ENGEN CORPORATION
Geotechnical & Environmental Services

Geotechnical Feasibility Study

Collier Avenue Project
(APN 389-220-003 through APN 389-220-006)

Project Number: 4626GS

February 1, 2021

41625 Enterprise Circle S., B-2
Temecula, CA 92590

info@engencorp.com

engencorp.com

ph | 951.296.3511

fx | 951.240.3380

SDVOSB | DVBE

In Memory Of
Chris Stevens
Tyrone Woods
Ann Smedingoff
Glen Doherty
Sean Smith

Prepared for:

Saddleback Associates Inc.
27405 Puerta Real, #120
Mission Viejo, California 92691

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February 1, 2021

Mr. Mark Severson
Saddleback and Associates, Inc.
27405 Puerta Real, Suite #120
Mission Viejo, California 92691

Subject: **Geotechnical Feasibility Study**
Collier Avenue Project, APN 389-220-003 through APN 389-220-006
Project Number: 4626GS

References: 1. **IE Surveying & Engineering**, Precise Grading Plan, APN 389-220-003 through APN 389-220-004, Saddleback Industrial, Lake Elsinore, CA, dated: December 8, 2020, scale: 1"=40'

Mr. Severson:

In accordance with your request and signed authorization, a representative of this firm has visited the subject site on November 13 and 16, 2020 to visually observe the surface conditions of the subject site, perform subsurface exploration and testing and collect samples of representative site earth materials. Laboratory testing was performed on these samples. Recommendations for grading operations and preliminary foundation design are provided in the subsequent sections of this report.

1.0 EXECUTIVE SUMMARY

1.1 Feasibility for development:

It is the opinion of this firm the proposed improvements are feasible from a geotechnical standpoint, provided that the recommendations presented in this report are incorporated in the design and construction of the project.

1.2 Demolition Operations:

All demolition operations should be conducted under the observation and documentation and testing of the project geotechnical engineer. **Failure to coordinate the demolition operations with the project geotechnical consultant of record may result in additional fieldwork beyond that represented herein**. It is the owner or the owner's authorized representative responsibility to ensure that the geotechnical consultant is informed of the demolition operations so that a qualified representative can be dispatched for observation and testing.

1.3 Unsuitable Soils:

The site is underlain by undocumented man-made fill, alluvium, Pauba Formation and Santiago Formation bedrock. A suspected trash pit that is considered undocumented fill was discovered near the center of the site. The undocumented fill, and upper portions of the alluvium are considered unsuitable for support and they should be removed to competent alluvium or competent bedrock in the areas of the proposed development.

1.4 Expansive and Corrosive Soils:

Based on the remediation recommendations for site grading operations, the expansive and corrosive properties of the soils that will be in contact with and supporting concrete foundations and slabs is not known. However, based on the laboratory test results conducted on representative samples of the on-site earth materials, we anticipate low expansive non-corrosive properties.

2.0 SITE AND PROJECT DESCRIPTION

2.1 General:

The site consists of an approximately 7.5-acres irregularly shaped lot located approximately 225 to 650 feet northwest of Riverside Drive, on the northeast side of Collier Avenue and the southwest side of

FIGURE 1 - SITE LOCATION MAP

terminus of El Toro Road, in the City of Lake Elsinore, County of Riverside, California. Topographic relief across the site is approximately 20-feet. A prominent flat knob is located at the northern corner of the site while the remainder of the site is very gently sloping. Site drainage is generally through sheet flow to the southwest toward Collier Avenue. Vegetation consisted of a moderate cover of weeds and grasses, along with scattered bushes, and several mature trees. Two areas within the property were enclosed with chain link fences at the time of our investigation, both adjacent to the east side of the site. The southerly fenced area appeared in a mostly natural state. The central fenced area appears to have been utilized as a construction storage yard. At the time of our investigation the ground surface within the central fenced area was covered by crushed rock, and various construction materials were stored within the fenced area, including lumber, various diameter plastic pipes, traffic control devices, and assorted hardware.

2.2 Background:

Based on a review of historical aerial photos, it appears that the knob at the northern corner had been previously developed with a single-family residence, with associated out buildings, free standing shade structures, a driveway and low retaining walls as recently as 2018. At the time of our investigation the structures had been removed, however, the driveway, concrete slabs and retaining walls, along with some wooden posts and supports for the shade structures remained.

2.3 Existing Septic System and Water Well:

A vitrified clay pipe was encountered on the knob. This clay pipe likely is connected to a septic system on the property. What appears to be a pressure vessel for a water well was observed near western corner of the site.

2.4 Project Description:

Based on our review of the grading plan, it is our understanding that the proposed earthwork will include typical cut and fill type grading. All cut and fill slopes are planned to be constructed at a ratio of 2:1 (horizontal to vertical) or flatter. The proposed development will consist of 6 commercial/light industrial tilt-up or steel-framed structures constructed with a slab on grade foundations, along with associated hardscape and landscape improvements.

Foundation plans were not available prior to this writing and should be reviewed by this office once available so that supplemental recommendations can be given if necessary. For the purposes of this report foundations bearing load criteria will be based on the following criteria:

Maximum Structure Bearing Loads	
Description	**Maximum Loads**
Maximum Wall Loads	2 kips per linear foot
Maximum Column Loads	30 kips
Maximum Floor Slab Pressure	150 pounds per cubic foot
Parking and Traffic Structural Loads (Design Life of 20 Years)	
Description	**Maximum Loads**
Concrete and Asphalt Pavement Areas	Equivalent Single Axle Loads = 18 kips
Concrete and Asphalt Pavement Areas	Maximum Loads = 60,000

It is represented that the proposed development will include infrastructure such as street, storm drains and utility improvements.

2.5 Scope of Work:

The scope of this study was to provide a preliminary geotechnical assessment of the surface and subsurface conditions within the proposed development area, and to provide recommendations for the development of the site from a geotechnical point of view. The scope included: 1) site reconnaissance and geologic mapping, 2) subsurface exploration and field testing, 3) sampling and laboratory testing of on-site materials, 4) engineering analysis of field and laboratory data, and 5) preparation of this report.

2.6 Field Study:

Field Reconnaissance: Field reconnaissance, geologic mapping and subsurface exploration was conducted at the subject property on November 13 and 16, 2020. The purpose of the subsurface exploration was to assess the underlying earth materials' existing condition and geotechnical properties as well as the presence of groundwater.

Borings: Four exploratory borings (B1 through B4) and seven exploratory test pits (TP1 through TP7) were excavated at the study site. The borings were advanced by Martini Drilling using a CME 75 truck-mounted drill rig equipped with 8-inch outside diameter hollow-stem augers. The maximum depth explored was approximately 48-feet below the existing ground surface at the boring locations. Bulk and relatively undisturbed samples of the earth materials encountered were obtained at various depths in the exploratory borings and transported to our soils laboratory for verification of field classifications and testing. Bulk samples were obtained from cutting developed during the excavation process and represent a mixture of the soils within the depth indicated on the logs. Relatively undisturbed samples of the earth materials encountered were obtained by driving a thin-walled steel sampler lined with 1.0-inch high, 2.42-inch inside diameter brass rings. Disturbed samples were obtained at various depths within the borings utilizing a Standard Penetration Test (SPT) sampler. The samplers were driven with successive drops of a 140-pound weight having a free fall of approximately 30-inches. The blow counts for each successive 6.0-inches of penetration, or fraction thereof, are shown in the Geotechnical Boring Logs presented in the Appendix. The ring samples were retained in close-fitting moisture-proof containers and transported to our laboratory for testing.

Test Pits: The test pits were excavated by Tiger Equipment Grading & Excavation utilizing a Caterpillar 420D wheel-mounted backhoe. The maximum test pit excavation depth was 14-feet below the existing ground surface. In-situ moisture and density were determined at various depths within the test pits utilizing a nuclear moisture-density gauge. Bulk soil samples were collected from the cuttings generated during the test pit excavation. The approximate locations of the exploratory borings and test pits are shown on the Geotechnical Site Plan (Plate 1).

2.7 Exploratory Test Pit Backfill Compaction:

The exploratory test pits were backfilled with loose soil cuttings after completion of logging, testing and sampling operations. No compaction efforts were applied during the backfill operations, and tests were not performed to determine the compaction of the backfilled material. The exploratory test pit backfill should be removed and re-compacted during grading and verified as meeting a minimum density of the surrounding earth materials within the body of the final grading report for the proposed project.

3.0 <u>LABORATORY TESTING</u>

3.1 General:

The results of laboratory tests performed on samples of earth material obtained during the site visit are presented in the attached Exhibits. Following is a listing and brief explanation of the laboratory tests performed. The samples obtained during the field study will be discarded 30 days after the date of this report. This office should be notified immediately if retention of samples will be needed beyond 30 days.

3.2 Classification:

The field classification of soil materials encountered during our site visit were verified in the laboratory in general accordance with the Unified Soils Classification System, ASTM D 2488, Standard Practice for Determination and Identification of Soils (Visual-Manual Procedures). The final classification is shown in the Moisture Density Test Report presented in the Appendix.

3.3 Maximum Dry Density/Optimum Moisture Content Relationship Test:

Maximum dry density/optimum moisture content relationship determinations were performed on samples of near-surface earth material in general accordance with ASTM 1557 procedures using a 4.0-inch diameter mold. Samples were prepared at various moisture contents and compacted in five (5) layers using a 10-pound weight dropping 18-inches and with 25 blows per layer. A plot of the compacted dry density versus the moisture content of the specimens is constructed and the maximum dry density and optimum moisture content determined from the plot. The plot is shown in the Moisture Density Test Report presented in the Appendix.

3.4 Expansion Test:

Laboratory expansion tests were performed on samples of near-surface earth material in general accordance with CBC 18-2. In this testing procedure, a remolded sample is compacted in two (2) layers in a 4.0-inch diameter mold to a total compacted thickness of approximately 1.0-inch by using a 5.5-pound weight dropping 12-inches and with 15 blows per layer. The sample should be compacted at a saturation between 49 and 51 percent. After remolding, the sample is confined under a pressure of 144 pounds per square foot (psf) and allowed to soak for 24 hours. The resulting volume change due to the increase in moisture content within the sample is recorded and the Expansion Index (EI) calculated.

3.5 Direct Shear Test:

Direct shear tests were performed on select samples of near-surface earth material in general accordance with ASTM D 3080 procedures. The shear machine is of the constant strain type. The shear machine is designed to receive a 1.0-inch high, 2.42-inch diameter ring sample. Specimens from the sample were sheared at various pressures normal to the face of the specimens. The specimens were tested in a submerged condition. The maximum shear stresses were plotted versus the normal confining stresses to determine the shear strength (cohesion and angle of internal friction).

3.6 Grain Size Distribution Test:

An evaluation was performed on selected representative soil samples in general accordance with ASTM D 422. This “grain-size” or “sieve analysis” test method determines the distribution of particle sizes in soils which allows for the proper classification according to the Unified Soils Classification System (USCS). In this test procedure, a weighed sample is processed through multiple sieves designated by their size generally ranging from a No. 4 (0.25-inch) to a No. 200 sieve by means of a lateral and vertical motion of the sieve on a mechanical shaker. The percentage of material passing each sieve is weighed and recorded with the results plotted in graph form.

3.7 Hydrometer Analysis:

An evaluation was performed on selected representative soil samples in general accordance with ASTM D 7928. This "particle size" or "gradation" test method determines the distribution of fine-grained particle sizes by means of the sedimentation hydrometer analysis which separates silts and clay fractions. In this procedure the particle size distribution of material that finer than No. 200 sieve are determined, and results are presented as the mass percent finer versus the log of particle diameter.

3.8 Consolidation Test:

Settlement predictions of the on-site soil and compacted fill behavior under load were made, based on consolidation tests that were performed in general accordance with ASTM D 2435 procedures. The consolidation apparatus is designed to receive a 1.0-inch high, 2.416-inch diameter ring sample. Porous stones are placed in contact with the top and bottom of each specimen to permit addition and release of pore water and pore pressure. Loads normal to the face of the specimen are applied in several increments in a geometric progression under both field moisture and submerged conditions. The resulting changes in sample thickness are recorded at selected time intervals. Water was added to the test apparatus at various loads to create a submerged condition and to measure the collapse potential (hydroconsolidation) of the sample. The resulting change in sample thickness was recorded.

