LGC Geo-Environmental, Inc.

Updated Preliminary Geotechnical Investigation Report for the Proposed Single-Family Residential Development, Located at 800 North Girard Street, City of Hemet, Riverside County, California

Dated: February 10, 2021
Project No. G18-1647-10

Prepared For:

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GEOTECHNICAL • ENVIRONMENTAL • MATERIALS TESTING

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Subject: Updated Preliminary Geotechnical Investigation Report for the Proposed Single-Family Residential Development, Located at 800 North Girard Street, City of Hemet, Riverside County, California.

LGC Geo-Environmental, Inc. (LGC) is pleased to submit herewith our preliminary geotechnical investigation report for the proposed single-family residential development at 800 North Girard Street in the city of Hemet, Riverside County, California. This report presents the results of our research of published geologic/geotechnical reports and/or maps, review of aerial photographs, field exploration, geologic mapping, and laboratory testing; in addition to our geotechnical and geologic judgment, opinions, conclusions and preliminary recommendations associated with the proposed residential development.

Based on the results of our field exploration, geologic mapping, laboratory testing, geologic and geotechnical engineering evaluations, along with review of published literature and the preliminary grading plan, it is our opinion that the subject site is suitable for the proposed single-family residential development, provided that the recommendations presented herein are utilized during design and implemented during grading and construction. LGC should review all pertinent grading plans, as well as any foundation/structural plans when these become available, and revise the recommendations presented herein, if necessary.

It has been a pleasure to be of service to you during the design stages of this project. If you have any questions regarding the contents of this report or require additional information, please do not hesitate to contact us.

Respectfully submitted,

LGC Geo-Environmental, Inc.

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1.0 INTRODUCTION

This report presents the results of LGC Geo-Environmental, Inc.'s (LGC) preliminary geotechnical investigation report for the proposed single-family residential development located at 800 N. Girard Street in the city of Hemet, Riverside County, California. The purposes of this geotechnical investigation are to determine the nature of surface and subsurface soil conditions, evaluate the soil characteristics, and provide geotechnical recommendations with respect to grading, construction, foundation design, and other relevant aspects to the proposed commercial development. The referenced preliminary site plan, which was provided, was utilized as the base map for our Geotechnical Map (Plate 1) of the site.

Our scope of services consists of:

- Review of available previous geologic/geotechnical literature, geologic maps, and aerial photographs pertinent to the site (Appendix A).
- Geologic mapping of the site.
- Subsurface exploration consisting of the sampling and logging of five (5) trenches to depths of approximately 5.5′ feet to 17.5′ feet, using a backhoe. Logs of the trenches as presented in Appendix B, with approximate locations depicted on the Geotechnical Map (Plate 1). The trenches were excavated to evaluate the general characteristics of the subsurface geologic/geotechnical conditions on the subject project site including classification of site soil, determination of depth to groundwater (if present), and to obtain representative soil samples.
- Core two (2) borings in the pavement on Menlo Park to depths of 5.0 feet and 6.0 feet.
- Laboratory testing of representative soil samples obtained during our current subsurface exploration (Appendix C).
- Geotechnical engineering and geologic analysis of the data with respect to the proposed single-family residential development.
- Preparation of General Earthwork and Grading Specifications (Appendix D).
- Preparation of this report presenting our findings, conclusions and preliminary geotechnical design recommendations for the proposed single-family residential development.

1.1 Proposed Construction and Grading

The referenced "Preliminary Site Plan", prepared by Sikand Engineering Associates, indicates that the proposed residential single-family development will be comprised of 51 graded pads, associated roadways, one water quality detention basin, and landscape and hardscape areas. The development is proposed to be single-family dwelling units per graded pad at this time. Based on the referenced preliminary site plan, maximum proposed cut and fill depths are approximately 12.0 feet to 2.5 feet, respectively. Slope and retaining walls are not proposed at this time.

When a rough grading plan is available, LGC should review and make any additional recommendations.

1.2 Location and Site Description

The subject site is irregular in shape and is located on the northwest corner of East Menlo Avenue and Park Avenue in the City of Hemet, Riverside County, California. The site is bounded on the north by residential development, on the west by Girard Street and residential development, on the south by East Menlo Avenue and residential development, and east by Park Avenue. The general location and configuration of the site is shown on the Site Location Map (Figure 1).

1.3 Topography and Drainage

The topography of the site is slightly inclined with sheet drainage appearing to flow from east to west. The existing site elevations vary from approximately 1,637 feet above mean sea level (msl) near the northeast corner of the site, to approximately 1,607 msl at the northwest corner of the site.

FIGURE 1
SITE LOCATION MAP

Project Name	SIKAND
Project No.	G18-1647-10
Geol./ Eng.	MB/RS
Scale	NOT TO SCALE
Date	FEBRUARY 2021

1.4 *Existing Improvements and Vegetation*

The subject site is a vacant property with several concrete pads, a roadway, and various small concrete structures. Annual weeds are abundant on the project site, along with trees, shrubs, and debris.

1.5 *Research of Previous Geological and Geotechnical Data*

This firm researched and reviewed available published and unpublished geotechnical and geologic reports, maps and data. Based on this firm's research, pertinent information was incorporated into the conclusions and recommendations presented in our report.

2.0 *FIELD INVESTIGATION*

2.1 *Geologic Mapping*

Surface geologic mapping of the site and accessible surrounding areas was accomplished by a geologist from this firm on October 3, 2019, utilizing the referenced "Preliminary Site Plan" for plotting geologic units. This information is plotted on the enclosed Geotechnical Map (Plate 1).

2.2 *Field Exploration*

Subsurface exploration was performed on October 3, 2019 and involved the excavation of five (5) exploratory trenches (Trenches TR-1 through TR-5) to depths of approximately 5.5 feet to 17.5 feet utilizing a rubber tire backhoe. Additionally, two (2) core borings were excavated within East Menlo Ave to evaluate existing pavement design.

Prior to our subsurface work, an underground utilities clearance was obtained from Underground Services Alert of Southern California. At the conclusion of the subsurface exploration, all of the exploratory trenches were backfilled with on-site materials with some compactive effort. Minor settlement of the backfill soil may occur over time.

Earth materials encountered within the trenches were classified and logged by a geologist from LGC in accordance with the visual-manual procedures of the Unified Soil Classification System. The approximate locations of the exploratory trenches and core borings are shown on the Geotechnical Map (Plate 1) and descriptive logs are presented in Appendix B.

Bulk samples of soil associated with the initial subsurface exploration were collected for laboratory testing. Bulk samples consisted of selected soil materials obtained at various depth intervals from the exploratory trenches.

2.3 *Laboratory Testing*

During our subsurface exploration, relatively undisturbed and bulk samples were retained for laboratory testing. Laboratory testing was performed on selected representative samples of onsite soil materials and included maximum dry density and optimum water content, expansion index, sulfate content, chloride content, pH, resistivity, shear strength, R-Value, and Atterberg limits. A brief description of the laboratory test criteria and test data are presented in Appendix C. In-situ water contents and dry densities are included in the exploration trench and core logs (Appendix B).

3.0 FINDINGS

3.1 Regional Geologic Setting

Regionally, the site is located in the Peninsular Ranges Geomorphic Province of California. The Peninsular Ranges are characterized by steep, elongated valleys that trend west to northwest. The northwest-trending topography is controlled by the Elsinore Fault Zone, which extends from the San Gabriel River Valley southeasterly to the United States/Mexico border. The Santa Ana Mountains lie along the western side of the Elsinore Fault Zone, while the Perris Block is located along the eastern side of the fault zone. The mountainous regions are underlain by Pre-Cretaceous, metasedimentary and metavolcanic rocks and Cretaceous plutonic rocks of the Southern California Batholith. Holocene to Pleistocene-aged alluvium overlie Quaternary and Tertiary rocks, which are generally comprised of non-marine sediments consisting of sandstone, mudstones, conglomerates, and occasional volcanic units. A map of the regional geology is presented on the Regional Geologic Map (Figure 2).

3.2 Local Geology and Soil Conditions

Based on our review of available geological and geotechnical literature, current field mapping, and exploratory trenches conducted at the site, it is our understanding that the site is primarily underlain by undocumented artificial fill, alluvium, and Bautista Formation bedrock. Each unit is described in greater detail below and presented within the exploratory trench logs (Appendix B). The approximate locations of the observed geologic units are depicted on the Geotechnical Map (Plate 1).

- Artificial Fill, Undocumented (Afu) – Undocumented artificial fill was encountered in Trenches TR-1 through TR-5 to depths ranging from approximately 1.5 feet to 2.5 feet below the surface. These materials consisted of silty sand which was various shades of brown; dry; loose to medium dense; very fine to coarse grained with some gravels; roots; roothairs; blocky; and desiccated.
- Alluvium (Qal) – Alluvium was encountered on the site during our subsurface exploration and was observed at depths ranging from approximately 2.0 feet to 17.5 feet below the surface, in all trenches except trench TR-3, below the undocumented artificial fill. The alluvium generally consists of alternating layers of poorly graded sand, silty sand, clayey sand, and silty clay, and is various shades of brown and gray; moist and loose to medium dense. The material was also noted to be very fine to medium grained with occasional coarse grains and gravels; roothairs; caliche nodules and stringers; pinhole porosity; trace oxidation staining; micaceous; and minor clay in trench TR-5 at a depth of approximately 12.0 feet below the surface.
- Quaternary Bautista Formation (Qts) – Pleistocene age Bautista Formation was encountered in TR-3 below the undocumented artificial fill throughout the entire depth of the trench. This bedrock is generally sandstone with some interbedded siltstone, and is characterized as being various shades of white, gray, and brown; moist; moderately hard; medium to coarse grained with gravels; some highly weathered granitic clasts; manganese staining; and oxidation staining.

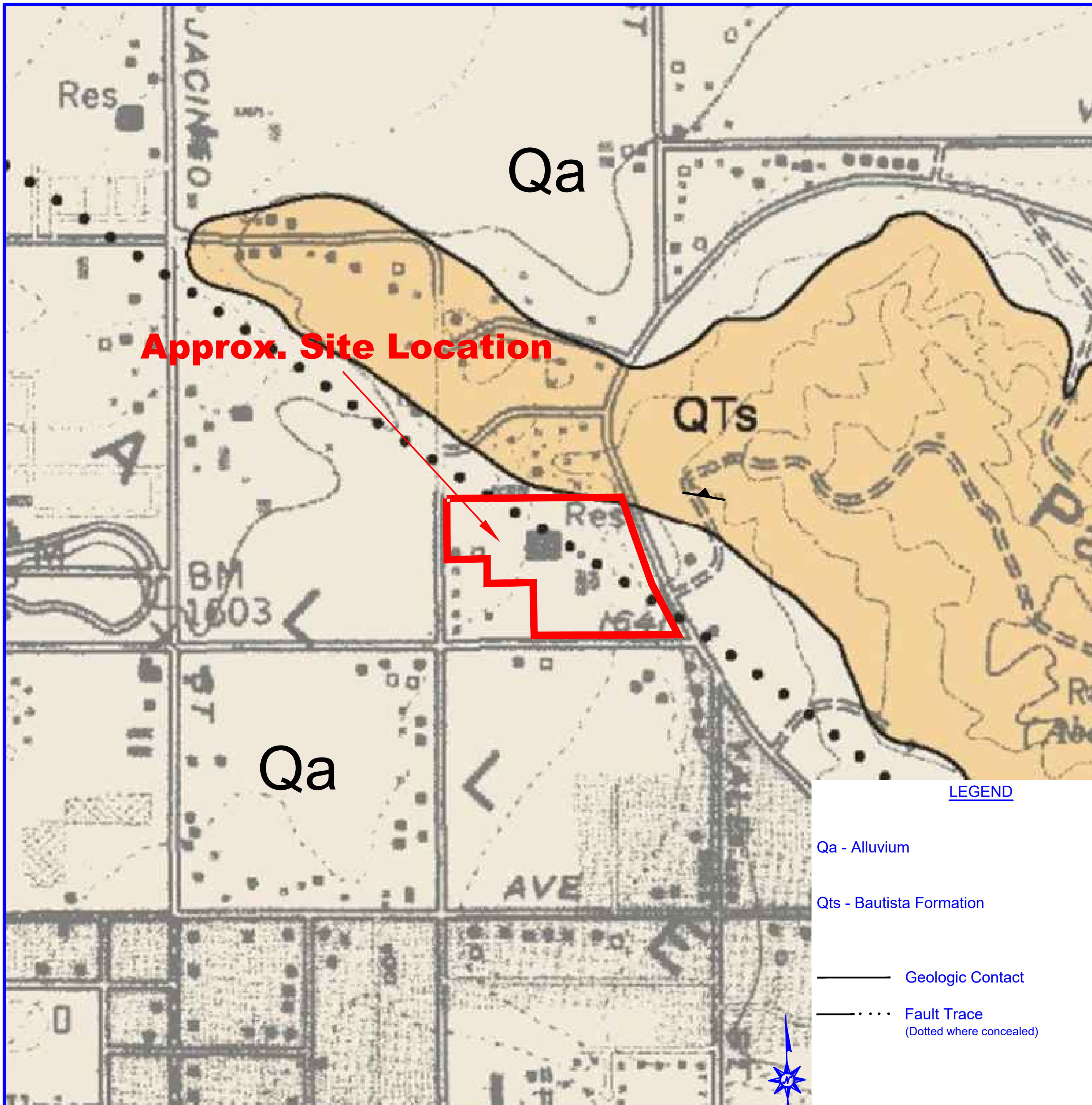
3.3 Landslides

Our investigation did not indicate the presence of landslides on or directly adjacent to the site.

3.4 Groundwater

Groundwater was not encountered during the subsurface exploration.

A review of the California Department of Water Resources, Water Data Library online database indicates the presence of groundwater less than a mile away from the general site area at approximately 267 feet below the existing ground surface according to historical records at an elevation of approximately 1,588 above mean sea level (Well ID: Station 337574N1169698W001).



GEOLOGIC MAP OF THE SAN JACINTO QUADRANGLE RIVERSIDE COUNTY, CALIFORNIA
By Thomas W. Dibble, Jr.. 2003 Edited by John A. Minch

FIGURE 2
REGIONAL GEOLOGIC MAP

Project Name	SIKAND
Project No.	G18-1647-10
Geol./ Eng.	MB/RS
Scale	NOT TO SCALE
Date	FEBRUARY 2021

3.5 Caving

Caving was not encountered in the exploratory trenches.

3.6 Surface Water

Surface water runoff relative to project design is within the purview of the project civil engineer and should be designed to be directed away from all structures and walls.

3.7 Faulting

The geologic structure of the Southern California area is dominated mainly by northwest-trending faults associated with the San Andreas system. Faults, such as the Whittier, Elsinore, San Jacinto and San Andreas, are major faults in this system and are known to be active and may produce moderate to strong ground shaking during an earthquake. In addition, the San Andreas, Elsinore and San Jacinto faults are known to have ruptured the ground surface in historic times.

The following table is comprised of a list of the significant faults located within 20 miles of the proposed project site. We have also included the Maximum Earthquake Magnitude predicted for each of these faults.

TABLE 1
Significant Faults in Proximity of the Project Site

ABBREVIATED FAULT NAME	*APPROXIMATE DISTANCE (mi)*	*MAXIMUM EARTHQUAKE MAGNITUDE (Mw)*
Casa Loma*	Onsite	6.9
San Jacinto-San Jacinto Valley	1.9	6.9
San Jacinto-Anza	3.1	7.2
San Andreas-Southern	17.6	7.4
San Andreas-San Bernardino	17.6	7.3

Source: EQFAULT for Windows Version 3.00b
*Casa Loma located on subject property.

Previous fault investigations conducted by Rasmussen (1988) and LGC (2018) concluded that active or potentially active faulting related to the Casa Loma fault are known to project through the site (Appendix A). The site does lie within an Alquist-Priolo Earthquake Fault Hazard Zone as defined by the State of California in the Alquist-Priolo Earthquake Fault Hazard Zoning Act. According to these reports, the potential for damage because of ground surface rupture is considered a possibility since active or potentially faults are known to cross the site. Accordingly, a structural setback zone has been established for the property as shown on the accompanying Geotechnical Map, Plate 1. No structures for human occupancy should be constructed in this setback zone.

3.8 Seismicity

Secondary effects of seismic shaking resulting from large earthquakes on the major faults in the southern California region, which may affect the site, include soil liquefaction and dynamic settlement. Liquefaction is a seismic phenomenon in which loose, saturated, granular soil behave similarly to a fluid when subject to high-intensity ground shaking. Liquefaction occurs when three general conditions exist: 1) shallow groundwater; 2) low density non-cohesive (granular) soil; and 3) high-intensity ground motion. Studies indicate that saturated, loose to medium dense, near surface cohesionless soil exhibit the highest liquefaction potential, while dry, dense, cohesionless soil and cohesive soil exhibit low to negligible liquefaction potential.

