

December 15, 2020

Project No. 20145-01

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**Subject: Preliminary Geotechnical Evaluation for Proposed Winery and Hotel Development,
41325 Calle Encantado, Temecula, California**

In accordance with your request, LGC Geotechnical, Inc. has performed a preliminary geotechnical evaluation for the proposed Winery and Hotel development located at 41325 Calle Encantado in the City of Temecula, California. The purpose of our study was to evaluate the site geotechnical conditions in the context of the preliminary site plans and to provide appropriate preliminary geotechnical design parameters and recommendations for the proposed development.

Should you have any questions regarding this report, please do not hesitate to contact our office. We appreciate this opportunity to be of service.

Sincerely,

LGC Geotechnical, Inc.



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ARN/RLD/amm

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1.0 INTRODUCTION

LGC Geotechnical has performed a preliminary geotechnical evaluation for the proposed Winery and Hotel development located at 41325 Calle Encantado in the City of Temecula, California (Figure 1). This report summarizes our findings, conclusions, and preliminary geotechnical design recommendations relative to the proposed development.

It should be noted that our evaluation and this report only address geotechnical issues associated with the site and do not address any environmental issues.

1.1 Project Description

The subject site is an approximately 20-acre, partially developed hillside property located at 41325 Calle Encantado in the City of Temecula, California. The site is bound to the north by Rancho California Road, to the east by agricultural fields, to the south by existing residential developments and to the west by agricultural fields. The site is currently utilized for agricultural purposes and contains two residential structures and an entry driveway. (Figure 1).

The site is located on a gentle hillside that generally descends to the north-northwest. Two existing, relatively level pads, descend in a tiered fashion to the north-northwest. Abandoned unpaved (dirt) roads run across the site toward the two pads, and along the southern property line. The site is currently accessed via Calle Encantado, which runs northwest/southeast through the site.

We understand that the proposed project will include the grading of one large building pad for the construction of a winery, hotel and associated improvements (4M Engineering, 2020). Proposed grades are expected to include design fills up to approximately 15 feet with the inclusion of a new fill slope on the northern side, and design cuts up to approximately 20 feet deep, with the inclusion of cut slopes along the southern portions of the site. The proposed building structures are anticipated to be light to medium-weight at-grade structures with maximum column and wall loads of approximately 50 kips and 5 kips per linear foot, respectively. Please note no structural loads were provided to us at the time of this report.

The preliminary recommendations given in this report are based upon the provided preliminary grading information and estimated structural loading and layout information provided above. We understand that the project plans are currently being developed at this time; LGC Geotechnical should be provided with updated project plans and any changes to grades and building layout when they become available, in order to either confirm or modify the recommendations provided herein. Additional field work and/or laboratory testing may be necessary.

1.2 Subsurface Evaluation

A geotechnical field evaluation was performed by LGC Geotechnical consisting of excavation of

seven hollow-stem auger borings. Borings HS-1 through HS-5 were excavated using a truck-mounted drill rig equipped with 6-inch-diameter hollow-stem augers to depths ranging from approximately 10 to 36 feet below existing grade. Borings I-1 and I-2 were drilled utilizing 8-inch-diameter hollow-stem augers to depths of approximately 15 feet and 5 feet, respectively, for field infiltration testing. An LGC Geotechnical representative observed the drilling operations, logged the borings, and collected soil samples for laboratory testing. Driven soil samples were collected by means of the Standard Penetration Test (SPT) and Modified California Drive (MCD) sampler. The SPT sampler (1.4-inch ID) and MCD sampler (2.4-inch ID, 3.0-inch OD) were driven using a 140-pound automatic hammer falling 30 inches to advance the sampler a total depth of 18 inches or until refusal. Bulk samples were also collected and logged for laboratory testing at select depths. The raw blow counts for each 6-inch increment of penetration were recorded on the boring logs. The borings were backfilled with cuttings. At the completion of infiltration testing of I-1 and I-2, the perforated pipe was removed, and the resulting void backfilled with the excavated soils.

The approximate locations of our subsurface explorations are provided on Figure 2. The boring logs and previous boring and test pit logs are provided in Appendix B. Elevations provided on the boring logs should be considered approximate.

1.3 Laboratory Testing

Representative bulk and driven (relatively undisturbed) samples were obtained for laboratory testing during our field evaluation. Laboratory testing included in-situ moisture content and in-situ dry density, fines content, Atterberg limits, expansion index, consolidation, direct shear, laboratory compaction and corrosion (sulfate, chloride, pH and minimum resistivity).

The following is a summary of the laboratory test results:

- Dry density of the samples collected ranged from approximately 93 pounds per cubic foot (pcf) to 126 pcf, with an average of 111 pcf. Field moisture contents ranged from approximately 2 to 28 percent, with an average of 11 percent.
- One fines content test was performed and indicated fines content (passing No. 200 sieve) of approximately 35 percent. Based on the Unified Soils Classification System (USCS), the tested sample is classified as “coarse-grained”.
- One Atterberg Limit (liquid limit and plastic limit) test was performed. Results indicated a Plasticity Index (PI) value of 28.
- Expansion potential testing indicated an expansion index value of 64, corresponding to “Medium” expansion potential.
- One consolidation test was performed. The load versus deformation plot is provided in Appendix C.
- One direct shear test was performed. The shear stress versus normal stress plot is provided in Appendix C.
- One laboratory compaction test of a near surface sample indicated a maximum dry density of 131.0 pcf with an optimum moisture content of 8.0 percent.
- Corrosion testing indicated soluble sulfate contents of less than 0.01 percent, a chloride content of 286 parts per million (ppm), pH of 8.62, and a minimum resistivity of 780 ohm-

centimeters.

A summary of the laboratory test results is presented in Appendix C. The moisture and dry density results are presented on the boring logs in Appendix B.

1.4 Field Infiltration Testing

Two falling head field infiltration tests (I-1 through I-2) were performed to approximate depths of 15 and 5 feet below existing grade, respectively. Estimation of infiltration rates was accomplished in general accordance with the guidelines set forth by the County of Riverside (2011). In general, a 3-inch diameter perforated PVC pipe was placed in the borehole, and the annulus was backfilled with gravel, including placement of approximately 2 inches of gravel at the bottom of the borehole. The infiltration wells were pre-soaked prior to testing. Based on the County of Riverside methodology, the calculated (observed) infiltration rates are provided in Table 1. These infiltration rates do not include any factor of safety (to be determined by the project Civil Engineer); however, they have been normalized to correct the 3-D flow that occurs within the field test to 1-D flow out of the bottom of the boring only. The locations and depths of the infiltration tests were coordinated with the civil engineer. The approximate infiltration test locations are shown on the Geotechnical Map (Figure 2) and the infiltration test data is included in Appendix D and summarized below.

TABLE 1

Summary of Infiltration Testing

Infiltration Test Location	Infiltration Test Depth Below Existing Grade (ft)	Observed Infiltration Rate* (Inch/Hr.)
I-1	15	4.8
I-2	5	0.6

*Normalized to One-Dimensional Flow, does not include any Factor of Safety

It should be emphasized that infiltration test results are only representative of the location and depth where they are performed. Varying subsurface conditions may exist outside of the test locations which could alter the calculated infiltration rates indicated above. Infiltration tests are performed using relatively clean water free of particulates, silt, etc. Refer to Section 4.9 for subsurface water infiltration recommendations.

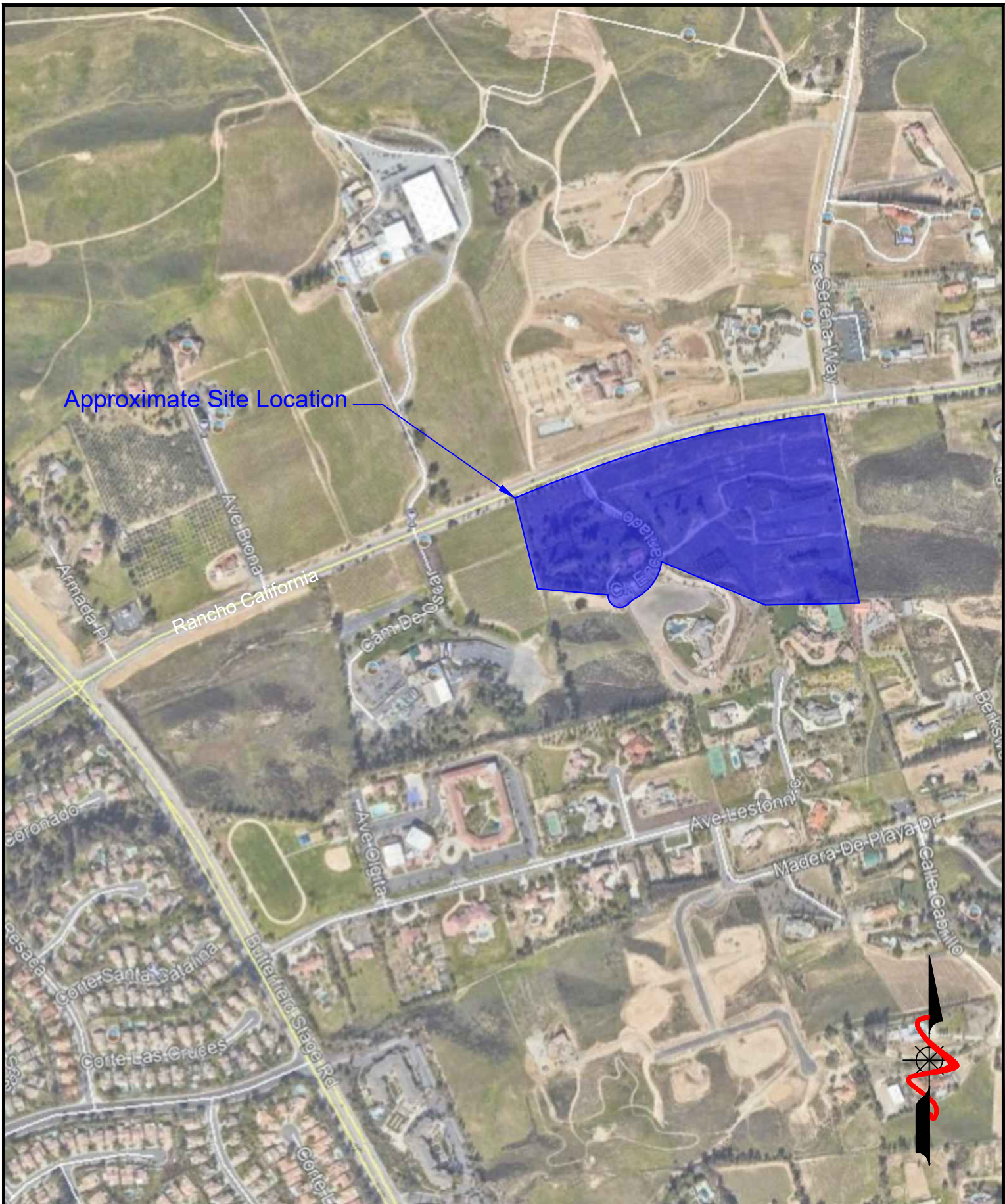


FIGURE 1
Site Location Map

PROJECT NAME	EPD - Temecula
PROJECT NO.	20145-01
ENG. / GEOL.	RLD/ARN
SCALE	Not to Scale
DATE	December 2020

2.0 GEOTECHNICAL CONDITIONS

2.1 Regional Geology

The property is regionally located in the Peninsular Ranges geomorphic province which extends from the Los Angeles Basin south to Baja California. The province is characterized by numerous southwest trending mountain ranges and valleys that are geologically controlled by a series of paralleling major active faults. More specifically, the site is located in the southern portion of the Perris block which is bordered to the northeast by the San Jacinto Fault Zone and to the southwest by the Elsinore Fault Zone. The region is underlain by young sedimentary units that have eroded into gentle badlands topography (CGS, 2003). The site is situated along a gently sloping property between two geomorphic features locally referred to as Long Canyon and Long Valley.

2.2 Site-Specific Geology

Based on the Bachelor Mountain 7.5 Quadrangle Geologic Map (CGS, 2003) the subject site is predominately underlain by sands, silts and clay of the Pleistocene Pauba Formation. Geologic revisions were incorporated during the updated Seismic Hazard Zone report (CGS, 2018b), which identified additional zones of Young Alluvial deposits. Quaternary Young Alluvium is mapped onsite in the lower elevations of the project roughly parallel to Rancho California Rd. The typical onsite characteristics of the materials are described in the following subsections (from youngest to oldest). The approximate lateral extent of the geologic units encountered is presented on the Geotechnical Map (Sheet 1). The topographic base utilized for our Geotechnical Map was provided by 4M Engineering and Development, inc. (2020).

A brief description of the encountered materials is presented in the following section. Descriptions of the subsurface conditions are presented on the exploratory excavation logs presented in Appendix B.

2.2.1 Quaternary Young Alluvial Channel Deposits (Map Symbol – Qya)

Quaternary young channel deposits are mapped at lower elevations of the site nearing gentle valley bottom along Rancho California Road. These deposits consist of unconsolidated sand, silt, and clay-bearing alluvium.

2.2.2 Pleistocene Pauba Formation – Sandstone Member (Map Symbol – Qpfs)

The Sandstone Member of the Pleistocene Pauba Formation is exposed on the majority of slopes within the site and underlies the entire site at depth. The Pauba Formation consist of brown, moderately well-indurated, cross bedded sandstone containing sparse cobble- to boulder-conglomerate beds. As encountered, these materials generally consist of medium to coarse-grained sands, with variable amounts of clayey sands and

silty sands, and occasional clay and sandy clay beds. The material is generally brown and gray in color.

2.3 Geologic Structure

Onsite material is observed to be massive to thickly bedded, with no apparent dip direction. Few regional attitudes have been taken within the formation (CGS, 2003). Refer to the Geotechnical Map (Figure 2) and Boring Logs (Appendix B) for details.

2.4 Landslides

Document research and field observations of the surficial conditions do not indicate the presence of landslides on the site or in the immediate vicinity. Review of the Seismic Hazards Zone Map (CGS, 2018c) for the Bachelor Mountain 7.5 Minute Quadrangle does not indicate the site to be within a mapped area considered susceptible to seismically induced slope instability. As outlined in the CGS Seismic Hazards Report, landsliding has occurred on a similar slope less than ½ mile to the east. The City of Temecula's general plan cites that potential landslide conditions exist in this area where slopes are greater than 15 percent (Public Safety Element, 2020). These maps were prepared by the State to raise awareness of the potential for such hazards and to prompt appropriate investigation to evaluate these potentials on a site-by-site basis.

Slope stability analysis of the anticipated design cut, and fill slopes was not performed as part of this entitlement level scope. Based on our preliminary evaluation and knowledge of the site and regional soil conditions, future proposed cut and fill slopes are anticipated to be generally stable following implementation of earthwork recommendations (current and future). Replacement slopes and/or buttress fills may be required based upon further evaluation.

LGC Geotechnical should perform a slope stability evaluation once grading plans have been made available. Additional field work and/or laboratory testing may be necessary.

2.5 Groundwater

Groundwater was not encountered in our geotechnical boring to the maximum depth excavated of approximately 35 feet below existing grade. Historic high groundwater has not been evaluated by the CGS in the vicinity of the site (CGS, 2018b).

Groundwater and/or groundwater seepage conditions may occur in the future due to changes in land use and/or following periods of heavy rain. Seasonal fluctuations of groundwater elevations should be expected over time. In general, groundwater levels fluctuate with the seasons and local zones of perched groundwater may be present within the near-surface deposits due to local landscape and vineyard irrigation or precipitation especially during rainy seasons.

2.6 **Seismicity and Faulting**

California is located on the boundary between the Pacific and North American Lithospheric Plates. The average motion along this boundary is on the order of 50-mm/yr. in a right-lateral sense. The majority of the motion is expressed at the surface along the northwest trending San Andreas Fault Zone with lesser amounts of motion accommodated by sub-parallel faults located predominantly west of the San Andreas including the Elsinore, Newport-Inglewood, Rose Canyon, and Coronado Bank Faults. Within Southern California, a large bend in the San Andreas Fault north of the San Gabriel Mountains has resulted in a transfer of a portion of the right-lateral motion between the plates into left-lateral displacement and vertical uplift. Compression south and west of the bend has resulted in folding, left-lateral, reverse thrust faulting, and regional uplift creating the east-west trending Transverse Ranges and several east-west trending faults. Further south within the Los Angeles Basin, “blind thrust” faults are believed to have developed below the surface also as a result of this compression, which have resulted in earthquakes such as the 1994 Northridge event along faults with little to no surface expression.

Prompted by damaging earthquakes in Northern and Southern California, State legislation and policies concerning the classification and land-use criteria associated with faults have been developed. The Alquist-Priolo Earthquake Fault Zoning Act was implemented in 1972 to prevent the construction of urban developments across the trace of active faults. California Geologic Survey Special Publication 42 was created to provide guidance for following and implementing the law requirements. Special Publication 42 was most recently revised in 2018 (CGS, 2018a). According to the State Geologist, an “active” fault is defined as one which has had surface displacement within Holocene time (roughly the last 11,700 years). Regulatory Earthquake Fault Zones have been delineated to encompass traces of known, Holocene-active faults to address hazards associated with surface fault rupture within California. Where developments for human occupation are proposed within these zones, the state requires detailed fault evaluations be performed so that engineering-geologists can identify the locations of active faults and recommend setbacks from locations of possible surface fault rupture.

No indication of active faulting was observed during our evaluation. The site is not located within a mapped State of California Earthquake Fault-Rupture Hazard Zone per compiled maps released by the CGS (2018a), and no known active or potentially active faults cross the site. The possibility of damage due to ground rupture is considered low since no active faults are known to cross the site.

No faults have been depicted as running through the site on the regional geologic map for the area (CGS, 2003 & 2018a).

Secondary effects of seismic shaking resulting from large earthquakes on the major faults in the Southern California region, which may affect the site, include ground lurching, and shallow ground rupture, soil liquefaction, and dynamic settlement. 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 major active nearby fault that could produce these secondary effects includes the Elsinore Fault Zone – with a branch known locally as the Wildomar fault section located approximately 4 miles west of the site. A discussion of these secondary effects and their potential impact on the site is provided in

the following sections.

2.6.1 Liquefaction and Dynamic Settlement

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 coexist: 1) shallow groundwater; 2) low density non-cohesive (granular) soils; and 3) high-intensity ground motion. Studies indicate that saturated, loose near-surface cohesionless soils exhibit the highest liquefaction potential, while dry, dense, cohesionless soils and cohesive soils exhibit low to negligible liquefaction potential. In general, cohesive soils are not considered susceptible to liquefaction, depending on their plasticity and moisture content (Bray & Sancio, 2006). Effects of liquefaction on level ground include settlement, sand boils, and bearing capacity failures below structures. Dynamic settlement of dry loose sands can occur as the sand particles tend to settle and densify as a result of a seismic event.

The northern strip of the site, closest to Rancho California road is located within a State of California Seismic Hazard Zone (CGS, 2018) for liquefaction potential. However, this liquefaction potential zone is mapped at the lower elevations of the site closest to Rancho California Road, which are outside of the proposed limits of grading and development. Additionally, according to the Riverside County Online Liquefaction Map (2018), the site is not located within an area mapped as having liquefaction susceptibility.

The post development site conditions will consist of compacted fill overlying dense/hard soil and formational materials, and these soils are not considered susceptible to liquefaction. The potential for liquefaction is considered very low.