3.9 In-Situ Moisture Content and Density Test:

The in-situ moisture content and dry density were determined in general accordance with ASTM D 2216 and ASTM D 2937 procedures, respectively, for each selected undisturbed sample obtained. The dry density is determined in pounds per cubic foot and the moisture content is determined as a percentage of the oven dry weight of the soil.

3.10 In-Situ Moisture Content and Density Test (Nuclear Method):

Relative compaction testing was performed in general accordance with ASTM D 2922 and ASTM D 3017 procedures for determining in-place density and moisture content, respectively, using nuclear density gauge equipment.

3.11 R-Value:

An evaluation was performed on a selected representative soil sample in general accordance with California Test Method 301. The resistance (R-Value) test method is used to measure the potential strength of subgrade, subbase, and base course materials for use in road pavements.

3.12 Soluble Sulfate Test:

Samples of the near-surface earth materials were obtained for soluble sulfate testing for the site. The concentration of soluble sulfates was determined in the general conformance with California Test Method 417 procedures.

3.13 pH/Minimum Resistivity:

Samples Sample(s) of near surface soils were tested for pH and minimum resistivity in general accordance with CTM 643.

3.14 Chloride Content:

Sample(s) of near surface soils were tested for chloride content in general conformance with CTM 422.

4.0 FINDINGS

4.1 Site Review:

At the time of our recent field study, no permanent structures were located on the site. Most of the site is nearly flat with a low knob at the northern corner of the site. It appears that previous structures that had once been located at the northern corner of the site had been removed, leaving remnants such as slabs, a driveway, retaining walls, wooden posts, etc. A water well, and likely a septic system are also located on the site. Vegetation on the site consisted of grasses, and bushes, as well as mature trees.

4.2 Subsurface Soil Profile:

Undocumented fill and alluvial deposits are exposed across the site. Pauba Formation sandstone bedrock underlies the undocumented fill and alluvium beneath the knob at the northern corner of the site, while the sandstone member of the Santiago Formation underlies the undocumented fill and alluvium in the remaining flat portions of the site. Excavation in the undocumented fill and alluvium is expected to be relatively easy with conventional heavy grading equipment. Based on our experience on similar projects near the subject site, the Pauba Formation bedrock is expected to be moderate to difficult ripping within the upper 6 feet. Based on the expected depths of removal, Santiago Formation will not likely be encountered during grading. A more detailed description of the earth materials encountered at the site are presented in the **Earth Materials** section of this report. The exploratory boring and test pit logs of earth materials encountered during the subsurface exploration are included in the Appendix.

4.3 Transition Areas:

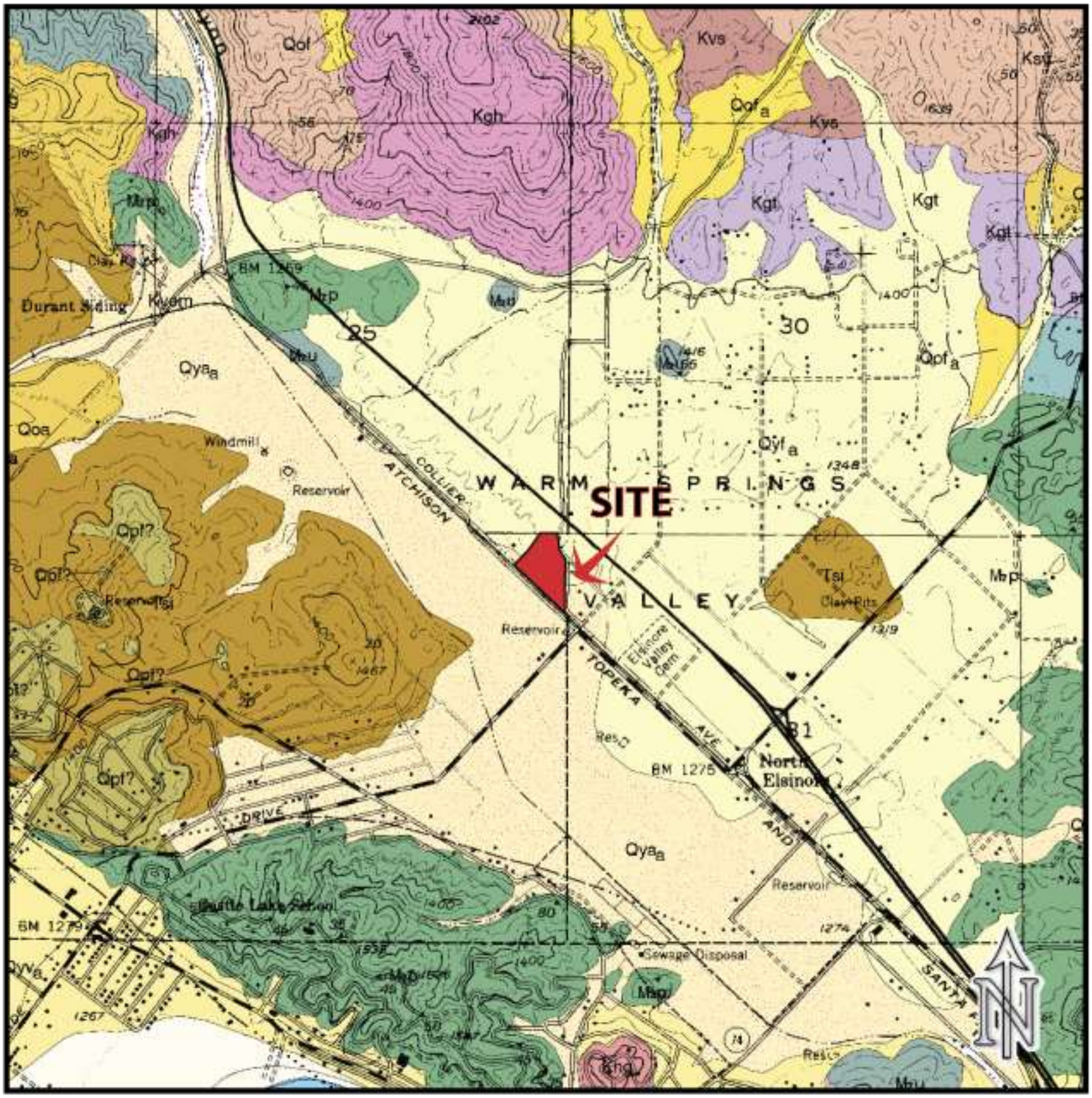
Transitions between cut and fill areas on the building pads are identified on the Referenced No. 1 grading plan. To guard against potential differential settlement, the footprint of the future structures should be over-excavated and recompacted to a minimum of 90 percent relative compaction (See § 7.1).

5.0 GEOLOGY AND SEISMICITY

5.1 Geologic Setting:

The site is located in the Northern Peninsular Range on the southern sector of the structural unit known as the Perris Block. The Perris Block is bounded on the northeast by the San Jacinto Fault Zone, on the southwest by the Elsinore Fault Zone, and on the north by the Cucamonga Fault Zone. The southern boundary of the Perris Block is believed to coincide with a complex group of faults trending southeast from the Murrieta, California area (Kennedy, 1977). The Peninsular Range is characterized by large Mesozoic age intrusive rock masses flanked by volcanic, metasedimentary, and sedimentary rocks. Various thicknesses of colluvial/alluvial sediments derived from the erosion of the elevated portions of the region fill the low-lying areas.

FIGURE 2 - REGIONAL GEOLOGIC MAP



Base Map: Preliminary Geologic Map of the the Elsinore 7.5' Quad., Riv Co, Open File Rpt. 03-281, Scale 1:24,000, DR Morton, 1978, 1998

LEGEND

- Qyw = **Young Alluviual-Wash deposits (Holocene & late Pleistocene)** - Unconsolidated bouldery to sandy alluvium. Recently active washes
- Qya_a = **Young Alluvial Channel deposits (Holocene & late Pleistocene)** - Fluvial deposits along valley floors. Consists of sand, silt, and clay -bearing alluvium
- Qof_a = **Old Alluvial Channel deposits (middle to early Pleistocene)**- Fluvial sediments on broad canyon floors. Consists of moderately to well-indurated, gravel, sand, silt
- Kt = **Tonalite, undifferentiated (Cretaceous)** Gray, medium grained biotite hornblende tolanite, typicaly foliated. Western edge of large body exposed at eastern edge of quadrngle.
- Qpf = **Pauba Formation Sandstone (Pleistocene)** - Siltstone, sandstone and conglomerate. subdivided into two informal members; only sandstone member exposed in quadrangle
- Qvoa = **Very old alluvial channel deposits (middle to early Pleistocene)** - Fluvial sediments deposited on broad canyon floors. Moderate to well-indurated
- Tsi = **Silverado Formation (Paleocene)** - Nonmarine and marine sandstone, siltstone, and conglomerate. Thoroughly weathered. Claymont clay bed underlies conglomerate
- Khg = **Hypabyssal Tonalite (Cretaceous)** - Includes a wide variety of heterogeneous granitic rocks. Mostly of granodioritic and tonalitic composition.

5.2 Seismic Hazards:

Because the proposed development is located in tectonically active southern California, it will likely experience some effects from earthquakes. The type or severity of seismic hazards affecting the site is mainly dependent upon the distance to the causative fault, the intensity of the seismic event, and the soil characteristics. The seismic hazard may be primary, such as surface rupture and/or ground shaking, or secondary, such as liquefaction or dynamic settlement. The following is a site-specific discussion about ground motion parameters, earthquake induced settlement hazards, and liquefaction. The purpose of this analysis is to identify potential seismic hazards and propose mitigations, if necessary, to an acceptable level of risk.

5.3 Seismic Design Parameters:

The 2019 California Building Code (CBC) seismic design parameters for the subject site were obtained from the seismic design mapping web application by the Structural Engineers Association of California (SEAOC) using ASCE 7-16 seismic maps and by using 2019 CBC Tables 11.4-1 and 11.4-2. Site Class D was assumed. Obtaining site-specific shear wave velocities were not within the scope of work, however, they may be obtained on request. The project Structural Engineer should determine the actual footing widths and depths necessary to resist vertical, horizontal, and uplift forces using the following seismic criteria:

Description	**Design Parameters**
Site Latitude:	33.697263° North
Site Longitude:	-117.348679° West
Site Class:	D (default)
Spectral Response – Short (0.2 sec):	S_S = 2.209
Spectral Response - One Second:	S_1 = 0.789
Short Period Site Coefficient:	Fa = 1.2
1-Second Period Site Coefficient:	Fv = 1.7
Adjusted Spectral Response – Short (0.2 sec):	S_{MS} = 2.651
Adjusted Spectral Response – One Second:	S_{M1} = 1.341
Design Spectral Response – Short (0.2 sec):	S_{DS} = 1.768
Design Spectral Response – One Second:	S_{D1} = 0.894

5.4 Surface Fault Rupture:

The fault mapping application “Earthquake Zones of Required Investigation” by the California Geological Survey (CGS) was viewed at https://maps.conservation.ca.gov/cgs/EQZApp. The “Map My County”, version 10, page by the County of Riverside Geographic Information System was viewed at https://gis.countyofriverside.us. Based on those maps the site is not located within an Alquist-Priolo Earthquake Fault Zone or a County Fault Zone. The nearest zoned fault is the Elsinore Fault which trends northwest-southeast and is located approximately 1.25 miles southwest of the site. Based on the County maps an unnamed splay of the Elsinore Fault, which is not within a County Fault Zone, is located approximately 3,300 feet south of the site. The splay also trends generally northwest-southeast. The mapped faulting is oblique to the site and does not trend toward the site. Therefore, no known active faults exist on the subject site or trend toward the site. Accordingly, the potential for fault surface rupture on the site is considered unlikely.