Due to the shallow depth of bedrock, dense alluvium, and groundwater depth being greater than 50 feet, liquefaction is considered nil.

Other secondary seismic effects include shallow ground rupture, seiches, and tsunamis. In general, these secondary effects of seismic shaking are a possibility throughout the Southern California region and are dependent on the distance between the site and causative fault and the onsite geology. A risk assessment of these secondary effects is provided in the following sections.

3.9 Earthwork and Structural Settlements

The results of our subsurface exploration and laboratory testing indicate that the site is underlain by approximately 1.5 feet to 5 feet of potentially compressible soil, consisting of non-engineered, undocumented artificial fill. These materials exhibit the potential to settle under the surcharge of proposed fill loads, anticipated future structural loads, and improvements.

Where overexcavation to competent underlying alluvium is accomplished, total static settlement from the earthwork and from proposed fill loads is estimated to be 3/4-inch total and 1/2-inch differential over 30 feet.

4.0 CONCLUSIONS AND RECOMMENDATIONS

4.1 General

Based on the results of our current geotechnical investigation, it is our opinion that the proposed single-family residential development, as indicated on the referenced site plan, is feasible from a geotechnical and geologic standpoint, provided that the following recommendations are incorporated into the design criteria and project specifications. When actual grading plans for the site and foundation/structural plans for the proposed development are available, a comprehensive plan review should be performed by this firm. Depending on the results, additional recommendations may be necessary for geotechnical design parameters for both earthwork and foundations. Grading should be conducted in accordance with local codes, the recommendations within this report, and future plan reviews. It is also our opinion that the proposed construction and grading will not adversely impact the geologic stability of adjoining properties.

The following is a summary of the primary geotechnical factors determined from our geotechnical investigation.

- Based on our current subsurface exploration and review of pertinent geological maps and reports, the site is underlain by undocumented artificial fill, alluvium, and Bautista Formation.
- There are not any known landslides impacting the site.
- Groundwater is not considered a constraint for the proposed single-family development.
- Active or potentially active faults are known to exist on the site.
- Laboratory test results of the upper soil (undocumented artificial fill and alluvium) indicate a medium expansion potential and negligible potential for soluble sulfate effects on normal concrete and chloride effects on reinforcing steel.
- Laboratory test results of the soil encountered indicated a moderate corrosion potential to buried metals.
- The majority of the site is underlain by approximately 6.0 feet of potentially compressible undocumented artificial fill and portions of the upper alluvium which may be prone to potential intolerable post-grading settlement under the surcharge of the future proposed fill loads and/or

structural loads. These materials should be overexcavated to underlying competent alluvium deposits.

- From a geotechnical perspective, the existing onsite soil appears to be suitable material for use as fill, provided the soil are relatively free from rocks (larger than 6 inches in maximum dimension), construction debris, and organic material. It is anticipated that the onsite soil may be excavated with conventional heavy-duty construction equipment.

5.0 GEOLOGIC CONSIDERATIONS

5.1 Slopes

Natural slopes or existing cut/fill slopes with adverse conditions are not anticipated.

5.2 Faulting

Geologic hazards due to fault rupture are known to be present on the subject site. Potentially active faulting related to the Casa Loma fault was observed within fault trenches located within the site. The fault trenches and actual fault location are shown on the Geotechnical Map (Plate 1).

5.3 Groundwater

Adverse effects on the proposed development resulting from groundwater are not anticipated.

5.4 Subsidence

In consideration of the anticipated grading, recommended overexcavations and subsurface material types and soil conditions, unfavorable ground subsidence is not anticipated.

5.5 Landsliding

Landslides or surface failures were not observed on or directly adjacent to the site. As a result, the possibility of the site being affected by landsliding is not anticipated.

5.6 Ground Rupture

Ground rupture from active faulting could possibly occur on site from the presence of observed, potentially active faulting related to the Casa Loma. Cracking from shaking from distant seismic events is not considered a significant hazard, although it is a possibility at any site.

5.7 Tsunamis and Seiches

Based on the elevation and location of the proposed residential development on the site with respect to sea level and its distance from large open bodies of water, the potential for seiches and/or tsunamis is not considered to be a possibility.

5.8 Liquefaction

Liquefaction is a seismic phenomenon in which loose, saturated, granular soils behave similarly to a fluid when subject to high-intensity ground shaking. Liquefaction occurs when three general conditions exist: 1) shallow groundwater; 2) low density non-cohesive (granular) soil; and 3) high-intensity ground motion. Studies indicate that saturated, loose to medium dense, near surface cohesionless soils exhibit the highest liquefaction potential, while dry, dense, cohesionless soils and cohesive soils exhibit low to negligible liquefaction potential. Bedrock was found as shallow as 1.5 feet in Trench FT-3. With shallow bedrock, the potential for liquefaction is considered nil.

6.0 SEISMIC-DESIGN CONSIDERATIONS

6.1 Ground Motions

The site will probably experience ground shaking from moderate to large size earthquakes during the life of the proposed development. Furthermore, it should be recognized that the Southern California region is an area of high seismic risk, and that it is not considered feasible to make structures totally resistant to seismic-related hazards.

Structures within the site should be designed and constructed to resist the effects of seismic ground motions as provided in the 2016 CBC, Section 1613. The method of design is dependent on the seismic zoning, site characterizations, occupancy category, building configuration, type of structural system, and building height.

The following seismic design parameters, presented in Table 2, were developed based on the CBC 2016 and should be used for the proposed structures. A site coordinate of 33.5322° N, 117.1795° W was used to derive the seismic parameters presented below. The Mean Peak Ground Acceleration (PGAm) is 0.97 below.

TABLE 2
Seismic Design Soil Parameters

SEISMIC DESIGN SOIL PARAMETERS **(2016 CBC Section 1613)**	
Site Class Definition ASCE 7; Chapter 20 (Table 20.3-1)	D
Mapped Spectral Response Acceleration Parameter S_s (for 0.2 second) (Figure 1613.5.3.(1)	2.53
Mapped Spectral Response Acceleration Parameter, S_1 (for 1.0 second) (Figure 1613.5.3.(2)	1.14
Site Coefficient F_a (short period) [Table 1613.3.3.(1)]	1.00
Site Coefficient F_v (1-second period) [Table 1613.3.3.(2)]	1.50
Adjusted Maximum Considered Earthquake (MCE) Spectral Response Acceleration Parameter S_{MS} (short period) (Eq. 16-37)	2.53
Adjusted Maximum Considered Earthquake (MCE) Spectral Response Acceleration Parameter S_{M1} (1-second period) (Eq. 16-38)	1.71
Design Spectral Response Acceleration Parameter, S_{DS} (short period) (Eq. 16-39)	1.68
Design Spectral Response Acceleration Parameter, S_{D1} (1-second period) (Eq. 16-40)	1.14
Mean Peak Ground Acceleration (PGA_m)	0.97

6.2 Secondary Seismic Hazards

Secondary effects of seismic activity normally considered as possible hazards to a site include several types of ground failure, as well as induced flooding. Various general types of ground failures which might occur as a consequence of severe ground shaking of the site include liquefaction, landsliding, ground subsidence, ground lurching, and shallow ground rupture. The probability of occurrence of each type of ground failure depends on the severity of the earthquake, distance from faults, topography, subsoils and groundwater conditions, in addition to other factors. Based on the depth to groundwater, proposed grading and recommended overexcavation of potentially compressible materials within areas of proposed development, the secondary effects of liquefaction are considered unlikely.

Seismically induced flooding, which might be considered a potential hazard to a site, normally includes flooding due to a tsunami (seismic sea wave), a seiche (i.e., a wave-like oscillation of the surface of water in an enclosed basin that may be initiated by a strong earthquake) or failure of a major reservoir or retention structure upstream of the site. Since the site is located several miles inland from the nearest coastline of the Pacific Ocean and elevation exceeds 1,600 feet above msl, there is no potential for seismically induced flooding from a tsunami. Since enclosed bodies of water do not lie adjacent to the site, the potential for induced flooding at the site due to a seiche is also considered nonexistent.

7.0 GEOTECHNICAL DESIGN PARAMETERS

7.1 Shrinkage/Bulking and Subsidence

Volumetric changes in earth quantities will occur when excavated onsite soil are replaced as properly compacted fill. The following table, Table 3, is an estimate of the shrinkage and bulking factors for the various geologic units present onsite. These estimates are based on in-place densities of the various materials and on the estimated average degree of relative compaction that will be achieved during grading.

TABLE 3
Estimated Shrinkage/Bulking

GEOLOGIC UNIT	*SHRINKAGE/BULKING PERCENT*
Artificial Fill, Undocumented (Afu)	10%-19%
Alluvium (Qal)	11%-20%
Bautista Formation (Qts)	0%-5%

Subsidence of the alluvium deposits is estimated to be about 0.25 to 0.30 feet.

The above estimates of shrinkage are intended as an aid for project engineers in determining earthwork quantities. **However, these estimates should be used with some caution since they are not absolute values.** These are preliminary rough estimates which may vary with depth of removal, stripping losses, field conditions at the time of grading, etc. Handling losses, and reduction in volume due to removal of oversized material, are not included in the estimates.

7.2 Compressible/Collapsible Soil

The results of our laboratory testing indicate that the existing undocumented artificial fill is susceptible to varying degrees of intolerable settlement when a load is applied, or the soil is saturated. Consequently, these materials should be collectively overexcavated to underlying competent alluvium (Qal) and Bautista Formation (Qts) and replaced as engineered compacted fill.

8.0 SITE EARTHWORK

8.1 General Earthwork and Grading Specifications

Earthwork and grading should be performed in accordance with applicable requirements of the grading code of the City of Hemet and in accordance with the following recommendations prepared by this firm. Grading should also be performed in accordance with the applicable provisions of the attached "General Earthwork and Grading Specifications" prepared by LGC (Appendix D), unless specifically revised or amended herein. In case of conflict, the following recommendations shall supersede those included in as part of Appendix D.

8.2 Geotechnical Observations and Testing

Prior to the start of grading, a meeting should be held at the site with the owner, developer, grading contractor, civil engineer and geotechnical consultant to discuss the work schedule and geotechnical aspects of the grading. Rough grading, which includes clearing, overexcavation, scarification/processing and fill placement, should be accomplished under the full-time observation and testing of the geotechnical consultant. Fills should not be placed without prior approval from the geotechnical consultant.

A representative of the project geotechnical consultant should also be present onsite during grading operations to document proper placement and compaction of fills, as well as to document excavations and compliance with the other recommendations presented herein.

8.3 Clearing and Grubbing

The project geotechnical consultant or his qualified representative should be notified at the appropriate times to provide observation and testing services during clearing and grubbing operations to observe and document compliance with the above recommendations. In addition, buried structures, unusual or adverse soil conditions encountered that are not described or anticipated herein should be brought to the immediate attention of the geotechnical consultant.

8.4 Overexcavation and Ground Preparation

The site is underlain by approximately 3 feet to 6 feet of compressible undocumented artificial fill and portions of the upper alluvium which is considered unsuitable for support of fill, structures, and/or improvements, and should be overexcavated to expose underlying competent alluvium or bedrock. Overexcavation must provide at least 5 feet or more of compacted fill below finished grade within areas of proposed structures or walls. Therefore, those areas should be overexcavated to at least 6 feet or more below proposed grade. Actual depths of overexcavation should be evaluated upon review of final grading and foundation plans, as well as during grading on the basis of observations and testing during grading by the project geotechnical consultant.

Across the site are twelve (12) fault trenches that were excavated in 1988 and 2018. These trenches range in depths of 9 feet to 14 feet. The locations of the trenches can be found on the Geotechnical Map (Plate 1) and should be over excavated and recompacted to each trench depth.

Prior to placing engineered fill, exposed bottom surfaces in each overexcavated area should first be scarified to a depth of approximately 6 inches, watered or air-dried as necessary to achieve a uniform water content of optimum or higher and then compacted in place to a relative compaction of 90 percent or more (based on American Standard of Testing and Materials [ASTM] Test Method D1557).

The estimated locations, extent, and approximate depths for overexcavation of unsuitable materials are indicated on the enclosed Geotechnical Map (Plate 1). The geotechnical consultant should be provided with appropriate survey staking during grading to document that depths and/or locations of recommended overexcavation are adequate.

Sidewalls for overexcavations greater than 5 feet in height should not be steeper than 1:1 horizontal to vertical (h:v) and should be periodically slope-boarded during the excavation to remove loose surficial debris and facilitate mapping. Flatter excavations may be necessary for stability.

The grading contractor will need to consider appropriate measures necessary to excavate existing improvements adjacent to the site without endangering them from caving or sloughing.

8.5 Fill Suitability

Soil materials excavated during grading are generally considered suitable for use as compacted fill provided that they do not contain significant amounts of trash, vegetation, organic material, construction debris, and oversize material.

8.6 *Oversized Material*

Oversized material that may be encountered during grading, greater than 6 inches, should be reduced in size or removed from the site

8.7 *Cut/Fill Transitions and Differential Fill Thicknesses*

To mitigate distress to structures related to the potential adverse effects of excessive differential settlement, cut/fill transitions should be eliminated from all building areas where the depth of fill placed within the "fill" portion exceeds proposed footing depths. The entire structure should be founded on a uniform bearing material. This should be accomplished by overexcavating the "cut" portion and replacing the excavated materials as properly compacted fill. Recommended depths of overexcavation are provided in the following table:

Cut/Fill Transition

DEPTH OF FILL ("fill" portion)	*DEPTH OF OVEREXCAVATION ("cut" portion)*
Up to 5 feet	Equal Depth
5 to 10 feet	5 feet
Greater than 10 feet	One-half the maximum thickness of fill placed on the "fill" portion (20 feet maximum)

Overexcavation of the "cut" portion should extend beyond the perimeter building lines to a horizontal distance equal to the depth of overexcavation or to a minimum distance of 5 feet, whichever is greater.

8.8 *Benching*

Where compacted fills are to be placed on natural slope surfaces inclining at 5:1 (h:v) or greater, the ground should be excavated to create a series of level benches, which are at least a minimum height of 4 feet, excavated into competent bedrock or existing compacted engineered materials. Typical benching details are described in the attached LGC "Standard Grading Specifications" (Appendix D).

8.9 *Fill Placement*

Fills should be placed in lifts not greater than 6 inches in uncompacted thickness, watered or air-dried as necessary to achieve a uniform moisture content of at least optimum moisture content, and then compacted in place to relative compaction of 90 percent or more. Fills should be maintained in a relatively level condition. The laboratory maximum dry density and optimum moisture content for each change in soil type should be determined in accordance with ASTM Test Method D1557.

8.10 *Inclement Weather*

Inclement weather may cause rapid erosion during mass grading and/or construction. Proper erosion and drainage control measures should be taken during periods of inclement weather in accordance with City of Hemet, Riverside County, and California State requirements.

9.0 *SLOPE CONSTRUCTION*

9.1 *Slope Stability*

Any proposed cut or fill slopes constructed at a 2:1 horizontal to vertical (h:v) orientation or flatter should be grossly stable.

Portions of any proposed cut slopes may expose low-density, undocumented artificial fill as well as significant layers of relatively non-cohesive alluvium deposits which will likely require stabilization by overexcavation and replacement with compacted fill. During the grading plan review stages, a detailed slope stability analyses may be warranted.

9.2 Temporary Excavations

Temporary excavations varying up to a height of approximately 5 feet or more below existing grades will be necessary to accommodate the recommended overexcavation of the unsuitable soil materials. Based on the physical properties of the onsite soil, temporary excavations exceeding 5 feet in height should be cut back at a ratio of 1:1 (h:v) or flatter, for the duration of the overexcavation and recompaction of unsuitable soil material. Temporary slopes excavated at the above slope configurations are expected to remain stable during grading operations. However, the temporary excavations should be observed by a representative of the project geotechnical consultant for any evidence of potential instability. Depending on the results of these observations, revised slope configurations may be necessary. Job safety is the sole responsibility of the contractor or sub-contractor.

Other factors which should be considered with respect to the stability of the temporary slopes include construction traffic and storage of materials on or near the tops of the slopes, construction scheduling, presence of nearby walls or structures on adjacent properties, and weather conditions at the time of construction. Applicable requirements of the California Construction and General Industry Safety Orders, the Occupational Safety and Health Act of 1970, and the Construction Safety Act should also be followed.

10.0 POST-GRADING CONSIDERATIONS

10.1 Control of Surface Water and Drainage Control

Positive-drainage devices such as sloping sidewalks, graded-swales, and/or area drains, should be provided to collect and direct water away from the structure and any slopes. Neither rain nor excess irrigation water should be allowed to collect or pond against the building foundations. Drainage should be directed to adjacent driveways, adjacent streets or storm-drain facilities and maintained at all times. The site is in a semi-arid climate area, from a geotechnical standpoint, the ground surface adjacent to the structures should be sloped at a gradient of at least 2 percent for a distance of at least 10 feet. The graded lot should be further maintained by a swale or drainage path at a gradient of at least 1 percent. Where necessary, drainage paths may be shortened by use of area drains and collector pipes.

