommend that the following items are visually inspected, tested or documented during the construction (by appropriately qualified personnel which are engaged by the owner and independent of the contractor).

- Observe site preparation in areas to receive fill, observe ground conditions exposed in excavation of deep assess, and consultation regarding the need for over-excavation to remove unsuitable material
- Evaluate and approve material to be used as engineered fill
- Observation of fill placement and record measurements of relative compaction and moisture content of engineered fills
- Observation and documentation of the as-build excavation/drilling depth of footings, piers, or other foundation elements

6 Closing

This report has been prepared in accordance with generally accepted professional geotechnical engineering practice for the exclusive use of Coby Friedman in relation to the specified project brief described in this report. In the event that any changes are made in the character, design, or layout of the proposed project, the conclusions and recommendations contained in this report should be reviewed by Gray Geotech Inc. to determine whether modifications to the report are necessary.

No other warranty, express or implied, is made. No liability is accepted for the use of any part of the report for any other purpose or by any other person or entity. The analyses and recommendations submitted in this report are based on the ground conditions indicated from published sources and investigations described in this report based on accepted normal methods of site investigations. Only a limited amount of information has been collected to meet the specific financial and technical requirements of the Client's brief in accordance with our work agreement dated March 30, 2020, and this report does not purport to completely describe all the site characteristics and properties. The nature and extent of variations within the project site may not become evident until construction. In the event variations occur, it will be necessary to reevaluate the recommendations of this report.

Subsurface conditions relevant to construction works should be assessed by contractors who can make their own interpretation of the factual data provided. They should perform any additional tests as necessary for their own purposes.

We hope this provides the information that you require at this time. If you have any questions, please contact us.

Sincerely,

Joe Gray, GE

GRAY GEOTECH

APPENDIX 1

Bore Logs and Lab Test Data

LOG OF EXPLORATORY BORING

B1

GRAY GEOTECH

PROJECT: Lot 59 & Lot 60
LOCATION: Ross Street Terrace, San Rafael, CA
LAT (NORTHING): 37° 58' 08.9920" N
LONG (EASTING): 122° 32' 12.1678" W

DATE STARTED: 2/24/2020
DATE FINISHED: 2/24/2020
LOGGED BY: JMG

SURFACE ELEVATION: 82 ft
MEASURED TOTAL DEPTH: 2 feet
INCLINATION: 0 deg
AZIMUTH: -- deg (True North)

DRILLING CONTRACTOR: Gray Geotech
DRILLING METHOD: Hand Auger
BORE DIAMETER: 4 inch
HAMMER:
*blow counts are uncorrected (raw field value)

** PP: Pocket Penetrometer (in-situ)
** TOR: Torvane (in-situ)
** Qu: Unconfined Compresive Strength (lab)
** TxUU: Triaxial Shear, Unconsolidated Undrained (lab)
** TxCU: Triaxial Shear, Consolidated Undrained (lab)
** TxCD: Triaxial Shear, Consolidated Drained (lab)

DEPTH - TVD (feet)	ELEVATION (feet)	USCS GRAPHIC	SAMPLES: Sample Interval	Sample Type	Blows/ Foot*	MATERIAL DESCRIPTION	Unit Weight (pcf)	Moisture Content (%)	Liquid Limit	Plasticity Index	Strength**	Remarks
0	82											
0.1	81.9											
						Sandy Organic Soil (OL/OH)-- olive brown (2.5Y 4/3), moist. Organic Topsoil						
0.2	81.8											
0.3	81.7											
0.4	81.6											
0.5	81.5											
0.6	81.4											
0.7	81.3											
0.8	81.2											
0.9	81.1											
1	81					Sandy Silt (ML)-- light olive brown (2.5Y 5/6), moist. Abundant roots						
1.1	80.9											
1.2	80.8											
1.3	80.7											
1.4	80.6											
1.5	80.5											
1.6	80.4											
1.7	80.3											
1.8	80.2											
						Weathered Sandstone (SM)-- olive brown (2.5Y 4/3), dry. Highly fractured						
1.9	80.1											
	80											

LOG OF EXPLORATORY BORING

B2

GRAY GEOTECH

PROJECT: Lot 59 & Lot 60
LOCATION: Ross Street Terrace, San Rafael, CA
LAT (NORTHING): 37° 58' 09.9698" N
LONG (EASTING): 122° 32' 11.2708" W