5.5 Liquefaction Evaluation:

The “Map My County” page by the County of Riverside Geographic Information System was reviewed for the site’s susceptibility to liquefaction. Based on information provided by the Riverside County website, the site’s susceptibility to liquefaction is considered very high.

Liquefaction is a phenomenon where a sudden large decrease of shearing resistance takes place in fine-grained cohesionless and/or low plasticity cohesive soils due to the cyclic stresses produced by earthquakes causing a sudden, but temporary, increase of porewater pressure. The increased porewater pressure occurs below the water table, but can cause propagation of groundwater upward into overlying soil and possibly to the ground surface and cause sand boils as excess porewater escapes. Potential hazards due to liquefaction include significant total and/or differential settlements of the ground surface and structures as well as possible collapse of structures due to loss of support of foundations. It has been shown by laboratory testing and from the analysis of soil conditions at sites where liquefaction has occurred that the soil types most susceptible to liquefaction are saturated, fine sand to sandy silt with a mean grain size ranging from approximately 0.075mm to 0.5mm. These soils derive their shear strength from intergranular friction and do not drain quickly during earthquakes. Published studies and field and laboratory test data indicate that coarse sands and silty or clayey sands beyond the above-mentioned grain size range are considerably less vulnerable to liquefaction. To a large extent, the relative density of the soil also controls the susceptibility to liquefaction for a given number of cycles and acceleration levels during a seismic event. Other characteristics such as confining pressure and the stresses created within the soil during a seismic event also affect the liquefaction potential of a site. Liquefaction of soil does not generally occur at depths of 40 to 50-feet below ground surface due to the confining pressure at that depth. The potential for liquefaction of the site is considered to be high due to the following conditions:

1. The existence of nearby major active faults may cause exceptionally high ground accelerations at the site.
2. The fine-grained nature (fine- to medium-grained silty sands) of the earth materials encountered make them susceptible to liquefaction.
3. Low to medium relative densities of some of the in-situ soils above and below the groundwater table.
4. Relatively shallow (up to 9-feet below ground surface) groundwater was encountered.

Settlement: The total potential settlement in the event of liquefaction has been calculated at 10.8-inches, assuming a groundwater maximum elevation of 9-feet below ground surface, and no mitigation measures are undertaken. The proposed 10-foot minimum blanket of engineered fill in the alluvial areas with the addition of geogrid reinforcement is expected to aid in mitigating the potential effects of liquefaction to within tolerable limits from a life safety standpoint in accordance with CDMG SP 117.

5.6 Seismically Induced Landsliding and Rockfalls:

Due to the relatively low topographic relief and lack of large boulders at the site, the probability of seismically induced landsliding and rockfalls is considered very low.

5.7 Seismically Induced Flooding, Seiches, and Tsunamis:

The site is located approximately 1.25 miles north of the lake in the City of Lake Elsinore and is approximately 20 feet higher in elevation than the lake. Portions of the Elsinore Fault pass beneath the lake. The Elsinore Fault is a major right-lateral strike-slip fault. In order to cause seismically induced flooding that could affect the site, a sizeable vertical offset of the lake floor would need to occur, as may be expected from a dip-slip fault. Strike-slip faults do not cause large vertical offsets in a single seismic event that would cause a displacement of the water above the fault, leading to seismically induced flooding. The site is higher in elevation than lake in the City of Lake Elsinore. Due to the type of faulting that passes beneath the lake, the distance from the lake, and the elevation above the lake, the possibility of seismically induced flooding or seiches is considered low. Due to the large distance of the project site to the Pacific Ocean, the possibility for seismically induced tsunamis is considered nil.

6.0 EARTH MATERIALS

6.1 Undocumented Fill (Afu):

Undocumented fill is associated with the previous residential building pad near the northern corner of the site, and other undocumented fills appear to spread over much of the flat areas in the central, western and southern portions of the site. Relatively minor fills are associated with the driveway and building pad areas, and low retaining walls, on the order of 1 to 4 feet thick are thought to exist near the northern corner of the site, with the thicker fills near the southwestern end of the existing pad area. A 4-inch diameter vitrified clay pipe that was buried with clean crushed rock was encountered in TP1 near the northern corner of the site. This pipe likely connected the former residence to a buried septic system on the site, however, the septic system was not located during our investigation. The central, western, and southern portions of the site generally appear to have relatively minor fills on the order of 1 to 3 feet thick. One notable exception to the minor undocumented fill depths is in the central portion of the site, in the vicinity of B3 and TP3, where a trash pit was discovered to extend to depths of at least 7 to 8 feet below the existing ground surface. Man-made materials, including lumber, plastic and metal were found buried in the trash pit at this location. The lateral extent of trash pit was not determined, nor can we confirm the maximum depth of the trash pit, since the deepest part may not have been located during our investigation. Delineating the maximum depth and maximum lateral extent of the suspected trash pit is beyond the scope of this report. In general, the undocumented fill was found to consist of silty fine to medium grained sand that is dry to slightly moist, and loose in place.

6.2 Alluvium (Qal):

Alluvium underlies the undocumented fill, and it is exposed at the ground surface generally along the eastern side of the site, and on portions of the previous house pad area at the northern corner of the site. The alluvium was found to be on the order of 9 to 10 feet thick in the vicinity of the previous house pad near the northern corner of the site, and it was found to extend to depths of approximately 32 to 36 feet below the existing ground surface at the locations of B1 and B2 in the low-lying, generally flat areas of the site. The alluvium was found to consist of very loose to medium dense silty fine to medium grained sands and silty fine to medium grained sands that were dry to wet in-place. Pinhole pores were commonly observed within the alluvium to depths of up to approximately 10 feet below the existing ground surface.

6.3 Pauba Formation Sandstone (Qps):

Sandstone of the Pauba Formation was found to underlie the undocumented fill and alluvium at the knob that supported the previous house pad area at the northern corner of the site at depths of approximately 9 to 10 feet below the existing ground surface. The Pauba Formation was found to consist of medium grained sands and silty fine-grained sands that are moist and dense in place.

6.4 Santiago Formation (Tsi):

The sandstone member of the Santiago Formation underlies the entire site. It was encountered at depths of approximately 32 to 36 feet below the existing ground surface in the deeper borings, B1 and B2, that were advanced in the low-lying, relatively flat portions of the site. The Santiago Formation was recovered as medium to coarse grained sands and gravel that are wet and dense to very dense in place. Refusal to advance the drill was encountered at approximately 9 to 16 feet into the Santiago Formation, at depths of approximately 45 to 48 feet below the existing ground surface.

6.5 Groundwater:

Groundwater was encountered at depths of approximately 9 to 18 feet below the existing ground surface at the time of our investigation in November 2020. Seasonal fluctuations of the groundwater should be anticipated. The areas where the groundwater was encountered at shallower depths was generally in the flat, lower-lying alluvial areas on the southwestern side if the site, in the vicinity of Collier Avenue. No sounding was made at the existing water well near Collier Avenue. These depths below the ground surface correspond to elevations of approximately 1250 to 1254 above mean sea level, based on the referenced No. 1 plan. Groundwater may be encountered during grading and excavation.

7.0 CONCLUSIONS AND RECOMMENDATIONS

7.1 Earthwork Recommendations:

1. Demolition Operations: All demolition operations should be conducted under the observation and documentation and testing of the project geotechnical engineer. **Failure to coordinate the demolition operations with the project geotechnical consultant of record may result in additional fieldwork beyond that represented herein**. It is the owner or the owner's authorized representative responsibility to ensure that the geotechnical consultant id informed of the demolition operations so that a qualified representative can be dispatched for observation and testing.

2. **Vegetation:** All vegetation should be removed from areas to be graded and not used in fills, including tree roots.

3. **Man-made Debris:** All man-made material should be removed from the site and not used in fills.

4. **Water Well Removal**: Prior to grading the site, the existing water well should be destroyed and abandoned per Riverside County Department of Environmental Health standards.

5. **Unsuitable Soil Removal:** Undocumented fill and loose, compressible, and/or porous alluvial soils are considered unsuitable for support of structural fill. All existing undocumented fill should be removed. The undocumented fill in the area of the previous residence at the northern end of the site is anticipated to be on the order of approximately 1 to 4 feet thick. The undocumented fill in the central, western, and southern portions of the site are anticipated to be on the order of 1 to 3 feet thick. An on-site sewage disposal (septic) system is likely located on the site and should be removed. Undocumented fill comprising a suspected trash pit was encountered in the central portion of the site in the vicinity of B3 and TP3. The undocumented fill of the trash pit is at least 7 to 8 feet deep, but the maximum depth and maximum lateral extent of the suspected trash pit were not determined as a part of this study. All loose, compressible, and/or porous alluvium should be removed to competent alluvium, or competent bedrock.

6. **Alluvial Removals in Structure Areas:** In order to address both static settlement of loose, compressible, and/or porous alluvium, and dynamic settlement due to the effects of liquefaction, alluvial removals beneath the proposed structures should be made to a minimum depth of 10 feet below the existing ground surface in proposed fill areas and 10 feet below pad grade in proposed cut areas so that a minimum of 10 feet of engineered fill exists beneath the structure, unless competent bedrock is reached at a shallower depth. The removal bottoms should extend laterally beyond the structure perimeter a distance equal to the removal depth, with a minimum of 10 feet, so that a 1:1 plane may be projected from the structure perimeter to the bottom outside edge of the removal. Deeper removals may be necessary based upon exposed conditions during grading. Groundwater may be reached during the recommended alluvial removals. The soils that have been removed should be cleared of vegetation and man-made debris and may then be stockpiled for re-use as engineered fill. For reference, a representative cross section, X-X', showing proposed alluvial removals in structure areas is included as Plate 2. Note that there is vertical exaggeration in the cross section in order to show the removals in more detail.

7. **Alluvial Removals in Parking, Driveway and Other Hardscape Areas:** Where native alluvium is exposed at the ground surface removals for parking, driveway and other hardscape areas should be a minimum of 3 feet below the ground surface in proposed fill areas and 3 feet below finished grade in proposed cut areas so that a minimum of 3 feet of engineered fill underlies these areas. Where undocumented fill is exposed at the ground surface the undocumented fill should be removed. After all undocumented fills have been removed, a minimum of the upper 3 feet of

alluvium beneath the undocumented fills should be removed. Therefore, in areas where approximately 1 to 4 feet of undocumented fills exist, removals are anticipated to be approximately 4 to 7 feet below the existing ground surface (3 feet below the 1 to 4 feet of undocumented fill).

8. **Overexcavation:** Structures on shallow footings must not straddle a cut/fill transition. The cut and shallow fill portions of the building pad should be overexcavated so that building does not straddle a cut/fill transition. Overexcavation in the cut and shallow fill portions of the building pad should be performed to half the depth of the maximum fill thickness below proposed grade, with a minimum of 5 feet. The horizontal extent of the overexcavation should extend laterally outside of the perimeter footings a distance equal to the overexcavation depth, with a minimum of 5 feet. It is anticipated that cut to shallow fill transitions may exist primarily in the proposed structures located near the northern corner of the site which is likely the only area where bedrock of the Pauba Formation Sandstone may be reached at depths less than 10 feet below the ground surface. Overexcavation estimates are conditionally based upon the estimated removal depth, however, deeper overexcavation may be necessary based on conditions exposed during grading.
9. **Removal Bottoms:** All exposed removal bottoms should be inspected and probed by the Geotechnical Engineer or Engineering Geologist, or their representative prior to placement of any fill. Natural, undisturbed bottoms should expose competent Pauba Formation bedrock or competent alluvium. Competent alluvium should be defined as alluvium that has in-place density that is at least 85% of the maximum density. The approved exposed bottoms should be scarified 12-inches, brought to near optimum moisture content, and compacted to a minimum of 90 percent relative compaction before placement of fill.