Planters with open bottoms adjacent to buildings should be avoided. Planters should not be designed adjacent to buildings unless provisions for drainage are made, such as catch basins, liners, and/or area drains. Over watering must be avoided.

10.2 Utility Trenches

Utility-trench backfill within roadways, utility easements, under walls, sidewalks, driveways, floor slabs and any other structures or improvements should be compacted. The onsite soil should generally be suitable as trench backfill provided the soil is screened of rocks and other material over 3 inches in diameter and organic matter. Trench backfill should be compacted in uniform lifts (generally not exceeding 6 inches to 8 inches in uncompacted thickness) by mechanical means to at least 90 percent relative density (per ASTM Test Method D1557).

Where onsite soils are utilized as backfill, mechanical compaction should be used. Density testing, along with probing, should be performed by the project geotechnical consultant or his representative, to document proper compaction.

If trenches are shallow, the use of conventional equipment may result in damage to the utilities. Clean sand, having a sand equivalent (SE) of 30 or greater should be used to bed and shade the utilities. Sand backfill should be densified. The densification may be accomplished by jetting or flooding and then tamping to ensure adequate compaction. A representative from LGC should observe, probe, and test the backfill to verify compliance with the project specifications.

Utility-trench sidewalls deeper than 5 feet should be laid back at a ratio of 1:1 (h:v) or flatter or braced. A trench box may be used in lieu of shoring. If shoring is anticipated, LGC should be contacted to provide design parameters.

To avoid point-loads and subsequent distress to clay, cement or plastic pipe, imported sand bedding should be placed 1-foot or more above pipe in areas where excavated trench materials contain significant cobbles. Sand-bedding materials should be compacted and tested prior to placement of backfill.

Where utility trenches are proposed parallel to building footings (interior and/or exterior trenches), the bottom of the trench should not be located within a 1:1 (h:v) plane projected downward from the outside bottom edge of the adjacent footing.

11.0 PRELIMINARY FOUNDATION DESIGN RECOMMENDATIONS

11.1 General

Provided that site grading is performed in accordance with the recommendations of this report, conventional shallow foundations are considered feasible for support of the proposed commercial building. Tentative foundation recommendations are provided herein. However, these recommendations may require modification depending on as-graded conditions existing within the building sites upon completion of grading.

11.2 Allowable-Bearing Values

An allowable-bearing value of 1,500 pounds per square foot (psf) may be used for 12-inch wide or greater continuous footings or 24-inch square pad footings, founded completely within competent compacted fill at a depth of 12-inches or more below the lowest adjacent final grade. This value may be increased by 20 percent for each additional foot of width and depth, to a value no greater than 3,000 psf. The recommended allowable-bearing value includes both dead and live loads and may not be increased by one-third for short-duration wind and seismic forces. The bearing capacities should be re-evaluated when loads and footing sizes have been finalized.

11.3 Settlement

Based on the general settlement characteristics of compacted fill, the previous overexcavation recommendations in this report and anticipated fill loading, it is estimated the site would be subjected to a total static settlement about 0.75-inch, and a differential settlement of about 0.50-inch over a distance of about 30 feet. It is anticipated that the majority of the settlement will occur during construction or shortly thereafter as building loads are applied.

The above settlement estimates are based on the assumption that the proposed precise grading will be performed in accordance with the grading recommendations presented in this report and that the project geotechnical consultant will observe and/or test the soil conditions in the footing excavations.

11.4 Lateral Resistance

Lateral forces on footings should be resisted by passive earth resistance and friction at the bottom of the footing. Foundations should be designed for a passive earth pressure of 230 psf per foot of depth to a maximum 3,000 psf and a coefficient of friction of 0.30. The passive earth pressure incorporates a minimum factor of safety of 1.5. When combining passive and friction forces, passive resistance should be reduced by 1/3. The above values may not be increased by 1/3 when designing for short-duration wind or seismic forces.

The above values are based on footings placed directly against compacted fill soil. In the case where footing sides are formed, backfill placed against the footings should be compacted to 90 percent or more of maximum dry density as determined by ASTM D1557.

11.5 Footing Setbacks from Descending Slopes

Where structures are proposed near the tops of descending graded or natural slopes, the footing setbacks from the slope face should conform to the 2016 CBC, Figure 1808.7.1. The required setback is H/3 (one-third the slope height) measured along a horizontal line projected from the lower outside face of the footing to the slope face. The footing setbacks should be 5 feet or more where the slope height is 15 feet or less and vary up to 40 feet where the slope height exceeds 15 feet.

11.6 Building Clearances from Ascending Slopes

Building setbacks from ascending graded or natural slopes should conform with the 2016 CBC, Figure 1808.7.1, which requires a building clearance of H/2 (one-half the slope height) varying from 5 to 15 feet. The building clearance is measured along a horizontal line projected from the toe of the slope to the face of the building. A retaining wall may be constructed at the base of the slope to achieve the required building clearance.

11.7 Footing Observations

Footing trenches should be observed by the project geotechnical consultant to document that those have been excavated into competent bearing soil. The foundation trenches should be observed prior to the placement of forms, reinforcement or concrete. The trenches should be trimmed neat, level and square. Loose, sloughed or moisture-softened soil should be removed prior to concrete placement.

Excavated materials from footing trenches should not be placed in slab-on-ground areas unless the soil are compacted to 90 percent or more of maximum dry density as determined by ASTM D1557.

11.8 Expansive Soil Considerations

Results of preliminary laboratory tests by LGC indicate onsite soil materials exhibit expansion potentials of **MEDIUM** in accordance with 2016 CBC, Chapter 18. Expansive soil conditions of the near surface finish grade soil should be evaluated and tested for individual building pads on a pad-by-pad basis during and at the completion of rough grading to verify and/or modify the anticipated conditions. The design and construction details presented herein are intended to provide recommendations for the levels of expansion potential which may be evident at the completion of rough grading. Furthermore, it should be noted that additional slab thickness, footing sizes and/or reinforcement more stringent than the recommendations that follow should be provided as recommended by the project structural engineer.

11.9 Footing/Floor Slabs: Medium Expansion Potential

The following are our recommendations where foundation soil exhibits a **MEDIUM** expansion potential as classified in accordance with 2016 CBC, and it is recommended that footings and floors be constructed and reinforced in accordance with the following criteria.

- *Footings*

– Exterior continuous footings should be founded into compacted engineered fill below the lowest adjacent final grade at minimum depths of 18 inches deep for one-story to two-story construction and 24 inches deep for three-story to four-story construction. Interior continuous footings may be founded at a depth of 18 inches or greater into compacted engineered fill below the lowest adjacent final grade. Continuous footings should have a minimum width of 15 inches or more for one-story and two-story structures and 18-inches for three-story to four-story structures.

- Continuous footings should be reinforced with four (4) No. 4 bars, two near top and two near bottom.

- Interior isolated pad footings should be 24 inches or more square and founded at a depth of 18 inches or more below the lowest adjacent grade. Footings should be reinforced in accordance with the structural engineer’s recommendation.

- Exterior pad footings should be 24 inches or more square and founded at a depth of 24 inches or more below the lowest adjacent grade. Footings should be reinforced in accordance with the structural engineer’s recommendations.

- ***Floor Slabs***

- Concrete floor slabs should be 5 inches or more thick and reinforced with No. 3 bars spaced 18 inches or less on-centers, both ways. Slab reinforcement should be supported on concrete chairs or bricks so that the desired placement is near mid-depth.

- Concrete floors should be underlain with a moisture-vapor retarder consisting of 15-mil thick vapor barrier. Laps within the membrane should be sealed and overlapped 12 inches. Two inches or more of clean sand should be placed above and below the membrane to promote uniform curing of the concrete.

- Garage area floor slabs should be a minimum of 5 inches thick and should be reinforced in a similar manner as concrete interior living area floor slabs. Garage area floor slabs should be placed separately from adjacent wall footings with a positive separation maintained with 3/8-inch minimum felt expansion joint materials and quartered with weakened-plane joints. A 12-inch wide grade beam founded at the same depth as adjacent footings should be provided across garage entrances. The grade beam should be reinforced with a minimum of two No. 4 bars, one near top and one at bottom.

- Prior to placing concrete, the subgrade soils below all floor slabs should be pre-watered to achieve a moisture content that is equal to 120% of the optimum water content of the subgrade soils. The water content should penetrate to a minimum depth of 18 inches. This will promote uniform curing of the concrete and minimize the development of shrinkage cracks.

11.10 Post-Tensioned Foundation Slab Design

Post-tensioned slabs may be utilized for the support of the proposed residential structures. We recommend that the foundation engineer design the foundation system using the geotechnical parameters provided in the following Table 4. These parameters have been determined in general accordance with ACI 302 and the Post Tension Institute (PTI). In utilizing these parameters, the foundation engineer should design the foundation system in accordance with the allowable deflection criteria of applicable codes and the requirements of the structural engineer/architect. We recommend using a P.I. of 12 pertinent to the foundation/slab design.

Table 4
Preliminary Geotechnical Parameters for Post-Tensioned Foundation Slab Design

PARAMETER		VALUE
Expansion Index		Medium
Thornthwaite Moisture Index		-20
Constant Soil Suction		P.F. 4.0
Center Lift	Edge moisture variation distance, e_m Center lift, y_m	8.0 feet 0.25 inches
Edge Lift	Edge moisture variation distance, e_m Edge lift, y_m	4.0 feet 1.0 inches
Soluble Sulfate Content for Design of Concrete Mixtures in Contact with Site Soils in Accordance with ACI 318 R-05; Table 4.3.1		Low
Modulus of Subgrade Reaction, k (assuming presaturation as indicated below)		120 lbs/in^3
Minimum Perimeter Foundation Embedment		18 inches
Sand and Visqueen[a]		15 mil thick Visqueen or equivalent moisture retardant in conformance with ASTM 1745 Class A material

[a] The above sand and Visqueen recommendations are traditionally included with geotechnical foundation recommendations although they are generally not a major factor influencing the geotechnical performance of the foundation. The sand and Visqueen requirements are the purview of the foundation engineer/corrosion engineer and the homebuilder to ensure that the concrete cures correctly and is protected from corrosive environments and moisture penetration of the floor is acceptable to the future homeowners. Therefore, the above recommendations may be superceded by the requirements of the previously mentioned parties.

It is noteworthy that the post-tensioned design methodology reflected by the (PTI) is based on the assumption that soil-moisture changes around and beneath the post-tensioned slabs are primarily influenced by climatological conditions. The variability in soil moisture below slabs is the major factor in foundation damages relative to expansive soil. The design methodology does not take into consideration such factors as presaturation, homeowner irrigation, or other such artificial influences on the moisture content of subgrade soils. In recognition of these factors, LGC has modified the geotechnical parameters obtained from this methodology to introduce a more conservative design. In addition, we recommend that prior to foundation construction, the upper 18 inches of slab subgrade for each lot be presoaked to approximately ten percent above optimum moisture content prior to trenching and maintained to the associated pouring of concrete. Future homeowners should be informed of the importance of maintaining a constant level of soil-moisture. **The owners should be made aware of the potential negative consequences of both excessive watering, as well as allowing expansive soils to become too dry. The soil will undergo shrinkage of approximately 8% as it dries up, followed by swelling during the rainy winter season, or when irrigation is resumed. This may result in distress to the improvements and structures**.

12.0 RETAINING WALLS

12.1 Lateral Earth Pressures and Retaining Wall Design Parameters

Conventional footings for retaining walls founded in properly compacted fill within competent bedrock should be embedded at least 18 inches below lowest adjacent grade. At this depth, an allowable bearing capacity of 1,500 psf may be assumed for retaining walls founded in competent compacted fill.

The following are lateral earth pressures are recommended for retaining walls up to 10 feet high that may be proposed. The recommended lateral pressures for approved on-site or import soil **(with an**

expansion index of 20 or less and an angle of internal friction (phi) of at least 28 degrees) for level or sloping backfill are presented in Table 5. Onsite soil should be screened of rocks and other material over 3 inches in diameter.

TABLE 5
Lateral Earth Pressures

CONDITIONS	*EQUIVALENT FLUID WEIGHT (pcf)*			
	Level Backfill (up to 6 feet)	*Level Backfill Dynamic (>6 feet to 10 feet)*	*2:1 Backfill Ascending (up to 6 feet)*	*2:1 Backfill Ascending-Dynamic (>6 feet to 10 feet)*
Active	45	80	80	115
At-Rest	65	100	100	130
Passive	235	235	105	105

Notes:
1. Applicable to retaining walls only.
2. Active force applied a 1/3 wall height.
3. Seismic force applied to at 1/2 to 6/10 wall height.
4. Lateral pressure acts normally to vertical stem.

For sliding resistance, the friction coefficient of 0.30 may be used at the concrete and soil interface. Wall footings should be designed in accordance with structural considerations. The passive resistance value may be increased by one-third when considering loads of short duration such as wind or seismic loads.

Embedded structural walls should be designed for lateral earth pressures exerted on them. Restrained structural walls should be designed for at rest conditions. The magnitude of those pressures depends on the amount of deformation that the wall can yield under load. If the wall can yield enough to mobilize the full shear strength of the soil, it can be designed for "active" pressure. If the wall cannot yield under the applied load, the shear strength of the retained soil cannot be mobilized and the earth pressure will be higher. Such walls should be designed for "at-rest" conditions. If a structure moves toward the soil, the resulting resistance developed by the soil is the "passive" resistance.

The equivalent fluid pressure values assume free-draining conditions and a soil expansion index of 20 or less. If conditions other than those assumed above are anticipated, revised equivalent fluid pressure values should be provided on an individual-case basis by the geotechnical engineer.

Surcharge loading effects from the adjacent structures should be evaluated by the geotechnical and structural engineers.

12.2 Footing Embedments

The base of retaining wall footings constructed on level ground may be founded at a depth of 18 inches or more below the lowest adjacent final grade. Where retaining walls are proposed on or within 15 feet from the top of an adjacent descending fill slopes, the footings should be deepened such that a horizontal clearance of H/3 or more (one-third the slope height) is maintained between the outside bottom edges of the footings and the face of the slope but not to exceed 15 feet nor be less than 5 feet. The above recommended footing setbacks are preliminary and may be revised based on site specific soil conditions. Footing or pier excavations should be observed by the project geotechnical representative to document that the footing trenches have been excavated into competent bearing soil and to the embedments recommended above. These observations should be performed prior to placing forms or reinforcing steel.

12.3 Drainage

Surcharge loading effects from the adjacent structures should be evaluated by the geotechnical and structural engineers. All retaining wall structures should be provided with appropriate wall drainage and appropriately waterproofed. The outlet pipe should be sloped to drain to a suitable outlet. It should be noted that recommended wall drains do not provide protection against seepage through the face of the wall and/or efflorescence. If such seepage or efflorescence is undesirable, retaining walls should be waterproofed to reduce this potential.

Weep holes or open vertical masonry joints should be provided in retaining walls 3 feet or less in height to reduce the likelihood of entrapment of water in the backfill. Weep holes, if used, should be 3 inches or more in diameter and provided at intervals of 6 feet or less along the wall. Open vertical masonry joints, if used, should be provided at 32-inch or less intervals. A continuous gravel fill, 12 inches by 12 inches, should be placed behind the weep holes or open masonry joints. The gravel should be wrapped in filter fabric to reduce infiltration of fines and subsequent clogging of the gravel. Filter fabric may consist of Mirafi 140N or equivalent.

In lieu of weep holes or open joints, for retaining walls less than 3 feet, a perforated pipe and gravel subdrain may be used. Perforated pipe should consist of 4-inch or more diameter PVC Schedule 40 or ABS SDR-35, with the perforations laid down. The pipe should be embedded in 1.5 cubic feet per foot of 0.75 or 1.5-inch open graded gravel wrapped in filter fabric. Filter fabric may consist of Mirafi 140N equivalent.

Retaining walls greater than 3 feet high should be provided with a continuous backdrain for the full height of the wall. This drain could consist of geosynthetic drainage composite, such as Miradrain 6000 or equivalent, or a permeable drain material, placed against the entire backside of the wall. If a permeable drain material is used, the backdrain should be 1 or more feet thick. Caltrans Class II permeable material or open graded gravel or crushed stone may be used as permeable drain material. If gravel or crushed stone is used, it should have less than 5 percent material passing the No. 200 sieve. The drain should be separated from the backfill with a geofabric. The upper 1-foot of the backdrain should be covered with compacted fill. A drainage pipe consisting of 4-inch diameter perforated pipe (described above) surrounded by 1 cubic foot per foot of gravel or crushed rock wrapped in a filter fabric should be provided along the back of the wall. The pipe should be placed with perforations down, sloped at 2 percent or more and discharge to an appropriate outlet through a solid pipe. The pipe should outlet away from structures and slopes. The outside portions of retaining walls supporting backfill should be coated with an approved waterproofing compound to inhibit infiltration of moisture through the walls.