2.6.2 Lateral Spreading

Lateral spreading is a type of liquefaction-induced ground failure associated with the lateral displacement of surficial blocks of sediment resulting from liquefaction in a subsurface layer. Once liquefaction transforms the subsurface layer into a fluid mass, gravity plus the earthquake inertial forces may cause the mass to move downslope towards a free face (such as a river channel or an embankment). Lateral spreading may cause large horizontal displacements and such movement typically damages pipelines, utilities, bridges, and structures.

Due to very low potential for liquefaction the potential for lateral spreading is also considered very low.

2.7 Seismic Design Parameters

The site seismic characteristics were evaluated per the guidelines set forth in Chapter 16, Section 1613 of the 2019 California Building Code (CBC) and applicable portions of ASCE 7-16

which has been adopted by the CBC. **Please note that the following seismic parameters are only applicable for code-based acceleration response spectra and are not applicable for where site-specific ground motion procedures are required by ASCE 7-16.** Representative site coordinates of latitude 33.5209 degrees (north) and longitude -117.0872 degrees (west) were utilized in our analyses. The maximum considered earthquake (MCE) spectral response accelerations (S_{MS} and S_{M1}) and adjusted design spectral response acceleration parameters (S_{DS} and S_{D1}) for Site Class C are provided in Table 2. The structural designer should contact the geotechnical consultant if structural conditions (e.g., number of stories, seismically isolated structures, etc.) require site-specific ground motions.

TABLE 2

Seismic Design Parameters

Selected Parameters from 2019 CBC, Section 1613 - Earthquake Loads	Seismic Design Values	Notes/Exceptions
Distance to applicable faults classifies the site as a "Near-Fault" site.		Section 11.4.1 of ASCE 7
Site Class	C	Chapter 20 of ASCE 7
S_s (Risk-Targeted Spectral Acceleration for Short Periods)	1.472g	From SEAOC, 2020
S_1 (Risk-Targeted Spectral Accelerations for 1-Second Periods)	0.545g	From SEAOC, 2020
F_a (per Table 1613.2.3(1))	1.200	For Simplified Design Procedure of Section 12.14 of ASCE 7, F_a shall be taken as 1.4 (Section 12.14.8.1)
F_v (per Table 1613.2.3(2))	1.455	-
S_{MS} for Site Class C [Note: $S_{MS} = F_a S_s$]	1.766g	-
S_{M1} for Site Class C [Note: $S_{M1} = F_v S_1$]	0.793g	-
S_{DS} for Site Class C [Note: $S_{DS} = (2/3) S_{MS}$]	1.177g	-
S_{D1} for Site Class C [Note: $S_{D1} = (2/3) S_{M1}$]	0.529g	-
C_{RS} (Mapped Risk Coefficient at 0.2 sec)	0.903	ASCE 7 Chapter 22
C_{R1} (Mapped Risk Coefficient at 1 sec)	0.901	ASCE 7 Chapter 22

A deaggregation of the PGA based on a 2,475-year average return period (MCE) indicates that an earthquake magnitude of 7.71 at a distance of approximately 5.6 km from the site would contribute the most to this ground motion (USGS, 2014).

Section 1803.5.12 of the 2019 CBC (per Section 11.8.3 of ASCE 7) states that the maximum considered earthquake geometric mean (MCE_G) Peak Ground Acceleration (PGA) should be used for liquefaction potential. The PGA_M for the site is equal to 0.777g (SEAOC, 2020).

2.8 Rippability

In general, excavation for grading and removals should be achievable with heavy duty construction equipment in good working order. Lighter equipment for use in foundation and utility excavation, may encounter difficult excavation where oversize material is encountered (see below for further discussion).

2.9 Oversized Material

Oversized material (material greater than 8 inches in maximum dimension) may be encountered during site grading. Recommendations are provided for appropriate handling of oversized materials in Appendix E.

2.10 Expansion Potential

Based on the results of our laboratory testing, fills at the completion of grading are anticipated to have a “medium” expansion potential. Final expansion potential of site soils should be determined at the completion of grading. Results of expansion testing at finish grades will be utilized to confirm final foundation design.

3.0 CONCLUSIONS

Based on the results of our subsurface evaluation and geotechnical review of the proposed plan, it is our opinion that the proposed grading of the site is considered feasible from a geotechnical standpoint, provided that the recommendations provided here and in future reports are incorporated during site grading and development. A summary of our geotechnical conclusions are as follows:

- The bedrock geologic unit mapped on the site is the Sandstone Member of the Pauba Formation. Shallow young alluvial channel deposits are mapped along the northern portions of the site at lower elevations roughly parallel to Rancho California. Young alluvial channel deposits were not encountered in our borings and are not anticipated within the limits of grading.
- The existing site contains remnants of a nursery. There are a significant number of palm trees onsite in the vicinity of the proposed grading that will need to be removed prior to or during grading. These trees cannot be incorporated into the fill.
- Groundwater was not encountered in the area of proposed development to a maximum explored depth of 36 feet below existing grade and is not considered a constraint for the proposed development.
- The subject site is not located within a State of California Earthquake Fault Zone (Alquist-Priolo). The main seismic hazard that may affect the site is ground shaking from one of the active regional faults. The subject site will likely experience strong seismic ground shaking during its design life.
- Based on our review of the State of California Seismic Hazard Zones Bachelor Mountain 7.5 Minute Quadrangle, a portion of the site is located within a zone having a potential for liquefaction. However, this liquefaction potential zone is mapped at the lower elevations of the site closest to Rancho California Road, which are outside of the proposed limits of grading and development. The post development site conditions will consist of compacted fill overlying dense/hard soil and formational materials, and these soils are not considered susceptible to liquefaction. The potential for liquefaction is considered to be very low.
- Based on our preliminary evaluation and knowledge of the site and regional soil conditions, future proposed cut and fill slopes are anticipated to be generally stable following implementation of earthwork recommendations (current and future). Replacement slopes and/or buttress fills may be required based upon further evaluation. LGC Geotechnical should perform a slope stability evaluation once grading plans have been made available. Additional field work and/or laboratory testing may be necessary.
- In general, excavation for grading and removals should be achievable with heavy duty construction equipment in good working order. Lighter equipment for use in foundation and utility excavation, may encounter difficult excavation where oversize material is encountered.
- Oversized material (material greater than 8 inches in maximum dimension) may be encountered during site grading. Recommendations are provided for appropriate handling of oversized materials in Appendix E.
- Based on the results of our preliminary laboratory testing, site soils are anticipated to have "Medium" expansion potential. Mitigation measures will be required for any planned foundations and site improvements such as concrete flatwork to minimize the impacts of expansive soils.

- Pre-soaking of the subgrade for building slabs and site flatwork is recommended due to site expansive soils. The duration of this process varies greatly based on the chosen method and is also dependent on factors such as soil type and weather conditions. Time duration for presoaking should be accounted for in the construction schedule. (Typically, approximately one to two weeks for medium expansion). Additional time at the completion of presoaking may be necessary for the surface soils of the pad to dry back sufficiently to be capable of supporting trenching equipment.
- From a geotechnical perspective, the existing onsite soils are considered suitable material for use as general fill, provided that they are relatively free from oversized material (larger than 8 inches in maximum dimension), construction debris, and significant organic material. Moisture conditioning will be required to obtain the required compaction.
- The site contains soils that are not suitable for retaining wall backfill due to their fines content and expansion potential, therefore import of sandy soils will be required by the contractor for obtaining suitable backfill soil for planned site retaining walls.
- Based on the results of our evaluation and analysis provided herein, and provided our recommendations are properly implemented during construction, the proposed development of the site is not anticipated to significantly impact adjacent properties.

4.0 RECOMMENDATIONS

The following recommendations are to be considered preliminary and should be confirmed upon completion of grading and earthwork operations. In addition, they should be considered minimal from a geotechnical viewpoint, as there may be more restrictive requirements from the architect, structural engineer, building codes, governing agencies, or the owner.

It should be noted that the following geotechnical recommendations are intended to provide sufficient information to develop the site in general accordance with the 2019 CBC requirements. With regard to the potential occurrence of potentially catastrophic geotechnical hazards such as fault rupture, earthquake-induced landslides, liquefaction, etc. the following geotechnical recommendations should provide adequate protection for the proposed development to the extent required to reduce seismic risk to an “acceptable level.” The “acceptable level” of risk is defined by the California Code of Regulations as “that level that provides reasonable protection of the public safety, though it does not necessarily ensure continued structural integrity and functionality of the project” [Section 3721(a)]. Therefore, repair and remedial work of the proposed improvements may be required after a significant seismic event. With regards to the potential for less significant geologic hazards to the proposed development, the recommendations contained herein are intended as a reasonable protection against the potential damaging effects of geotechnical phenomena such as expansive soils, fill settlement, groundwater seepage, etc. It should be understood, however, that although our recommendations are intended to maintain the structural integrity of the proposed development and structures given the site geotechnical conditions, they cannot preclude the potential for some cosmetic distress or nuisance issues to develop as a result of the site geotechnical conditions.

The geotechnical recommendations contained herein must be confirmed to be suitable or modified based on the actual as-graded conditions.

4.1 Site Earthwork

We anticipate that earthwork at the site will consist of site preparation, removal of existing vegetation, remedial grading, excavation of the stability fill keys, construction of subdrains, and fill placement.

We recommend that earthwork onsite be performed in accordance with the following recommendations, future grading plan review report(s), the 2019 CBC, City of Temecula/County of Riverside grading requirements, and the General Earthwork and Grading Specifications for Rough Grading included in Appendix E. In case of conflict, the following recommendations shall supersede all previous recommendations and those included as part of Appendix E. The following recommendations should be considered preliminary and may be revised based upon future evaluation and review of the project plans and/or based on the actual conditions encountered during site grading and construction.

4.1.1 Site Preparation

Prior to grading of areas to receive structural fill or engineered improvements, the areas should be cleared of existing building structures, asphalt, surface obstructions, and demolition debris. Vegetation and debris should be removed and properly disposed of off-site. Holes resulting from the removal of buried obstructions, which extend below proposed finish grades, should be replaced with suitable compacted fill material. Any abandoned sewer or storm drain lines should be completely removed and replaced with properly placed compacted fill. Deeper demolition may be required in order to remove existing foundations. We recommend the trenches associated with demolition which extend below the remedial grading depth be backfilled and properly compacted prior to the demolition contractor leaving the site.

If cesspools or septic systems are encountered, they should be removed in their entirety. The resulting excavation should be backfilled with properly compacted fill soils. As an alternative, cesspools can be backfilled with lean sand-cement slurry. Any encountered wells should be properly abandoned in accordance with regulatory requirements. At the conclusion of the clearing operations, a representative of LGC Geotechnical should observe and accept the site prior to further grading.

4.1.2 Removal Depths and Limits and Over-Excavation of Pads

In order to provide a relatively uniform bearing condition for the planned improvements, the upper loose/compressible soils are to be removed and replaced as properly compacted fills.

For design fill areas, site remedial grading should include removal of unsuitable materials (topsoil/colluvium and weathered bedrock material) to suitable materials. Approximate anticipated removal depths are estimated to be approximately 5 feet below existing grades and are presented on the Geotechnical Map (Figure 2). For design cut areas, over-excavation should be performed as outlined below to provide the recommended thickness of compacted fill below proposed improvements.

Building Structures: Where not achieved by planned grading or remedial grading, we recommend that the proposed structures be over-excavated to a minimum depth of 5 feet below existing grade or 3-foot below proposed footings, whichever is deeper. In general, the envelope for over-excavation should extend laterally a minimum horizontal distance of 5 feet beyond the edges of the proposed structures. The excavation bottom should be observed by LGC Geotechnical to confirm the anticipated conditions. If the conditions differ from those expected additional remedial measures may be required.

Bedrock/Fill Transitions: Structural footings/mat foundations should not be placed on a bedrock and fill transition. In order to provide a uniform bearing, the bedrock portion of the bedrock/fill transition should be over-excavated a minimum of 3 feet below the bottom of proposed footings/mat and extend 3 feet laterally beyond the edges of the proposed footings/mat foundations. A maximum 3:1 differential fill thickness, up to a maximum over-excavation depth of 10 feet, underneath the proposed building structures

should be maintained in order to reduce the potential for future differential settlement.

Retaining/Free-Standing Wall Structures: Where not achieved by planned grading or remedial grading, we recommend planned retaining walls be over-excavated so that they are underlain by at least 2 feet of compacted fill below proposed footings. Where practical, the envelope for over-excavation should extend laterally a minimum lateral distance of 2 feet beyond the edges of the proposed footings.

Pavement and Hardscape Areas: Where not achieved by planned grading or remedial grading, we recommend that proposed pavement and hardscape areas be over-excavated so that they are underlain by at least 2-feet of compacted fill below the proposed finished subgrade (i.e., below planned aggregate base/asphalt concrete). Where practical, the envelope for pavement and hardscape over-excavation should extend laterally a minimum lateral distance of 2-feet beyond the edges of the proposed improvements.

Local conditions may be encountered during excavation that could require additional over-excavation beyond the above-noted minimum in order to obtain an acceptable subgrade. The actual depths and lateral extents of grading will be determined by the geotechnical consultant, based on subsurface conditions encountered during grading. The excavation bottoms should be observed by LGC Geotechnical to confirm the anticipated conditions. If the conditions differ from those expected additional remedial measures may be required. Removal areas should be accurately staked in the field by the Project Surveyor.

Over-excavations/undercuts must be confirmed and mapped by the geotechnical consultant prior to subsequent fill placement. The actual depth and lateral extents of over-excavation should be determined by the geotechnical consultant, based on subsurface conditions encountered during grading. Over-excavation areas should be accurately staked in the field by the Project Surveyor.

4.1.3 Temporary Excavations

Temporary excavations should be performed in accordance with project plans, specifications, and all Occupational Safety and Health Administration (OSHA) requirements. Excavations should be laid back or shored in accordance with OSHA requirements before personnel or equipment are allowed to enter. Based on our field investigation, the majority of site soils are anticipated to be OSHA Type "B" soils (refer to the attached boring logs). Sandy soils are present and should be considered susceptible to caving and raveling. Soil conditions should be regularly evaluated during construction to verify conditions are as anticipated. The contractor shall be responsible for providing the "competent person" required by OSHA standards to evaluate soil conditions. Close coordination with the geotechnical consultant should be maintained to facilitate construction while providing safe excavations. Excavation safety is the sole responsibility of the contractor.

Vehicular traffic, stockpiles, and equipment storage should be set back from the perimeter of excavations a minimum distance equivalent to a 1:1 (horizontal to vertical) projection

from the bottom of the excavation or 5 feet, whichever is greater, unless the cut is shored and designed for applicable surcharge load. Once an excavation has been initiated, it should be backfilled as soon as practical. Prolonged exposure of temporary excavations may result in some localized instability. Excavations should be planned so that they are not initiated without sufficient time to shore/fill them prior to weekends, holidays, or forecasted rain.

It should be noted that any excavation that extends below a 1:1 (horizontal to vertical) projection of an existing foundation will remove existing support of the structure foundation. If requested, temporary shoring parameters will be provided.

4.1.4 Subgrade Preparation

Removal bottoms, over-excavation bottoms and areas to receive fill should be observed and accepted by LGC Geotechnical prior to subsequent fill placement. Areas approved for fill placement should be scarified to a minimum depth of 6 to 8 inches, brought to a near-optimum moisture condition, and re-compacted in place per the project requirements.

4.1.5 Material for Fill

From a geotechnical perspective, the onsite soils are generally considered suitable for use as general compacted fill, provided they are relatively free of organic materials and construction debris. Any encountered oversized material (material larger than 8 inches in maximum dimension) must be appropriately handled as outlined in Appendix E.

From a geotechnical viewpoint, any required import soils for general fill (i.e., non-retaining wall backfill) should consist of clean, relatively granular soils of “Very Low” to “Medium” expansion potential (expansion index 90 or less based on American Society for Testing and Materials [ASTM] D 4829), and free of organic materials, construction debris and any material greater than 3 inches in maximum dimension. Import for any required retaining wall backfill should meet the criteria outlined in the following paragraph. Source samples should be provided to the geotechnical consultant for laboratory testing a minimum of 3 working days prior to any planned importation.

Conventional (masonry) retaining wall backfill should consist of sandy soils with a maximum of 35 percent fines (passing the No. 200 sieve) per American Society for Testing and Materials (ASTM) Test Method D1140 (or ASTM D6913/D422) and a “Very Low” expansion potential (EI of 20 or less per ASTM D4829). Soils should also be screened of organic materials, construction debris, and any material greater than 3 inches in maximum dimension. The site soils are not suitable for retaining wall backfill due to their fines content (i.e., silt and clay content) and expansion potential. Therefore, import of select sandy soils meeting the criteria outlined above will be required by the contractor for obtaining suitable retaining wall backfill soil. These preliminary findings will be confirmed during grading.

Aggregate base (crushed aggregate base or crushed miscellaneous base) should conform to the latest requirements of Section 200-2 of the Standard Specifications for Public Works Construction ("Greenbook") for untreated base materials (except processed miscellaneous base) or Caltrans Class 2 aggregate base.

The placement of demolition materials in compacted fill is acceptable from a geotechnical viewpoint provided the demolition material is broken up into pieces not larger than typically used for aggregate base (approximately 1-inch in maximum dimension) and well blended into fill soils with essentially no resulting voids. Demolition material placed in fills must be free of construction debris (wood, brick, etc.) and reinforcing steel. If asphalt concrete fragments will be incorporated into the demolition materials, approval from an environmental viewpoint may be required and is not the purview of the geotechnical consultant. From our previous experience, we recommend that asphalt concrete fragments be limited to fill areas within planned street areas below future utilities (i.e., not within building pad areas).

4.1.6 Subdrains

Construction of canyon style subdrains may be recommended. The locations of the recommended subdrains are generally controlled by the natural site topography within the alluvial canyons/swales. Canyon subdrains are typically placed following remedial grading and before fill placement within the "cleaned-out" channels on the exposed bedrock removal bottoms to collect future groundwater that may accumulate/migrate in these areas along the bedrock/fill contact. In areas where remedial grading will be deeper than available subdrain outlet elevations, fill placement will be performed until suitable subdrain flow elevations are achieved (minimum 2 percent flow towards the outlet location). In these areas, the primary purpose of the subdrains will be to reduce the potential for groundwater to rise above the subdrain elevations into the compacted fill. Recommended subdrain locations should be provided once grading plans are available in a grading plan review geotechnical report.

Installation of subdrains along backcuts of recommended stability fill buttresses should also be anticipated for site grading.

If unanticipated groundwater or areas of potential future groundwater seepage and/or accumulation are encountered during grading, additional subdrains may be recommended by the geotechnical consultant.

A representative of the project civil engineer should survey the installed subdrains for alignment and grade prior to fill placement above the subdrains.

4.1.7 Fill Placement

Material to be placed as fill should be brought above optimum moisture content (generally near optimum to about 2 percent above optimum moisture content) and recompacted to at least 90 percent relative compaction (per American Society for Testing

and Materials [ASTM] Test Method D1557). Moisture conditioning of site soils will be required in order to achieve adequate compaction. Drying and/or mixing the very moist soils will be required prior to reusing the materials in compacted fills. Soils are also present that will require additional moisture in order to achieve the required compaction.