7.2 Groundwater and Removal Bottom Stabilization:

Where removal bottoms expose bedrock no groundwater and no stabilization of the removal bottom is anticipated. Where alluvial bottoms are exposed, groundwater is anticipated at or near the elevation of the removal bottoms in the relatively flat areas of the site, especially the low-lying southwestern side of the site, near Collier Avenue. Saturated removal bottoms may exhibit excessive “pumping” and rutting of the bottoms, especially when wheel-mounted vehicles are used. Therefore, special techniques may be needed in order to achieve the recommended removal depth, and the removal bottom may have to be stabilized prior to the placement of geogrid reinforcement and engineered fill. Placement of clean crushed rock or other methods may be utilized to stabilize the bottom prior to placement of geogrid reinforcement and engineered fill. The grading contractor should select the methods and equipment that will be used in order to achieve the recommended removal depths.

7.3 Geogrid Reinforcement:

In order to mitigate for the effects of settlement due to liquefaction, two layers of geogrid reinforcement should be placed where the removal bottoms expose alluvium. Once the removal bottom is achieved, a layer of bi-directional geogrid reinforcement such as Tensar BX 1200 or equivalent should be placed across the entire bottom. Geogrid rolls should overlap per the manufacturer’s recommendations, with a minimum of 2 feet. Engineered fill should be placed to a height of 2 feet above the lower layer of geogrid reinforcement, and then a second layer of bi-directional geogrid should be placed across the entire bottom. After the second layer of geogrid has been placed, engineered fill may be placed in order to achieve the proposed grades without additional geogrid layers. Care must be taken so that the geogrid layers are not disturbed or damaged by grading equipment following their placement.

7.4 Engineered Fill:

Engineered fill should be compacted to a minimum of 90 percent relative compaction. Maximum dry density and optimum moisture content for compacted materials should be determined according to ASTM D 1557 procedures.

7.5 Oversize Material:

We anticipate that no oversize material, defined as rocks or boulders that cannot be reduced to less than 12-inches in diameter, will be encountered during the grading for the proposed development. Should oversize material be encountered, please contact our office for further recommendations.

7.6 Structural Fill:

All fill material, whether on-site material or import, should be accepted by the Project Geotechnical Engineer and/or his representative before placement. All fill should be free from vegetation, organic material, and other debris. Import fill should be no more expansive than the existing on-site material, unless approved by the Project Geotechnical Engineer. Approved fill material should be placed in horizontal lifts not exceeding 6.0 to 8.0-inches in thickness, and watered or aerated to obtain near-optimum moisture content (within 2.0 percent of optimum). Each lift should be spread evenly and should be thoroughly mixed to ensure uniformity of soil moisture. Structural fill should meet a minimum relative compaction of 90 percent of maximum dry density based upon ASTM D 1557 procedures. Moisture content of fill materials should not vary more than 2.0 percent of optimum, unless approved by the Project Geotechnical Engineer.

7.7 Soil Expansion Potential:

Preliminary Expansion Index testing was performed, yielding an EI of 0. This is classified as a **very low expansion** potential. Import soils or soils used near finish grade may have a different EI. Final foundation design parameters should be based on EI testing of near-surface soils and be performed at the conclusion of rough grading. Those results should be forwarded and incorporated into the final design by the Project Structural Engineer.

7.8 Soil Corrosive Potential:

The following table lists the corrosive tests performed and their results:

Soil Corrosion Parameter Test Results (California Test Methods 643, 417, and 422)					
Sample Number	**Location and Depth**	**pH**	**Minimum Resistivity (ohm-cm)**	**Chloride (ppm)**	**Sulfate (ppm)**
TP2@0'-9'	TP2@0'-9'	8.4	3195	250	60

CALTRANS considers soils that will be in contact with foundation elements to have corrosive properties if one or more of the following conditions exist:

(1) The pH is equal to or less than 5.5

(2) The resistivity is equal to or less than 1,000 ohm-cm

(3) Chloride concentration is equal to or greater than 500 parts per million (ppm)

(4) Sulfate concentration is equal to or greater than 2,000 ppm.

(5) Based on the test results from the samples obtained that will may be in contact with proposed footings, the soils are not considered corrosive to concrete foundations, thus, type II concrete may be used.

8.0 SLOPE RECOMMENDATIONS

8.1 Fill Slopes:

It is our opinion that 2:1 (horizontal to vertical) or flatter fill slopes possess gross and surficial stability in excess of generally accepted minimum engineering criteria (Factor of Safety at least 1.5) and therefore 2:1 or flatter fill slopes are anticipated to be suitable for their intended purpose provided that proper slope maintenance procedures are maintained. These procedures include but are not limited to installation and maintenance of drainage devices and planting of slope faces to protect from erosion in accordance with City and County standards. Any fill slopes should be constructed at slope ratios no steeper than 2:1 (horizontal to vertical). A keyway excavated into competent native earth materials should be constructed at the toe of all fill slopes that are proposed on natural grades of 5:1 (horizontal to vertical) or steeper. Keyways should be a minimum of 15 feet wide (equipment width) and tilted a minimum of two percent into the hillside. A series of level benches should be constructed into competent bedrock or native soil on natural grades of 5:1 or steeper prior to placing fill.

8.2 Cut Slopes:

It is our opinion that 2:1 (horizontal to vertical) or flatter cut slopes possess gross and surficial stability in excess of generally accepted minimum engineering criteria (Factor of Safety at least 1.5) and therefore 2:1 or flatter cut slopes are anticipated to be suitable for their intended purpose provided that proper slope maintenance procedures are maintained. These procedures include but are not limited to installation and maintenance of drainage devices and planting of slope faces to protect from erosion in accordance with City and County standards. All cut slopes should be inspected by the Project Engineering Geologist to verify stability. Cut slopes exposing significant amounts of alluvium or weathered bedrock may be unstable. Unstable cut slopes may require flattening or buttressing.

8.3 Slope Protection and Maintenance:

The following recommendations are presented for slope protection and maintenance.

8.3.1 Surface Drainage:

Surface water should not be allowed to flow over the slopes other than incidental rainfall. No alteration of pad gradients should be allowed that will prevent pad and roof run-off from being expediently directed to approved disposal areas away from the tops of slopes.

8.3.2 Off-Site Drainage:

Concentrated surface waters entering the property from off-site sources should be collected and directed to a permanent drainage system away from the tops of slopes.

8.3.3 Maintenance Responsibility:

The property owner is responsible for the maintenance and cleaning of all interceptor ditches, drainage terraces, downdrains and any other drainage devices that have been installed to promote slope stability.

8.3.4 Slope Protection:

It is recommended that slopes be planted with ground cover, shrubs, and trees that possess deep, dense root structures that require a minimum of irrigation. It should be the responsibility of the landscape architect to provide such plants initially and of the resident to maintain such planting. Alteration of the planting scheme is at the property owner's risk.

8.3.5 Excessive Irrigation:

If automatic sprinkler systems are installed on the slopes, their use should be adjusted to account for natural rainfall.

8.3.6 Burrowing Animals:

The resident and/or owner should maintain a program for the elimination of burrowing animals. This should be an on-going program to protect slope stability.

9.0 FOUNDATION DESIGN RECOMMENDATIONS:

9.1 General:

Foundations for the proposed structures may consist of conventional column footings and continuous wall footings founded on compacted fill. The recommendations presented in the subsequent paragraphs for foundation design and construction are based on geotechnical characteristics and upon a very low expansion potential for the supporting soils and should not preclude more restrictive structural requirements. The Structural Engineer for the project should determine the actual footing width and depth in accordance with the latest edition of the California Building Code to resist design vertical, horizontal, and uplift forces and should either verify or amend the design based on final expansion testing at the completion of grading.

9.2 Foundation Size:

Continuous footings should have a minimum width of 12-inches. Continuous footings should be continuously reinforced with a minimum of one (1) No. 4 steel reinforcing bar located near the top and near the bottom of the footings to minimize the effects of slight differential movements which may occur due to minor variations in the engineering characteristics or seasonal moisture change in the supporting soils. Column footings should have a minimum width of 18-inches by 18-inches and be suitably reinforced, based on structural requirements. A grade beam, founded at the same depths and reinforced the same as the adjacent footings, should be provided across doorway and garage entrances.

9.3 Depth of Embedment:

Exterior and interior footings founded in engineered fill material should extend to a minimum depth of 18-inches below lowest adjacent finish grade.

9.4 Bearing Capacity:

Provided the recommendations for site earthwork, minimum footing width, and minimum depth of embedment for footings are incorporated into the project design and construction, the allowable bearing value for design of continuous and column footings, for the residential structure for the total dead plus frequently-applied live loads is 1,500 psf for footings in competent engineered fill. The allowable bearing value has a Factor of Safety of at least 3.0 and may be increased by 33.3 percent for short durations of live and/or dynamic loading such as wind or seismic forces.

9.5 Settlement:

Based on the recommended mitigation measures for site earthwork in consideration of the life-safety standard guidelines of CDMG SP 117, the footings designed to the recommended bearing value limits described under § 2.1 of this report settlement is not expected to exceed a maximum of 0.75-inch or a differential settlement of 0.50-inch over a distance of 40-feet in compacted fill material under static load conditions.

9.6 Lateral Capacity:

Additional foundation design parameters for the residence based on compacted fill for resistance to static lateral forces, are as follows:

Allowable Lateral Pressure (Equivalent Fluid Pressure), Passive Case:
Engineered Fill – 200 pcf
Allowable Coefficient of Friction:
Engineered Fill - 0.35

Lateral load resistance may be developed by a combination of friction acting on the base of foundations and slabs and passive earth pressure developed on the sides of the footings and stem walls below grade when in contact with engineered fill material. The above values are allowable design values and may be used in combination without reduction in evaluating the resistance to lateral loads. The allowable values may be increased by 33.3 percent for short durations of live and/or dynamic loading, such as wind or seismic forces. For the calculation of passive earth resistance, the upper 1.0-foot of material should be neglected unless confined by a concrete slab or pavement. The maximum recommended allowable passive pressure is 5.0 times the recommended design value.

9.7 Slab-on-Grade Recommendations:

The recommendations for concrete slabs, both interior and exterior, excluding PCC pavement, are based upon the anticipated building usage and upon a very low expansion potential for the supporting material as determined by Chapter 18 of the California Building Code. Concrete slabs should be designed to minimize cracking as a result of shrinkage. Joints (isolation, contraction, and construction) should be placed in accordance with the American Concrete Institute (ACI) guidelines. Special precautions should be taken during placement and curing of all concrete slabs. Excessive slump (high water/cement ratio) of the concrete and/or improper curing procedures used during either hot or cold weather conditions could result in excessive shrinkage, cracking, or curling in the slabs. It is recommended that all concrete proportioning, placement, and curing be performed in accordance with ACI recommendations and procedures.

Slab-on-grade reinforcement and thickness should be provided by the structural engineer based on structural considerations, but as a minimum, it is recommended that concrete floor slabs subjected to crane loads for tilt-up buildings be at least 5-inches in actual thickness and reinforced with at least No. 3 reinforcing bars placed 18-inches on center, both ways, placed at mid-height of the slab cross-section.

9.8 Exterior Slabs:

All exterior concrete slabs cast on finish subgrade (patios, sidewalks, etc., with the exception of PCC pavement) should be a minimum of 4-inches nominal in thickness. Reinforcing in the slabs and the use of a compacted sand or gravel base beneath the slabs should be according to the current local standards. Subgrade soils should be moisture conditioned to at least optimum moisture content to a depth of 12-inches immediately before placing the concrete.

10.0 RETAINING WALL RECOMMENDATIONS

10.1 Earth Pressures:

Retaining walls should be backfilled with non-expansive granular soil (EI=0) or very low expansive potential materials (Expansion Index of 20 or less) within a zone extending upward and away from the heel of the footing at a slope of 0.5:1 (horizontal to vertical) or flatter can be designed to resist the following static lateral soil pressures:

Condition	Level Backfill	2:1 Slope	Seismic*
Active	35 pcf	50 pcf	K_u=0.2
At Rest	65 pcf	--	--

**For use on walls exceeding 6' in height. To be used with Mononobe-Okabe method.*

Further expansion testing of potential backfill material should be performed at the time of retaining wall construction to determine suitability. Walls that are free to deflect 0.01 radian at the top may be designed for the above-recommended active condition. Walls that need to be restricted from this amount of movement should be assumed rigid and designed for the at-rest condition. The above values assume well-drained backfill and no buildup of hydrostatic pressure. Surcharge loads, dead and/or live, acting on the backfill behind the wall should also be considered in the design.