DATE STARTED: 2/24/2020
DATE FINISHED: 2/24/2020
LOGGED BY: JMG

SURFACE ELEVATION: 90 ft
MEASURED TOTAL DEPTH: 3.8 feet
INCLINATION: 0 deg
AZIMUTH: -- deg (True North)

DRILLING CONTRACTOR: Gray Geotech
DRILLING METHOD: Hand Auger
BORE DIAMETER: 4 inch
HAMMER:
*blow counts are uncorrected (raw field value)

** PP: Pocket Penetrometer (in-situ)
** TOR: Torvane (in-situ)
** Qu: Unconfined Compresive Strength (lab)
** TxUU: Triaxial Shear, Unconsolidated Undrained (lab)
** TxCU: Triaxial Shear, Consolidated Undrained (lab)
** TxCD: Triaxial Shear, Consolidated Drained (lab)

DEPTH - TVD (feet)	ELEVATION (feet)	USCS GRAPHIC	SAMPLES Sample Interval	Sample Type	Blows/ Foot*	MATERIAL DESCRIPTION	Unit Weight (pcf)	Moisture Content (%)	Liquid Limit	Plasticity Index	Strength**	Remarks
0												
0.2	89.8											
0.4	89.6											
0.6	89.4											
0.8	89.2											
1	89											
1.2	88.8											
1.4	88.6											
1.6	88.4											
1.8	88.2					Sandy Elastic Silt (MH)-- olive brown (2.5Y 4/3), moist. Abundant roots						
2	88											
2.2	87.8											
2.4	87.6											
2.6	87.4											
2.8	87.2											
3	87											
3.2	86.8											
3.4	86.6											
3.6	86.4					Weathered Sandstone (SM)-- very dark grayish brown (2.5Y 3/2), dry. Highly fractured						
	86.2											

LOG OF EXPLORATORY BORING

B3

GRAY GEOTECH

PROJECT: Lot 59 & Lot 60
LOCATION: Ross Street Terrace, San Rafael, CA
LAT (NORTHING): 37° 58' 12.2332" N
LONG (EASTING): 122° 32' 10.2602" W

DATE STARTED: 2/24/2020
DATE FINISHED: 2/24/2020
LOGGED BY: JMG

SURFACE ELEVATION: 90 ft
MEASURED TOTAL DEPTH: 7.4 feet
INCLINATION: 0 deg
AZIMUTH: -- deg (True North)

DRILLING CONTRACTOR: Gray Geotech
DRILLING METHOD: Hand Auger
BORE DIAMETER: 4 inch
HAMMER:
*blow counts are uncorrected (raw field value)

** PP: Pocket Penetrometer (in-situ)
** TOR: Torvane (in-situ)
** Qu: Unconfined Compresive Strength (lab)
** TxUU: Triaxial Shear, Unconsolidated Undrained (lab)
** TxCU: Triaxial Shear, Consolidated Undrained (lab)
** TxCD: Triaxial Shear, Consolidated Drained (lab)

DEPTH - TVD (feet)	ELEVATION (feet)	USCS GRAPHIC	SAMPLES: Sample Interval	Sample Type	Blows/ Foot*	MATERIAL DESCRIPTION	Unit Weight (pcf)	Moisture Content (%)	Liquid Limit	Plasticity Index	Strength**	Remarks
0												
0.4	89.6											
0.8	89.2											
1.2	88.8											
1.6	88.4											
2	88					Silty Gravel with Sand (GM)-- olive brown (2.5Y 4/3), moist. Fill. Some imported gravel (3/4-inch crushed, clean), and roots. Trace glass and sandstone fragments.						
2.4	87.6											
2.8	87.2											
3.2	86.8											
3.6	86.4											
4	86											
4.4	85.6											
4.8	85.2											
5.2	84.8											
5.6	84.4					Sandy Lean Clay (CL)-- very dark grayish brown (2.5Y 3/2), medium plasticity, wet. Buried topsoil under fill						
6	84											
6.4	83.6											
6.8	83.2											
7.2	82.8					Clayey Sand (SC)-- yellowish brown (10YR 5/6), wet. Iron oxide staining and some sandstone fragments. Refusal at 7.4 ft BGS						

BORING 1

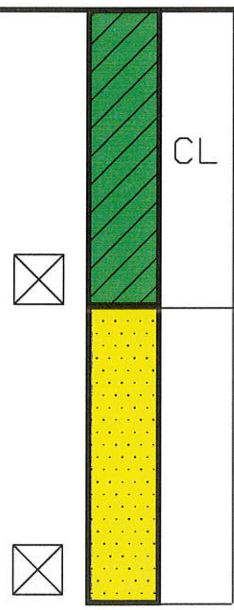
1" Diameter Percussion Hole

Depth
(Ft.)