The optimum lift thickness to produce a uniformly compacted fill will depend on the type and size of compaction equipment used. In general, fill should be placed in uniform lifts not exceeding 8 inches in compacted thickness. Each lift should be thoroughly compacted and accepted prior to subsequent lifts. Generally, placement and compaction of fill should be performed in accordance with local grading ordinances and under the observation and testing performed by the geotechnical consultant. Any encountered oversized material as previously defined must be appropriately handled as recommended in Section 4.1.7.1 and Appendix E.

Fill placed on any slopes greater than 5:1 (horizontal to vertical) should be properly keyed and benched into firm and competent soils as it is placed in lifts. During backfill of excavations, the fill should be properly benched into firm and competent soils of temporary backcut slopes as it is placed in lifts.

Fill slope faces should also be compacted to project requirements. This may require overbuilding of the slope face and trimming back to design grades. Placement of sand or gravel lacking cohesive soil for binder on the outer slope face should be avoided in order to reduce potential for surficial instability such as erosion rills. To improve surficial stability, vegetation specified by the landscape architect should be established on the slope face as soon as it is practical.

Aggregate base material (crushed aggregate base and crushed miscellaneous base) should be compacted to a minimum of 95 percent relative compaction at or slightly above optimum moisture content per ASTM D1557. Subgrade below aggregate base should be compacted to a minimum of 90 percent relative compaction at near-optimum moisture content (generally within optimum and 2 percent above optimum moisture content) per ASTM D1557.

If gap-graded ¾-inch rock is used for backfill (around storm drain storage chambers, retaining wall backfill, etc.) it will require compaction. Rock shall be placed in thin lifts (typically not exceeding 6 inches) and mechanically compacted with observation by geotechnical consultant. Backfill rock shall meet the requirements of ASTM D2321. Gap-graded rock is required to be wrapped in filter fabric (Mirafi 140N or approved alternative) to prevent the migration of fines into the rock backfill.

4.1.7.1 Oversized Placement

Oversized material (material larger than 8 inches in maximum dimension) may be generated during site grading. Recommendations are provided for appropriate handling of oversized materials in General Earthwork & Grading Specifications, Appendix E. Oversize material should not be placed in any deep fill areas where an increased minimum relative compaction is required. If feasible, crushing

oversized materials or exporting to an offsite location may be considered.

4.1.8 Trench and Conventional Retaining Wall Backfill and Compaction

The onsite soils may generally be suitable as trench backfill, provided the soils are screened of rocks and other materials greater than 3 inches in diameter and organic matter. If trenches are shallow, or the use of conventional equipment may result in damage to the utilities, sand having a sand equivalent (SE) of 30 (per Caltrans Test Method 217) or greater may be used to bed and shade the pipes. Sand backfill within the pipe bedding zone may be densified by jetting or flooding and then tamping to ensure adequate compaction. Subsequent trench backfill should be compacted in uniform thin lifts by mechanical means to at least 90 percent relative compaction (per ASTM D1557).

Conventional retaining wall backfill should consist of select sandy soils as outlined in preceding Section 4.1.5. The limits of select sandy backfill should extend at minimum $\frac{1}{2}$ the height of the retaining wall or the width of the heel (if applicable), whichever is greater (Figure 3). Retaining wall backfill soils should be compacted in relatively uniform thin lifts to at least 90 percent relative compaction (per ASTM D1557). Jetting or flooding of retaining wall backfill materials should not be permitted.

In backfill areas where mechanical compaction of soil backfill is impractical due to space constraints, typically sand-cement slurry may be substituted for compacted backfill. The slurry should contain about one sack of cement per cubic yard. When set, such a mix typically has the consistency of compacted soil. Sand cement slurry placed near the surface within landscape areas should be evaluated for potential impacts on planned improvements.

A representative from LGC Geotechnical should observe, probe, and test the backfill to verify compliance with the project recommendations.

4.1.9 Shrinkage and Bulking

Volumetric changes in earth quantities will occur when excavated onsite earth materials are replaced as properly compacted fill. The following is an estimate of factors for the geologic unit found onsite in the vicinity of grading. These estimates are based on in-place densities of the various materials and on the estimated average degree of relative compaction achieved during grading. Allowance in the earthwork volumes budget should be made for an estimated 5 to 15 percent reduction in volume of Pleistocene Pauba Formation (Qpfs). Subsidence due to earthwork equipment is expected to be on the order of 0.1 feet.

It should be stressed that these values are only estimates and that an actual shrinkage factor is extremely difficult to predetermine. The effective shrinkage of onsite soils will depend primarily on the type of compaction equipment and method of compaction used onsite by the contractor. Additionally, the onsite geology is somewhat complex, the above estimates are generalized groupings of similar lithologies and should be expected to vary

across the site and with depth. The above shrinkage and bulking estimates are intended as an aid for project engineers in determining preliminary earthwork quantities. However, these estimates should be used with some caution since they are not absolute values. Contingencies such as a balance pad should be made for balancing earthwork quantities based on actual shrinkage and subsidence that occurs during grading. Shrinkage and bulking are also expected to vary with variations in survey accuracy during rough grading.

4.2 Slope Stability

Global slope stability for the site has not been evaluated in this study. Recommendations for cut and fill slope construction should be provided as part of a future grading plan review report once grading plans are made available.

4.3 Provisional Foundation Recommendations

Given that the expansion index value corresponds to a “Medium” level of expansion (EI of 90 or less per ASTM D4829), the foundation system shall be designed for effects of expansive soil. Provided that the remedial grading recommendations provided herein are implemented, the site may be considered suitable for the support of the proposed structures using a conventional foundation system. Preliminary conventional foundation recommendations are provided in the following sections. Recommended soil bearing and estimated settlement due to structural loads are provided in Section 4.5.

Please note that the following foundation recommendations are preliminary and must be confirmed by LGC Geotechnical at the completion of project plans (i.e., foundation, grading, and site layout plans) as well as completion of grading.

4.3.1 Provisional Conventional Foundation Design Parameters

Conventional foundations may be designed in accordance with Wire Reinforcement Institute (WRI) procedure for slab-on-ground foundations per Section 1808 of the 2019 CBC to resist expansive soils. The following preliminary soil parameters may be used:

- Effective Plasticity Index: 25
- Climatic Rating: $C_w = 15$
- Reinforcement: Per structural designer.
- Minimum Perimeter Footing Depth: 18 inches below lowest adjacent grade.
- Moisture condition (presoak) slab subgrade to 120% of optimum moisture content to a minimum depth of 18 inches prior to trenching. The recommended moisture content should be maintained up to the time of concrete placement

Other types of stiff slabs may be used in place of the WRI design procedure provided that, in the opinion of the foundation structural designer, the alternative type of slab is at least as stiff and strong as that designed by the WRI to resist expansive soils.

4.3.2 Foundation Subgrade Preparation and Maintenance

Moisture conditioning (presoaking) of the subgrade soils is recommended prior to trenching the foundation. The duration of this process varies greatly based on the chosen method and is also dependent on factors such as soil type and weather conditions. Time duration for presoaking from completion of rough grading to trenching of foundations should be accounted for in the construction schedule (typically 1 to 2 weeks for medium expansion subgrade). The recommendations specific to the anticipated site soil conditions, including recommended presoak, are presented in Section 4.3.1. The subgrade moisture condition of the building pad soils should be maintained at near-optimum moisture content up to the time of concrete placement. This moisture content should be maintained around the immediate perimeter of the slab during construction and up to final sign off of the recreation center.

The geotechnical parameters provided herein assume that if the areas adjacent to the foundation are planted and irrigated, these areas will be designed with proper drainage and adequately maintained so that ponding, which causes significant moisture changes below the foundation, does not occur. Our recommendations do not account for excessive irrigation and/or incorrect landscape design. Plants should only be provided with sufficient irrigation for life and not overwatered to saturate subgrade soils. Sunken planters placed adjacent to the foundation, should either be designed with an efficient drainage system or liners to prevent moisture infiltration below the foundation. Some lifting of the perimeter foundation beam should be expected even with properly constructed planters.

In addition to the factors mentioned above, the owner should be made aware of the potential negative influences of trees and/or other large vegetation. Roots that extend near the vicinity of foundations can cause distress to foundations. The owner (and the owner's landscape architect) should not plant trees/large shrubs closer to the foundations than a distance equal to half the mature height of the tree or 20 feet, whichever is more conservative unless specifically provided with root barriers to prevent root growth below the foundation.

It is the owner's responsibility to perform periodic maintenance during hot and dry periods to ensure that adequate watering has been provided to keep soils from separating or pulling back from the foundation.

4.3.3 Slab Underlayment Guidelines

The following is for informational purposes only since slab underlayment (e.g., moisture retarder, sand or gravel layers for concrete curing and/or capillary break) is unrelated to the geotechnical performance of the foundation and thereby not the purview of the geotechnical consultant. Post-construction moisture migration should be expected below the foundation. The foundation engineer/architect should determine whether the use of a capillary break (sand or gravel layer), in conjunction with the vapor retarder, is necessary or required by code. Sand layer thickness and location (above and/or below vapor retarder) should also be determined by the foundation engineer/architect.

4.4 Foundation Setback from Top-of-Slope and Bottom-of-Slope

Foundations should have adequate setback from top and bottom of slopes in accordance with the City of Temecula, County of Riverside Grading Manual or the 2019 CBC, whichever is more conservative. Per the 2019 CBC, the minimum top-of-slope setback is $H/3$, with a maximum required setback of 40 feet, where H is the total height of the slope. As an alternative to moving the building footprint, setback requirements may be accomplished by deepened footings or deep foundations. The minimum bottom-of-slope setback is $H/2$, with a maximum required setback of 15 feet. Refer to Chapter 18 of the 2019 CBC for more information. It is the purview of the project civil engineer to implement the appropriate foundation setbacks.

4.5 Soil Bearing and Lateral Resistance

Provided our earthwork recommendations are implemented, an allowable soil bearing pressure of 2,000 pounds per square foot (psf) may be used for the design of footings having a minimum width of 12 inches and minimum embedment of 18 inches below lowest adjacent ground surface. This value may be increased by 300 psf for each additional foot of embedment or 150 psf for each additional foot of foundation width to a maximum value of 3,000 psf. An allowable soil bearing pressure of 1,200 psf may be used for a mat slab a minimum of 6 inches below lowest adjacent grade. These allowable bearing pressures are applicable for level (ground slope equal to or flatter than 5H:1V) conditions only. Bearing values indicated above are for total dead loads and live loads. The above vertical bearing may be increased by one-third for short durations of loading which will include the effect of wind or seismic forces.

In utilizing the above-mentioned allowable bearing capacity and provided our earthwork recommendations are implemented, foundation settlement due to structural loads is anticipated to be 1-inch or less. Differential settlement may be taken as half of the total settlement (i.e., $\frac{1}{2}$ -inch over a horizontal span of 40 feet).

Resistance to lateral loads can be provided by friction acting at the base of foundations and by passive earth pressure. For concrete/soil frictional resistance, an allowable coefficient of friction of 0.35 may be assumed with dead-load forces. An allowable passive lateral earth pressure of 250 psf per foot of depth (or pcf) to a maximum of 2,500 psf may be used for lateral resistance. Allowable passive pressure may be increased to 340 pcf to a maximum of 3,400 psf for short duration seismic loading. These passive pressures are applicable for level (ground slope equal to or flatter than 5H:1V) conditions only. For a 2:1 (horizontal to vertical) downward sloping condition, a reduced passive lateral earth pressure of 90 pcf to a maximum of 900 psf may be used. This allowable passive pressure may be increased to 120 pcf to a maximum of 1,200 psf for short duration seismic loading. We recommend that the upper foot of passive resistance be neglected if finished grade will not be covered with concrete or asphalt. Frictional resistance and passive pressure may be used in combination without reduction. The provided allowable passive pressures are based on a factor of safety of 1.5 and 1.1 for static and seismic loading conditions, respectively. The structural designer should incorporate appropriate factors of safety and/or load factors in their design.

4.6 **Lateral Earth Pressures for Conventional Retaining Wall Design**

The following lateral earth pressures are presented in Table 3 for approved granular soils a maximum of 35 percent fines (passing the No. 200 sieve per ASTM D1140) and Very Low expansion potential (EI of 20 or less per ASTM D4829). Retaining wall backfill should also be limited to fill material not exceeding 3 inches in greatest dimension. Please note that the on-site soils are not suitable for use as retaining wall backfill; therefore, import of select sandy soils meeting the criteria outlined above will be required by the contractor for obtaining suitable retaining wall backfill soil. The retaining wall designer should clearly indicate on the retaining wall plans the required sandy backfill.

Lateral earth pressures are provided as equivalent fluid unit weights, in psf/ft of depth or pcf. These values do not contain an appreciable factor of safety, so the retaining wall designer should apply the applicable factors of safety and/or load factors during design. A soil unit weight of 120 pcf may be assumed for calculating the actual weight of soil over the wall footing.

TABLE 3
Lateral Earth Pressures – Select Sandy Backfill

Conditions	Equivalent Fluid Unit Weight (pcf)	
	Level Backfill	2:1 Backfill Sloping Upwards
	Approved Sandy Backfill	Approved Sandy Backfill
Active	35	55
At-Rest	55	70

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 earth pressure will be higher. The equivalent fluid pressure values assume free-draining conditions. Retaining wall structures should be provided with appropriate drainage and appropriately waterproofed (Refer to Figure 3). Please note that waterproofing and outlet systems are not the purview of the geotechnical consultant. If conditions other than those assumed above are anticipated, the equivalent fluid pressure values should be provided on an individual-case basis by the geotechnical consultant.

Retaining wall structures should be provided with appropriate drainage and appropriately waterproofed. To reduce, but not eliminate, saturation of near surface (upper approximate 1-foot) soils in front of the retaining walls, the perforated subdrain pipe should be located as low as possible behind the retaining wall. The outlet pipe should be sloped to drain to a suitable outlet.

Surcharge loading effects from any adjacent structures should be evaluated by the retaining wall

designer. In general, structural loads within a 1:1 (horizontal to vertical) upward projection from the bottom of the proposed basement/retaining wall footing will surcharge the proposed retaining structure. In addition to the recommended earth pressure, retaining walls adjacent to streets should be designed to resist vehicular traffic if applicable. Uniform surcharges may be estimated using the applicable coefficient of lateral earth pressure using a rectangular distribution. A factor of 0.35 and 0.5 may be used for the active and at-rest conditions, respectively. The vertical traffic surcharge may be determined by the structural designer. The retaining/basement wall designer should contact the geotechnical engineer for any required geotechnical input in estimating any applicable surcharge loads.

If required, the retaining wall designer may use a seismic lateral earth pressure increment of 10 pcf for level backfill conditions. This increment should be applied in addition to the provided static lateral earth pressure using a “normal” triangular distribution with the resultant acting at $H/3$ in relation to the base of the retaining structure (where H is the retained height). For the restrained, at-rest condition, the seismic increment may be added to the applicable active lateral earth pressure (in lieu of the at-rest lateral earth pressure) when analyzing short duration seismic loading. Per Section 1803.5.12 of the 2019 CBC, the seismic lateral earth pressure is applicable to structures assigned to Seismic Design Category D through F for retaining wall structures supporting more than 6 feet of backfill height. This seismic lateral earth pressure is estimated using the procedure outlined by the Structural Engineers Association of California (Lew, et al, 2010).

Soil bearing and lateral resistance (friction coefficient and passive resistance) are provided in Section 4.5. Earthwork considerations (temporary backcuts, backfill, compaction, etc.) for retaining walls are provided in Section 4.1 (Site Earthwork) and the subsequent earthwork related sub-sections.

4.7 Soil Corrosivity to Concrete and Metal

Although not corrosion engineers (LGC Geotechnical is not a corrosion consultant), several governing agencies in Southern California require the geotechnical consultant to determine the corrosion potential of soils on buried concrete and metal facilities. We therefore present the results of our testing with regard to corrosion for the use of the client and other consultants, as they determine necessary.

Preliminary corrosion testing of a near-surface bulk sample indicated a soluble sulfate content less than approximately 0.01 percent, a chloride content of 286 parts per million (ppm), pH of 8.62, and a minimum resistivity of 780 ohm-centimeters. Based on Caltrans Corrosion Guidelines (Caltrans, 2018), soils are considered corrosive to structural elements if the pH is 5.5 or less, or the chloride concentration is 500 ppm or greater, or the sulfate concentration is 1,500 ppm (0.15 percent) or greater. Based on the preliminary test results, soils are not considered corrosive using Caltrans criteria.

Based on preliminary laboratory sulfate test results, the near surface soils are designated to a class “S0” per ACI 318, Table 19.3.1.1 with respect to sulfates. Concrete in direct contact with the onsite soils can be designed according to ACI 318, Table 19.3.2.1 using the “S0” sulfate classification.

Laboratory testing may need to be performed at the completion of grading by the project corrosion engineer to further evaluate the as-graded soil corrosivity characteristics. Accordingly, revision of the corrosion potential may be needed, should future test results differ substantially from the conditions reported herein. The client and/or other members of the development team should consider this during the design and planning phase of the project and formulate an appropriate course of action.

4.8 Control of Surface Water and Drainage Control

From a geotechnical perspective, we recommend that compacted finished grade soils adjacent to proposed buildings be sloped away from the proposed buildings and towards an approved drainage device or unobstructed swale. Drainage swales, wherever feasible, should not be constructed within 5 feet of buildings. Where lot and building geometry necessitates that the side yard drainage swales be routed closer than 5 feet to structural foundations, we recommend the use of area drains together with drainage swales. Drainage swales used in conjunction with area drains should be designed by the project civil engineer so that a properly constructed and maintained system will prevent ponding within 5 feet of the foundation. Code compliance of grades is not the purview of the geotechnical consultant.

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

4.9 Subsurface Water Infiltration

Recent regulatory changes have occurred that mandate storm water be infiltrated below grade rather than collected in a conventional storm drain system. Typically, a combination of methods are implemented to reduce surface water runoff and increase infiltration including; permeable pavements/pavers for roadways and walkways, directing surface water runoff to grass-lined swales, retention areas, and/or drywells, etc. It should be noted that collecting and concentrating surface water for the purpose of intentionally infiltrating below grade, conflicts with the geotechnical engineering objective of directing surface water away from slopes, structures and other improvements. The geotechnical stability and integrity of a site is reliant upon appropriately handling surface water. In general, the vast majority of geotechnical distress issues are directly related to improper drainage. In general, distress in the form of movement of improvements could occur as a result of soil saturation and loss of soil support, expansion, internal soil erosion, collapse and/or settlement.

The results of the infiltration testing were found to be somewhat variable. If required, stormwater may be infiltrated into the subsurface soils at the depths tested below existing grade, using the observed infiltration rates presented in Table 1 and the appropriate Riverside County (2011) safety factors. The Civil Engineer should determine the appropriate safety factor applicable to the proposed infiltration system. Results of field infiltration testing are provided in Appendix D.