10.2 Retaining Wall Design:

Retaining wall footings should be founded to the same depths into firm, competent, undisturbed, engineered fill as standard foundations and may be designed for an allowable bearing value of 1,500 psf (as long as the resultant force is located in the middle one-third of the footing), and with an allowable static lateral bearing pressure of 200 psf/ft and allowable sliding resistance coefficient of friction of 0.35. When using the allowable lateral pressure and allowable sliding resistance, a Factor of Safety of 1.5 should be achieved.

10.3 Subdrain:

A subdrain system should be constructed behind and at the base of retaining walls equal to or in excess of 4-feet in height to allow drainage and to prevent the buildup of excessive hydrostatic pressures. Gravel galleries and/or filter rock, if not properly designed and graded for the on-site and/or import materials, should be enclosed in a geotextile fabric such as Mirafi 140N, Supac 4NP, or a suitable substitute in order to prevent infiltration of fines and clogging of the system. The perforated pipes should be at least 4.0-inches in diameter. Pipe perforations should be placed downward. Gravel filters should have volume of at least 1.0 cubic foot per lineal foot of pipe. For retaining walls with an overall height of less than 4-feet, subdrains may include weep holes with a continuous gravel gallery, perforated pipe surrounded by filter rock, or some other approved system. Subdrains should maintain a positive flow gradient and have outlets that drain in a non-erosive manner.

10.4 Backfill:

Backfill directly behind retaining walls (if backfill width is less than 3 feet) may consist of 0.5 to 0.75-inch diameter, rounded to subrounded gravel enclosed in a geotextile fabric such as Mirafi 140N, Supac 4NP, or a suitable substitute or a clean sand (Sand Equivalent Value greater than 50) water jetted into place to obtain proper compaction. If water jetting is used, the subdrain system should be in place. Even if water jetting is used, the sand should be densified to a minimum of 90 percent relative compaction. If the specified density is not obtained by water jetting, mechanical methods will be required. If other types of soil or gravel are used for backfill, mechanical compaction methods will be required to obtain a relative compaction of at least 90 percent of maximum dry density. Backfill directly behind retaining walls should not be compacted by wheel, track or other rolling by heavy construction equipment unless the wall is designed for the surcharge loading. If gravel, clean sand or other imported backfill is used behind retaining walls, the upper 18-inches of backfill in unpaved areas should consist of typical on-site material compacted to a minimum of 90 percent relative compaction in order to prevent the influx of surface runoff into the granular backfill and into the subdrain system. Maximum dry density and optimum moisture content for backfill materials should be determined in accordance with ASTM D 1557 procedures.

10.5 Pavement Design

The following structural pavement section is for proposed street improvements for the subject development and are presented for preliminary design purposes only. *The final design should be based on R-Values testing performed at subgrade upon completion of grading*. The preliminary pavement sections as presented below are based on the County of Riverside Standards and Specifications and an R-Value of 10. The sections listed are provided for reference purposes and are calculated as a minimum based on varying Traffic Indexes:

Traffic Index	Calculated Section
5.0	3-inches AC over 7.5-inches AB, placed on properly prepared subgrade.
6.0	3-inches AC over 12.0-inches AB, placed on properly prepared subgrade.
6.5	3-inches AC over 13.6-inches AB, placed on properly prepared subgrade.
7.0	3-inches AC over 15.2-inches AB, placed on properly prepared subgrade.

10.6 CalTrans Standard Specification:

Asphalt concrete pavement materials should be as specified in Sections 39-2.01 and 39-2.02 of the current **Caltrans** Standard Specifications or a suitable equivalent. Aggregate base should conform to 3/4-inch Class II material as specified in Section 26-01.02B of the current **Caltrans** Standard Specifications or a suitable equivalent. To properly prepare the subgrade, the soil should be recompacted to a minimum 90 percent relative compaction in asphalt pavement areas and 95 percent relative compaction in Portland cement concrete areas, to a minimum depth of 12-inches below finish subgrade elevation. The aggregate base material should be compacted to at least 95 percent relative compaction. Maximum dry density and optimum moisture content for subgrade and aggregate base materials should be determined according to ASTM D 1557 procedures. If pavement subgrade soils are prepared and aggregate base material is not placed immediately, or the aggregate base material is placed and the area is not paved immediately, additional observations and testing will be required prior to placing aggregate base material or asphaltic concrete to locate areas that may have been damaged by construction traffic, construction activities, and/or seasonal wetting and drying.

11.0 MISCELLANEOUS RECOMMENDATIONS

11.1 Utility Trench Recommendations:

Utility trenches within the zone of influence of foundations or under building floor slabs, hardscape, and/or pavement areas should be backfilled with properly compacted soil. It is recommended that all utility trenches excavated to depths of 5.0-feet or deeper be cut back to an inclination not steeper than 1:1 (horizontal to vertical) or be adequately shored during construction. Where interior or exterior utility trenches are proposed parallel and/or perpendicular to any building footing, the bottom of the trench should not be located below a 1:1 plane projected downward from the outside bottom edge of the adjacent footing unless the utility lines are designed for the footing surcharge loads. Backfill material should be placed in a lift thickness appropriate for the type of backfill material and compaction equipment used. Backfill material should be compacted to a minimum of 90 percent relative compaction by mechanical means. Jetting of the backfill material will not be considered a satisfactory method for compaction. Maximum dry density and optimum moisture content for backfill material should be determined according to ASTM D 1557 procedures.

11.2 Finish Lot Drainage Recommendations:

Finish lot surface gradients in unpaved areas should be provided next to tops of slopes and buildings to direct surface water away from foundations and slabs and from flowing over the tops of slopes. The surface water should be directed toward suitable drainage facilities. Ponding of surface water should not be allowed next to structures or on pavements. In unpaved areas, a minimum positive gradient of 2.0 percent away from the structures and tops of slopes for a minimum distance of 10.0-feet and a minimum of 1.0 percent pad drainage off the property in a non-erosive manner should be provided.

11.3 Bio-Retention Basin:

Based on the referenced grading plan, no bio-retention basins are proposed.

11.4 Planter Recommendations:

Planters around the perimeter of the structure should be designed with proper surface slope or a sufficient number of area drains should be installed within the planters to ensure that adequate drainage is maintained, and minimal irrigation water is allowed to percolate into the soils underlying the building. Planters in parking areas should be avoided or should at least be very well-drained because percolation into the parking subgrade may significantly reduce the lifespan of the pavement adjacent to the planters.

11.5 Supplemental Construction Observations and Testing:

Any subsequent grading for development of the subject property should be performed under engineering observation and testing performed by EnGEN Corporation. Subsequent grading includes, but is not limited to, any additional overexcavation of cut and/or cut/fill transitions, fill placement, and excavation of temporary and permanent cut and fill slopes. In addition, EnGEN Corporation should observe all foundation excavations. Observations should be made prior to installation of concrete forms and/or reinforcing steel to verify and/or modify, if necessary, the conclusions and recommendations in this report. Observations of overexcavation cuts, fill placement, finish grading, utility or other trench backfill, pavement subgrade and base course, retaining wall backfill, slab pre-saturation, or other earthwork completed for the development of subject property should be performed by EnGEN Corporation. If any of the observations and testing to verify site geotechnical conditions are not performed by EnGEN Corporation, liability for the safety and performance of the development is limited to the actual portions of the project observed and/or tested by EnGEN Corporation.

12.0 PLAN REVIEW

Subsequent to formulation of final plans and specifications for the project but before bids for construction are requested, grading and foundation plans for the proposed development should be reviewed by EnGEN Corporation to verify compatibility with site geotechnical conditions and conformance with the recommendations contained in this report. If EnGEN Corporation is not accorded the opportunity to make the recommended review, we will assume no responsibility for misinterpretation of the recommendations presented in this report.

13.0 CONFERENCES

13.1 Pre-Bid Conference:

It is recommended that a pre-bid conference be held with the owner or an authorized representative, the Project Architect, the Project Civil Engineer, the Project Geotechnical Engineer and the proposed contractors present. This conference will provide continuity in the bidding process and clarify questions relative to the supplemental grading and construction requirements of the project.

13.2 Pre-Grading Conference:

Before the start of any grading, a conference should be held with the owner or an authorized representative, the contractor, the Project Architect, the Project Civil Engineer, and the Project Geotechnical Engineer present. The purpose of this meeting should be to clarify questions relating to the intent of the supplemental grading recommendations and to verify that the project specifications comply with the recommendations of this geotechnical engineering report. Any special grading procedures and/or difficulties proposed by the contractor can also be discussed at that time.

14.0 CLOSURE

This report has been prepared for use by the parties or project named or described in this document. It may or may not contain sufficient information for other parties or purposes. In the event that changes in the assumed nature, design, or location of the proposed structure and/or project as described in this report, are planned, the conclusions and recommendations contained in this report will not be considered valid unless the changes are reviewed and the conclusions and recommendations of this report are modified or verified in writing. This study was conducted in general accordance with the applicable standards of our profession and the accepted soil and foundation engineering principles and practices at the time this report was prepared. No other warranty, implied or expressed beyond the representations of this report, is made. Although every effort has been made to obtain information regarding the geotechnical and subsurface conditions of the site, limitations exist with respect to the knowledge of unknown regional or localized off-site conditions that may have an impact at the site. The recommendations presented in this report are valid as of the date of the report. However, changes in the conditions of a property can occur with the passage of time, whether they are due to natural processes or to the works of man on this and/or adjacent properties. If conditions are observed or information becomes available during the design and construction process that are not reflected in this report, EnGEN Corporation should be notified so that supplemental evaluations can be performed and the conclusions and recommendations presented in this report can be modified or verified in writing. Changes in applicable or appropriate standards of care or practice occur, whether they result from legislation or the broadening of knowledge and experience. Accordingly, the conclusions and recommendations presented in this report may be invalidated, wholly or in part, by changes outside of the control of EnGEN Corporation which occur in the future.

Thank you for the opportunity to provide our services. Often, because of design and construction details which occur on a project, questions arise concerning the geotechnical conditions on the site. If we can be of further service or should you have questions regarding this report, please do not hesitate to contact this office at your convenience. Because of our involvement in the project to date, we would be pleased to discuss engineering testing and observation services that may be applicable on the project.

Respectfully submitted,
EnGEN Corporation

Wayne Baimbridge, Principal
General Manager, REPA 467279

Osbjorn Bratene, Principal
GE 162

Colby Matthews, Project Engineering Geologist
CEG 2460

WB/CM/OB:ch

Distribution: (2) Addressee

APPENDIX 1 - GENERAL TECHNICAL REFERENCES

1. **California Building Code (CBC)**, 2019, State of California, California Code of Regulations, Title 24, California Building Code.

2. **California Division of Mines and Geology (CDMG)**, 1997, Guidelines for Evaluating and Mitigating Seismic Hazards in California, Special Publication 117.

3. **California Geological Survey (CGS)**, 2002, California Geomorphic Provinces: CDMG, Note 36.

4. **California Geological Survey (CGS)**, 2014, Guidelines for Evaluating and Mitigating Seismic Hazards in California, 2008, Special Publication 117A.

5. **California Geological Survey (CGS),** 2021, Earthquake Zones of Required Investigation web application viewed at https://maps.conservation.ca.gov/cgs/EQZApp.

6. **Hart, Earl W., and Bryant, William A**., Revised 2007, Fault-Rupture Hazard Zones in California, Alquist-Priolo Earthquake Fault Zoning Act with Index to Earthquake Fault Zone Maps: State of California, Department of Conservation, Division of Mines and Geology, Special Publication 42.

7. **Kennedy, M.P**., 1977, Recency and Character of Faulting Along the Elsinore Fault Zone in Southern Riverside County, California: California Division of Mines and Geology, Special Report 131, 12 p., 1 Plate, Scale 1:24,000.