12.4 Temporary Excavations

Retaining walls should be constructed and backfilled as soon as possible after backcut excavations are constructed. Prolonged exposure of backcut slopes may result in some localized slope instability. To facilitate retaining wall construction, the lower 5 feet of temporary slopes may be cut vertical and the upper portions exceeding a height of 5 feet should be cut back at a gradient of 1:1 (h:v) or flatter for the duration of construction. However, temporary slopes should be observed by the project geotechnical consultant for evidence of potential instability. Depending on the results of these observations, flatter slopes may be necessary. The potential effects of various parameters such as weather, heavy equipment travel, storage near the tops of the temporary excavations and construction scheduling should also be considered in the stability of temporary slopes. Water should not be permitted to drain away from the slope. Surcharges due to equipment, spoil piles, etc., should not be allowed within 10 feet of the top of the slope.

All excavations should be made in accordance with Cal/OSHA. Excavation safety is the sole responsibility of the contractor.

12.5 Retaining Wall Backfill

The retaining wall backfill soil (with an angle of internal friction of at least 33 degrees) should be placed in 6 to 8 inch loose lifts, watered or air-dried as necessary to achieve near optimum moisture conditions, and compacted to at least 90 percent relative density (based on ASTM Test Methods D2922 and D3017).

13.0 MASONRY GARDEN WALLS

13.1 Construction on Level Ground

Where masonry screen walls or garden walls are proposed on level ground and 5 feet or more from the tops of descending slopes, the footings for these walls may be founded at a depth of 18 inches or more below the lowest adjacent final grade. These footings should also be reinforced with four No. 4 bars, two top and two bottom and in accordance with the structural engineer's recommendations.

13.2 Construction Joints

In order to mitigate the potential for unsightly cracking related to the effects of differential settlement, positive separations (construction joints) should be provided in the walls at horizontal intervals of approximately 25 feet and at each corner. The separations should be provided in the blocks only and not extend through the footings. The footings should be placed monolithically with continuous rebar to serve as effective "grade beams" along the full lengths of the walls.

14.0 CONCRETE FLATWORK

14.1 Nonstructural Concrete Flatwork

Concrete flatwork (such as walkways, bicycle trails, etc.) has a high potential for cracking because of changes in soil volume related to soil-moisture fluctuations. To reduce the potential for excessive cracking and lifting, concrete should be designed in accordance with the minimum guidelines outlined in Table 6. These guidelines will reduce the potential for irregular cracking and promote cracking along construction joints but will <u>not</u> eliminate all cracking or lifting. Thickening the concrete and/or adding additional reinforcement will further reduce cosmetic distress.

TABLE 6
Nonstructural Concrete Flatwork for Medium Expansive Soils

	Private Sidewalks	*Private Drives*	*Patios/Entryways*	*City Sidewalk Curb and Gutters*
Minimum Thickness (in.)	4 (nominal)	5 (full)	4 (full)	City/Agency Standard
Presaturation	Presoak to 18 inches	Presoak to 18 inches	Presoak to 18 inches	City/Agency Standard
Reinforcement	—	No. 3 at 18 inches on centers	No. 3 at 18 inches on centers	City/Agency Standard
Thickened Edge	—	8″ x 8″	8″ X 8″	City/Agency Standard
Crack Control	Saw cut or deep open tool joint to a minimum of 1/3 the concrete thickness	Saw cut or deep open tool joint to a minimum of 1/3 the concrete thickness	Saw cut or deep open tool joint to a minimum of 1/3 the concrete thickness	City/Agency Standard
Maximum Joint Spacing	5 feet	10 feet or quarter cut whichever is closer	6 feet	City/Agency Standard

14.2 Joint Spacing

To reduce the potential for unsightly cracking, concrete sidewalks and patio type slabs should be provided with construction or expansion joints every 6 feet or less. Concrete driveway slabs should be provided with construction or expansion joints every 10 feet or less.

14.3 Subgrade Preparation

As a further measure to reduce cracking of concrete flatwork, the upper 12 inches of subgrade soil below concrete-flatwork areas should first be compacted to a relative density of 90 percent of more and then thoroughly wetted to achieve a moisture content that is equal to or slightly greater than optimum moisture content. This moisture should extend to a depth of 12 inches or more below subgrade and maintained in the soil during placement of concrete. Pre-watering of the soil will promote uniform curing of the concrete and reduce the potential for the development of shrinkage cracks. A representative of the project geotechnical consultant should observe and document the density and moisture content of the soil and depth of moisture penetration prior to placing concrete.

15.0 PLANTERS

Area drains should be extended into planters that are located within 5 feet of building walls, foundations, retaining walls and masonry garden walls to reduce excessive infiltration of water into the adjacent foundation soil. The surface of the ground in these areas should also be sloped at a gradient of 2 percent or more away from the walls and foundations. Drip-irrigation systems are also recommended to reduce overwatering and subsequent saturation of the adjacent foundation soil.

16.0 SOIL CORROSIVITY

16.1 Corrosivity to Concrete and Metal

The National Association of Corrosion Engineers (NACE) defines corrosion as "a deterioration of a substance or its properties because of a reaction with its environment". From a geotechnical viewpoint, the "environment" is the prevailing foundation soil and the "substances" are the reinforced concrete foundations or various buried metallic elements such as rebar, piles, pipes, etc., which are in direct contact with or within close vicinity of the foundation soil.

In general, soil environments that are detrimental to concrete have high concentrations of soluble sulfates. ACI 318R-05, Table 4.3.1 provides specific guidelines for the concrete mix design based on different amount of soluble sulfate content. The minimum amount of chloride ions in the soil environment that are corrosive to steel, either in the form of reinforcement protected by concrete cover, or plain steel substructures such as steel pipes or piles, is 500 ppm per California Test 532 and ACI 318R-05, Table 4.4.1.

The corrosion potential of the onsite materials was evaluated for its effect on steel and concrete. The corrosion potential was evaluated using the results of laboratory tests on representative samples obtained during our field exploration. Laboratory testing was performed to evaluate pH, minimum electrical resistivity and chloride and soluble sulfate content. Based on testing performed during this investigation within the project site, the onsite soil are classified as having a **negligible** sulfate exposure condition in accordance with ACI 318R-05, Table 4.3.1, and **negligible** chloride exposure condition in accordance with ACI 318R-05, Table 4.4.1. Based on laboratory testing of on-site soil it is also our opinion that onsite soil should be considered **moderately** corrosive to buried metals due to the low resistivity. Metal piping should be corrosion-protected or consideration should be given to using plastic piping instead of metal or plastic sleeving around the metal pipe.

Despite the minimum recommendation above, LGC is not a corrosion-engineering firm. Therefore, we recommend that you consult with a competent corrosion engineer and conduct additional testing (if required) to evaluate the actual corrosion potential of the site and to provide recommendations to reduce the corrosion potential with respect to the proposed improvements. The recommendations of the corrosion engineer may supersede the above requirements.

These recommendations are based on the current and previous samples of the subsurface soil or bedrock. The initiation of grading at the site could blend various soil types and import soil may be used locally. These changes made to the foundation soil could alter sulfate-content levels. Accordingly, it is recommended that additional testing be performed at the completion of grading.

17.0 PRELIMINARY PAVEMENT DESIGN

17.1 Visual Inspection and Mapping

Surface mapping of the existing pavement conditions was accomplished utilizing the Google Earth imagery for assessing the magnitude of the existing distress and field mapping. This was performed by an engineer from this firm on October 3, 2019.

Overall, the existing pavement exhibits moderate to severe shrinkage cracking, slight to moderate potholing, slight rutting, and local areas of moderate pavement settlement along Menlo Avenue and Girard Street. Shrinkage cracks were present in the form of longitudinal cracking, transverse cracking, and alligator cracking with an average maximum thickness of 1.5 inches. Typical factors that can cause shrinkage cracking within the AC include volume change in the AC mix, volume changes in the subgrade materials, age of the pavement, and/or combinations of these factors. Rutting was observed within severe areas of shrinkage cracking along within localized areas of pavement settlement. Rutting and pavement settlement in AC is usually caused by consolidation or lateral movement of subgrade materials from traffic loading and/or inadequate pavement section thickness. Potholing was observed in areas where cracks were present throughout entire thicknesses of asphaltic concrete representing complete structural failure of pavement section.

Generally, Menlo Avenue was observed to have moderate to severe shrinkage cracking, slight rutting, slight to moderate rutting, and local areas of moderate pavement settlement was observed in the following areas:

- Moderate to severe shrinkage cracking within Menlo Avenue was observed in both the east and west bound lanes.
- Severe longitudinal cracking and alligator cracking was observed along the centerline of Menlo Avenue.
- Moderate to severe transverse cracking was observed throughout Menlo Avenue.
- Slight rutting was observed along the centerline of Menlo Avenue within areas of severe shrinkage cracking.
- A localized area of pavement settlement was observed in the east bound lane that appeared to be in line within an existing utility trench.
- Moderate cracking ranged from ¼ inches to ¾ inches and severe cracking ranged from ¾ inches to 2 inches wide.

Light to severe shrinkage cracking was observed within Girard Avenue along with localized areas of moderate pavement section settlement:

- Light to moderate longitudinal shrinkage cracking was observed along the centerline of Girard Avenue.
- Moderate to severe transverse shrinkage cracking was observed within Girard Avenue. Severe cracks were observed to have widths up to 3.5 inches.
- Severe alligator cracking and moderate pavement settlement was observed within southbound lane of Girard Avenue.

17.2 Subsurface Exploration

The site subsurface conditions were explored by LGC on October 3, 2019 by means of two (2) cores excavated with a hand auger within Menlo Avenue. Continuous logs of the subsurface borings are presented within Appendix B. The approximate locations of the cores, with respect to the subject streets, are presented on the Geotechnical Map (Figure 1). Cores C-1 and C-2 were distributed throughout critical areas of the proposed street improvements and ranged in depth from approximately 5.0 feet below ground surface to approximately 6.0 feet below ground surface. Excavations were performed in such a manner to expose, observe, and measure the contacts between the pavement, base materials, and the subgrade. Additionally, the pavement structural sections were measured during our subsurface exploration. Field measurements of the core log thicknesses are presented within Appendix B of this report. The materials encountered in the exploratory cores were continuously logged and visually classified by a geologist from this firm, in accordance with the visual manual procedures of the Unified Soil Classification System.

A representative bulk sample was collected during the field exploration for pavement evaluation.

17.3 Preliminary Pavement Structural Section Designs

Structural pavement section design recommendations presented herein are based on soil samples recovered during our subsurface exploration. However, it should be understood that the soil material exposed during grading may differ from the materials sampled and tested during this investigation. Therefore, preliminary pavement recommendations are subject to verification and possible revision based on observations and possible sampling and testing of subgrade soils that exist after grading.

For purposes of design, we have prepared the following pavement structural sections based on R-values acquired during our recent laboratory testing (Appendix C). The assigned Traffic Indices (T.I.) of 5.5, 7.0, and 8.5 utilized in pavement section calculations was taken from the City of Hemet General Plan 2030, Chapter 4, and County of Riverside Roadway Design Requirements. Since the subgrade R-Value quality of the soils may change, laterally, based on the available laboratory testing, Table 7 proposes the following pavement designs for the areas indicated below:

TABLE 7
Preliminary Pavement Design

AREA	ASSUMED TRAFFIC INDEX	DESIGN R-VALUE	ASPHALTIC CONCRETE (AC) Inches)	AGGREGATE BASE (AB) (AB)(Inches)
Menlo Avenue	8.5	30	6.0	11.0
N. Girard Street	7.0	17	5.0	11.0
Park Avenue	8.5	67	6.0	11.0
Interior Roads	5.5	67	3.0	6.0

Aggregate base materials may consist of crushed miscellaneous base (CMB) or Class 2 aggregate base materials.

Subgrade soil immediately below the aggregate base (base) should be compacted to a minimum of 95 percent relative compaction based on ASTM Test Method D1557 to a minimum depth of 12 inches. Final subgrade compaction should be performed prior to placing base or asphaltic concrete and after all utility trench backfills have been compacted and tested.

Base materials should consist of crushed aggregate base conforming to Section 200-2 of Greenbook and should be compacted to at least 95 percent of the laboratory maximum dry density determined in accordance with ASTM D1557.

Our preliminary pavement recommendations should be considered as minimum and can be revised once actual T.I.'s are known or superseded by the City of Hemet.

17.4 *Pavement Rehabilitation*

Based on existing pavement section thicknesses observed within Menlo Avenue, lack of base material, and the extent of observed cracking within Menlo Avenue and Girard Avenue, the existing pavement does not meet current standards and has started failing due to traffic loads. Areas of moderate to severe cracking and localized areas of moderate settlement should be overexcavated at least 3 feet below existing AC grade and 3 feet outside the distressed area; then replaced with 2 feet of compacted fill to 90% compaction, except for the upper 12 inches of subgrade which should be 95% compaction within all streets. Table 7 above provides preliminary pavement designs to assist in the rehabilitation.

18.0 PLAN REVIEWS AND CONSTRUCTION SERVICES

This is a preliminary geotechnical investigation report prepared for the exclusive use of **Sikand Engineering,** to assist the project engineer and architect in the design of the proposed development. It is recommended that LGC be engaged to review the rough grading plans, foundation plans and other pertinent final design drawings and specifications prior to construction. This is to document that the recommendations contained in this report have been properly interpreted and are incorporated into the project specifications. LGC's review of the final grading plans may indicate that additional subsurface exploration, laboratory testing and analysis should be performed to address areas of concern. If LGC is not accorded the opportunity to review these documents, we can take no responsibility for misinterpretation of our recommendations.

We recommend that LGC be retained to provide geotechnical engineering services during both the rough grading and construction phases of the work. This is to document compliance with the design, specifications or recommendations and to allow design changes in the event that subsurface conditions differ from those anticipated prior to start of construction.

If the project plans change significantly (e.g., building loads or type of structures), LGC should be retained to review our original design recommendations and their applicability to the revised construction. If conditions are encountered during construction that appear to be different than those indicated in this report, this office should be notified immediately. Design and construction revisions may be required.

19.0 LIMITATIONS

Our services were performed using the degree of care and skill ordinarily exercised, under similar circumstances, by reputable engineers and geologists practicing in this or similar localities. The professional opinions contained herein have been derived in accordance with current standards of practice for preliminary reports. No other warranty, expressed or implied, is made as to the conclusions and professional advice included in this report. The samples taken and submitted for laboratory testing, the observations made and the in-situ field testing performed are believed representative of the entire project; however, soil and geologic conditions can vary in characteristics between excavations, both laterally and vertically and may be different than our preliminary findings.

If this occurs, the changed conditions must be evaluated by the project geotechnical engineer and engineering geologist and design(s) adjusted as required or alternate design(s) recommended.

This report is issued with the understanding that it is the responsibility of the owner, or of his/her representative, to ensure that the information and recommendations contained herein are brought to the attention of the architect and/or project engineer and incorporated into the plans, and the necessary steps are taken to see that the contractor and/or subcontractor properly implements the recommendations in the field. The contractor and/or subcontractor should notify the owner if they consider any of the recommendations presented herein to be unsafe.

The conclusions and opinions contained in this report are based on the results of the described geotechnical evaluations and represent our professional judgment. The findings, conclusions and recommendations contained in this report are to be considered preliminary only and subject to confirmation by the undersigned during the construction process. Without this confirmation, this report is to be considered incomplete and LGC or the undersigned professionals assume no responsibility for its use.

The conclusions and opinions contained in this report are valid up to a period of 2 years from the date of this report. Changes in the conditions of a property can and do occur with the passage of time, whether they be because of natural processes or the works of man on this or adjacent properties. In addition, changes in applicable or appropriate codes or standards may occur, whether they result from legislation or the broadening of knowledge. Accordingly, the findings of this report may be invalidated wholly or partially by changes outside our control. Therefore, if any of the above-mentioned situations occur, an update of this report should be completed.

This report has not been prepared for use by parties or projects other than those named or designed above. It may not contain sufficient information for other parties or other purposes.

The opportunity to be of service is appreciated. Should you have any questions regarding the content of this report, or should you require additional information, please do not hesitate to contact this office at your earliest convenience.

APPENDIX A

REFERENCES

APPENDIX A

References

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APPENDIX B

FIELD EXPLORATION TRENCHES AND CORE LOGS

APPENDIX B

Field Exploration

B-1 General

Geologic mapping of the site was carried out by LGC's personnel. The locations of the exploratory excavations were chosen to obtain subsurface information needed to achieve the objective for this investigation.