The following should be considered for design of any required infiltration system:

- Water discharge from any infiltration systems should not occur within the zone of influence of foundation footings (column and load bearing wall locations). For preliminary purposes we recommend a minimum setback of 15 feet from the structural improvements or the County recommended minimum setback, whichever is more conservative.
- An adequate setback distance between any infiltration facility and adjacent private property should be maintained.
- Infiltration systems should not be placed over compacted fill or at the top of descending slopes.
- It may be prudent to provide an overflow system directly connected to the storm drain system in order to prevent failure of the infiltration system, either as a result of lower than anticipated infiltration and/or very high flow volumes. It should be noted that if pretreatment of runoff to remove debris, soil particles, etc., cannot be performed, design infiltration rates may need to be further reduced.
- The infiltration values provided are based on clean water and this requires the removal of trash, debris, soil particles, etc., and on-going maintenance. Over time, siltation and plugging may reduce the infiltration rate and subsequent effectiveness of the infiltration system. It should be noted that methods to prevent this shall be the responsibility of the infiltration designer and are not the purview of the geotechnical consultant. If adequate measures cannot be incorporated into the design and maintenance of the system, then the infiltration rates may need to be further reduced. These and other factors should be considered in selecting a design infiltration rate.
- Any designed infiltration system will require routine periodic maintenance.
- As with any systems that are designed to concentrate the surface flow and direct the water into the subsurface soils, some type of nuisance water and/or other water-related issues should be expected.
- Contamination and environmental suitability of the site for infiltration was not evaluated by us and should be evaluated by others (environmental consultant). We only addressed the geotechnical issues associated with stormwater infiltration.

LGC Geotechnical should be provided with details for any planned required infiltration system early in the design process for geotechnical input.

4.10 Preliminary Asphalt Concrete Pavement Sections

The following preliminary minimum asphalt concrete (AC) street sections are provided in Table 4 for Traffic Indices (TI) of 5.0 and 6.0. These sections are based on an assumed R-value of 20. These recommendations must be confirmed with R-value testing of representative near-surface soils at the completion of grading and after underground utilities have been installed and backfilled. Final pavement sections should be confirmed by the project civil engineer based upon the final design Traffic Index. If requested, LGC Geotechnical will provide sections for alternate TI values. Should the City of Temecula have more stringent requirements, updated pavement recommendation can be provided.

TABLE 4

Preliminary Asphalt Concrete Pavement Section Options

Assumed Traffic Index	5.0	6.0
R -Value Subgrade	20	20
AC Thickness	4.0 inches	5.0 inches
Aggregate Base Thickness	6.0 inches	7.5 inches

The pavement section thicknesses provided above are considered minimum thicknesses. Increasing the thickness of any or all of the above layers will reduce the likelihood of the pavement experiencing distress during its service life. The above recommendations are based on the assumption that proper maintenance and irrigation of the areas adjacent to the roadway will occur throughout the design life of the pavement. Failure to maintain a proper maintenance and/or irrigation program may jeopardize the integrity of the pavement.

Earthwork recommendations regarding aggregate base and subgrade are provided in the previous Section 4.1 (Site Earthwork) and the related sub-sections of this report.

4.11 Nonstructural Concrete Flatwork

Nonstructural concrete flatwork has a potential for cracking due to changes in soil volume related to soil-moisture fluctuations. To reduce the potential for excessive cracking and lifting, concrete may be designed in accordance with the minimum guidelines outlined in Table 5 on the following page. These guidelines will reduce the potential for irregular cracking and promote cracking along construction joints, but will not eliminate all cracking or lifting. Thickening the concrete and/or adding additional reinforcement will further reduce cosmetic distress.

TABLE 5

**Preliminary Geotechnical Parameters for Nonstructural Concrete Flatwork
Placed on Medium Expansion Potential Subgrade**

	Flatwork/ Patios	City Sidewalk Curb and Gutters
Minimum Thickness (in.)	5 (full)	City/Agency Standard
Presoaking	Presoak to 12 inches	City/Agency Standard
Reinforcement	No. 3 at 24 inches on centers	City/Agency Standard
Thickened Edge (in.)	—	City/Agency Standard
Crack Control Joints	Saw cut or deep open tool joint to a minimum of $\frac{1}{3}$ the concrete thickness	City/Agency Standard
Maximum Joint Spacing	6 feet	City/Agency Standard
Aggregate Base Thickness (in.)	2	City/Agency Standard

4.12 Geotechnical Plan Review

Grading plans, retaining wall plans, foundation plans and final project drawings should be reviewed by this office prior to grading to verify that our geotechnical recommendations, provided herein, have been appropriately incorporated. Additional or modified geotechnical recommendations may be required based on the proposed design.

4.13 Geotechnical Observation and Testing During Construction

The recommendations provided in this report are based on limited subsurface observations and geotechnical analysis. The interpolated subsurface conditions should be checked in the field during grading by a representative of LGC Geotechnical. Geotechnical observation and testing is required per Section 1705 of the 2019 California Building Code (CBC).

Geotechnical observation and/or testing should be performed by LGC Geotechnical at the following stages:

- During grading (key excavations, removal bottoms, remedial grading, fill placement, etc.);
- Subdrain construction;

- During retaining wall backfill and compaction;
- During utility trench backfill and compaction;
- After presoaking building pads and other concrete-flatwork subgrades, and prior to placement of aggregate base or concrete;
- After building and wall footing excavation and prior to placing steel reinforcement and/or concrete;
- Preparation of pavement subgrade and placement of aggregate base; and
- When any unusual soil conditions are encountered during any construction operation subsequent to issuance of this report.

5.0 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. 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 revealed by excavation may be different than our preliminary findings. If this occurs, the changed conditions must be evaluated by the project soils engineer and 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 findings of this report are valid as of the present date. However, changes in the conditions of a property can and do occur with the passage of time, whether they be due to natural processes or the works of man on this or adjacent properties. Therefore, the findings, conclusions, and recommendations presented in this report can be relied upon only if LGC Geotechnical has the opportunity to observe the subsurface conditions during grading and construction of the project, in order to confirm that our preliminary findings are representative for the site.

In addition, changes in applicable or appropriate 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, this report is subject to review and modification, and should not be relied upon after a period of 3 years.

Qya

Quaternary Young Alluvial Channel Deposits

Qpfs

Pleistocene Pauba Formation - Sandstone Member, Circled Where Buried

HS-5

T.D. = 26.5'

I-2

T.D. = 5'

Approximate Location of Hollow Stem Auger Boring by LGC Geotechnical, With Total Depth in Feet

Approximate Location of Hollow Stem Auger Infiltration Boring by LGC Geotechnical, With Total Depth in Feet

Approximate Location of Geologic Contact

Approximate Limits of this Report

5

Approximate Anticipated Depth of Remedial Grading in Feet Below Existing Ground Surface

The map displays a site with topographic contours ranging from 1280 to 1350 feet. A dashed black line indicates the approximate location of a geologic contact between Quaternary Young Alluvial Channel Deposits (Qya) and Pleistocene Pauba Formation - Sandstone Member (Qpfs). A blue dashed line outlines the approximate limits of the report. Several boring locations are marked: HS-1 (T.D. = 36.5'), HS-2 (T.D. = 36.5'), HS-3 (T.D. = 36.5'), HS-4 (T.D. = 11.5'), HS-5 (T.D. = 26.5'), I-1 (T.D. = 15'), and I-2 (T.D. = 5'). Red triangles with the number 5 indicate the approximate anticipated depth of remedial grading in feet below the existing ground surface. The map also shows various engineering data points, including bearings and distances for property lines and curves.

Basemap provided by 4M Engineering and Developement, Inc.

LGC

Geotechnical, Inc.

LGC Geotechnical, Inc.

131 Calle Iglesia, Ste. 200

San Clemente, CA 92672

TEL (949) 369-6141 FAX (949) 369-6142

FIGURE 2

Geotechnical Map

PROJECT NAME	EPD - Temecula
PROJECT NO.	20145-01
ENG. / GEOL.	RLD/ARN
SCALE	1" = 100'
DATE	December, 2020

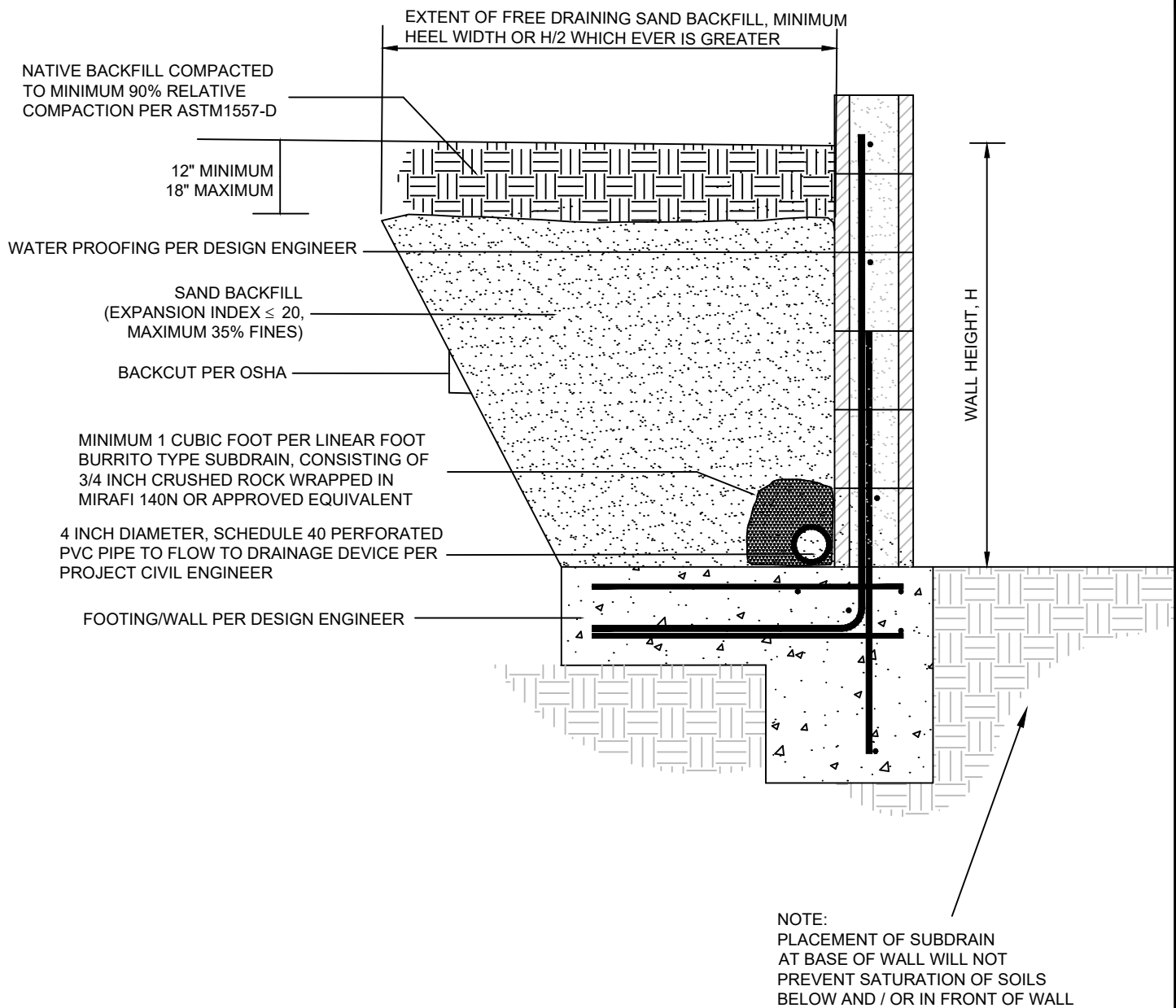


FIGURE 3
Retaining Wall
Backfill Detail

PROJECT NAME	EPD - Temecula
PROJECT NO.	20145-01
ENG. / GEOL.	RLD / ARN
SCALE	Not to Scale
DATE	December 2020

Appendix A

References

References

- 4M Engineering and Development, Inc. (4M Engineering), 2020, Preliminary Site Plan, 41325 Calle Encantado, Temecula, California, dated October 23, 2020.
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APPENDIX A (Cont'd)

References

Structural Engineers Association of California (SEAOC), 2020, Seismic Design Maps, Retrieved December 2, 2020, from <https://seismicmaps.org/>

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Appendix B

Boring Logs

Geotechnical Boring Log Borehole HS-1

Date: 11/12/2020	Drilling Company: Cal Pac Drilling
Project Name: EDP - Temecula	Type of Rig: CME-75 Truck Mounted Drill Rig
Project Number: 20145-01	Drop: 30" Hole Diameter: 6"
Elevation of Top of Hole: ~1328' MSL	Drive Weight: 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 2

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	Logged By ARN Sampled By ARN Checked By RLD DESCRIPTION	Type of Test
1325	0		R-1	10 14 21	123.0	8.4	SC	0' to T.D. - Pauba Formation (Qpfs): @2.5' - Clayey SAND: brown, moist, medium dense	
	5		R-2	8 17 24	97.2	11.7		@5' - Clayey SAND: brown, moist, dense	
1320			R-3	6 16 23	116.6	12.5		@7.5' - Clayey SAND: light olive brown, moist, dense	#200 DS
	10		R-4	11 13 23	102.7	4.7	SP	@10' - SAND: light brown, slightly moist, dense	
1315									
	15		SPT-1	4 7 9		23.7	CL	@15' - CLAY: gray brown with red orange, very moist, very stiff; minor iron oxide staining	
1310									
	20		R-5	20 50/6"	117.3	6.1	SC	@20' - Clayey SAND: light gray and brown, slightly moist, very dense	
1305									
	25		SPT-2	8 12 19		12.6		@25' - Clayey SAND: olive gray, moist, dense	
1300									
	30								



THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF THE ACTUAL CONDITIONS ENCOUNTERED. THE DESCRIPTIONS PROVIDED ARE QUALITATIVE FIELD DESCRIPTIONS AND ARE NOT BASED ON QUANTITATIVE ENGINEERING ANALYSIS.

SAMPLE TYPES:
B BULK SAMPLE
R RING SAMPLE (CA Modified Sampler)
G GRAB SAMPLE
SPT STANDARD PENETRATION TEST SAMPLE

GROUNDWATER TABLE

TEST TYPES:
DS DIRECT SHEAR
MD MAXIMUM DENSITY
SA SIEVE ANALYSIS
S&H SIEVE AND HYDROMETER
EI EXPANSION INDEX
CN CONSOLIDATION
CR CORROSION
AL ATTERBERG LIMITS
CO COLLAPSE/SWELL
RV R-VALUE
-#200 % PASSING # 200 SIEVE

Geotechnical Boring Log Borehole HS-1

Date: 11/12/2020	Drilling Company: Cal Pac Drilling
Project Name: EDP - Temecula	Type of Rig: CME-75 Truck Mounted Drill Rig
Project Number: 20145-01	Drop: 30" Hole Diameter: 6"
Elevation of Top of Hole: ~1328' MSL	Drive Weight: 140 pounds
Hole Location: See Geotechnical Map	Page 2 of 2

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	Logged By ARN Sampled By ARN Checked By RLD DESCRIPTION	Type of Test
1295	30		R-6	10 17 50/4"	99.0	10.0	SC	@30' - Clayey SAND: dusky gray brown, moist, very dense	
	35		SPT-3	10 14 29		10.0		@35' - Clayey SAND: red brown, moist, dense	
1290	40							Total Depth = 36.5' Groundwater Not Encountered Backfilled with Cuttings on 11/12/2020	
1285	45								
1280	50								
1275	55								
1270	60								



THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF THE ACTUAL CONDITIONS ENCOUNTERED. THE DESCRIPTIONS PROVIDED ARE QUALITATIVE FIELD DESCRIPTIONS AND ARE NOT BASED ON QUANTITATIVE ENGINEERING ANALYSIS.

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GROUNDWATER TABLE

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 EI EXPANSION INDEX
 CN CONSOLIDATION
 CR CORROSION
 AL ATTERBERG LIMITS
 CO COLLAPSE/SWELL
 RV R-VALUE
 -#200 % PASSING # 200 SIEVE

Geotechnical Boring Log Borehole HS-2

Date: 11/12/2020	Drilling Company: Cal Pac Drilling
Project Name: EDP - Temecula	Type of Rig: CME-75 Truck Mounted Drill Rig
Project Number: 20145-01	Drop: 30" Hole Diameter: 6"
Elevation of Top of Hole: ~1312' MSL	Drive Weight: 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 2

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	Logged By ARN Sampled By ARN Checked By RLD DESCRIPTION	Type of Test
1310	0	B-1						0' to T.D. - Pauba Formation:(Qpfs):	
			R-1	8 7	107.8	6.8	SC	@2.5' - Clayey SAND: brown, slightly moist, medium dense	MD
	5		R-2	5 7 13	109.1	3.5	SM	@5' - Silty SAND: brown, dry, medium dense	
1305			R-3	6 8 9	113.9	4.5		@7.5' - Silty SAND: dusky brown, slightly moist, medium dense; scarce rootlets	
	10		R-4	8 10 10	116.3	4.1		@10' - Silty SAND: dusky brown, dry, medium dense; occasional pea gravel	
1300									
	15		SPT-1	8 12 10		4.3		@15' - Silty SAND: light brown, dry, medium dense	
1295									
	20		R-5	14 23 50/5"	123.7	12.8		@20' - Silty SAND: dusky brown, moist, very dense	
1290									
	25		SPT-2	6 11 18		11.4	CL	@25' - Sandy CLAY: gray brown, slightly moist, very stiff	
1285									
	30								



THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF THE ACTUAL CONDITIONS ENCOUNTERED. THE DESCRIPTIONS PROVIDED ARE QUALITATIVE FIELD DESCRIPTIONS AND ARE NOT BASED ON QUANTITATIVE ENGINEERING ANALYSIS.

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 G GRAB SAMPLE
 SPT STANDARD PENETRATION TEST SAMPLE

GROUNDWATER TABLE

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 S&H SIEVE AND HYDROMETER
 EI EXPANSION INDEX
 CN CONSOLIDATION
 CR CORROSION
 AL ATTERBERG LIMITS
 CO COLLAPSE/SWELL
 RV R-VALUE
 -#200 % PASSING # 200 SIEVE

Geotechnical Boring Log Borehole HS-2

Date: 11/12/2020	Drilling Company: Cal Pac Drilling
Project Name: EDP - Temecula	Type of Rig: CME-75 Truck Mounted Drill Rig
Project Number: 20145-01	Drop: 30" Hole Diameter: 6"
Elevation of Top of Hole: ~1312' MSL	Drive Weight: 140 pounds
Hole Location: See Geotechnical Map	Page 2 of 2

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	Logged By ARN Sampled By ARN Checked By RLD DESCRIPTION	Type of Test
1280	30		R-6	14 31 45	121.7	12.4	SC	@30' - Clayey SAND: dusky brown, moist, very dense	
1275	35		SPT-3	7 16 26		7.9	SM	@35' - Silty SAND: light brown, moist, dense	
1270	40							Total Depth = 36.5' Groundwater Not Encountered Backfilled with Cuttings on 11/12/2020	
1265	45								
1260	50								
1255	55								
1250	60								



THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF THE ACTUAL CONDITIONS ENCOUNTERED. THE DESCRIPTIONS PROVIDED ARE QUALITATIVE FIELD DESCRIPTIONS AND ARE NOT BASED ON QUANTITATIVE ENGINEERING ANALYSIS.