Drilled 6/23/16

Depth in Feet

0 —



6" Top Soils

Light brown Silty Clay,
with many sand, lsightly
plastic, medium
to stiff

1 —

Ave. Moisture Content = 9%

Light yellow brown
Sandstone,
weathered, fractured
and jointed,
very stiff

2 —

Ave. Moisture Content = 7%

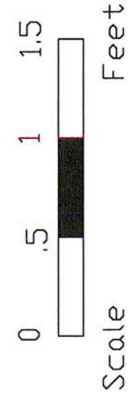
3 —

Hard,
Refusal @ 3'

Note: Free ground water not encountered
Refusal @ 3' on 6/23/16

4 —

LOG OF BORING



GEOTECHNICAL ENGINEERING, INC.
PLATE 3

BORING 2

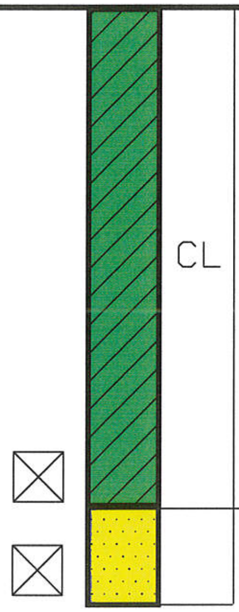
1" Diameter Percussion Hole

Drilled 6/23/16

Depth (Ft.)

Depth in Feet

0 —



6" Top Soils

1 —

Light brown Silty Clay, with many sand, lsightly plastic, medium stiff to stiff

2 —

Ave. Moisture Content = 8%

Ave. Moisture Content = 7%

3 —

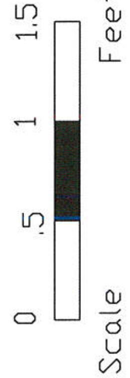
Light yellow brown Sandstone, weathered, fractured and jointed, very stiff

Hard, Refusal @ 3'

4 —

Note: Free ground water not encountered
Refusal @ 3' on 6/23/16

LOG OF BORING



GEOTECHNICAL ENGINEERING, INC.
PLATE 4

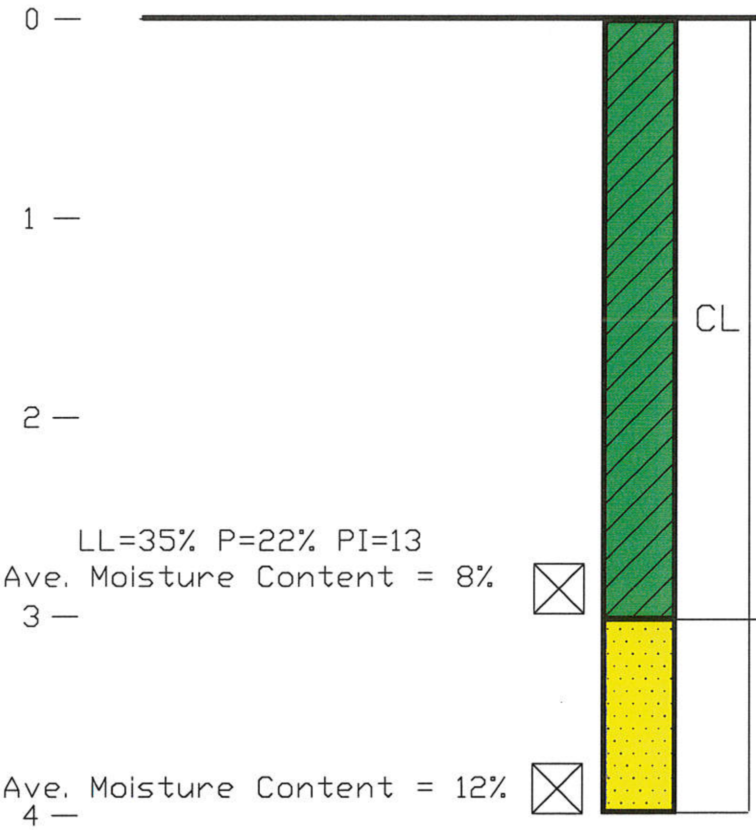
BORING 3

1" Diameter Percussion Hole

Drilled 6/23/16

Depth (Ft.)