A visual survey was conducted to verify that the proposed excavations would not encounter any subsurface utility lines. No underground lines were encountered during the field exploratory program.

B-2 Excavation, Trenching and Sampling

Our initial subsurface exploration was performed on October 3, 2019, which included trenching, logging and sampling five (5) trenches, to depths ranging from 5.5 feet to 17.5 feet, and hand augering 2 cores on Menlo Ave to depths of 5 feet and 6 feet, for pavement evaluation. Logs of the trenches and cores are presented in Appendix B, and their approximate locations are depicted on the Geotechnical Map (Plate 1).

Prior to the subsurface work, an underground utilities clearance was obtained from Underground Service Alert of Southern California. At the conclusion of the subsurface investigation, all borings were backfilled with native materials. Minor settlement of the backfill soil may occur over time.

During our subsurface investigation, representative bulk and relatively undisturbed samples were retained for laboratory testing. Laboratory testing was performed on selected representative samples of onsite soil samples and included maximum dry density and optimum moisture content, expansion index, sulfate content, chloride content, pH, resistivity, direct shear, Atterberg limit, and R-Values. A discussion of the tests performed and a summary of the results are presented in Appendix C. Moisture and density test results are presented on the trench logs which are presented on the following pages.

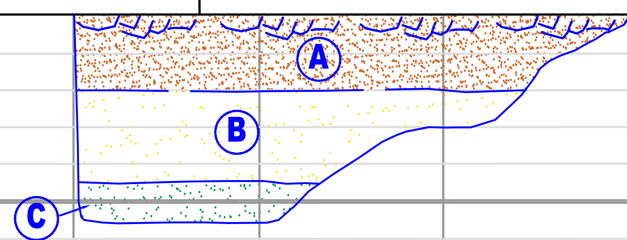
B-3 Miscellaneous

The trench logs describe the earth materials encountered, sampling method used, and field and laboratory tests performed. The logs also show the trench number, date of completion, and the name of the logger. A geologist logged the trenches in accordance with the Standard Practice for Description and Identification of Soils (Visual-Manual Procedure) ASTM D2488-93. The boundaries between soil types shown on the logs are approximate and the transition between different soil layers may be gradual. The logs of the trenches are presented on the following pages.

Project Name: Sikand	Logged by: JL	LOG OF TRENCH TR-1			
Project Number: G18-1647-10	Elevation: 1607'	Engineering Properties			
Equipment: BACKHOE	Location/Grid: SEE INFILTRATION TEST MAP				

Depth	Date: 10/3/2019	Description:	Geologic Unit	USCS	Sample No.	Moisture (%)	Dry Density (pcf)
0.0'-2.0'	A	<u>ARTIFICIAL FILL, UNDOCUMENTED:</u> Silty SAND; light brown, dry, loose to medium dense, very fine to fine grained, roots, roothairs, blocky, desiccated	Afu	SM	Bulk @ 0.0'-3.0' Nuke @0.0'	1.5	94.3
2.0'-4.5'	B	<u>ALLUVIUM:</u> Silty SAND; yellowish brown, moist, loose, very fine to fine grained with occasional coarse grains, roothairs	Qal		Bulk @ 2.0'-5.0' Nuke @ 3.0'	3.2	85.7
4.5'-5.5'	C	@4.5'; brown, caliche nodules			Nuke @ 5.5'	2.3	91.9

GRAPHICAL REPRESENTATION: EAST WALL	SCALE: 1" = 5'	SURFACE SLOPE: LEVEL	TREND: N5W

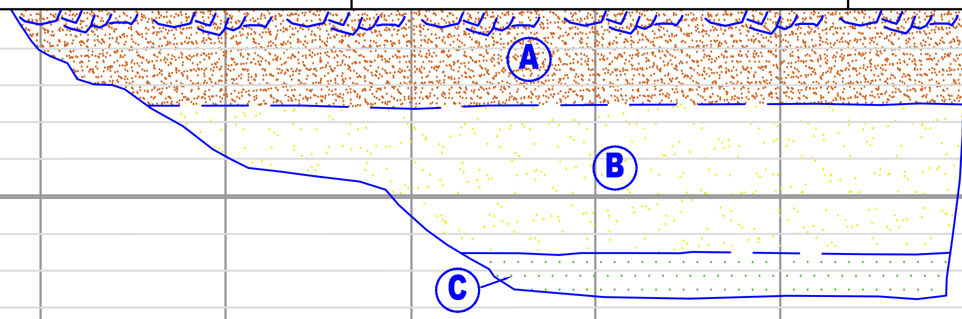


TOTAL DEPTH= 5.5 FEET
NO GROUNDWATER ENCOUNTERED

Project Name: Sikand	**Logged by: JL**	**LOG OF TRENCH TR-2**
Project Number: G18-1647-10	**Elevation: 1611'**	**Engineering Properties**
Equipment: BACKHOE	**Location/Grid: SEE INFILTRATION TEST MAP**	

Depth	Date: 10/3/2019	Description:	Geologic Unit	USCS	Sample No.	Moisture (%)	Dry Density (pcf)
0.0'-2.5'	A	ARTIFICIAL FILL, UNDOCUMENTED: Silty SAND; yellowish brown, dry, loose to medium dense, very fine to fine grained with some coarse grains, roots, roothairs, blocky, desiccated	Afu	SM	Nuke @ 0.0'	1.4	99.4
2.5'-6.5'	B	ALLUVIUM: Silty SAND; light brown to brown, moist, loose, very fine to fine grained, roothairs, tree roots, pinhole pores	Qal		Nuke @ 6.5'	5.7	83.8
6.5'-7.5'	C	@6.5'; orangish brown, moist, medium dense, fine grained, caliche stringers		SM/ML	Nuke @ 7.5'	4.8	97.3

GRAPHICAL REPRESENTATION: EAST WALL	**SCALE: 1" = 5'**	**SURFACE SLOPE: LEVEL**	**TREND: N70E**

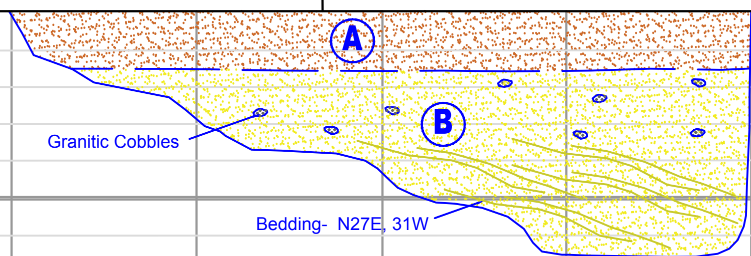


TOTAL DEPTH= 7.5 FEET
NO GROUNDWATER ENCOUNTERED

Project Name: Sikand	Logged by: JL	LOG OF TRENCH TR-3			
Project Number: G18-1647-10	Elevation: 1616'	Engineering Properties			
Equipment: BACKHOE	Location/Grid: SEE INFILTRATION TEST MAP				

Depth	Date: 10/3/2019	Description:	Geologic Unit	USCS	Sample No.	Moisture (%)	Dry Density (pcf)
0.0'-1.5'	A	ARTIFICIAL FILL, UNDOCUMENTED: Silty SAND; light brown, dry, medium dense, very fine to coarse grained with some gravels, roots, roothairs	Afu	SM	Nuke @ 0.0'	0.2	102.5
1.5'-6.5'	B	BAUTISTA FORMATION: SANDSTONE/SILTSTONE; gray to light brown, moist, moderately hard, medium to coarse grained with gravels, weathered granitic clasts, manganese staining, oxidation staining, bedding	Qts		Bulk @ 1.0'-5.0' Nuke @ 6.5'	3.0	107.0

GRAPHICAL REPRESENTATION: SOUTH WALL	SCALE: 1" = 5'	SURFACE SLOPE: LEVEL	TREND: N59E

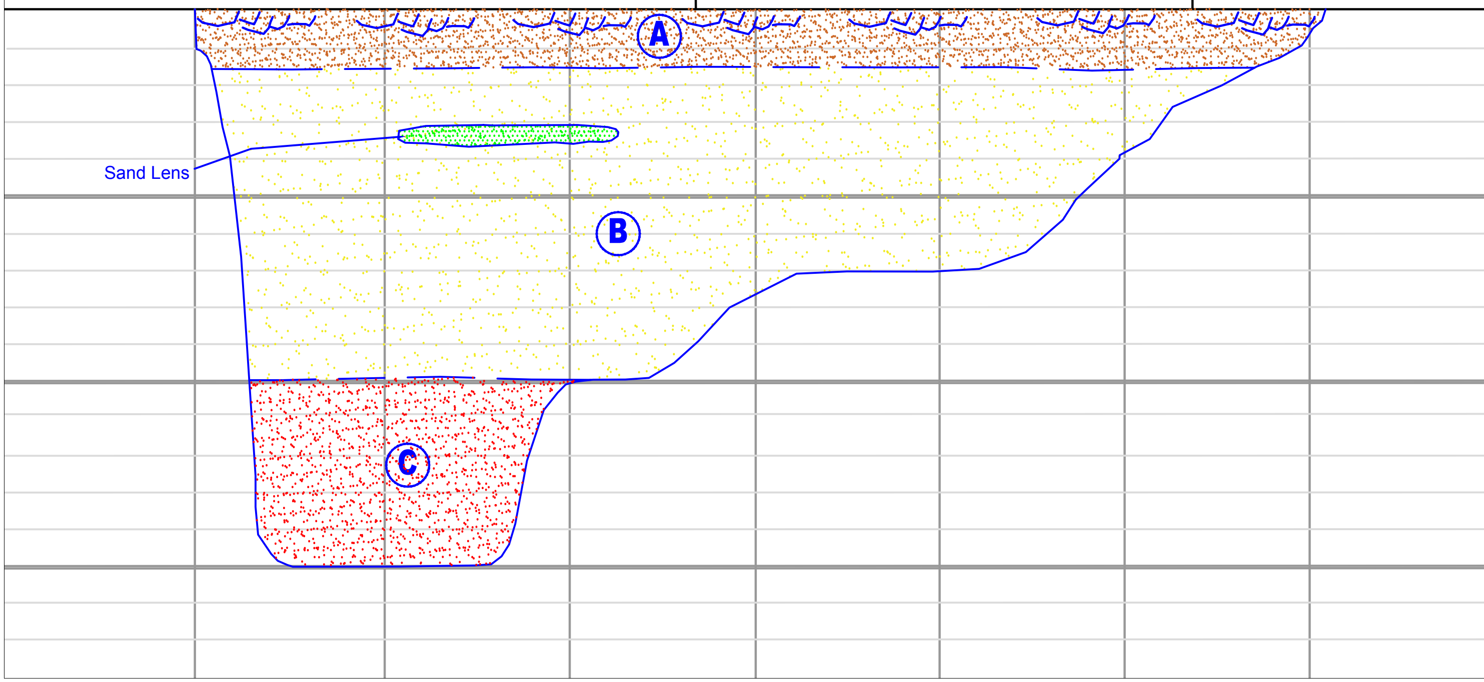


TOTAL DEPTH= 6.5 FEET
NO GROUNDWATER ENCOUNTERED

Project Name: Sikand	Logged by: JL	LOG OF TRENCH TR-4
Project Number: G18-1647-10	Elevation: 1629'	Engineering Properties
Equipment: BACKHOE	Location/Grid: SEE GEOTECHNICAL MAP	

Depth	Date: 10/3/2019	Description:	Geologic Unit	USCS	Sample No.	Moisture (%)	Dry Density (pcf)
0.0'-1.5'	A	ARTIFICIAL FILL, UNDOCUMENTED: Silty SAND; light brown, dry, loose to medium dense, fine to medium grained with some coarse grains, roots, roothairs, blocky, desiccated	Afu	SM	Bulk @ 0.0'-3.0' Nuke @0.0'	0.2	94.7
1.5'-10.0'	B	ALLUVIUM: Silty SAND; yellowish brown, dry to moist, medium dense, fine to medium grained with occasional coarse grains, roothairs, pinhole pores, trace oxidation staining, caliche nodules, sand lens	Qal		Bulk @ 2.0'-5.0' Nuke @ 6.0'	1.8	103.1
10.0'-15.0'	C	Poorly-graded SAND with SILT; orangish brown, moist, medium dense, fine to medium grained with some coarse grains, trace oxidation, micaceous		SP/SM	Bulk @ 10.0'-15.0' Nuke @ 10.0'	2.5	105.3

GRAPHICAL REPRESENTATION: EAST WALL | **SCALE: 1" = 5'** | **SURFACE SLOPE: LEVEL** | **TREND: N29W**

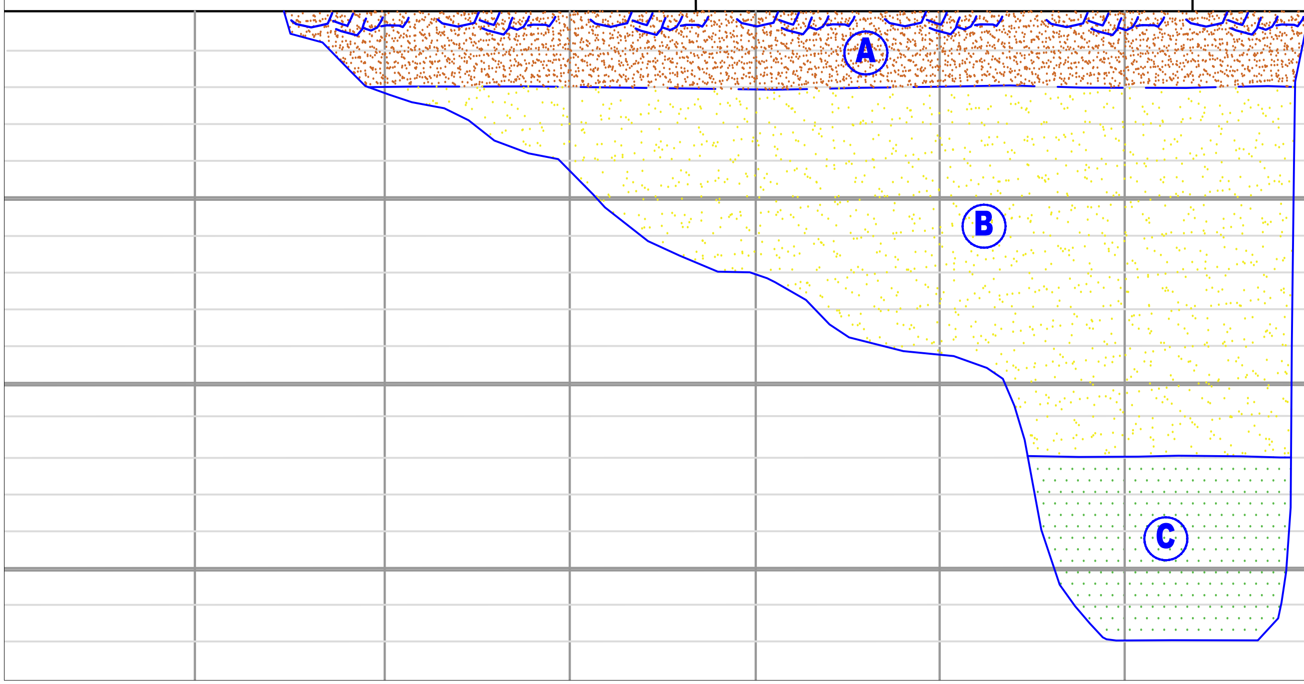


TOTAL DEPTH=15.0 FEET
NO GROUNDWATER ENCOUNTERED

Project Name: Sikand	Logged by: JL	LOG OF TRENCH TR-5
Project Number: G18-1647-10	Elevation: 1617'	Engineering Properties
Equipment: BACKHOE	Location/Grid: SEE GEOTECHNICAL MAP	

Depth	Date: 10/3/2019	Description:	Geologic Unit	USCS	Sample No.	Moisture (%)	Dry Density (pcf)
0.0'-2.0'	A	ARTIFICIAL FILL, UNDOCUMENTED: Silty SAND; light brown, dry, loose to medium dense, very fine to fine grained with occasional coarse grains, roots, roothairs, blocky, desiccated	Afu	SM	Bulk @ 0.0'-6.0' Nuke @0.0'	0.3	92.8
2.0'-12.0'	B	ALLUVIUM: Silty SAND; yellowish brown, moist, loose to medium dense, very fine to fine grained with occasional coarse grains and gravel, roothairs, caliche nodules, pinhole pores	Qal		Nuke @ 6.0'	7.1	88.2
12.0'-17.0'	C	SILT with CLAY; orangish brown to grayish brown, moist, loose to medium dense, very fine, roothairs, micaceous		ML/CL	Bulk @ 12.0'-17.0'		

GRAPHICAL REPRESENTATION: SOUTH WALL	SCALE: 1" = 5'	SURFACE SLOPE: LEVEL	TREND: N79E



TOTAL DEPTH=17.5 FEET
NO GROUNDWATER ENCOUNTERED

Geotechnical Boring Log C-1

Date: 10/3/19	**Project Name:** SIKAND — Page 1 of 1
Project Number: G18-1647-10	**Logged By:** JW/AJR
Drilling Company:	**Type of Rig:** HAND AUGER
Drive Weight (lbs.):	**Drop (in.):** **Hole Dia. (in.):** 5"
Top of Hole Elevation (ft): 1,624'	**Hole Location:** SEE GEOTECHNICAL MAP

Elevation (MSL) and Depth (ft.)	Blow Count / 6"	Sample No.	Soil Graphic	Geologic / Group Symbol	DESCRIPTION	In-Situ Moist. (%)	Dry Density (pcf)	Standard Penetration Test SPT Depth	N	CURVE 10 30 50	Type of Test
0					**ASPHALT** grayish black A/C						
1620 — 5				Afu SM	**ARTIFICIAL FILL, UNDOCUMENTED** Silty SAND; grayish brown, dry, very fine to medium grained with some coarse grains and occasional very coarse grains, micaceous @4.0'; dark olive brown, dry to damp, very fine to fine grained with some medium grains						
1615 — 10					Total Depth: '6						
1610 — 15											
1605 — 20											
1600 — 25											
1595 — 30											

Sample Legend

- SPT
- Ring Sample (CA modified)

LGC GEO-ENVIRONMENTAL, INC.