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 SPT STANDARD PENETRATION TEST SAMPLE

GROUNDWATER TABLE

TEST TYPES:
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 S&H SIEVE AND HYDROMETER
 EI EXPANSION INDEX
 CN CONSOLIDATION
 CR CORROSION
 AL ATTERBERG LIMITS
 CO COLLAPSE/SWELL
 RV R-VALUE
 -#200 % PASSING # 200 SIEVE

Geotechnical Boring Log Borehole HS-3

Date: 11/12/2020 **Drilling Company:** Cal Pac Drilling
Project Name: EDP - Temecula **Type of Rig:** CME-75 Truck Mounted Drill Rig
Project Number: 20145-01 **Drop:** 30" **Hole Diameter:** 6"
Elevation of Top of Hole: ~1311' MSL **Drive Weight:** 140 pounds
Hole Location: See Geotechnical Map

Page 1 of 2

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	Logged By ARN Sampled By ARN Checked By RLD DESCRIPTION	Type of Test
1310	0	B-1						0' to T.D. - Pauba Formation:(Qpfs):	
			R-1	10 15 21	120.0	6.8	SM	@2.5' - Silty SAND: dusky brown, slightly moist, dense	
1305	5		R-2	13 25 50/5"	125.8	5.0		@5' - Silty SAND: dusky brown, slightly moist, very dense; scattered gravel	
			R-3	19 19 19	124.3	9.4	SC	@7.5' - Clayey SAND: dusky brown, moist, dense	
1300	10		R-4	10 13 21	100.5	24.8	ML	@10' - Sandy SILT: dusky gray brown with red brown, very moist, very stiff	
1295	15		SPT-1	5 9 16		13.0	SM	@15' - Silty SAND: gray, very moist, medium dense	
1290	20		R-5	18 25 40	121.6	3.1	SP	@20' - SAND: pinkish brown, dry, very dense; scarce rounded gravel	
1285	25		SPT-2	9 17 22		10.4	SC	@25' - Clayey SAND: gray brown, moist, dense	
	30								



THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF THE ACTUAL CONDITIONS ENCOUNTERED. THE DESCRIPTIONS PROVIDED ARE QUALITATIVE FIELD DESCRIPTIONS AND ARE NOT BASED ON QUANTITATIVE ENGINEERING ANALYSIS.

SAMPLE TYPES:
 B BULK SAMPLE
 R RING SAMPLE (CA Modified Sampler)
 G GRAB SAMPLE
 SPT STANDARD PENETRATION TEST SAMPLE

GROUNDWATER TABLE

TEST TYPES:
 DS DIRECT SHEAR
 MD MAXIMUM DENSITY
 SA SIEVE ANALYSIS
 S&H SIEVE AND HYDROMETER
 EI EXPANSION INDEX
 CN CONSOLIDATION
 CR CORROSION
 AL ATTERBERG LIMITS
 CO COLLAPSE/SWELL
 RV R-VALUE
 -#200 % PASSING # 200 SIEVE

Geotechnical Boring Log Borehole HS-3

Date: 11/12/2020	Drilling Company: Cal Pac Drilling
Project Name: EDP - Temecula	Type of Rig: CME-75 Truck Mounted Drill Rig
Project Number: 20145-01	Drop: 30" Hole Diameter: 6"
Elevation of Top of Hole: ~1311' MSL	Drive Weight: 140 pounds
Hole Location: See Geotechnical Map	Page 2 of 2

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	Logged By ARN Sampled By ARN Checked By RLD DESCRIPTION	Type of Test
1280	30		R-6	7 22 37	110.1	6.5	SP	@30' - SAND: rusty brown, moist, dense	
1275	35		SPT-3	14 16 14		7.0	SM	@35' - Silty SAND: red brown, slightly moist, medium dense	
1270	40							Total Depth = 36.5' Groundwater Not Encountered Backfilled with Cuttings on 11/12/2020	
1265	45								
1260	50								
1255	55								
	60								



THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF THE ACTUAL CONDITIONS ENCOUNTERED. THE DESCRIPTIONS PROVIDED ARE QUALITATIVE FIELD DESCRIPTIONS AND ARE NOT BASED ON QUANTITATIVE ENGINEERING ANALYSIS.

SAMPLE TYPES:
B BULK SAMPLE
R RING SAMPLE (CA Modified Sampler)
G GRAB SAMPLE
SPT STANDARD PENETRATION TEST SAMPLE

GROUNDWATER TABLE

TEST TYPES:
DS DIRECT SHEAR
MD MAXIMUM DENSITY
SA SIEVE ANALYSIS
S&H SIEVE AND HYDROMETER
EI EXPANSION INDEX
CN CONSOLIDATION
CR CORROSION
AL ATTERBERG LIMITS
CO COLLAPSE/SWELL
RV R-VALUE
-#200 % PASSING # 200 SIEVE

Geotechnical Boring Log Borehole HS-4

Date: 11/12/2020	Drilling Company: Cal Pac Drilling
Project Name: EDP - Temecula	Type of Rig: CME-75 Truck Mounted Drill Rig
Project Number: 20145-01	Drop: 30" Hole Diameter: 6"
Elevation of Top of Hole: ~1297' MSL	Drive Weight: 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 1

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	Logged By ARN Sampled By ARN Checked By RLD DESCRIPTION	Type of Test
1295	0	B-1	R-1	6 13 17	113.3	7.5	SM	0' to T.D. - Pauba Formation:(Qpfs): @2.5' - Silty SAND: red brown, moist, medium dense	
	5		R-2	8 10 20	105.6	18.1	SC	@5' - Clayey SAND: dusky brown, very moist, medium dense	
1290			R-3	7 13 19	100.8	24.3	ML	@7.5' - Sandy SILT: brown, very moist, very stiff	
	10		R-4	8 21 30	118.1	15.4		@10' - Sandy SILT: brown, very moist, hard	
1285								Total Depth = 11.5' Groundwater Not Encountered Backfilled with Cuttings on 11/12/2020	
	15								
1280									
	20								
1275									
	25								
1270									
	30								



THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF THE ACTUAL CONDITIONS ENCOUNTERED. THE DESCRIPTIONS PROVIDED ARE QUALITATIVE FIELD DESCRIPTIONS AND ARE NOT BASED ON QUANTITATIVE ENGINEERING ANALYSIS.

SAMPLE TYPES:
 B BULK SAMPLE
 R RING SAMPLE (CA Modified Sampler)
 G GRAB SAMPLE
 SPT STANDARD PENETRATION TEST SAMPLE

GROUNDWATER TABLE

TEST TYPES:
 DS DIRECT SHEAR
 MD MAXIMUM DENSITY
 SA SIEVE ANALYSIS
 S&H SIEVE AND HYDROMETER
 EI EXPANSION INDEX
 CN CONSOLIDATION
 CR CORROSION
 AL ATTERBERG LIMITS
 CO COLLAPSE/SWELL
 RV R-VALUE
 #200 % PASSING # 200 SIEVE

Geotechnical Boring Log Borehole HS-5

Date: 11/12/2020	Drilling Company: Cal Pac Drilling
Project Name: EDP - Temecula	Type of Rig: CME-75 Truck Mounted Drill Rig
Project Number: 20145-01	Drop: 30" Hole Diameter: 6"
Elevation of Top of Hole: ~1312' MSL	Drive Weight: 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 1

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	Logged By ARN Sampled By ARN Checked By RLD DESCRIPTION	Type of Test
1310	0	B-1						0' to T.D. - Pauba Formation:(Qpfs):	
			R-1	12 17 18	92.7	8.8	SC	@2.5' - Silty to Clayey SAND: brown, moist, medium dense	CR, EI
	5		R-2	10 11 14	121.4	10.7	CL	@5' - CLAY: olive brown, slightly moist, very stiff	CN
1305			R-3	7 11 15	108.9	6.5	SM	@7.5' - Silty SAND: gray brown, slightly moist, medium dense	
	10		R-4	7 17 30	113.2	6.5		@10' - Silty SAND: gray brown, slightly moist, dense	
1300									
	15		SPT-1	13 14 16		1.7	SW	@15' - SAND: gray, dry, dense	
1295									
	20		R-5	14 18 25	113.4	13.7	CL	@20' - CLAY with Sand: gray and brown, moist, very stiff	
1290									
	25		SPT-2	6 18 24		28.1	ML	@25' - Sandy SILT: olive gray with red orange, very moist, hard	
1285								Total Depth = 26.5'	
								Groundwater Not Encountered	
								Backfilled with Cuttings on 11/12/2020	
	30								



THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF THE ACTUAL CONDITIONS ENCOUNTERED. THE DESCRIPTIONS PROVIDED ARE QUALITATIVE FIELD DESCRIPTIONS AND ARE NOT BASED ON QUANTITATIVE ENGINEERING ANALYSIS.

SAMPLE TYPES:
 B BULK SAMPLE
 R RING SAMPLE (CA Modified Sampler)
 G GRAB SAMPLE
 SPT STANDARD PENETRATION TEST SAMPLE

TEST TYPES:
 DS DIRECT SHEAR
 MD MAXIMUM DENSITY
 SA SIEVE ANALYSIS
 S&H SIEVE AND HYDROMETER
 EI EXPANSION INDEX
 CN CONSOLIDATION
 CR CORROSION
 AL ATTERBERG LIMITS
 CO COLLAPSE/SWELL
 RV R-VALUE
 -#200 % PASSING # 200 SIEVE

GROUNDWATER TABLE

Geotechnical Boring Log Borehole I-1

Date: 11/12/2020	Drilling Company: Cal Pac Drilling
Project Name: EDP - Temecula	Type of Rig: CME-75 Truck Mounted Drill Rig
Project Number: 20145-01	Drop: 30" Hole Diameter: 8"
Elevation of Top of Hole: ~1320' MSL	Drive Weight: 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 1

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number		Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	Logged By ARN Sampled By ARN Checked By RLD DESCRIPTION	Type of Test
1315	0		R-1		8 9 10	96.5	17.3	CH	0' to T.D. - Pauba Formation:(Qpfs): @2.5' - CLAY: gray brown, moist, stiff @5' - Sandy SILT: gray brown, very moist, very stiff @7.5' - Silty SAND: gray brown, moist, medium dense @10' - Silty SAND: gray, slightly moist, medium dense @13' - SAND: gray brown, dry, dense	AL
	5		R-2		5 10 21	97.7	26.2	ML		
			R-3		5 9 15	105.4	7.9	SM		
	10		R-4		8 13 22	101.2	6.4			
			R-6		15 19 28	111.0	1.7	SW		
1305	15								Total Depth = 15' Groundwater Not Encountered Infiltration Well Installed Per County Guidelines Pipe Pulled and Boring Backfilled with Cuttings on 11/13/2020	
1300	20									
1295	25									
	30									



THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF THE ACTUAL CONDITIONS ENCOUNTERED. THE DESCRIPTIONS PROVIDED ARE QUALITATIVE FIELD DESCRIPTIONS AND ARE NOT BASED ON QUANTITATIVE ENGINEERING ANALYSIS.

SAMPLE TYPES: B BULK SAMPLE R RING SAMPLE (CA Modified Sampler) G GRAB SAMPLE SPT STANDARD PENETRATION TEST SAMPLE	TEST TYPES: DS DIRECT SHEAR MD MAXIMUM DENSITY SA SIEVE ANALYSIS S&H SIEVE AND HYDROMETER EI EXPANSION INDEX CN CONSOLIDATION CR CORROSION AL ATTERBERG LIMITS CO COLLAPSE/SWELL RV R-VALUE -#200 % PASSING # 200 SIEVE
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GROUNDWATER TABLE

Geotechnical Boring Log Borehole I-2

Date: 11/12/2020	Drilling Company: Cal Pac Drilling
Project Name: EDP - Temecula	Type of Rig: CME-75 Truck Mounted Drill Rig
Project Number: 20145-01	Drop: 30" Hole Diameter: 8"
Elevation of Top of Hole: ~1304' MSL	Drive Weight: 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 1

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	Logged By ARN Sampled By ARN Checked By RLD DESCRIPTION	Type of Test
1300	0		R-1	3 4 5	114.3	9.0	SC	0' to T.D. - Pauba Formation (Qpfs): @3' - Clayey SAND: brown, moist, loose	
1295	5							Total Depth = 5' Groundwater Not Encountered Infiltration Well Installed Per County Guidelines Pipe Pulled and Boring Backfilled with Cuttings on 11/13/2020	
1290	10								
1285	15								
1280	20								
1275	25								
	30								



THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF THE ACTUAL CONDITIONS ENCOUNTERED. THE DESCRIPTIONS PROVIDED ARE QUALITATIVE FIELD DESCRIPTIONS AND ARE NOT BASED ON QUANTITATIVE ENGINEERING ANALYSIS.

SAMPLE TYPES:
 B BULK SAMPLE
 R RING SAMPLE (CA Modified Sampler)
 G GRAB SAMPLE
 SPT STANDARD PENETRATION TEST SAMPLE



TEST TYPES:
 DS DIRECT SHEAR
 MD MAXIMUM DENSITY
 SA SIEVE ANALYSIS
 S&H SIEVE AND HYDROMETER
 EI EXPANSION INDEX
 CN CONSOLIDATION
 CR CORROSION
 AL ATTERBERG LIMITS
 CO COLLAPSE/SWELL
 RV R-VALUE
 -#200 % PASSING # 200 SIEVE

Appendix C
Laboratory Test Results

APPENDIX C

Laboratory Test Results

The laboratory testing program was directed towards providing quantitative data relating to the relevant engineering properties of the soils. 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.

Moisture and Density Determination Tests: Moisture content (ASTM D2216) and dry density determinations (ASTM D2937) were performed on driven samples obtained from the test borings. The results of these tests are presented in the boring logs. Where applicable, only moisture content was determined from undisturbed or disturbed samples.

Grain Size Distribution/Fines Content: Representative samples were dried, weighed, and soaked in water until individual soil particles were separated (per ASTM D421) and then washed on a No. 200 sieve (ASTM D1140). Where applicable, the portion retained on the No. 200 sieve was dried and then sieved on a U.S. Standard brass sieve set in accordance with ASTM D6913 (sieve) or ASTM D422 (sieve and hydrometer).

Sample Location	Description	% Passing # 200 Sieve
HS-1 @ 7.5 ft	Clayey SAND	35

Atterberg Limits: The liquid and plastic limits ("Atterberg Limits") were determined per ASTM D4318 for engineering classification of fine-grained material and presented in the table below. The USCS soil classification indicated in the table below is based on the portion of sample passing the No. 40 sieve and may not necessarily be representative of the entire sample.

Sample Location	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	USCS Soil Classification
I-1 @ 3 ft	51	23	28	CH

APPENDIX C (Cont'd)

Laboratory Test Results

Expansion Index: The expansion potential of a selected representative sample was evaluated by the Expansion Index Test per ASTM D4829. The result is presented in the table below.

Sample Location	Expansion Index	Expansion Potential*
HS-5 @ 1-5 ft	64	Medium

* Per ASTM D4829

Direct Shear: One direct shear test was performed on a remolded sample compacted to 90 percent relative compaction. The ring samples were soaked for a minimum of 24 hours prior to testing. The samples were tested under various normal loads using a motor-driven, strain-controlled, direct-shear testing apparatus (ASTM D3080). The plot is provided in this Appendix.

Consolidation: One consolidation test was performed per ASTM D2435. A sample (2.4 inches in diameter and 1 inch in height) was placed in a consolidometer and increasing loads were applied. The sample was allowed to consolidate under "double drainage" and total deformation for each loading step was recorded. The percent consolidation for each load step was recorded as the ratio of the amount of vertical compression to the original sample height. The consolidation pressure curve is provided in this Appendix.

Laboratory Compaction: The maximum dry density and optimum moisture content of typical materials were determined in accordance with ASTM D1557. The results are presented in the table below.

Sample Location	Sample Description	Maximum Dry Density (pcf)	Optimum Moisture Content (%)
HS-2 @ 1-5 ft	Brown Silty SAND	131.0	8.0

Soluble Sulfates: The soluble sulfate contents of selected samples were determined by standard geochemical methods (CTM 417). The test results are presented in the table below.

Sample Location	Sulfate Content, %
HS-5 @ 1-5 ft	< 0.01

APPENDIX C (Cont'd)

Laboratory Test Results

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

Sample Location	Chloride Content, ppm
HS-5 @ 1-5 ft	286

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

Sample Location	pH	Minimum Resistivity (ohms-cm)
HS-5 @ 1-5 ft	8.62	780

DIRECT SHEAR TEST
Consolidated Drained - ASTM D 3080

Project Name: Temecula	Tested By: G. Bathala	Date: 11/18/20
Project No.: 20145-01	Checked By: A. Santos	Date: 11/23/20
Boring No.: HS-1	Sample Type: Ring	
Sample No.: R-3	Depth (ft.): 7.5	
Soil Identification: Light olive brown clayey sand (SC)		

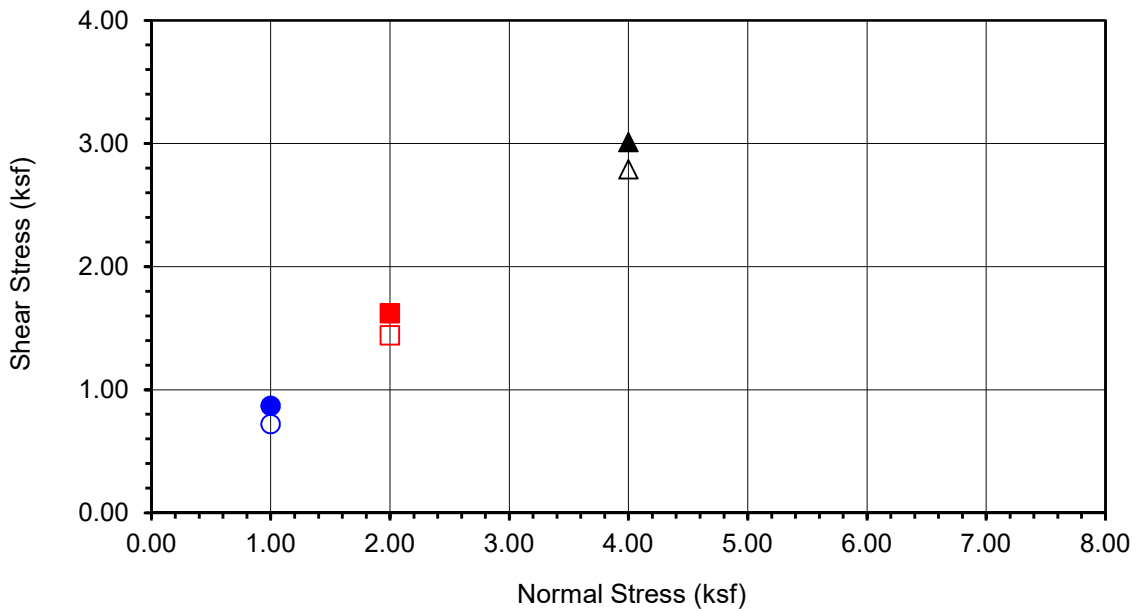
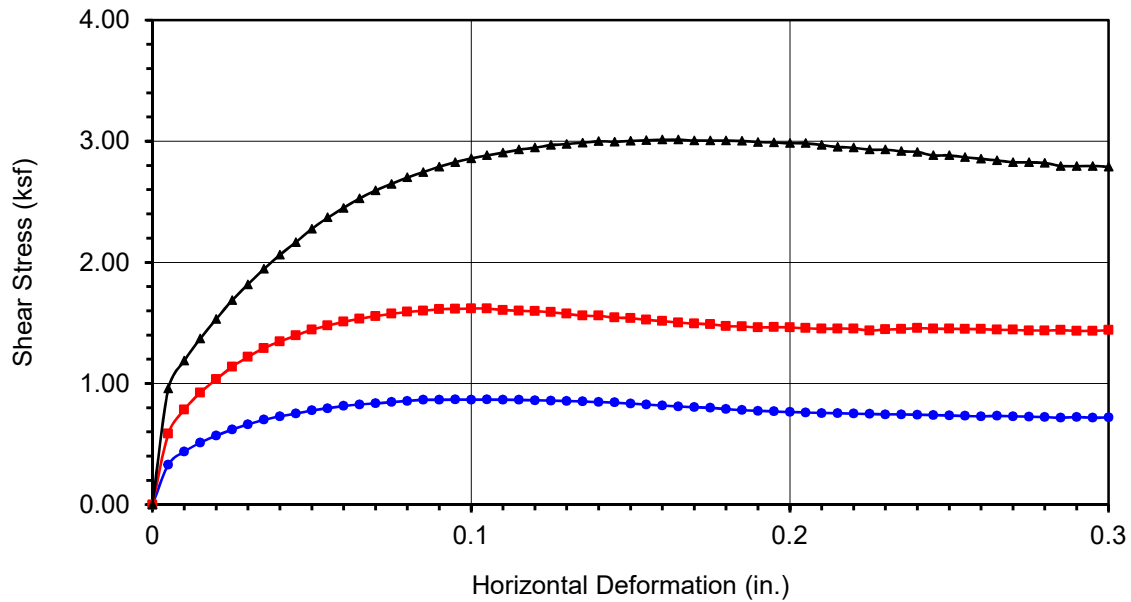
Sample Diameter(in):	2.415	2.415	2.415
Sample Thickness(in.):	1.000	1.000	1.000
Weight of Sample + ring(gm):	204.83	206.90	207.78
Weight of Ring(gm):	45.56	45.51	45.47