8. **Mann, J.F., Jr**., October 1955, Geology of a Portion of the Elsinore Fault Zone, California: State of California, Department of Natural Resources, Division of Mines, Special Report 43.

9. **Morton, D.M. and Weber, F.H. Jr.,** 2003, Geologic Map of the Elsinore 7.5 Quadrangle, California, Version 1.0, Scale 1:24,000, Digital Database by Rachel Alvarez and Diane Burns, USGS Open File Report 03-281.

10. **Riverside County Transportation and Land Management Agency**, 2000, Technical Guidelines for Review of Geotechnical and Geologic Reports.

11. **Riverside County Planning Department**, January 1983, Riverside County Comprehensive General Plan - County Seismic Hazards Map, Scale 1 Inch = 2 Miles.

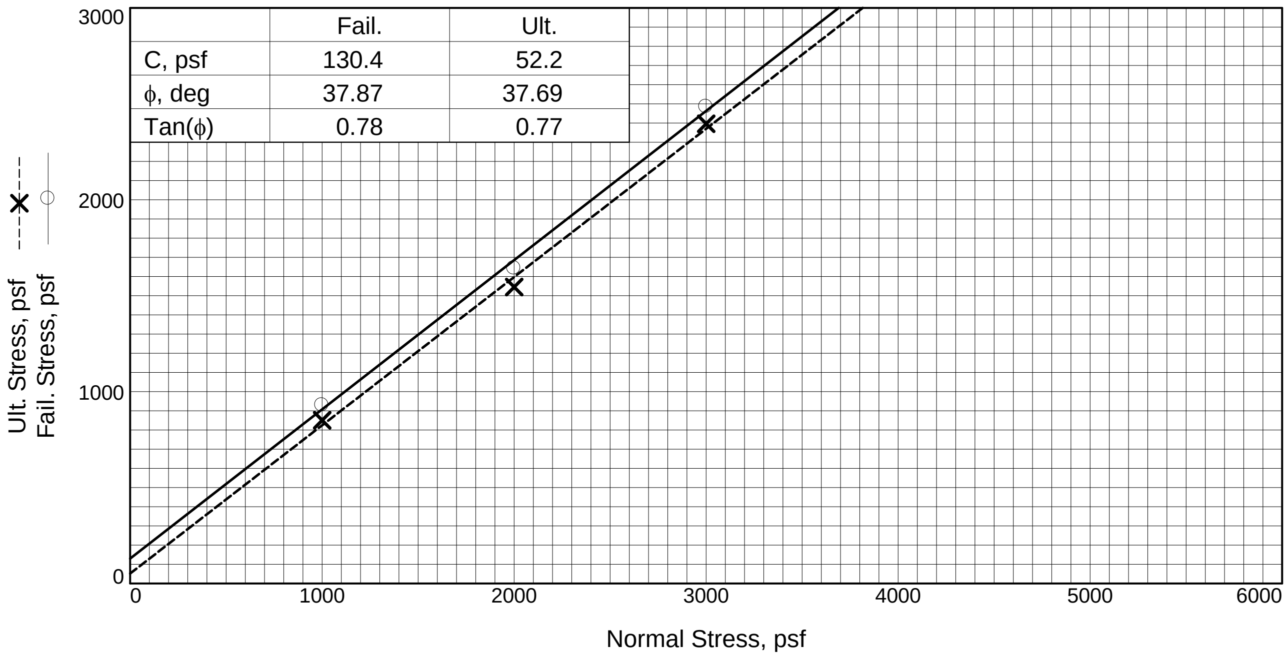
12. **Riverside County Land Information System**: HTTP://WWW3.TLMA.CO.RIVERSIDE.CA.US/PA/RCLIS/

13. **Riverside County Geographic Information System,** 2021, Map My County web application, version 10, viewed at https://gis.countyofriverside.us.

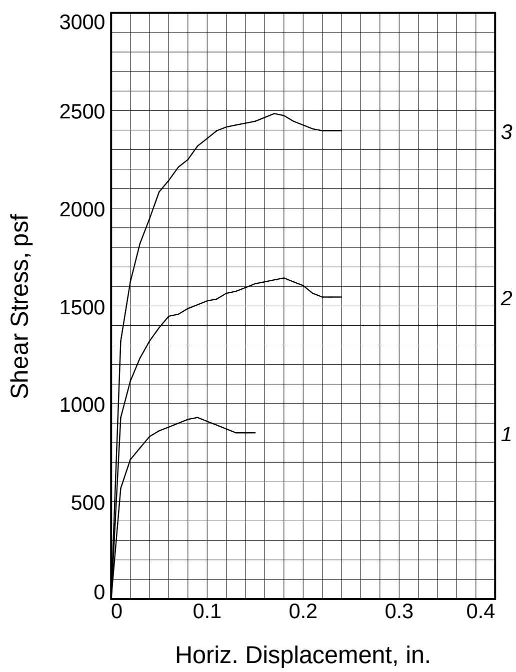
14. **Southern California Earthquake Data Center (SCEDC),** 2021, Southern California Earthquake Data Center Website, HTTP://WWW.SCECDC.SCEC.ORG.

15. **Structural Engineers Association of California (SEAOC),** 2021, Seismic Design Mapping Tool, web application viewed at: WWW.SEAOC.ORG/PAGE/SEISMICDESIGNMAPTOOL

APPENDIX 2 - LABORATORY TEST RESULTS



	Fail.	Ult.
C, psf	130.4	52.2
ϕ, deg	37.87	37.69
Tan(ϕ)	0.78	0.77



	Sample No.	1	2	3
Initial	Water Content, %	8.5	8.5	8.5
	Dry Density, pcf	118.9	118.9	118.9
	Saturation, %	52.9	52.9	52.9
	Void Ratio	0.4438	0.4438	0.4438
	Diameter, in.	2.416	2.416	2.416
	Height, in.	1.000	1.000	1.000
At Test	Water Content, %	14.5	13.9	13.4
	Dry Density, pcf	118.9	118.9	118.9
	Saturation, %	90.0	86.4	83.0
	Void Ratio	0.4438	0.4438	0.4438
	Diameter, in.	2.416	2.416	2.416
	Height, in.	1.000	1.000	1.000
	Normal Stress, psf	1000.0	2000.0	3000.0
	Fail. Stress, psf	929.2	1643.3	2484.5
	Displacement, in.	0.090	0.180	0.170
	Ult. Stress, psf	851.0	1545.4	2396.4
	Displacement, in.	0.130	0.220	0.220
	Strain rate, in./min.	0.100	0.100	0.100

Sample Type: BULK

Description: Silty fine sand, light brown (5YR 6/4)

Specific Gravity= 2.75

Remarks:

Figure ________

Client: Saddleback & Assc.

Project: Saddleback LAKE ELSINORE

Source of Sample: Shears **Depth:** 0'-5'

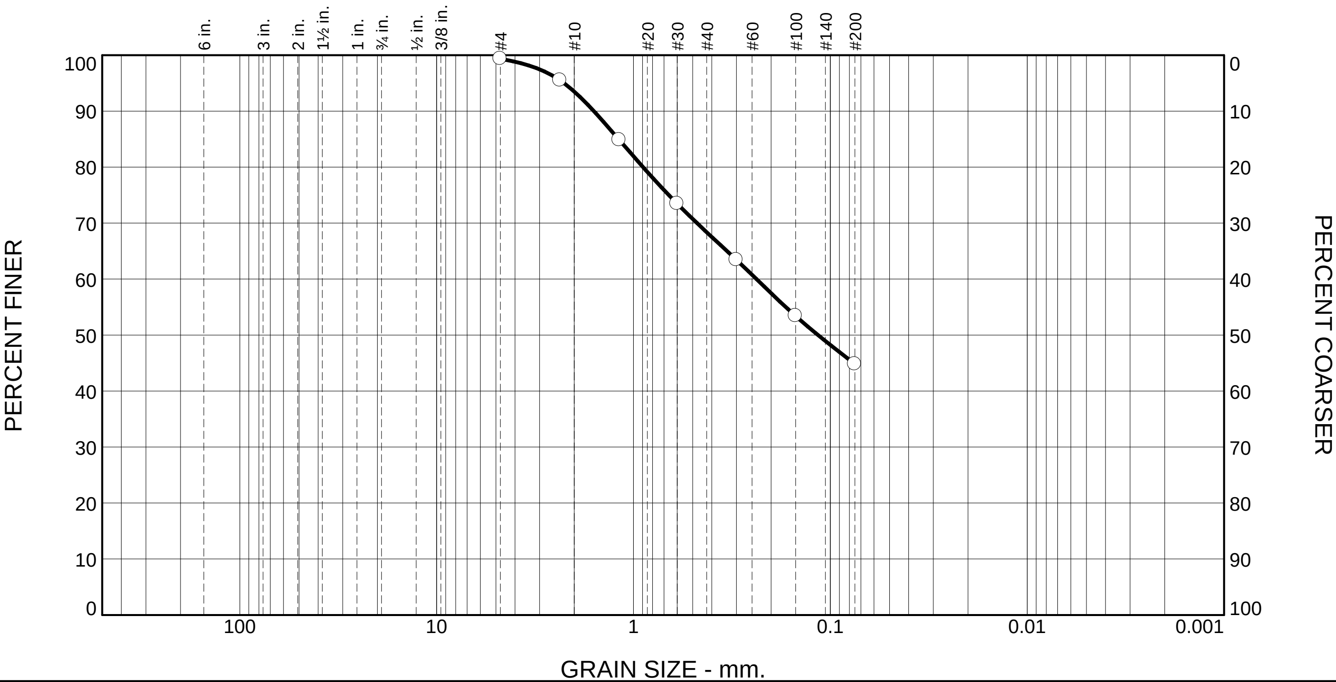
Sample Number: B1@0'-5'

Proj. No.: 4626 GFS **Date Sampled:** 11/13/20

DIRECT SHEAR TEST REPORT

EnGEN Corporation

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
			5.9	25.1	23.6	44.8	

TEST RESULTS			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
#4	99.4		
#8	95.5		
#16	84.8		
#30	73.5		
#50	63.4		
#100	53.4		
#200	44.8		

* (no specification provided)

Material Description

Silty fine sand, light brown (5YR 5/6)

Atterberg Limits (ASTM D 4318)

PL= **LL=** **PI=**

Classification

USCS (D 2487)= **AASHTO (M 145)=**

Coefficients

D_{90}= 1.5901 D_{85}= 1.1909 D_{60}= 0.2374
D_{50}= 0.1154 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Remarks

F.M.=1.30

Date Received: **Date Tested:** 1/21/21

Tested By: GK

Checked By: GK

Title:

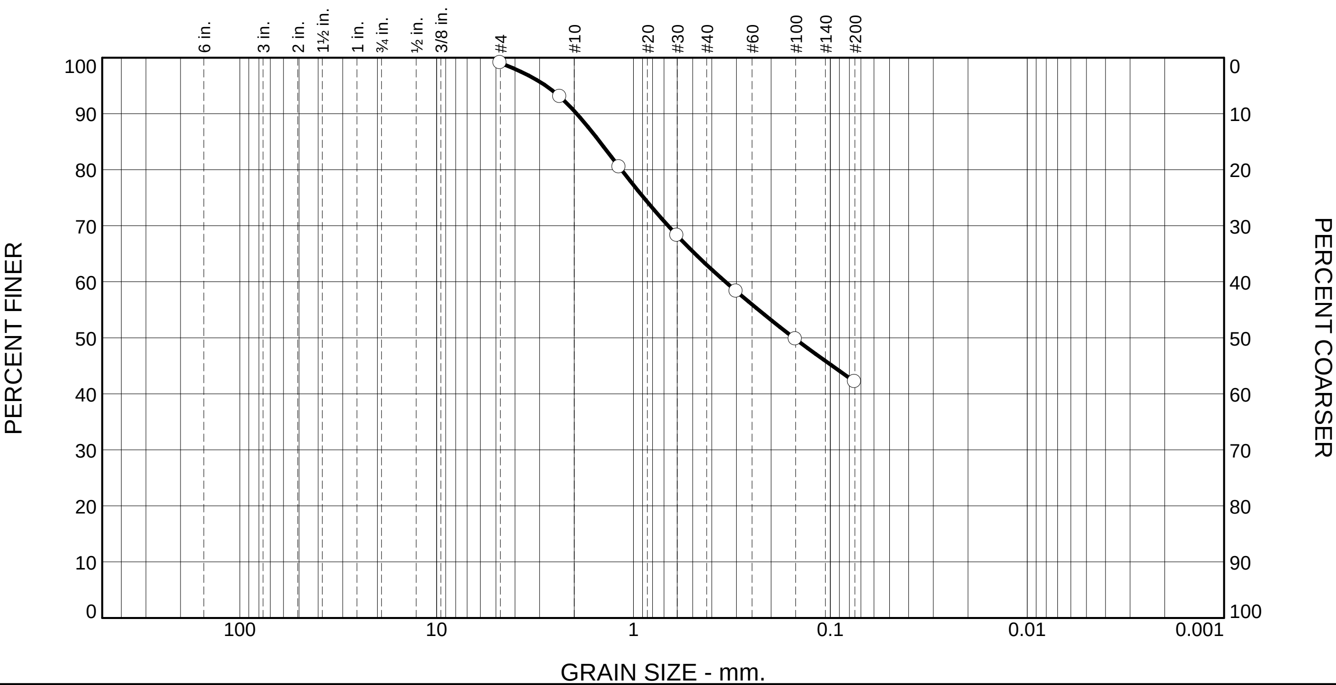
Source of Sample: B2 (Gradation) **Depth:** 2.5' **Date Sampled:** 11/13/20
Sample Number: B2@2.5

EnGEN Corporation

Client: Saddleback & Assc.
Project: Saddleback LAKE ELSINORE

Project No: 4626 GFS **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
			8.6	27.5	20.9	42.1	

TEST RESULTS

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
#4	99.1		
#8	93.0		
#16	80.5		
#30	68.2		
#50	58.3		
#100	49.8		
#200	42.1		

* (no specification provided)

Material Description

Silty sand, light brown (5YR 5/6)

Atterberg Limits (ASTM D 4318)

PL= **LL=** **PI=**

Classification

USCS (D 2487)= **AASHTO (M 145)=**

Coefficients

D_{90}= 1.9434 D_{85}= 1.4841 D_{60}= 0.3413
D_{50}= 0.1532 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Remarks

F.M.=1.51

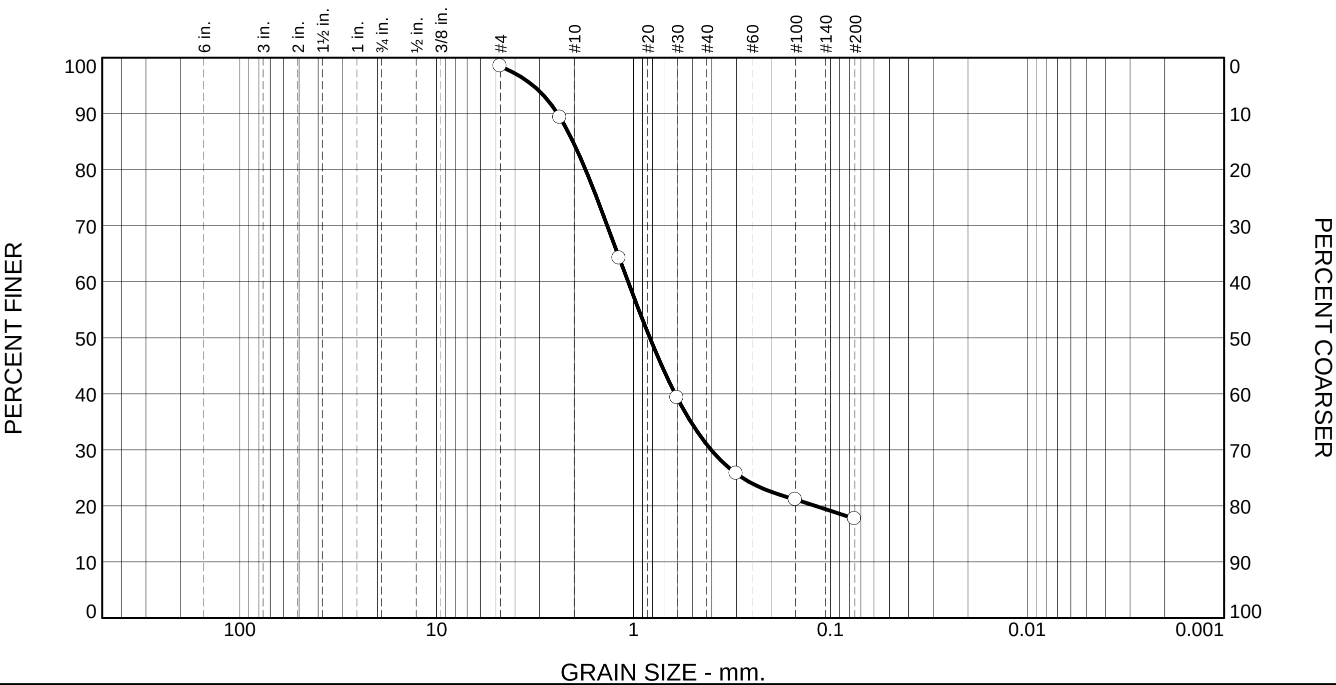
Date Received: **Date Tested:** 1/21/21
Tested By: GK
Checked By: GK
Title:

Source of Sample: B2 (Gradation) **Depth:** 5' **Date Sampled:** 11/13/20
Sample Number: B2@5

EnGEN Corporation

Client: Saddleback & Assc.
Project: Saddleback LAKE ELSINORE
Project No: 4626 GFS **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
			14.0	53.5	13.3	17.7	

TEST RESULTS			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
#4	98.5		
#8	89.3		
#16	64.2		
#30	39.3		
#50	25.7		
#100	21.1		
#200	17.7		

* (no specification provided)

Material Description

Medium sand, medium yellowish brown (10YR 5/4)

Atterberg Limits (ASTM D 4318)

PL= **LL=** **PI=**

Classification

USCS (D 2487)= **AASHTO (M 145)=**

Coefficients

D_{90}= 2.4280 D_{85}= 2.0284 D_{60}= 1.0652
D_{50}= 0.8254 D_{30}= 0.4031 D_{15}=
D_{10}= C_u= C_c=

Remarks

F.M.=2.62

Date Received: **Date Tested:** 1/19/21

Tested By: GK

Checked By: GK

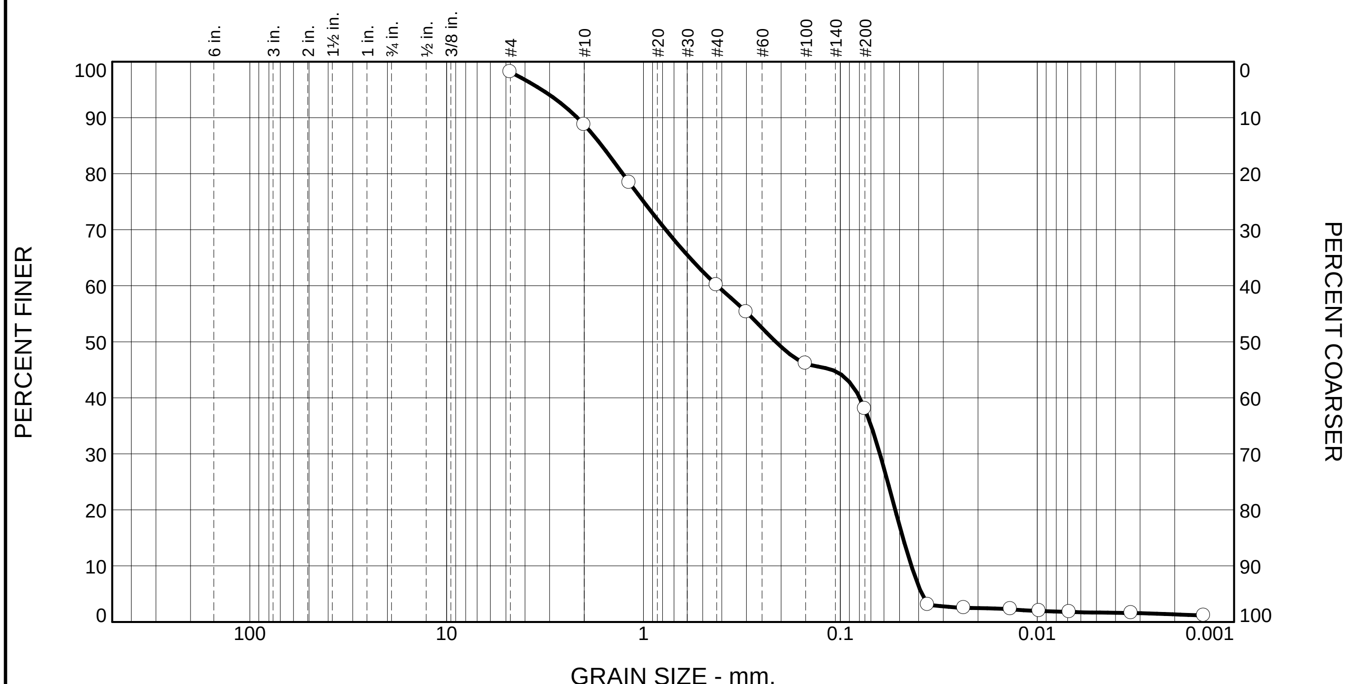
Title:

Source of Sample: B2 (Gradation) **Depth:** 25' **Date Sampled:** 11/13/20
Sample Number: B2@25

EnGEN Corporation

Client: Saddleback & Assc.
Project: Saddleback LAKE ELSINORE
Project No: 4626 GFS **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
			9.5	28.6	22.1	36.3	1.7

TEST RESULTS

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
#4	98.2		
#10	88.7		
#16	78.4		
#40	60.1		
#50	55.3		
#100	46.1		
#200	38.0		
0.0359 mm.	3.1		
0.0235 mm.	2.5		
0.0136 mm.	2.3		
0.0098 mm.	2.0		
0.0069 mm.	1.8		
0.0033 mm.	1.6		
0.0014 mm.	1.1		

* (no specification provided)

Material Description

Silty fine to medium sand, light brown (5YR 5/6)

Atterberg Limits (ASTM D 4318)

PL= **LL=** **PI=**

Classification

USCS (D 2487)= **AASHTO (M 145)=**

Coefficients

D_{90}= 2.1688 D_{85}= 1.6287 D_{60}= 0.4215
D_{50}= 0.2129 D_{30}= 0.0629 D_{15}= 0.0481
D_{10}= 0.0436 C_u= 9.67 C_c= 0.22