Geotechnical Boring Log C-2

Date: 10/3/19	**Project Name:** SIKAND — Page 1 of 1
Project Number: G18-1647-10	**Logged By:** JW/AJR
Drilling Company:	**Type of Rig:** HAND AUGER
Drive Weight (lbs.):	**Drop (in.):** **Hole Dia. (in.):** 5"
Top of Hole Elevation (ft):	**Hole Location:** SEE GEOTECHNICAL MAP

Elevation (MSL) and Depth (ft.)	Blow Count / 6"	Sample No.	Soil Graphic	Geologic / Group Symbol	DESCRIPTION	In-Situ Moist. (%)	Dry Density (pcf)	Standard Penetration Test SPT Depth	N	CURVE 10 30 50	Type of Test
0					**ASPHALT** grayish black A/C						
				Afu SM	**ARTIFICIAL FILL, UNDOCUMENTED** Silty SAND; grayish brown, dry, very fine to medium grained with some coarse grains and occasional very coarse grains, micaceous						
					@4.0'; dark olive brown, dry to damp, very fine to fine grained with some medium grains						
5					Total Depth: '5						
10											
15											
20											
25											
30											

Sample Legend

- SPT
- Ring Sample (CA modified)

LGC GEO-ENVIRONMENTAL, INC.

APPENDIX C

LABORATORY TESTING PROCEDURES AND TEST RESULTS

APPENDIX C

Laboratory Testing Procedures and Test Results

The laboratory testing program was directed towards providing quantitative data relating to the relevant engineering properties of the soil. Samples considered representative of site conditions were tested in general accordance with American Society for Testing and Materials (ASTM) procedure and/or California Test Methods (CTM), where applicable. The following summary is a brief outline of the test type and a table summarizing the test results.

Soil Classification: Soil were classified according the Unified Soil Classification System (USCS) in accordance with ASTM Test Methods D2487 and D2488. The soil classifications (or group symbol) are shown on the laboratory test data, and boring logs.

Maximum Dry Density Tests: The maximum dry density and optimum moisture content of typical materials were determined in accordance with ASTM test method D1557. The test results are presented in the table below:

SAMPLE LOCATION	SAMPLE DESCRIPTION (USCS)	MAXIMUM DRY DENSITY (% by weight)	OPTIMUM MOISTURE CONTENT (%)
TR-4 @ 10′-15′	Poorly Graded SAND (SP)	127.5	8.0
TR-5 @ 0′-6′	Silty SAND (SM)	124.2	11.0

Expansion Index: The expansion potential of a selected sample was evaluated by the Expansion Index Test, U.B.C. Standard No. 18-2 and/or ASTM test method D4829. Specimens are molded under a given compactive energy at or near the optimum moisture content and approximately 50 percent saturation or approximately 90 percent relative compaction. The prepared 1-inch thick by 4-inch diameter specimens are loaded to an equivalent 144 psf surcharge and are inundated with tap water until volumetric equilibrium is reached. The results of these tests are presented in the table below:

SAMPLE LOCATION	*SAMPLE DESCRIPTION* **(USCS)**	*EXPANSION INDEX*	*EXPANSION POTENTIAL**
TR-5 @ 0′-6′	Silty SAND (SM)	67	Medium

*Per ASTM D4829

Soluble Sulfates: The soluble sulfate content of selected samples was determined by standard geotechnical methods (CTM 417). The soluble sulfate content is used to determine the appropriate cement type and maximum water-cement ratios. The test results are presented in the table below:

SAMPLE LOCATION	*SAMPLE DESCRIPTION* **(USCS)**	*SULFATE CONTENT (ppm)*	*SULFATE EXPOSURE**
TR-5 @ 0′-6′	Silty SAND (SM)	Non-Detected	Negligible

*Per ACI 318R-05 Table 4.3.1

Chloride Content: Chloride content was tested with CTM 422. The results are presented below:

SAMPLE LOCATION	*SAMPLE DESCRIPTION* **(USCS)**	*CHLORIDE CONTENT (ppm)*
TR-5 @ 0′-6′	Silty SAND (SM)	21

Minimum Resistivity and pH Tests: Minimum resistivity and pH tests were performed with CTM 643. The results are presented in the table below:

SAMPLE LOCATION	*SAMPLE DESCRIPTION* **(USCS)**	*pH*	*MINIMUM RESISTIVITY (ohm-cm)*
TR-5 @ 0′-6′	Silty SAND (SM)	8.2	2,100

Direct Shear: Direct shear tests were performed on selected remolded samples, which were soaked for a minimum of 24 hours under a surcharge equal to the applied normal force during testing. After transfer of the sample to the shear box, and reloading the sample, pore pressures set up in the sample due to the transfer were allowed to dissipate for a period of approximately 1 hour prior to application of shearing force. The samples were tested under various normal loads, a motor-driven, strain-controlled, direct-shear testing apparatus at a strain rate of less than 0.001 to 0.5 inch per minute (depending upon the soil type). The graphical test results are presented in the table below:

SAMPLE LOCATION	*SAMPLE DESCRIPTION*	*ANGLE OF INTERNAL FRICTION (degrees)*	*COHESION (psf)*
TR-5 @ 0′-6′	Silty SAND (SM)	28	160

Atterberg Limits: The liquid and plastic limits ("Atterberg Limits") were determined with ASTM D4318 for engineering classification of fine material and presented in the table below:

SAMPLE LOCATION	*LIQUID LIMIT*	*PLASTIC LIMIT*	*PLASTICITY INDEX*	*USCS SOIL SYMBOL*
TR-5 @ 0′-6′	31	19	12	SM

R-Value: The resistance R-value was determined by the ASTM test method D2844 for base, sub-base, and basement soil. The samples were prepared and exudation pressure and R-value were determined. These results were used for pavement design:

SAMPLE LOCATION	*SAMPLE DESCRIPTION* **(USCS)**	*R-VALUE*
TR-1 @ 0′-3′	Silty SAND (SM)	17
TR-4 @ 0′-3′	Silty SAND (SM)	67
Menlo Ave 0.5′-1.5′	Silty SAND (SM)	30

APPENDIX D

GENERAL EARTHWORK AND GRADING SPECIFICATIONS

APPENDIX D

General Earthwork and Grading Specifications

1.0 General

1.1 ***Intent:*** These General Earthwork and Grading Specifications are for the grading and earthwork shown on the approved grading plan(s) and/or indicated in the geotechnical report(s). These Specifications are a part of the recommendations contained in the geotechnical report(s). In case of conflict, the specific recommendations in the geotechnical report shall supersede these more general Specifications. Observations of the earthwork by the project Geotechnical Consultant during the course of grading may result in new or revised recommendations that could supersede these specifications or the recommendations in the geotechnical report(s).

1.2 ***The Geotechnical Consultant of Record:*** Prior to commencement of work, the owner shall employ a qualified Geotechnical Consultant of Record (Geotechnical Consultant). The Geotechnical Consultant shall be responsible for reviewing the approved geotechnical report(s) and accepting the adequacy of the preliminary geotechnical findings, conclusions, and recommendations prior to the commencement of the grading.

Prior to commencement of grading, the Geotechnical Consultant shall review the "work plan" prepared by the Earthwork Contractor (Contractor) and schedule sufficient personnel to perform the appropriate level of observation, mapping, and compaction testing.

During the grading and earthwork operations, the Geotechnical Consultant shall observe, map, and document the subsurface exposures to verify the geotechnical design assumptions. If the observed conditions are found to be significantly different than the interpreted assumptions during the design phase, the Geotechnical Consultant shall inform the owner, recommend appropriate changes in design to accommodate the observed conditions, and notify the review agency where required.

The Geotechnical Consultant shall observe the moisture-conditioning and processing of the subgrade and fill materials and perform relative compaction testing of fill to confirm that the attained level of compaction is being accomplished as specified. The Geotechnical Consultant shall provide the test results to the owner and the Contractor on a routine and frequent basis.

1.3 ***The Earthwork Contractor:*** The Earthwork Contractor (Contractor) shall be qualified, experienced, and knowledgeable in earthwork logistics, preparation and processing of ground to receive fill, moisture-conditioning and processing of fill, and compacting fill. The Contractor shall review and accept the plans, geotechnical report(s), and these Specifications prior to commencement of grading. The Contractor shall be solely responsible for performing the grading in accordance with the project plans and specifications. The Contractor shall prepare and submit to the owner and the Geotechnical Consultant a work plan that indicates the sequence of earthwork grading, the number of "equipment" of work and the estimated quantities of daily earthwork contemplated for the site prior to commencement of grading.

The Contractor shall inform the owner and the Geotechnical Consultant of changes in work schedules and updates to the work plan at least 24 hours in advance of such changes so that appropriate personnel will be available for observation and testing. The Contractor shall not assume that the Geotechnical Consultant is aware of all grading operations.

The Contractor shall have the sole responsibility to provide adequate equipment and methods to accomplish the earthwork in accordance with the applicable grading codes and agency ordinances, these Specifications, and the recommendations in the approved geotechnical report(s) and grading plan(s). If, in the opinion of the Geotechnical Consultant, unsatisfactory conditions, such as unsuitable soil, improper moisture condition, inadequate compaction, insufficient buttress key size, adverse weather, etc., are resulting in a quality of work less than required in these specifications, the Geotechnical Consultant shall reject the work and may recommend to the owner that construction be stopped until the conditions are rectified. It is the contractor's sole responsibility to provide proper fill compaction.

2.0 *Preparation of Areas to be Filled*

2.1 ***Clearing and Grubbing:*** Vegetation, such as brush, grass, roots, and other deleterious material shall be sufficiently removed and properly disposed of in a method acceptable to the owner, governing agencies, and the Geotechnical Consultant.

The Geotechnical Consultant shall evaluate the extent of these removals depending on specific site conditions. Earth fill material shall not contain more than 1 percent of organic materials (by volume). No fill lift shall contain more than 10 percent of organic matter. Nesting of the organic materials shall not be allowed.

If potentially hazardous materials are encountered, the Contractor shall stop work in the affected area, and a hazardous material specialist shall be informed immediately for proper evaluation and handling of these materials prior to continuing to work in that area.

As presently defined by the State of California, most refined petroleum products (gasoline, diesel fuel, motor oil, grease, coolant, etc.) have chemical constituents that are considered to be hazardous waste. As such, the indiscriminate dumping or spillage of these fluids onto the ground may constitute a misdemeanor, punishable by fines and/or imprisonment, and shall not be allowed. The contractor is responsible for all hazardous waste relating to his work. The Geotechnical Consultant does not have expertise in this area. If hazardous waste is a concern, then the Client should acquire the services of a qualified environmental assessor.

2.2 ***Processing:*** Existing ground that has been declared satisfactory for support of fill by the Geotechnical Consultant shall be scarified to a minimum depth of 6 inches. Existing ground that is not satisfactory shall be overexcavated as specified in the following section. Scarification shall continue until soil are broken down and free of oversize material and the working surface is reasonably uniform, flat, and free of uneven features that would inhibit uniform compaction.

2.3 ***Overexcavation:*** In addition to removals and overexcavations recommended in the approved geotechnical report(s) and the grading plan, soft, loose, dry, saturated, spongy, organic-rich, highly fractured or otherwise unsuitable ground shall be overexcavated to competent ground as evaluated by the Geotechnical Consultant during grading.

2.4 ***Benching:*** Where fills are to be placed on ground with slopes steeper than 5:1 (horizontal to vertical units), the ground shall be stepped or benched. The lowest bench or key shall be a minimum of 15 feet wide and at least 2 feet deep, into competent material as evaluated by the Geotechnical Consultant. Other benches shall be excavated a minimum height of 4 feet into competent material or as otherwise recommended by the Geotechnical Consultant. Fill placed on ground sloping flatter than 5:1 shall also be benched or otherwise overexcavated to provide a flat subgrade for the fill.

2.5 ***Evaluation/Acceptance of Fill Areas:*** All areas to receive fill, including removal and processed areas, key bottoms, and benches, shall be observed, mapped, elevations recorded, and/or tested prior to being accepted by the Geotechnical Consultant as suitable to receive fill. The Contractor shall obtain a written acceptance from the Geotechnical Consultant prior to fill placement. A licensed surveyor shall provide the survey control for determining elevations of processed areas, keys, and benches.

3.0 Fill Material

3.1 ***General:*** Material to be used as fill shall be essentially free of organic matter and other deleterious substances evaluated and accepted by the Geotechnical Consultant prior to placement. Soil of poor quality, such as those with unacceptable gradation, high expansion potential, or low strength shall be placed in areas acceptable to the Geotechnical Consultant or mixed with other soil to achieve satisfactory fill material.

3.2 ***Oversize:*** Oversize material defined as rock, or other irreducible material with a maximum dimension greater than 8 inches, shall not be buried or placed in fill unless location, materials, and placement methods are specifically accepted by the Geotechnical Consultant. Placement operations shall be such that nesting of oversized material does not occur and such that oversize material is completely surrounded by compacted or densified fill. Oversize material shall not be placed within 10 vertical feet of finish grade or within 2 feet of future utilities or underground construction.

3.3 ***Import:*** If importing of fill material is required for grading, proposed import material shall meet the requirements of this Section. The potential import source shall be given to the Geotechnical Consultant at least 48 hours (2 working days) before importing begins so that its suitability can be determined and appropriate tests performed.

4.0 Fill Placement and Compaction

4.1 ***Fill Layers:*** Approved fill material shall be placed in areas prepared to receive fill (per Section 3.0) in near-horizontal layers not exceeding 8 inches in loose thickness. The Geotechnical Consultant may accept thicker layers if testing indicates the grading procedures can adequately compact the thicker layers. Each layer shall be spread evenly and mixed thoroughly to attain relative uniformity of material and moisture throughout.

4.2 ***Fill Moisture Conditioning:*** Fill soil shall be watered, dried back, blended, and/or mixed, as necessary to attain relatively uniform moisture content at or slightly over optimum. Maximum density and optimum soil moisture content tests shall be performed in accordance with the American Society of Testing and Materials (ASTM Test Method D1557-91).

4.3 ***Compaction of Fill:*** After each layer has been moisture-conditioned, mixed, and evenly spread, it shall be uniformly compacted to not less than 90 percent of maximum dry density (ASTM Test Method D1557-91). Compaction equipment shall be adequately sized and be either specifically designed for soil compaction or of proven reliability to efficiently achieve the specified level of compaction with uniformity.

4.4 ***Compaction of Fill Slopes:*** In addition to normal compaction procedures specified above, compaction of slopes shall be accomplished by backrolling of slopes with sheepsfoot rollers at increments of 3 to 4 feet in fill elevation, or by other methods producing satisfactory results acceptable to the Geotechnical Consultant. Upon completion of grading, relative compaction of the fill, out to the slope face, shall be at least 90 percent of maximum density per ASTM Test Method D1557-91.

4.5 ***Compaction Testing:*** Field tests for moisture content and relative compaction of the fill soil shall be performed by the Geotechnical Consultant. Location and frequency of tests shall be at the Consultant's discretion based on field conditions encountered. Compaction test locations will not necessarily be selected on a random basis. Test locations shall be selected to verify adequacy of compaction levels in areas that are judged to be prone to inadequate compaction (such as close to slope faces and at the fill/bedrock benches).