Before Shearing

Weight of Wet Sample+Cont.(gm):	175.81	175.81	175.81
Weight of Dry Sample+Cont.(gm):	162.67	162.67	162.67
Weight of Container(gm):	57.31	57.31	57.31
Vertical Rdg.(in): Initial	0.2714	0.0000	0.0000
Vertical Rdg.(in): Final	0.2799	-0.0149	0.0227

After Shearing

Weight of Wet Sample+Cont.(gm):	236.65	218.55	213.48
Weight of Dry Sample+Cont.(gm):	215.98	199.08	194.94
Weight of Container(gm):	75.85	57.97	52.67
Specific Gravity (Assumed):	2.70	2.70	2.70
Water Density(pcf):	62.43	62.43	62.43



Boring No.	HS-1
Sample No.	R-3
Depth (ft)	7.5
<u>Sample Type:</u>	
Ring	
<u>Soil Identification:</u>	
Light olive brown clayey sand (SC)	

Normal Stress (kip/ft²)	1.000	2.000	4.000
Peak Shear Stress (kip/ft²)	● 0.868	■ 1.619	▲ 3.012
Shear Stress @ End of Test (ksf)	○ 0.720	□ 1.440	△ 2.789
Deformation Rate (in./min.)	0.0013	0.0013	0.0013
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	12.47	12.47	12.47
Dry Density (pcf)	117.8	119.3	120.0
Saturation (%)	78.1	81.6	83.3
Soil Height Before Shearing (in.)	0.9915	0.9851	1.0227
Final Moisture Content (%)	14.8	13.8	13.0

DIRECT SHEAR TEST RESULTS
Consolidated Drained - ASTM D 3080

Project No.: 20145-01

Temecula

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS ASTM D 2435

Project Name: Temecula

Project No.: 20145-01

Boring No.: HS-5

Sample No.: R-2

Soil Identification: Olive brown lean clay (CL)

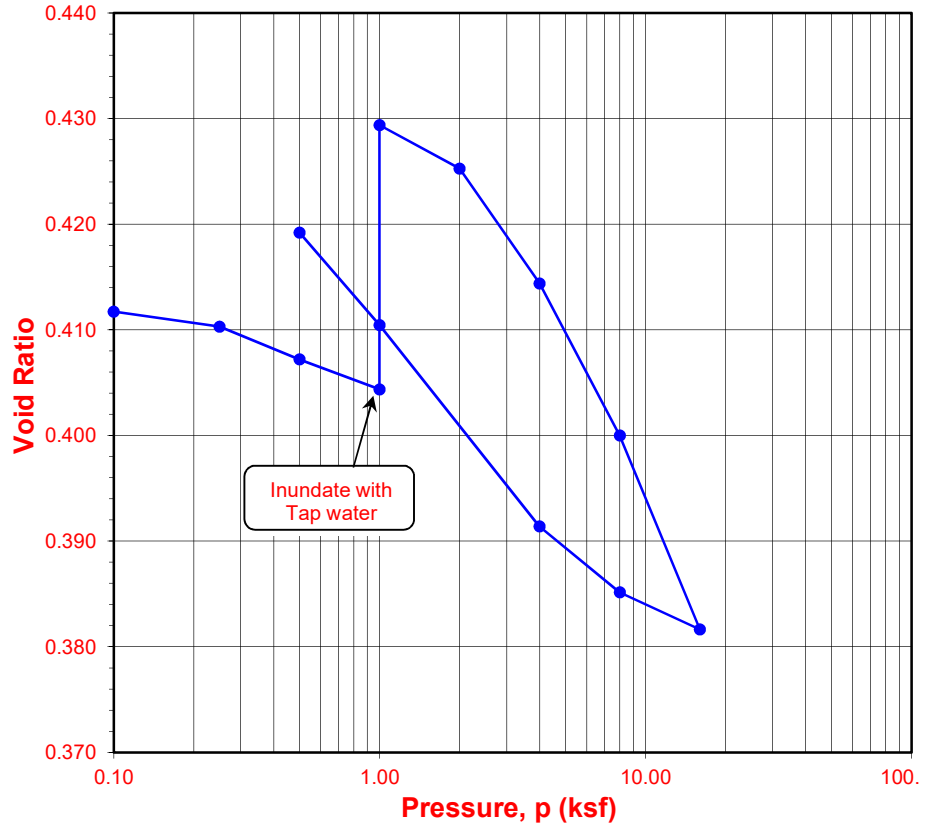
Tested By: G. Bathala Date: 11/17/20

Checked By: A. Santos Date: 11/24/20

Depth (ft.): 5.0

Sample Type: Ring

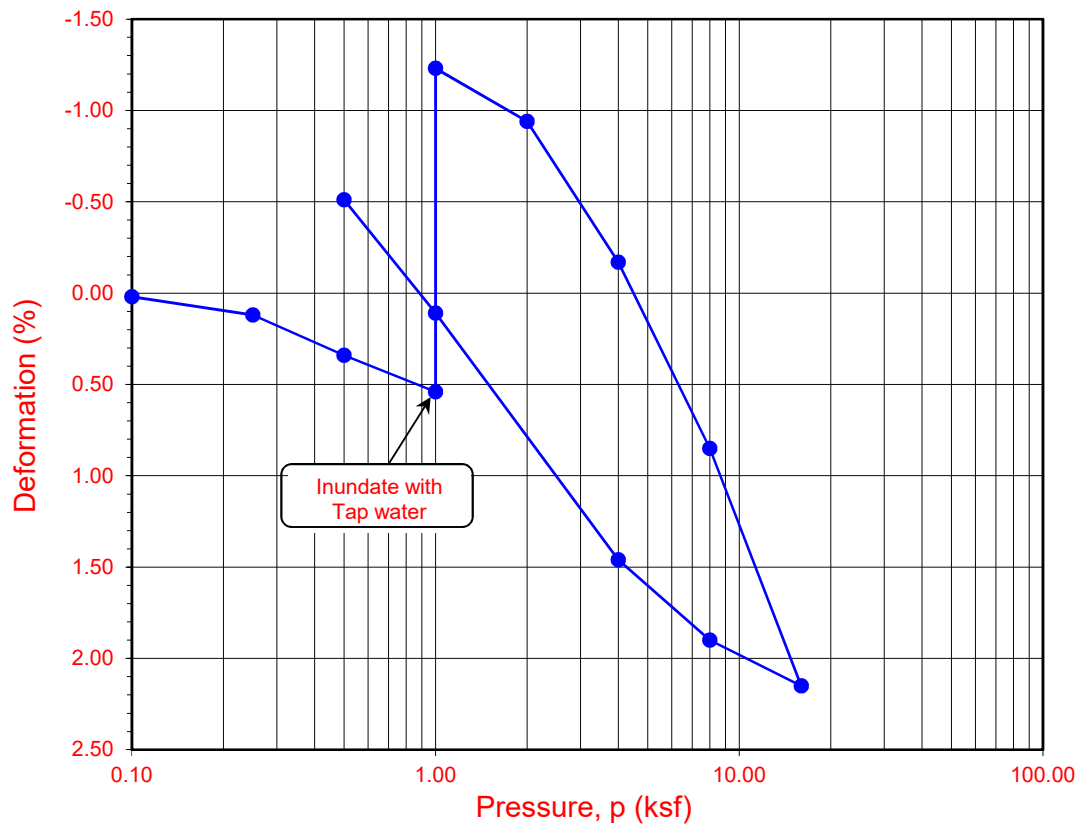
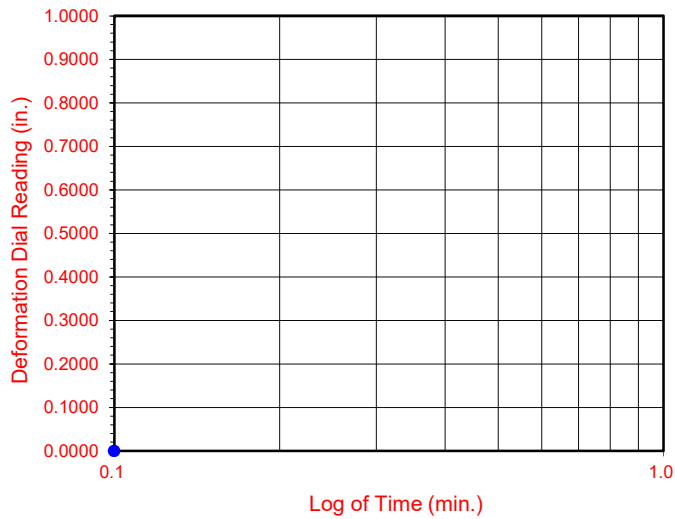
Sample Diameter (in.)	2.415
Sample Thickness (in.)	1.000
Wt. of Sample + Ring (g)	202.10
Weight of Ring (g)	43.22
Height after consol. (in.)	1.0051
Before Test	
Wt.Wet Sample+Cont. (g)	210.54
Wt.of Dry Sample+Cont. (g)	195.96
Weight of Container (g)	59.52
Initial Moisture Content (%)	10.7
Initial Dry Density (pcf)	119.4
Initial Saturation (%)	70
Initial Vertical Reading (in.)	0.3246
After Test	
Wt.of Wet Sample+Cont. (g)	262.63
Wt. of Dry Sample+Cont. (g)	241.43
Weight of Container (g)	55.74
Final Moisture Content (%)	14.88
Final Dry Density (pcf)	117.9
Final Saturation (%)	93
Final Vertical Reading (in.)	0.3254
Specific Gravity (assumed)	2.70
Water Density (pcf)	62.43



Pressure (p) (ksf)	Final Reading (in.)	Apparent Thickness (in.)	Load Compliance (%)	Deformation % of Sample Thickness	Void Ratio	Corrected Deforma- tion (%)
0.10	0.3244	0.9998	0.00	0.02	0.412	0.02
0.25	0.3227	0.9981	0.07	0.19	0.410	0.12
0.50	0.3199	0.9953	0.13	0.47	0.407	0.34
1.00	0.3171	0.9925	0.21	0.75	0.404	0.54
1.00	0.3348	1.0102	0.21	-1.02	0.429	-1.23
2.00	0.3307	1.0061	0.33	-0.61	0.425	-0.94
4.00	0.3217	0.9971	0.46	0.29	0.414	-0.17
8.00	0.3097	0.9851	0.64	1.49	0.400	0.85
16.00	0.2945	0.9699	0.86	3.01	0.382	2.15
8.00	0.2978	0.9732	0.78	2.68	0.385	1.90
4.00	0.3032	0.9786	0.68	2.14	0.391	1.46
1.00	0.3185	0.9939	0.50	0.61	0.410	0.11
0.50	0.3254	1.0008	0.43	-0.08	0.419	-0.51

[illegible]

Time Readings



Boring No.	Sample No.	Depth (ft.)	Moisture Content (%)		Dry Density (pcf)		Void Ratio		Degree of Saturation (%)	
			Initial	Final	Initial	Final	Initial	Final	Initial	Final
HS-5	R-2	5.0	10.7	14.9	119.4	117.9	0.412	0.419	70	93

Soil Identification: Olive brown lean clay (CL)

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS ASTM D 2435

Project No.: 20145-01

Temecula

Appendix D
Infiltration Test Results

Infiltration Test Data Sheet

LGC Geotechnical, Inc

131 Calle Iglesia Suite 200, San Clemente, CA 92672 tel. (949) 369-6141

Project Name: Temecula - Winery
Project Number: 20145-01
Date: 11/13/2020
Boring Number: I-1

Test hole dimensions (if circular)

Boring Depth (feet)*: 15
Boring Diameter (inches): 8
Pipe Diameter (inches): 3

*measured at time of test

Test pit dimensions (if rectangular)

Pit Depth (feet): _____
Pit Length (feet): _____
Pit Breadth (feet): _____

Pre-Test (Sandy Soil Criteria)*

Trial No.	Start Time (24:HR)	Stop Time (24:HR)	Time Interval (min)	Initial Depth to Water (feet)	Final Depth to Water (feet)	Total Change in Water Level (feet)	Greater Than or Equal to 0.5 feet (yes/no)
1	9:35	10:05	30.0	12.14	13.34	1.20	yes
2	10:05	10:35	30.0	12.04	13.28	1.24	yes

*If two consecutive measurements show that six inches of water seeps away in less than 25 minutes, the test shall be run for an additional hour with measurements taken every 10 minutes. Otherwise, pre-soak (fill) overnight, and then obtain at least twelve measurements per hole over at least six hours (approximately 30 minute intervals) with a precision of at least 0.25 inches

Main Test Data

Trial No.	Start Time (24:HR)	Stop Time (24:HR)	Time Interval, t (min)	Initial Depth to Water, D _o (feet)	Final Depth to Water, D _f (feet)	Change in Water Level, D (feet)	Observed Infiltration Rate(in/hr)
1	10:35	10:45	10.0	12.1	13.15	1.05	5.0
2	10:45	10:55	10.0	11.81	13.04	1.23	5.4
3	10:55	11:05	10.0	11.85	13.05	1.2	5.3
4	11:05	11:15	10.0	11.91	13.02	1.11	4.9
5	11:15	11:25	10.0	11.86	12.99	1.13	4.9
6	11:25	11:35	10.0	11.88	12.98	1.1	4.8
7							
8							
9							
10							
11							
12							

Observed Infiltration Rate (No Factor of Safety) **4.8**

Minimum Factor of Safety

Infiltration Rate (With Factor of Safety)

Sketch:

Notes:

Based on Guidelines from: Riverside County 09/2012

Spreadsheet Revised on: 10/30/2019



Infiltration Test Data Sheet

LGC Geotechnical, Inc

131 Calle Iglesia Suite 200, San Clemente, CA 92672 tel. (949) 369-6141

Project Name: Temecula - Winery
Project Number: 20145-01
Date: 11/13/2020
Boring Number: I-2

Test hole dimensions (if circular)

Boring Depth (feet)*: 5
Boring Diameter (inches): 8
Pipe Diameter (inches): 3

*measured at time of test

Test pit dimensions (if rectangular)

Pit Depth (feet):
Pit Length (feet):
Pit Breadth (feet):

Pre-Test (Sandy Soil Criteria)*

Trial No.	Start Time (24:HR)	Stop Time (24:HR)	Time Interval (min)	Initial Depth to Water (feet)	Final Depth to Water (feet)	Total Change in Water Level (feet)	Greater Than or Equal to 0.5 feet (yes/no)
1	9:39	10:09	30.0	2.99	3.44	0.45	no
2	10:09	10:39	30.0	2.94	3.28	0.34	no

*If two consecutive measurements show that six inches of water seeps away in less than 25 minutes, the test shall be run for an additional hour with measurements taken every 10 minutes. Otherwise, pre-soak (fill) overnight, and then obtain at least twelve measurements per hole over at least six hours (approximately 30 minute intervals) with a precision of at least 0.25 inches

Main Test Data

Trial No.	Start Time (24:HR)	Stop Time (24:HR)	Time Interval, t (min)	Initial Depth to Water, D _o (feet)	Final Depth to Water, D _f (feet)	Change in Water Level, D (feet)	Observed Infiltration Rate(in/hr)
1	10:39	11:09	30.0	2.95	3.34	0.39	0.8
2	11:09	11:39	30.0	2.99	3.38	0.39	0.8
3	11:39	12:09	30.0	2.9	3.24	0.34	0.6
4	12:09	12:39	30.0	2.96	3.3	0.34	0.7
5	12:39	13:09	30.0	2.98	3.29	0.31	0.6
6	13:09	13:39	30.0	2.79	3.12	0.33	0.6
7	13:39	14:09	30.0	2.84	3.15	0.31	0.6
8	14:09	14:39	30.0	2.87	3.21	0.34	0.6
9	14:39	15:09	30.0	2.91	3.23	0.32	0.6
10	15:09	15:39	30.0	2.88	3.21	0.33	0.6
11	15:39	16:09	30.0	2.9	3.21	0.31	0.6
12	16:09	16:39	30.0	2.89	3.19	0.3	0.6

Observed Infiltration Rate (No Factor of Safety) 0.6

Minimum Factor of Safety

Infiltration Rate (With Factor of Safety)

Sketch:

Notes:

Based on Guidelines from: Riverside County 09/2012

Spreadsheet Revised on: 10/30/2019



Appendix E
General Earthwork and
Grading Specifications for Rough Grading

General Earthwork and Grading Specifications for Rough Grading

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). 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 over-excavated as specified in the following section. Scarification shall continue until soils 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 Over-excavation

In addition to removals and over-excavations 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 over-excavated 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. Please see the Standard Details for a graphic illustration. 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 over-excavated 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. Soils 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 soils 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 the geotechnical consultant. 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 soils shall be watered, dried back, blended, and/or mixed, as necessary to attain a 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).

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). 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 sheepfoot 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.

4.5 Compaction Testing

Field tests for moisture content and relative compaction of the fill soils 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 soils embankment. In addition, as a guideline, at least one 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 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), the grading plan, and the Standard Details. The Geotechnical Consultant may recommend additional subdrains 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.

6.0 Excavation

Excavations, as well as over-excavation for remedial purposes, shall be evaluated by the Geotechnical Consultant during grading. Remedial removal depths shown on geotechnical plans are estimates only. The actual extent of removal shall be determined by the Geotechnical Consultant based on the field evaluation of exposed conditions during grading. Where fill-over-cut slopes are to be graded, the cut portion of the slope shall be made, evaluated, and accepted by the Geotechnical Consultant prior to placement of materials for construction of the fill portion of the slope, unless otherwise recommended by the Geotechnical Consultant.