Depth in Feet



6" Top Soils

Light brown Silty Clay, with many sand, lsightly plastic, medium stiff to stiff

LL=35% P=22% PI=13

Ave. Moisture Content = 8%

Light yellow brown Sandstone, weathered, fractured and jointed, very stiff

Ave. Moisture Content = 12%

Hard, Refusal @ 4'

Note: Free ground water not encountered
Refusal @ 4' on 6/23/16

LOG OF BORING

Scale 0 .5 1 1.5 Feet

GEOTECHNICAL ENGINEERING, INC.
PLATE 5

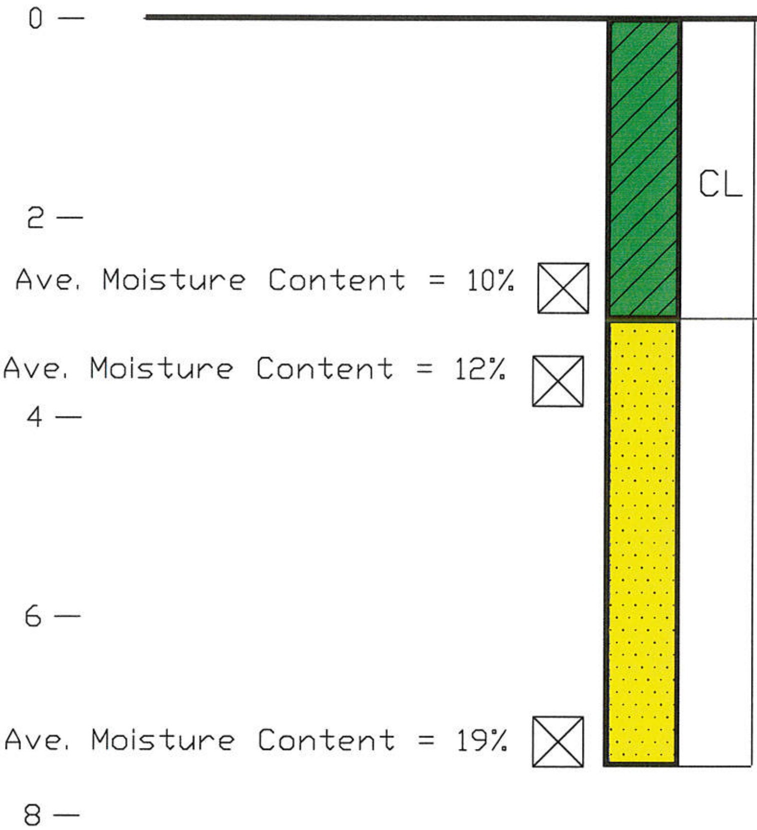
BORING 4

1" Diameter Percussion Hole

Drilled 6/23/16

Depth (Ft.)

Depth in Feet



Ave. Moisture Content = 10%

Ave. Moisture Content = 12%

Ave. Moisture Content = 19%

6" Top Soils

Light brown Silty Clay, with many sand, lsightly plastic, medium stiff to stiff

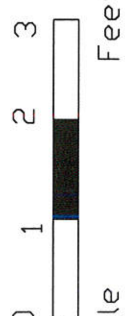
Light yellow brown Sandstone, weathered, fractured and jointed, very stiff

Light brown Sandstone, very stiff to hard

Hard, Refusal @ 7.5'