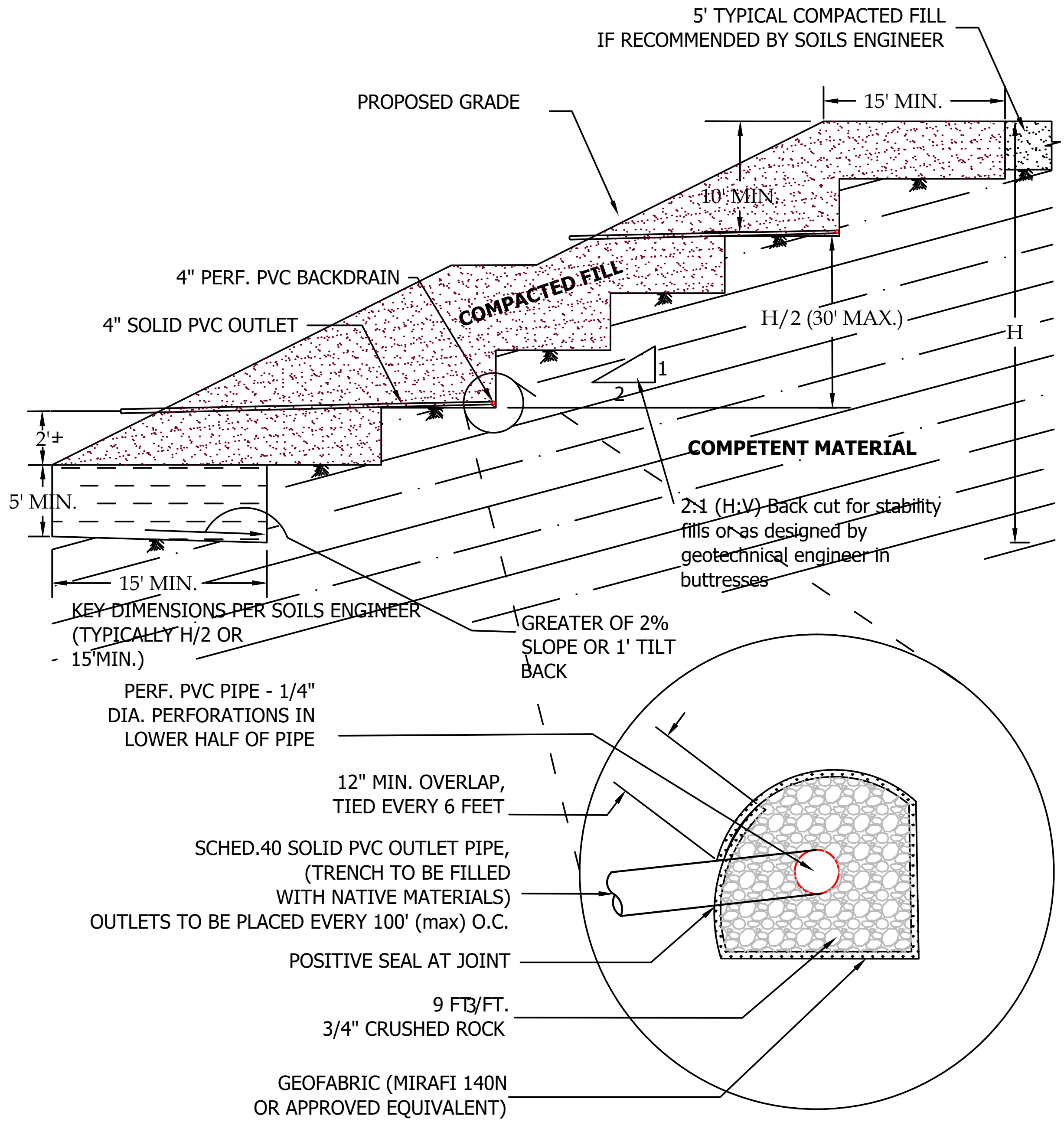
4.6 ***Frequency of Compaction Testing:*** Tests shall be taken at intervals not exceeding 2 feet in vertical rise and/or 1,000 cubic yards of compacted fill soil embankment. In addition, as a guideline, at least one (1) test shall be taken on slope faces for each 5,000 square feet of slope face and/or each 10 feet of vertical height of slope. The Contractor shall assure that fill construction is such that the testing schedule can be accomplished by the Geotechnical Consultant. The Contractor shall stop or slow down the earthwork construction if these minimum standards are not met.

4.7 *Compaction Test Locations*:

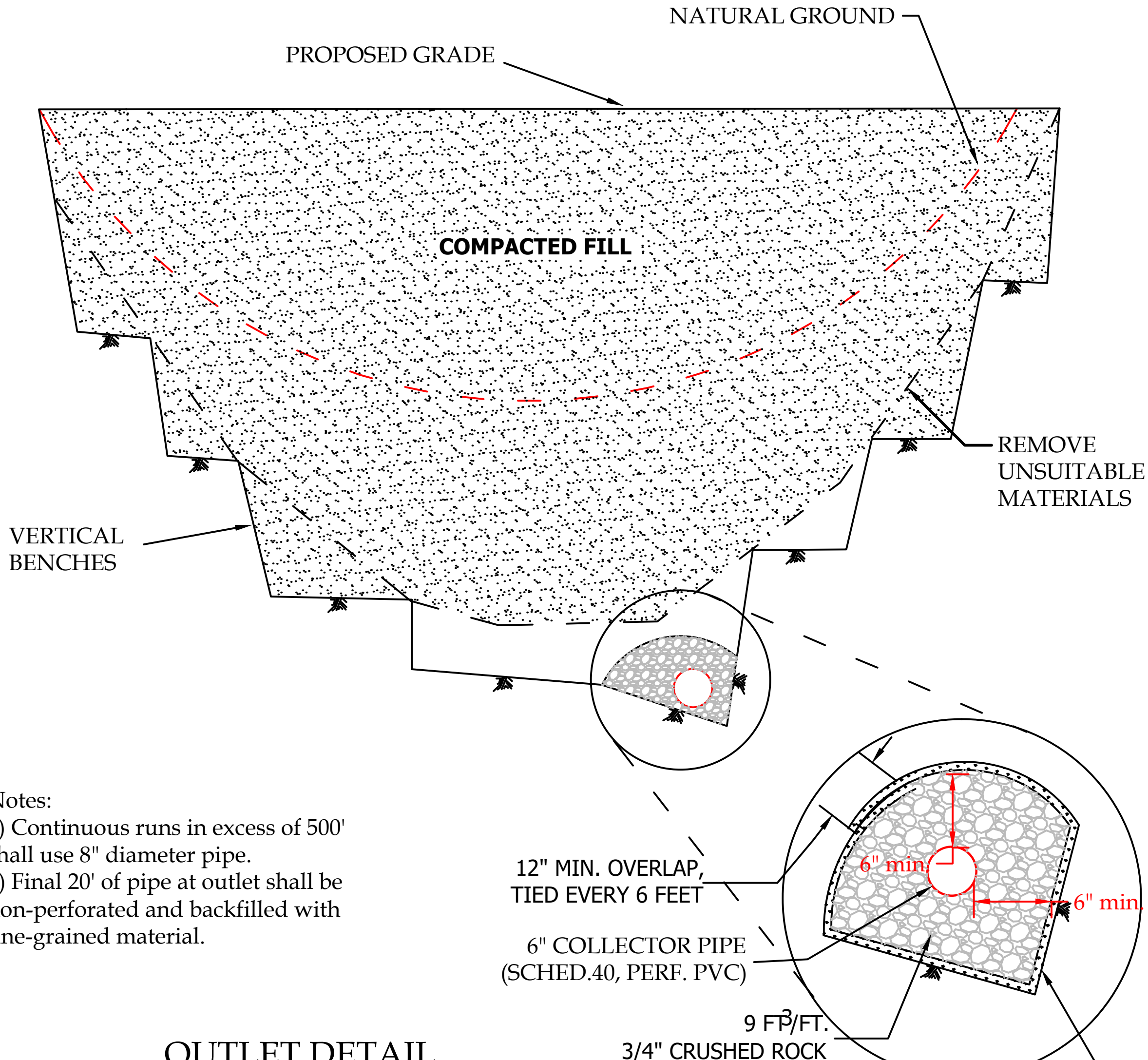
The Geotechnical Consultant shall document the approximate elevation and horizontal coordinates of each test location. The Contractor shall coordinate with the project surveyor to assure that sufficient grade stakes are established so that the Geotechnical Consultant can determine the test locations with sufficient accuracy. At a minimum, two (2) grade stakes within a horizontal distance of 100 feet and vertically less than 5 feet apart from potential test locations shall be provided.

5.0 *Subdrain Installation*

Subdrain systems shall be installed in accordance with the approved geotechnical report(s) and grading plan. The Geotechnical Consultant may recommend additional subdrain and/or changes in subdrain extent, location, grade, or material depending on conditions encountered during grading. All subdrains shall be surveyed by a land surveyor/civil engineer for line and grade after installation and prior to burial. Sufficient time should be allowed by the Contractor for these surveys.

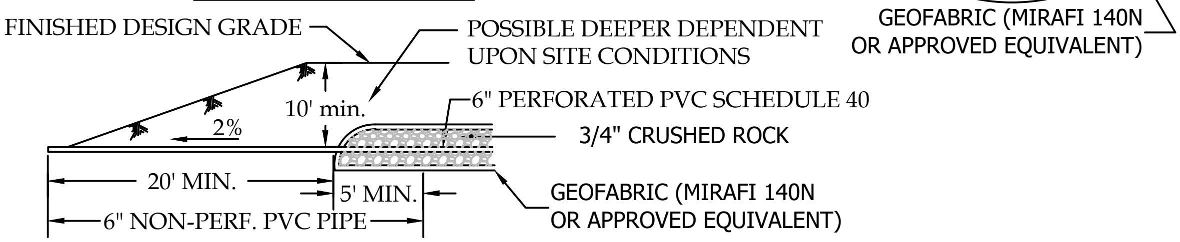


TYPICAL BUTTRESS/ STABILIZATION FILL DETAIL



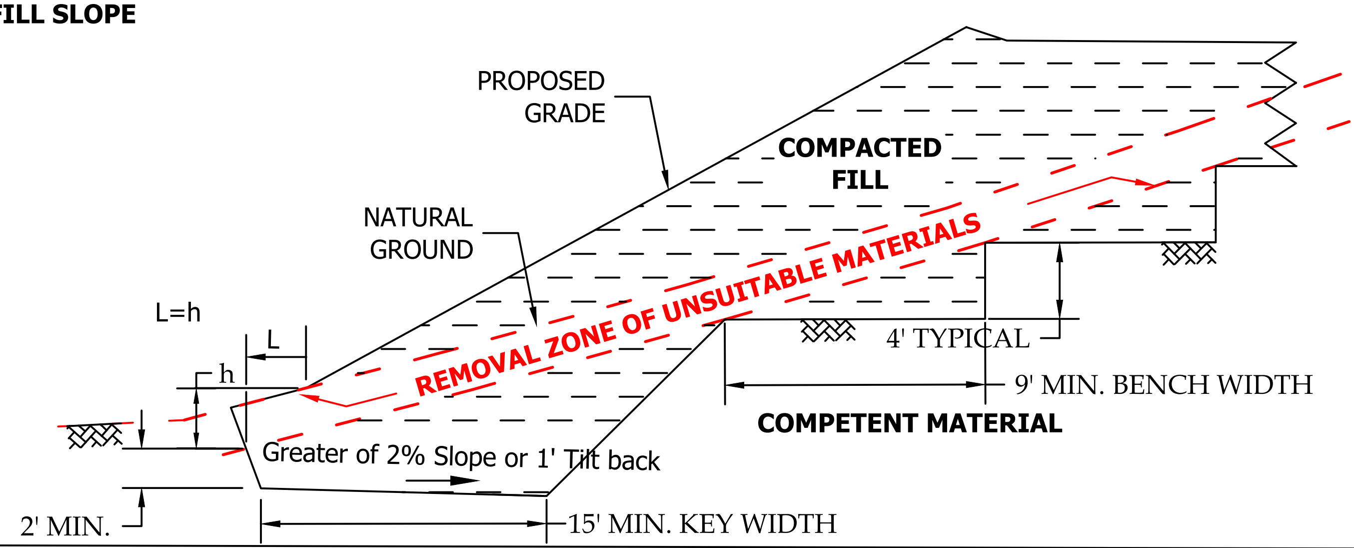
Notes:
1) Continuous runs in excess of 500' shall use 8" diameter pipe.
2) Final 20' of pipe at outlet shall be non-perforated and backfilled with fine-grained material.