7.0 Trench Backfills

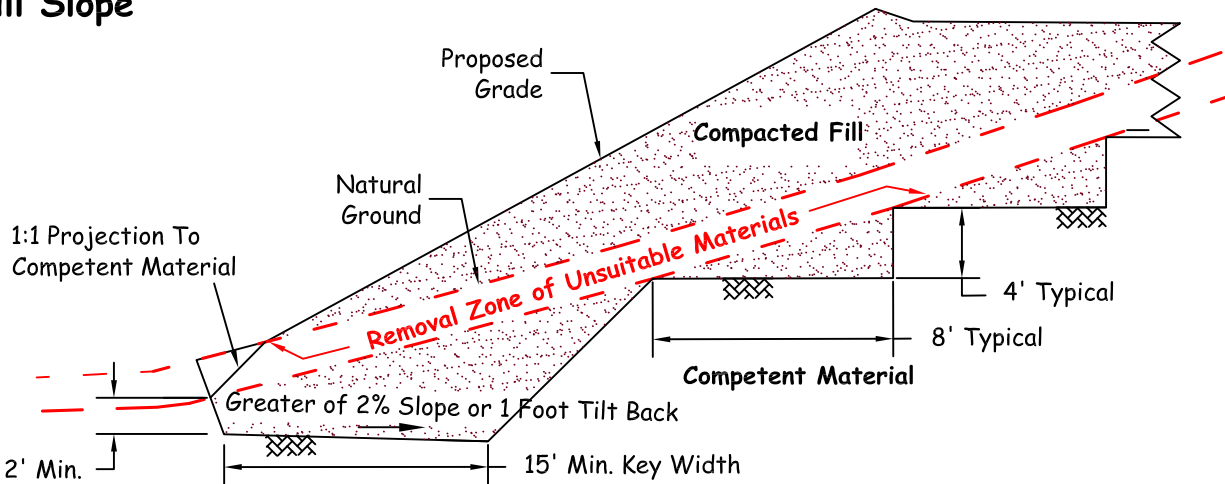
7.1 The Contractor shall follow all OHSA and Cal/OSHA requirements for safety of trench excavations.

7.2 All bedding and backfill of utility trenches shall be done in accordance with the applicable provisions of Standard Specifications of Public Works Construction. Bedding material shall have a Sand Equivalent greater than 30 (SE>30). The bedding shall be placed to 1 foot over

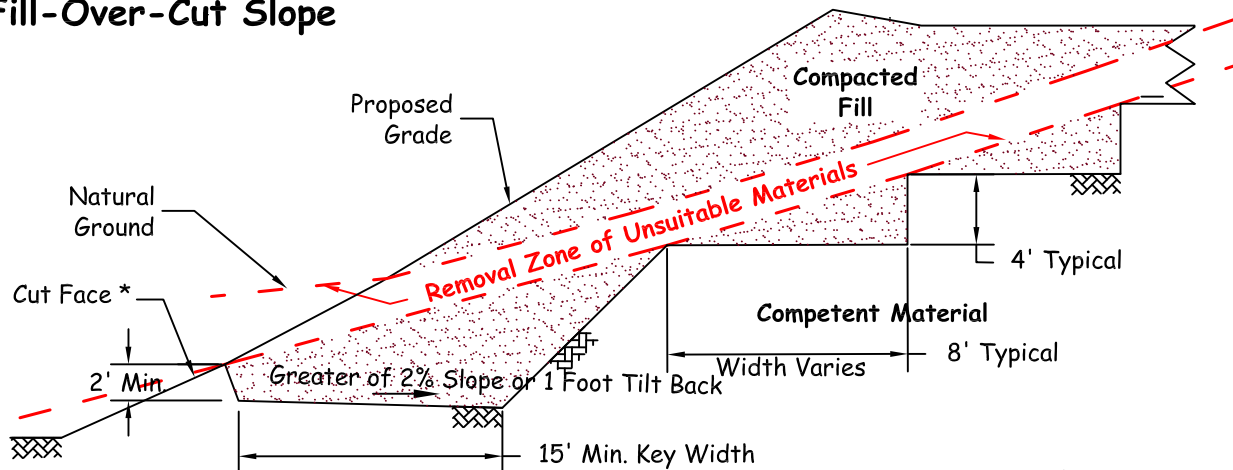
the top of the conduit and densified by jetting. Backfill shall be placed and densified to a minimum of 90 percent of maximum from 1 foot above the top of the conduit to the surface.

- 7.3** The jetting of the bedding around the conduits shall be observed by the Geotechnical Consultant.
- 7.4** The Geotechnical Consultant shall test the trench backfill for relative compaction. At least one test should be made for every 300 feet of trench and 2 feet of fill.
- 7.5** Lift thickness of trench backfill shall not exceed those allowed in the Standard Specifications of Public Works Construction unless the Contractor can demonstrate to the Geotechnical Consultant that the fill lift can be compacted to the minimum relative compaction by his alternative equipment and method.

Fill Slope

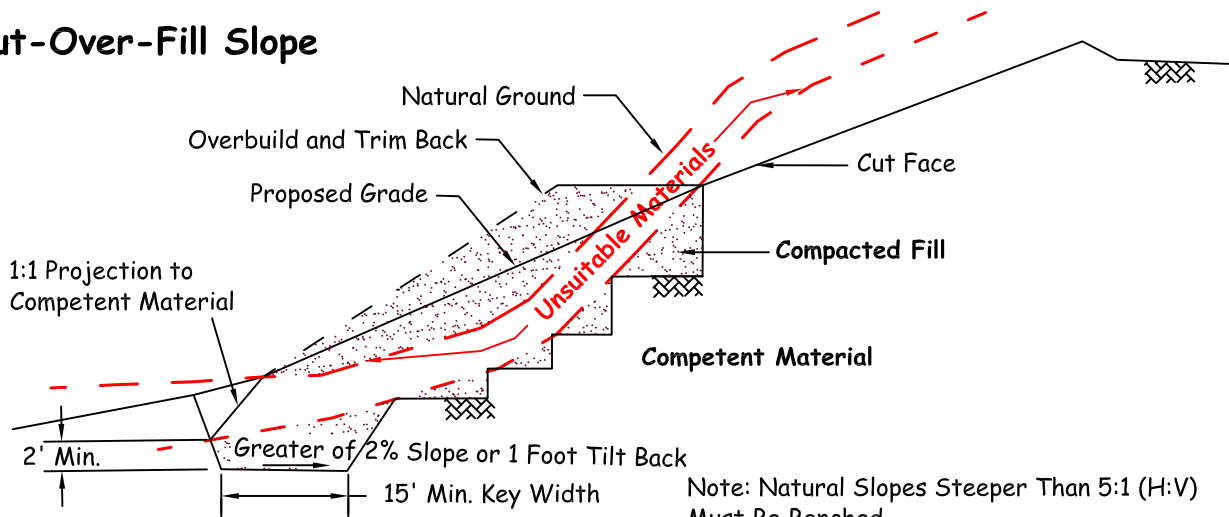


Fill-Over-Cut Slope

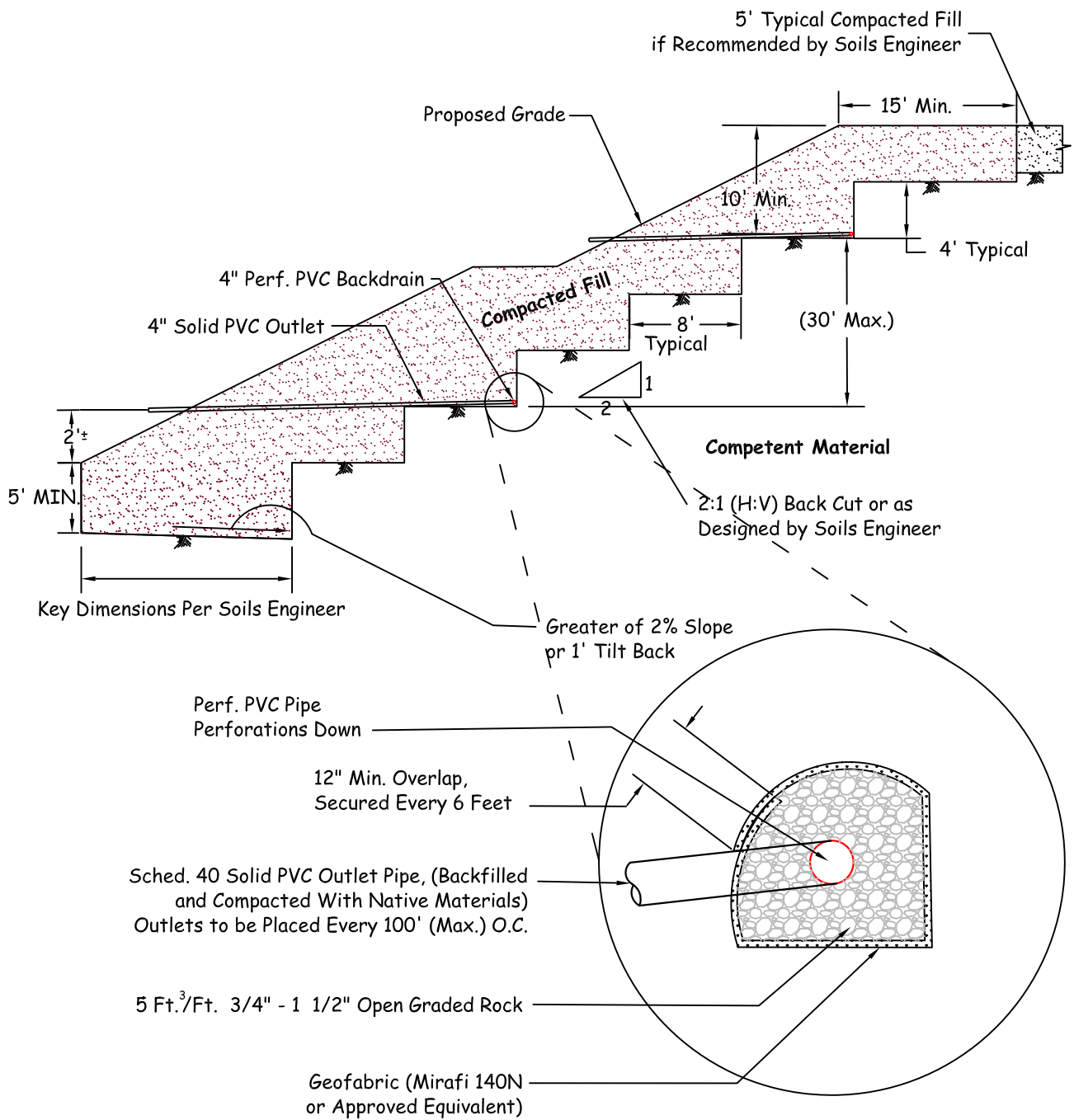


* Construct Cut Slope First

Cut-Over-Fill Slope



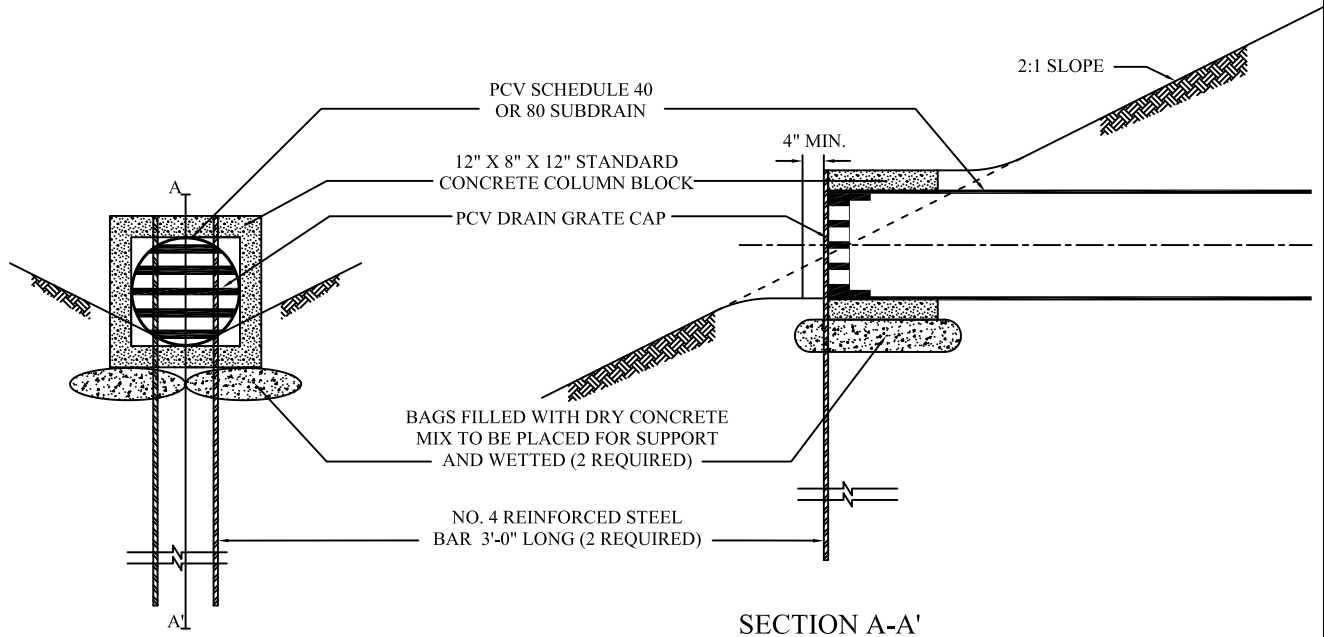
Note: Natural Slopes Steeper Than 5:1 (H:V) Must Be Benched.



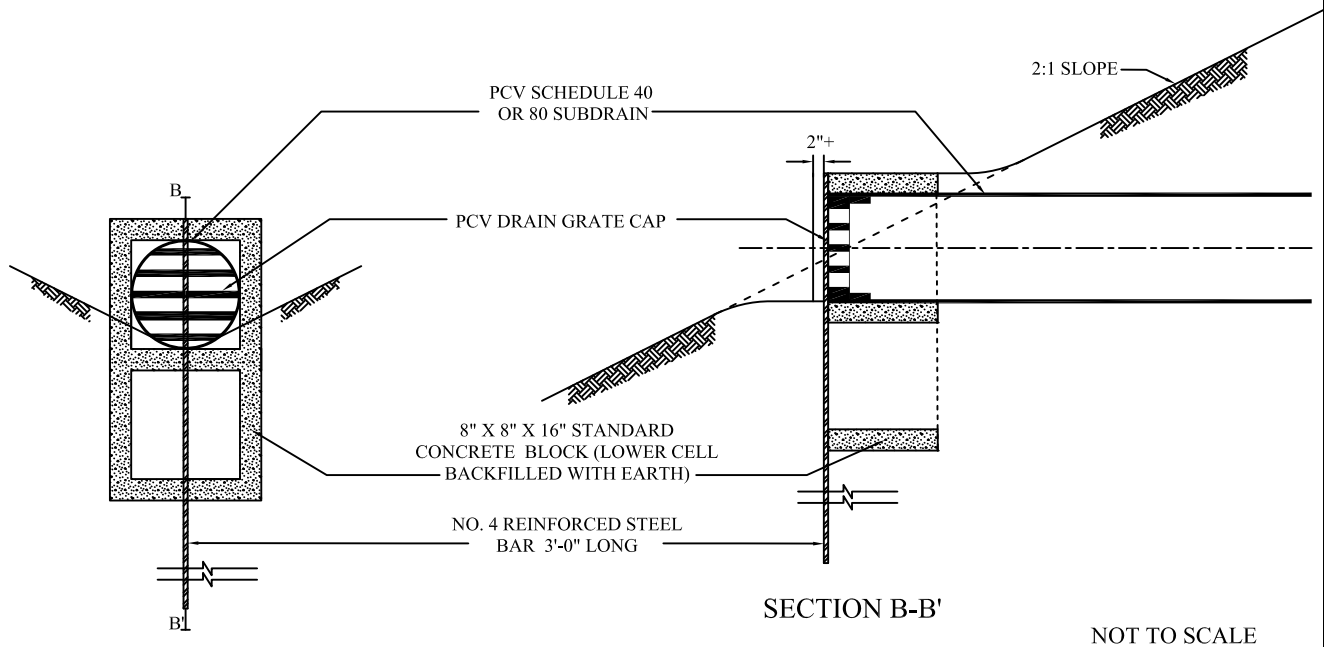


TYPICAL STABILIZATION FILL DETAIL

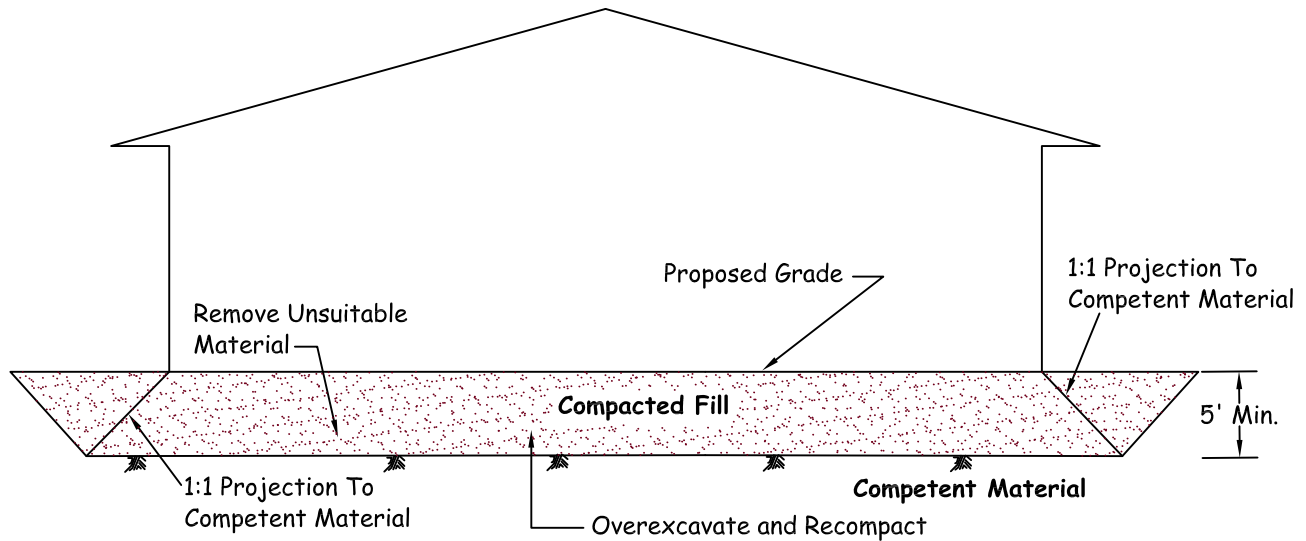
SUBDRAIN OUTLET MARKER -6" & 8" PIPE



SUBDRAIN OUTLET MARKER -4" PIPE



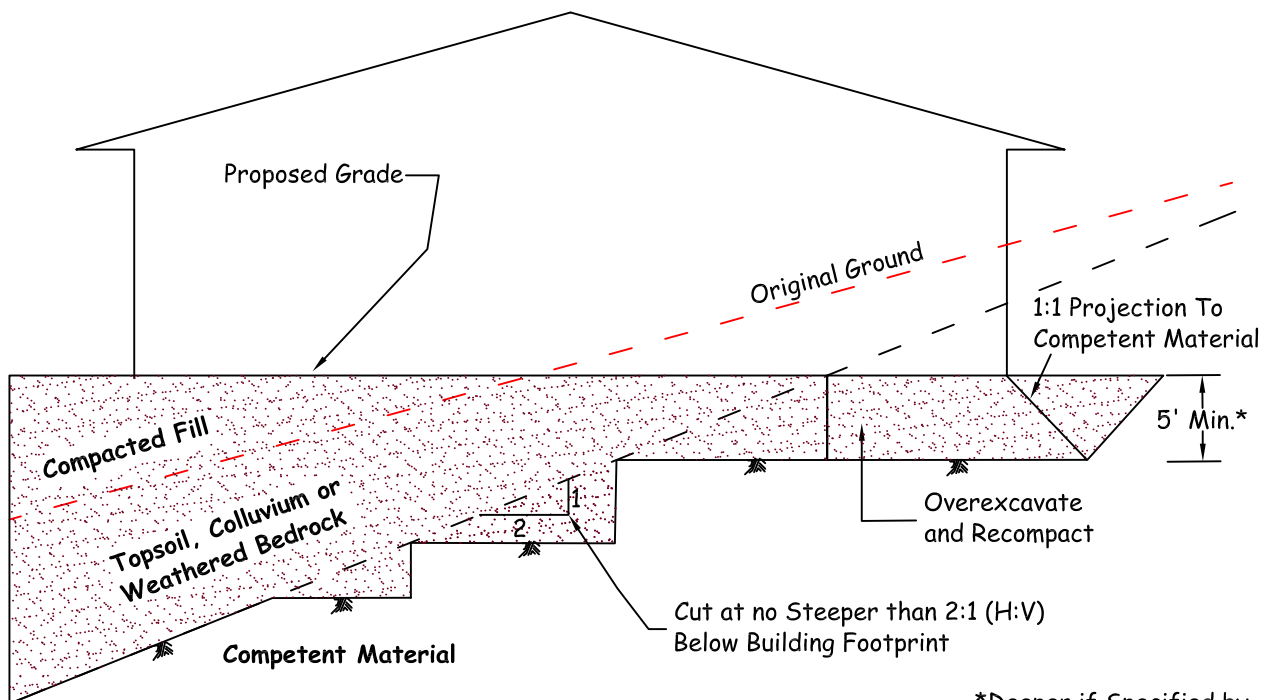
Cut Lot (Exposing Unsuitable Soils at Design Grade)



Note 1: Removal Bottom Should be Graded With Minimum 2% Fall Towards Street or Other Suitable Area (as Determined by Soils Engineer) to Avoid Ponding Below Building

Note 2: Where Design Cut Lots are Excavated Entirely Into Competent Material, Overexcavation May Still be Required for Hard-Rock Conditions or for Materials With Variable Expansion Characteristics.