Note: Free ground water not encountered
Refusal @ 7.5' on 6/23/16

LOG OF BORING



GEOTECHNICAL ENGINEERING, INC.
PLATE 6

BORING 4

SAMPLE 1

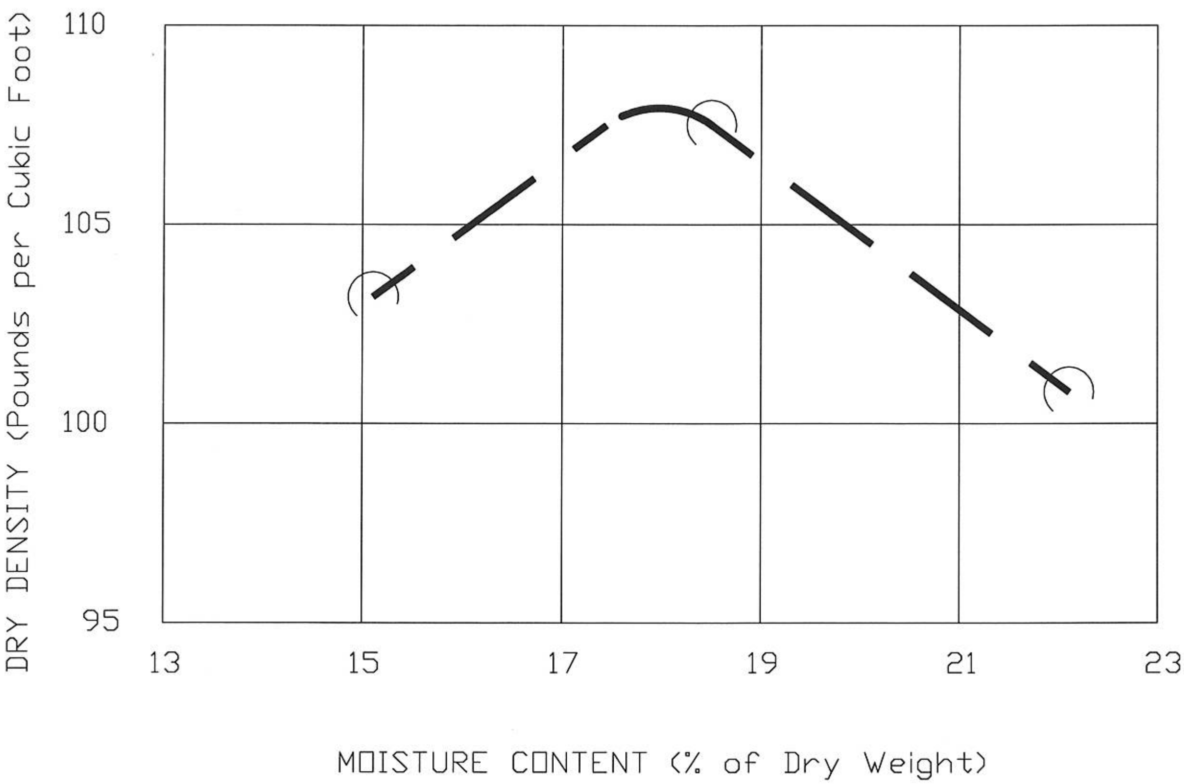
DEPTH .7'

MAXIMUM DRY DENSITY = 108 Pounds Per Cubic Foot

OPTIMUM MOISTURE CONTENT = 18 percent

AMERICAN SOCIETY FOR TESTING & MATERIALS DESIGNATION: D:1557-78

(Modified Proctor Compaction Method)



COMPACTION TEST DATA

GEOTECHNICAL ENGINEERING, INC.

PLATE 7

GRAY GEOTECH

ASTM D4318-10

Standard Test Method for Liquid Limit, Plastic Limit and Plasticity Index of Soils

Project Name: Ross Street Terrace

Project Number: 20-018-01

Boring Number/Sample Location: B3

Depth 5.2

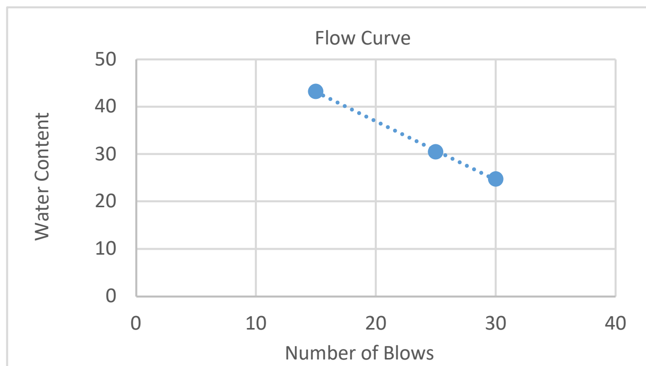
Sample Number:

SPECIMEN PREPARATION			
Wet:	x	Washed on #40 Sieve:	
Dry (Air):		Dry Sieved on #40 Sieve:	
Dry (Oven):		Mechanically Pushed Through #40 Sieve:	
		Mixed on Glass Plate and Removed Medium+ Sand Particles:	x
Mixing Water:	Distilled	x Demineralized	Other

TESTING EQUIPMENT USED		
Plastic Limit:	Hand Rolled	x
	Mech Rolling Device	
Liquid Limit:	Manual	x
	Mechanical	
Casagrande/ASTM Grooving Tool:	Metal	x
	Plastic	

AS-RECEIVED WATER CONTENT (OVEN DRIED)

Container No.	4
Mass Moist Soil + Container, M1 (g)	52.6
Mass Dry Soil + Container, M2 (g)	45.8
Mass Container, M3 (g)	13.3
WATER CONTENT, Wc (%)	20.9



PLASTIC LIMIT

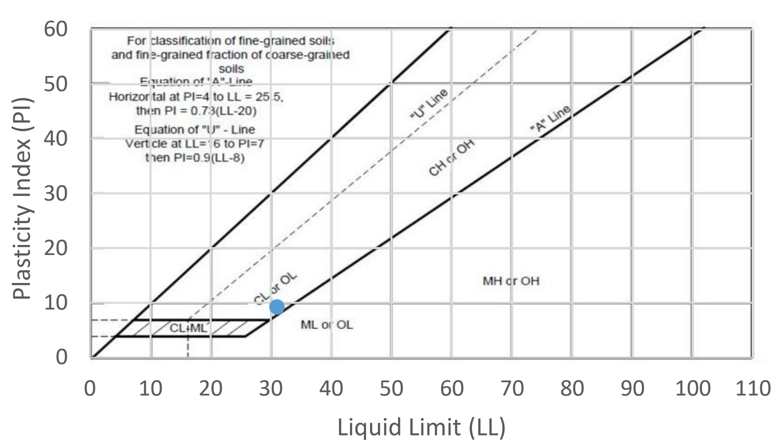
Container No.	12
Mass Moist Soil + Container, M1 (g)	68.6
Mass Dry Soil + Container, M2 (g)	58.7
Mass Container, M3 (g)	13
WATER CONTENT, Wc (%)	21.7

LIQUID LIMIT

Container No.	3	5	7	
Mass Moist Soil + Container, M1 (g)	62.5	55.4	69.5	
Mass Dry Soil + Container, M2 (g)	47.6	45.5	58.3	
Mass Container, M3 (g)	13.1	13	13	
WATER CONTENT, Wc (%)	43.2	30.5	24.7	
NUMBER OF BLOWS, N	15	25	30	

A	x	Wet	X
B		Dry	

AS-RECEIVED WATER CONTENT, Wc, (%)	**20.9**
LIQUID LIMIT, LL	**31**
PLASTIC LIMIT, PL	**22**
PLASTIC INDEX, PI	**9**
LIQUIDITY INDEX, LI	**n/a**
PERCENTAGE POINTS ABOVE/BELOW A-LINE	**2**
PLASTICITY CHART CLASSIFICATION	**CL**



GRAY GEOTECH

ASTM D422-63 (2007)

Particle Size Distribution Report

Project Name: Lot 59 & 60
Project Number: 20-018-01

Boring Number/Sample Location: B2
Depth 3.8 ft
Sample Number:

Weight of Container (g): 225.4
Weight of Container & Soil (g): 1191.0
Weight of Dry Sample (g): 965.6

Sieve Number	Diameter (mm)	Mass of Sieve (g)	Mass of Sieve & Soil (g)	Soil Retained (g)	Soil Retained (%)	Soil Passing (%)
1/2	12.7	0			0.0	100.0
3/8	9.5	0			0.0	100.0
#4	4.75	0	182.0	182.0	18.8	81.2
#10	2.00	0	173.5	173.5	18.0	63.2
#60	0.25	0	404.2	404.2	41.9	21.3
#200	0.075	0	84.0	84.0	8.7	12.6
Pan				121.90	12.6	
			TOTAL:	**965.6**	**100.0**