OUTLET DETAIL

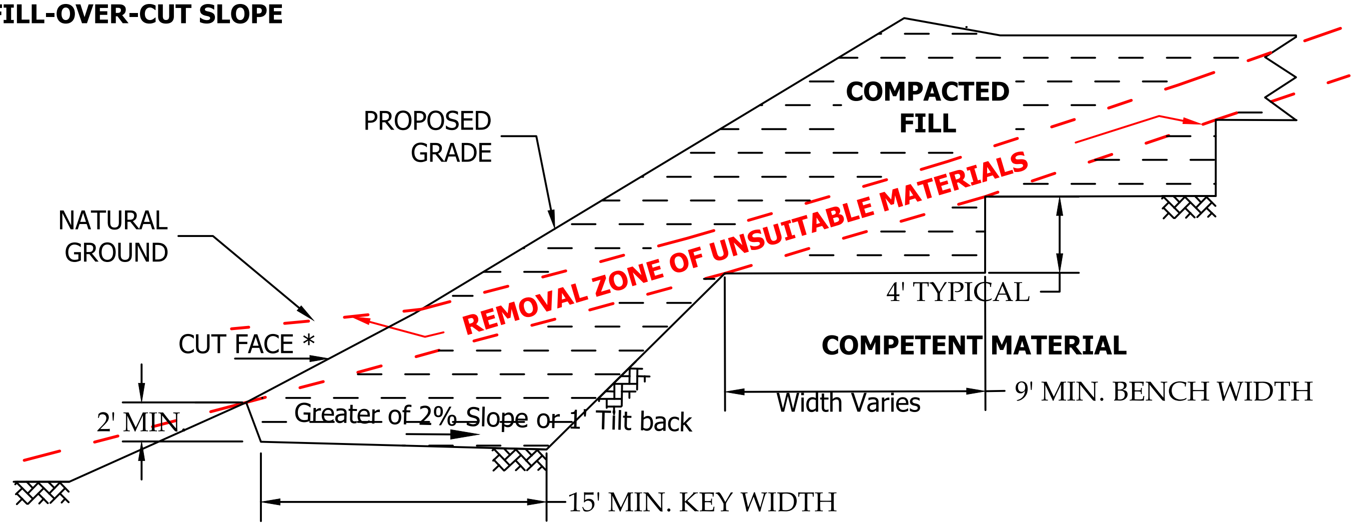


CANYON & STREET SUBDRAINS

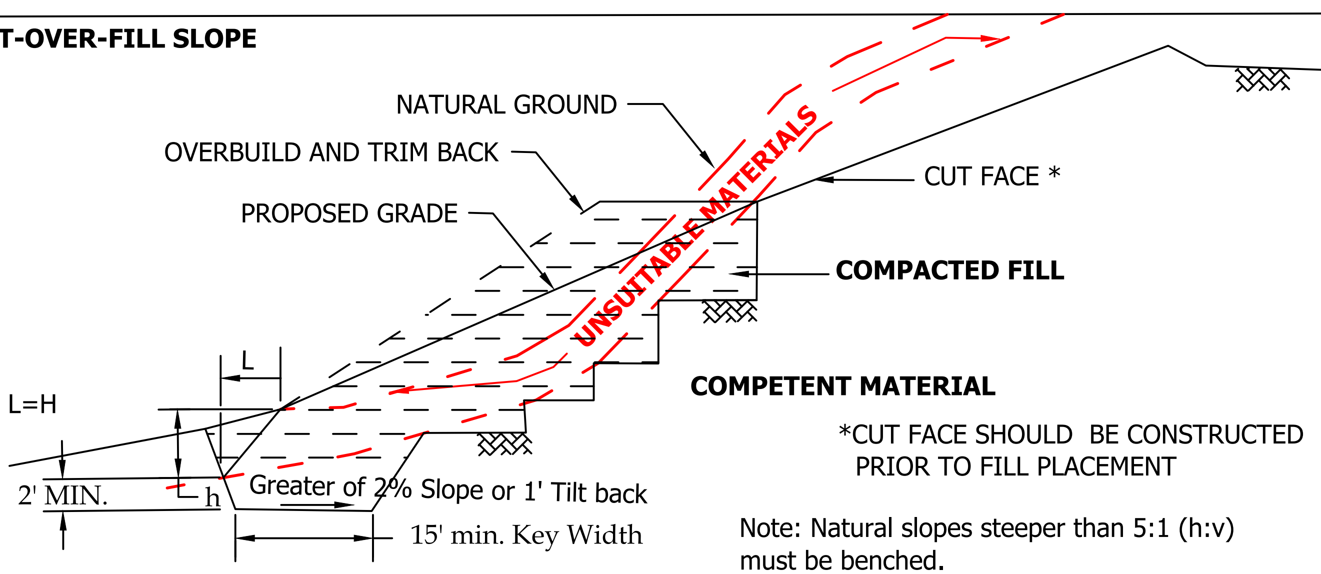
FILL SLOPE



FILL-OVER-CUT SLOPE



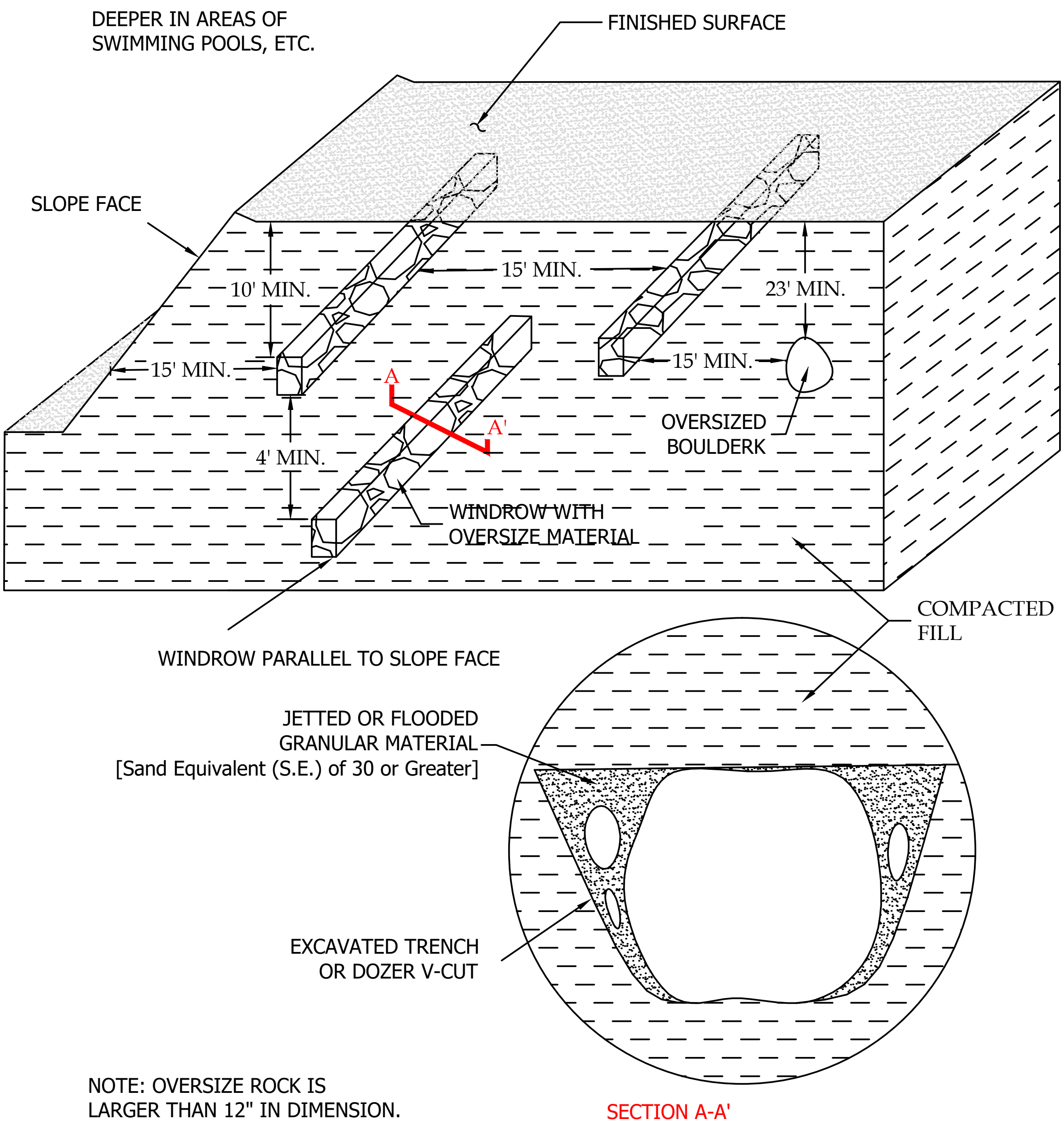
CUT-OVER-FILL SLOPE



*CUT FACE SHOULD BE CONSTRUCTED PRIOR TO FILL PLACEMENT

Note: Natural slopes steeper than 5:1 (h:v) must be benched.

KEYING AND BENCHING

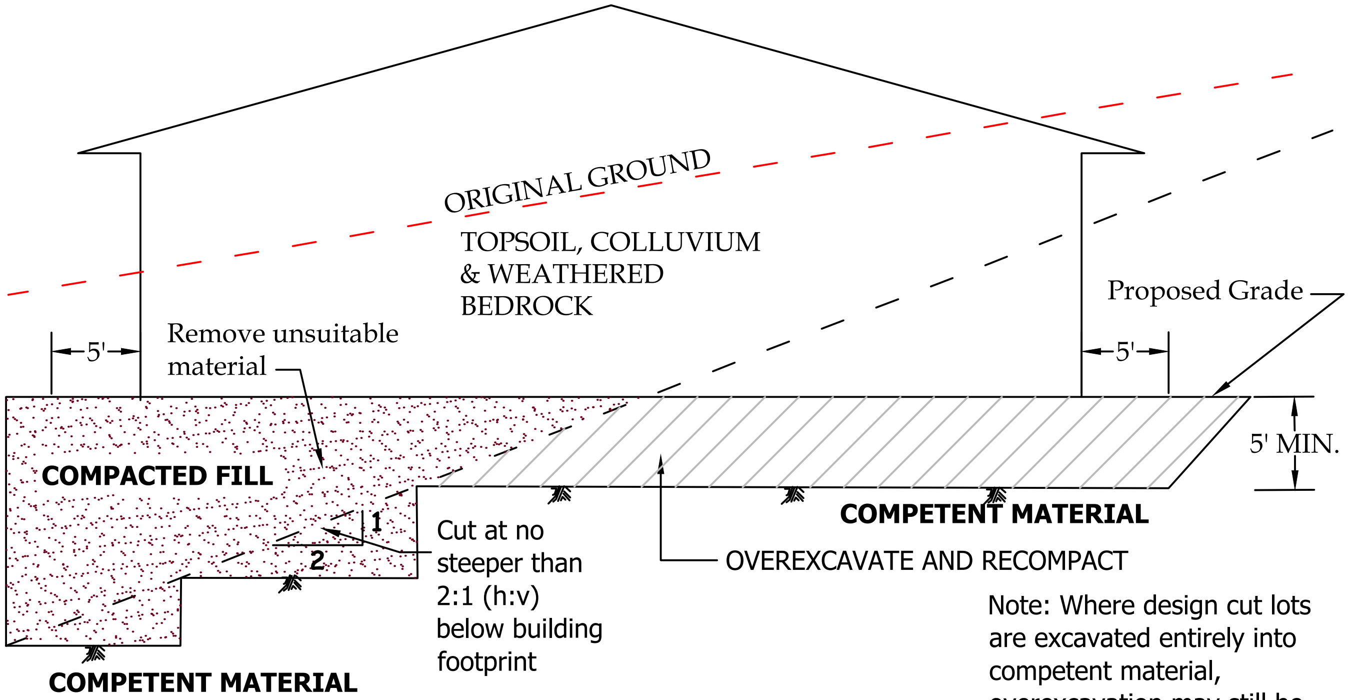


NOTE: OVERSIZE ROCK IS
LARGER THAN 12" IN DIMENSION.

OVERSIZE ROCK DISPOSAL

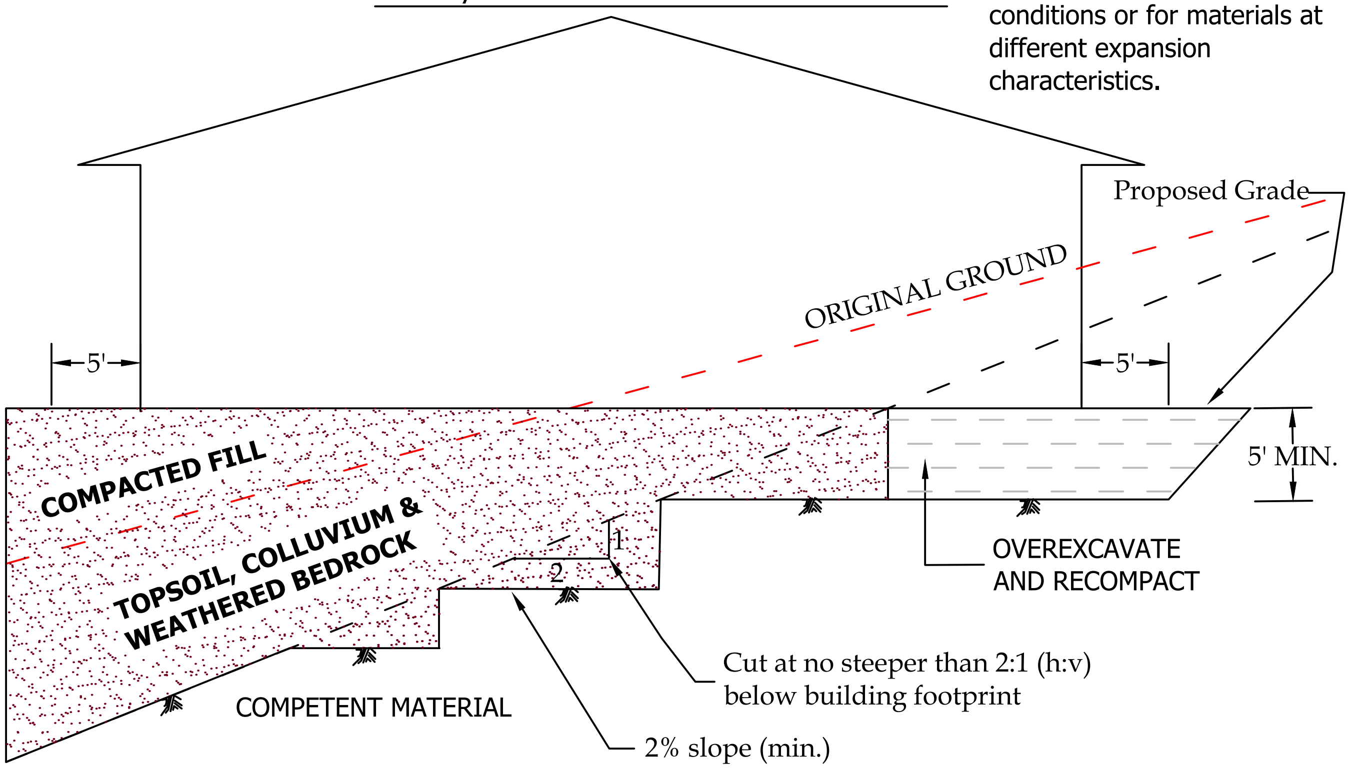
CUT LOT

(Exposing Unsuitable Soils/Bedrock @ Design Grade)

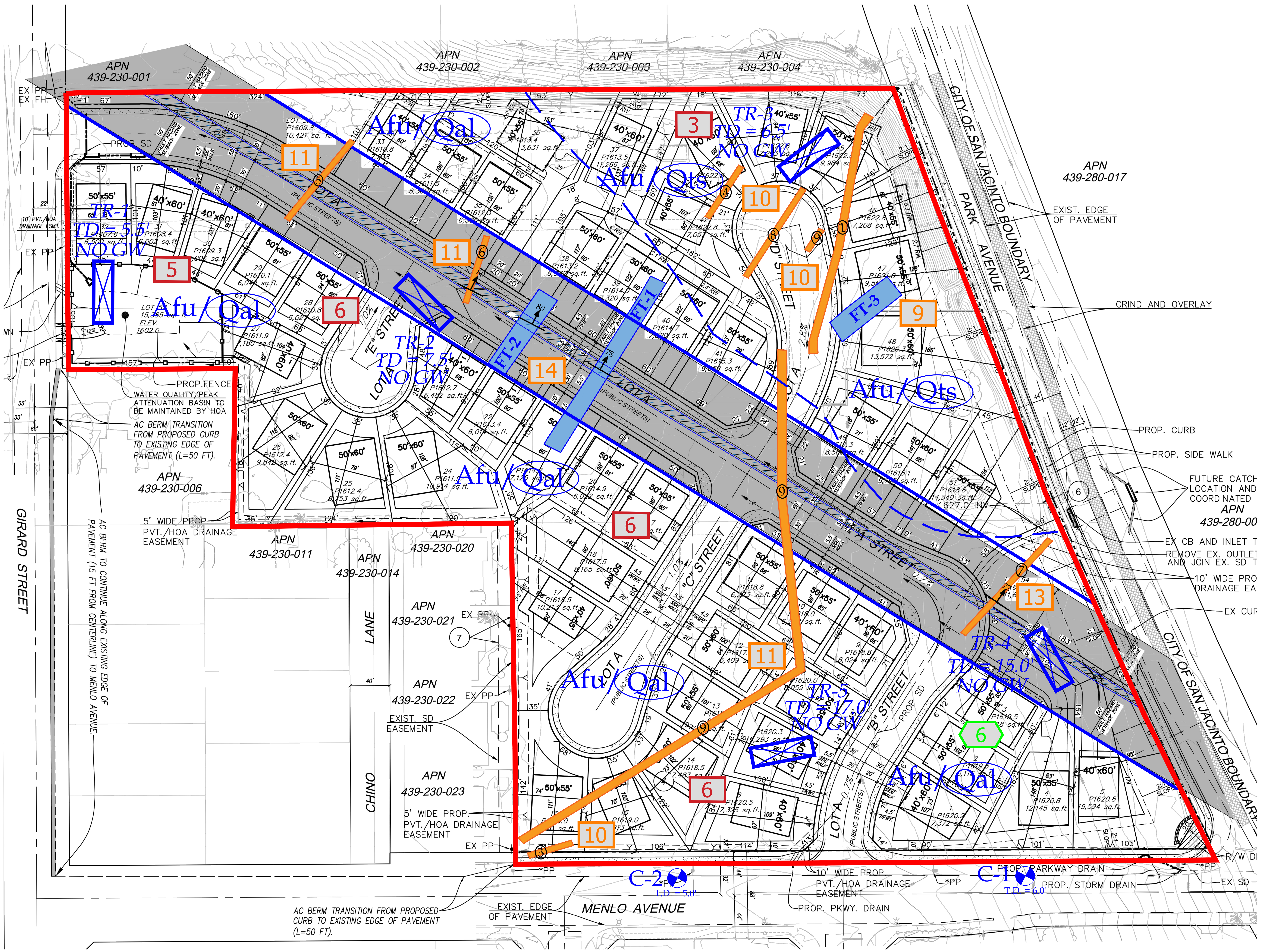


Note: Where design cut lots are excavated entirely into competent material, overexcavation may still be required for hard-rock conditions or for materials at different expansion characteristics.

CUT/FILL TRANSITION LOT



TRANSITION LOT OVEREXCAVATION DETAIL



LEGEND
(Locations are Approximate)

Geologic Earth Units

- Afu - Artificial Fill, Undocumented
- Qal - Alluvium (Circled Where Buried)
- Qts - Bautista Formation (Circled Where Buried)

Symbols

- Limits of This Report
- TR-5 TD=17.0' NO GW - Exploratory Trench Location
- C-2 T.D. = 5.0' NO GW - Exploratory Core Location
- Approximate Geologic Contact
- 6 - Approximate Depths of Overexcavation Below Existing Grade (ft.) in proposed fill areas
- 6 - Approximate Depths of Overexcavation Below Proposed Grade (ft.) in proposed cut areas
- 13 - Approximate Depths of Overexcavation Below Existing Grade (ft.) in fault trench areas
- FT-3 - Fault Trench Location (LGC, 2018)
- 9 - Fault Trench Location (Rasmussen, 1988)
- Limits of Structural Setback Zone Per (LGC, 2018)
- Fault Zone (Casa Loma Fault)
- 80 - Fault Attitude (LGC, 2018)

0 50' 100'

LOGICAL GEOTECHNICAL CONSULTANTS
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Mark Bergmann, Engineering Geologist
Robert Sargent, Project Engineer

GEOTECHNICAL MAP

800 North Girard Street
City of Hemet, County of Riverside, State of California

Name:	Sikand
Project No.	G18-1647-10
Client:	Sikand
Scale:	1" = 50'
Date:	February 2021
Reference:	Sikand Engineering, Vesting Tentative Tract 37558, Scale 1"=60' dated 9/9/1920
Plate No.	1 OF 1