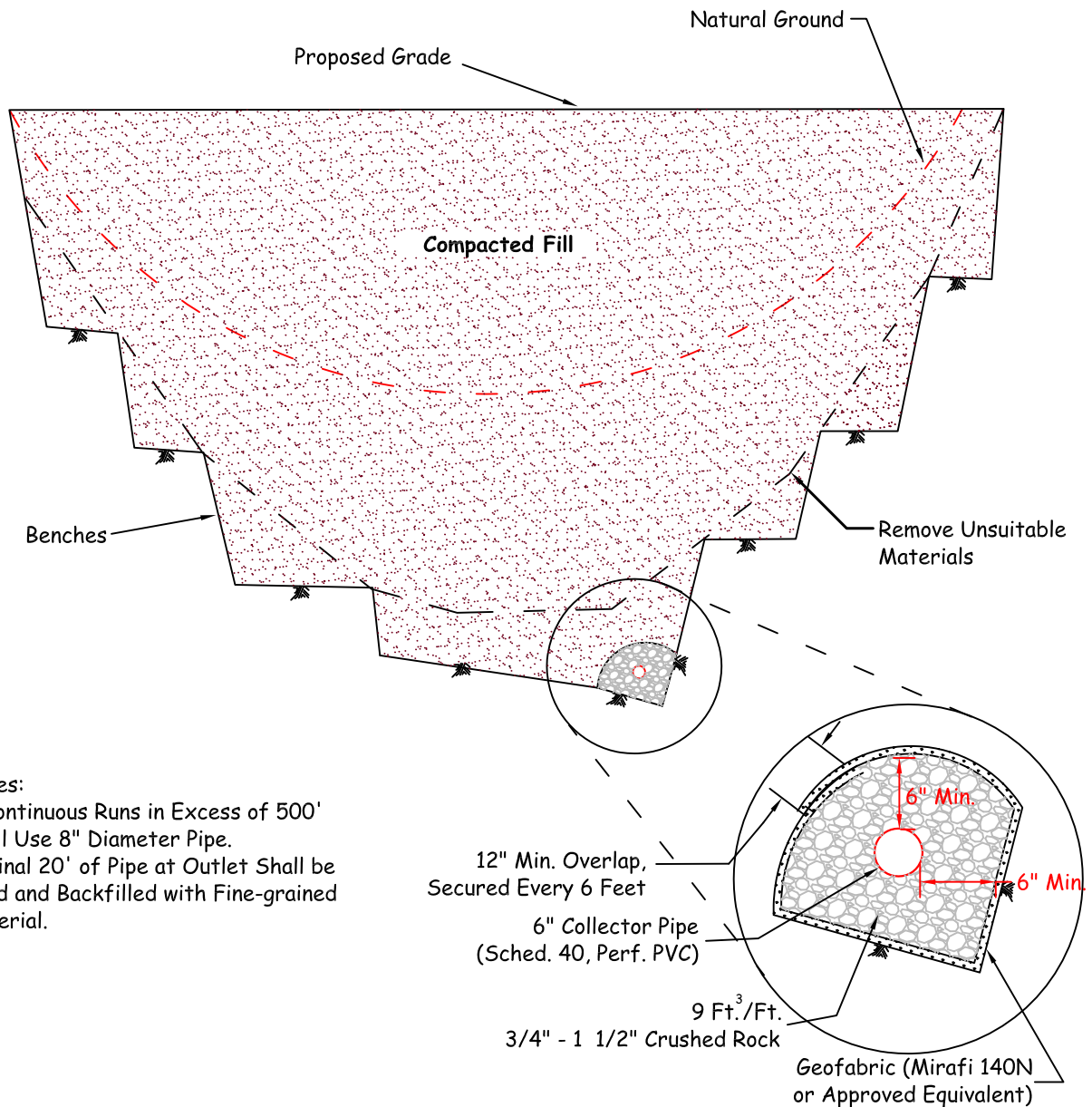
Cut/Fill Transition Lot



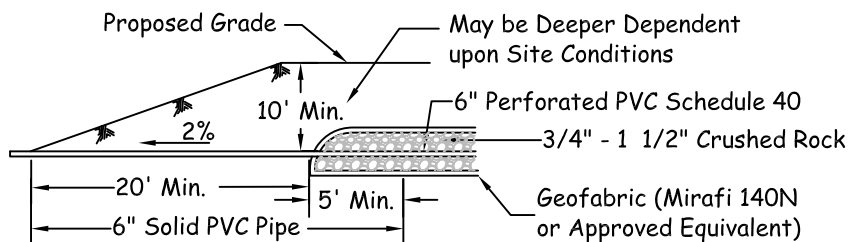
*Deeper if Specified by Soils Engineer

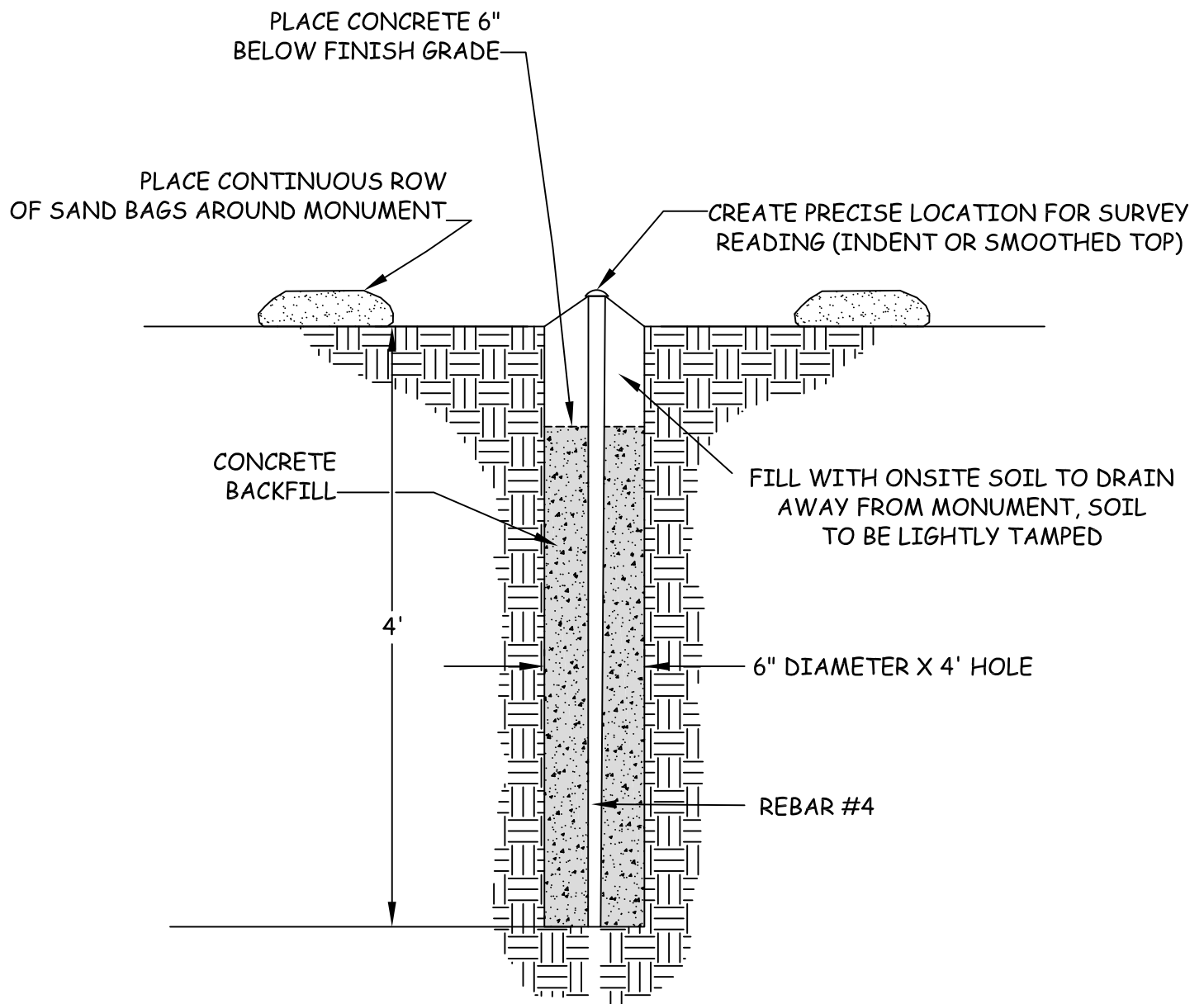


CUT AND TRANSITION LOT OVEREXCAVATION DETAIL



Proposed Outlet Detail

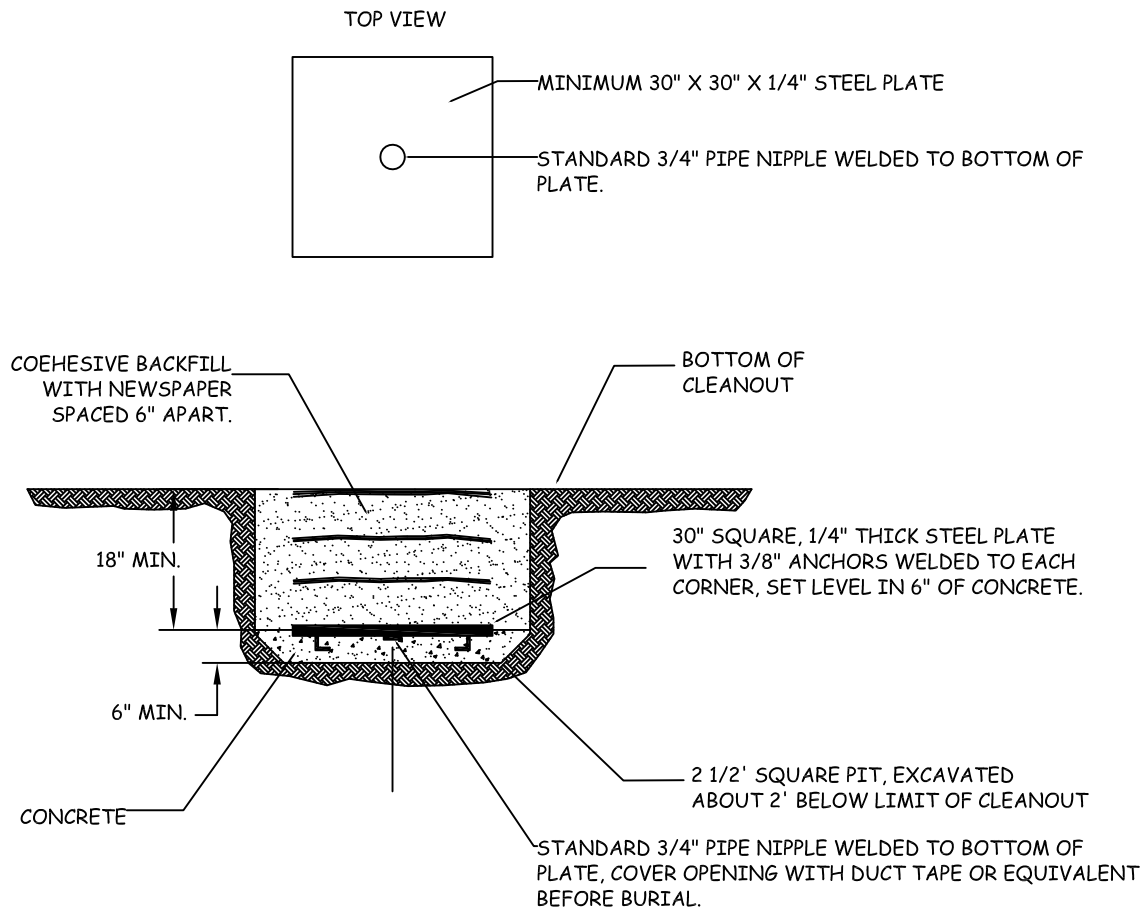




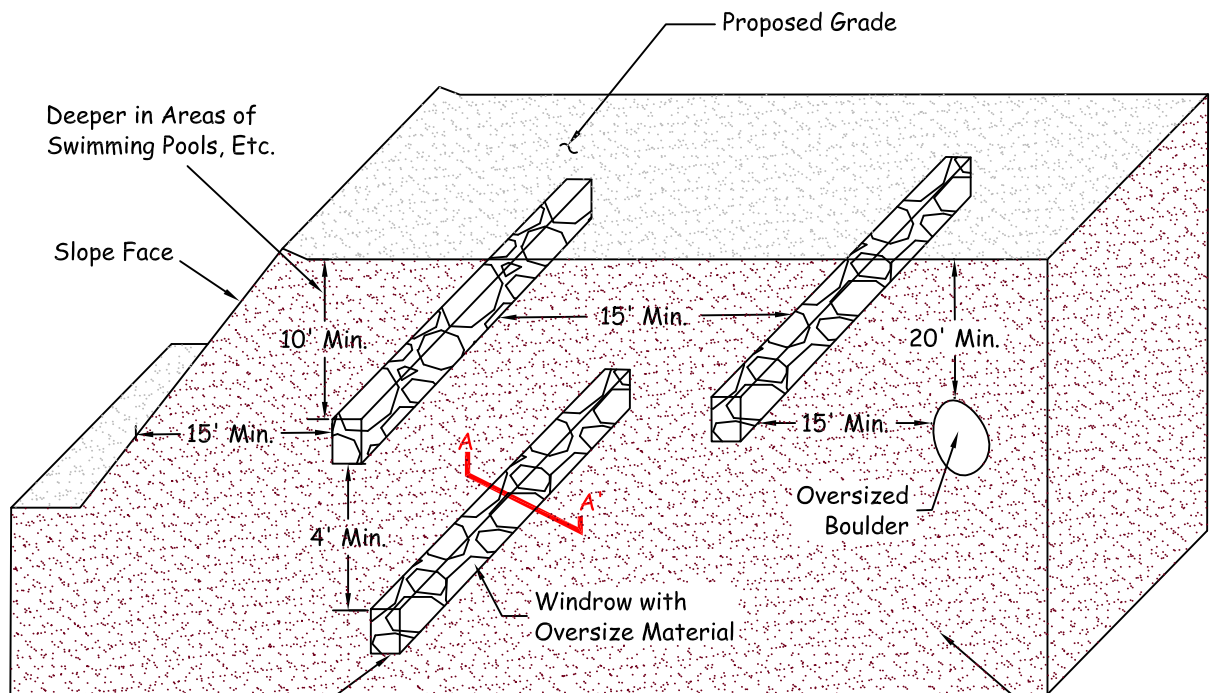
NO CONSTRUCTION EQUIPMENT WITHIN 25 FEET
OF ANY INSTALLED SETTLEMENT MONUMENTS



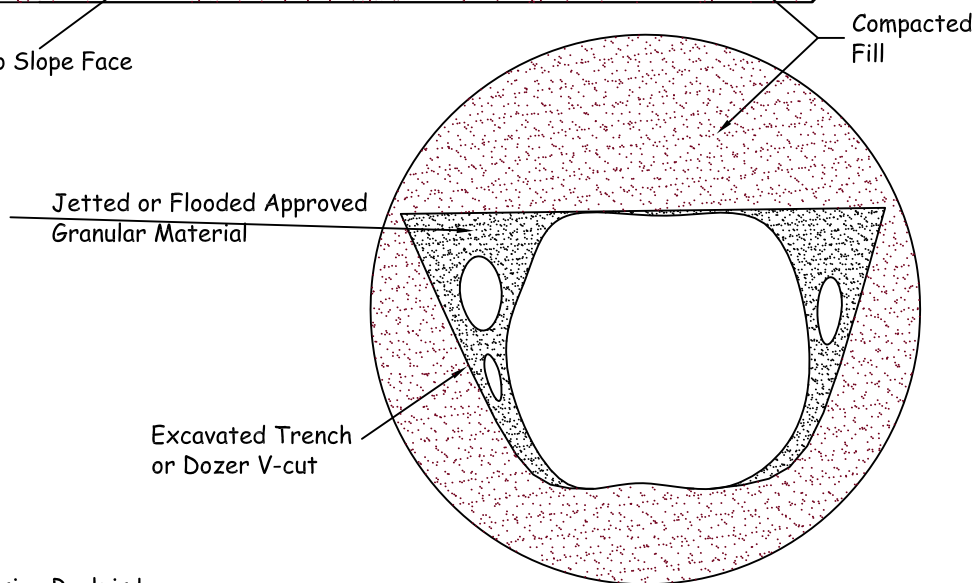
TYPICAL SURFACE SETTLEMENT MONUMENT



1. SURVEY FOR HORIZONTAL AND VERTICAL LOCATION TO NEAREST .01 INCH PRIOR TO BACKFILL USING KNOW LOCATIONS THAT WILL REMAIN INTACT DURING THE DURATION OF THE MONITORING PROGRAM. KNOW POINTS EXPLICITLY NOT ALLOWED ARE THOSE LOCATED ON FILL OR THAT WILL BE DESTROYED DURING GRADING.
2. IN THE EVENT OF DAMAGE TO SETTLEMENT PLATE DURING GRADING, CONTRACTOR SHALL IMMEDIATELY NOTIFY THE GEOTECHNICAL ENGINEER AND SHALL BE RESPONSIBLE FOR RESTORING THE SETTLEMENT PLATES TO WORKING ORDER.
3. DRILL TO RECOVER AND ATTACH RISER PIPE.



Windrow Parallel to Slope Face



Note: Oversize Rock is Larger than 8" in Maximum Dimension.

Section A-A'