Remarks

F.M.=1.65

Date Received: **Date Tested:** 1/11/21

Tested By: GK

Checked By: GK

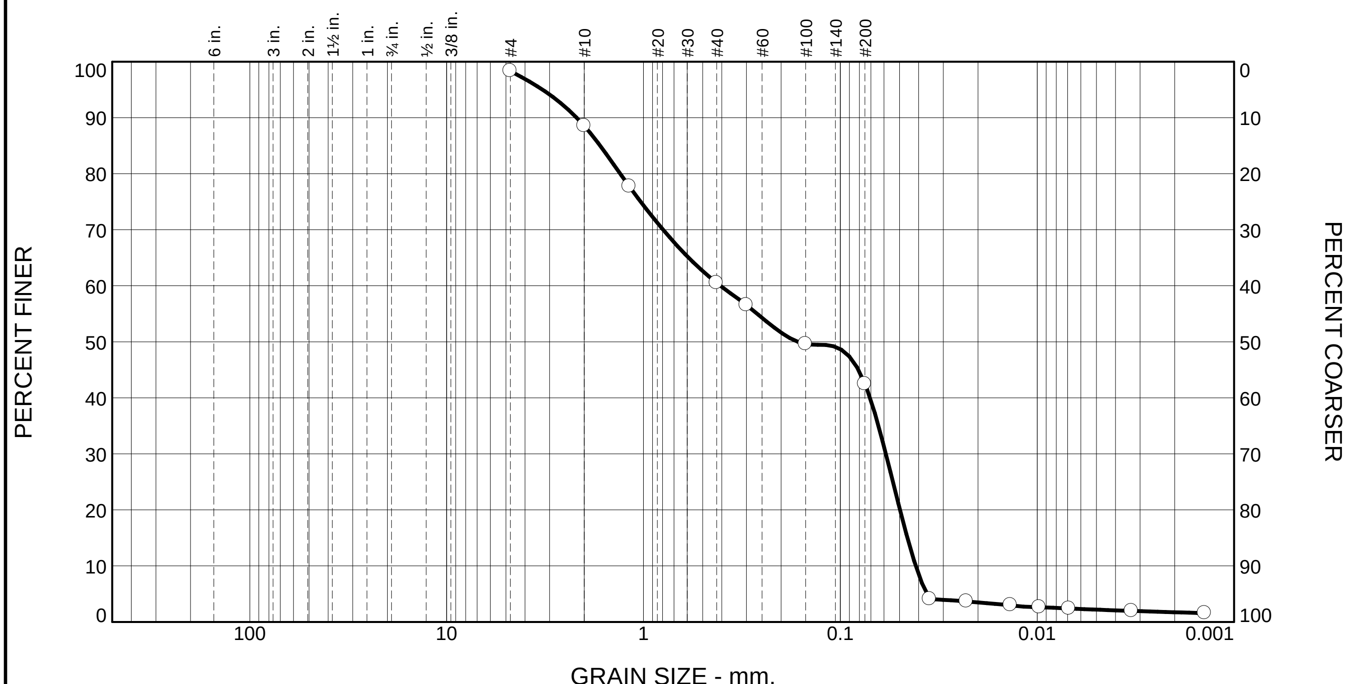
Title:

Location: B2@10'
Sample Number: B2@10 **Depth:** 10' **Date Sampled:** 11/13/20

EnGEN Corporation

Client: Saddleback & Assc.
Project: Saddleback LAKE ELSINORE
Project No: 4626 GFS **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
			9.8	28.0	18.0	40.3	2.2

TEST RESULTS

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
#4	98.3		
#10	88.5		
#16	77.7		
#40	60.5		
#50	56.6		
#100	49.6		
#200	42.5		
0.0351 mm.	4.1		
0.0228 mm.	3.7		
0.0137 mm.	3.0		
0.0098 mm.	2.6		
0.0069 mm.	2.4		
0.0033 mm.	2.0		
0.0014 mm.	1.6		

* (no specification provided)

Material Description

Silty fine sand, medium yellowish brown (5YR 5/6)

Atterberg Limits (ASTM D 4318)

PL= **LL=** **PI=**

Classification

USCS (D 2487)= **AASHTO (M 145)=**

Coefficients

D_{90}= 2.1851 D_{85}= 1.6609 D_{60}= 0.4066
D_{50}= 0.1642 D_{30}= 0.0587 D_{15}= 0.0456
D_{10}= 0.0414 C_u= 9.82 C_c= 0.20

Remarks

F.M.=1.61

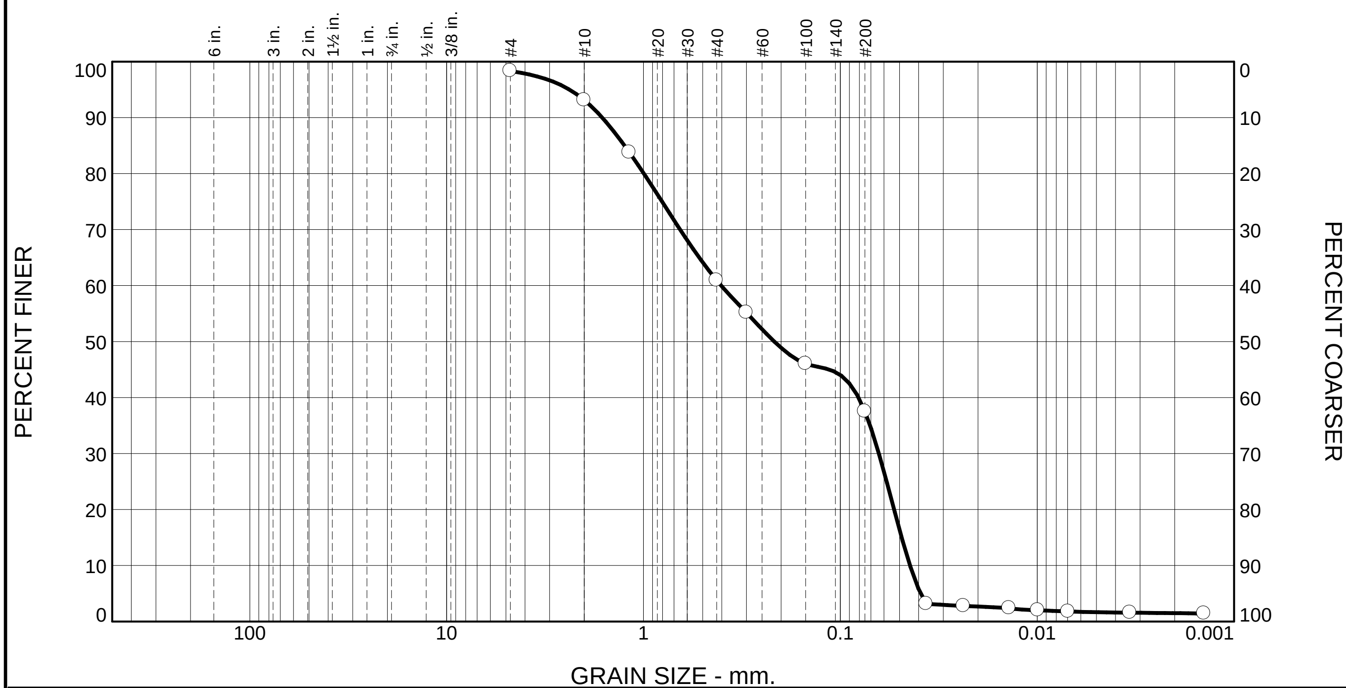
Date Received: **Date Tested:** 1/11/21
Tested By: GK
Checked By: GK
Title:

Location: B2@15'
Sample Number: B2@15 **Depth:** 15'
Date Sampled: 11/13/20

EnGEN Corporation

Client: Saddleback & Assc.
Project: Saddleback LAKE ELSINORE
Project No: 4626 GFS **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
			5.2	32.2	23.4	35.8	1.7

TEST RESULTS			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
#4	98.3		
#10	93.1		
#16	83.8		
#40	60.9		
#50	55.2		
#100	46.1		
#200	37.5		
0.0366 mm.	3.2		
0.0237 mm.	2.8		
0.0139 mm.	2.4		
0.0099 mm.	2.0		
0.0070 mm.	1.8		
0.0034 mm.	1.6		
0.0014 mm.	1.4		

* (no specification provided)

Material Description

Silty to clayey fine to medium sand, light brown (5YR 5/6)

Atterberg Limits (ASTM D 4318)

PL= **LL=** **PI=**

Classification

USCS (D 2487)= **AASHTO (M 145)=**

Coefficients

D_{90}= 1.6215 D_{85}= 1.2482 D_{60}= 0.4032
D_{50}= 0.2165 D_{30}= 0.0638 D_{15}= 0.0488
D_{10}= 0.0442 C_u= 9.11 C_c= 0.23

Remarks

F.M.=1.54

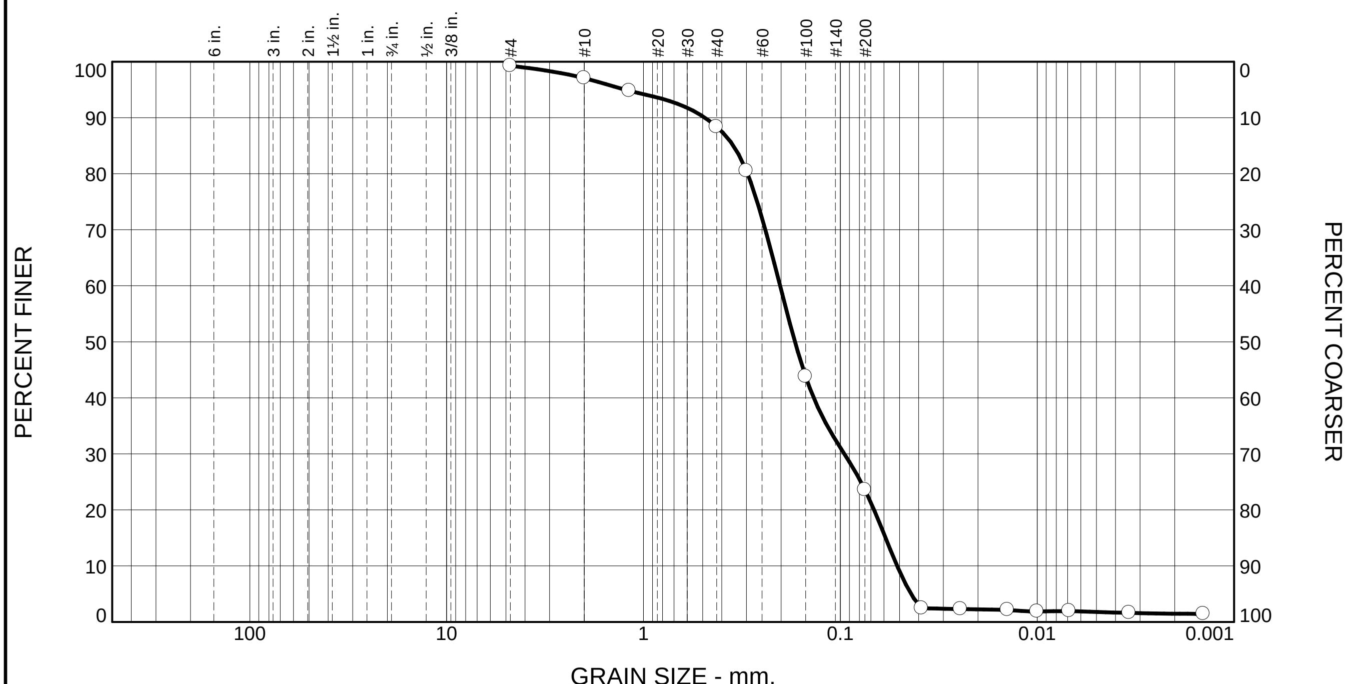
Date Received: **Date Tested:** 1/11/21
Tested By: GK
Checked By: GK
Title:

Location: B2@20'
Sample Number: B2@20 **Depth:** 20'
Date Sampled: 11/13/20

EnGEN Corporation

Client: Saddleback & Assc.
Project: Saddleback LAKE ELSINORE
Project No: 4626 GFS **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
			2.2	8.7	64.8	21.8	1.8

TEST RESULTS

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
#4	99.3		
#10	97.1		
#16	94.8		
#40	88.4		
#50	80.5		
#100	43.8		
#200	23.6		
0.0386 mm.	2.5		
0.0245 mm.	2.3		
0.0141 mm.	2.2		
0.0100 mm.	1.9		
0.0069 mm.	1.9		
0.0034 mm.	1.6		
0.0014 mm.	1.5		

* (no specification provided)

Material Description

Medium yellowish brown (5YR 5/6)

Atterberg Limits (ASTM D 4318)

PL= **LL=** **PI=**

Classification

USCS (D 2487)= **AASHTO (M 145)=**

Coefficients

D_{90}= 0.4880 D_{85}= 0.3492 D_{60}= 0.2024
D_{50}= 0.1700 D_{30}= 0.0955 D_{15}= 0.0589
D_{10}= 0.0514 C_u= 3.94 C_c= 0.88

Remarks

F.M.=0.92

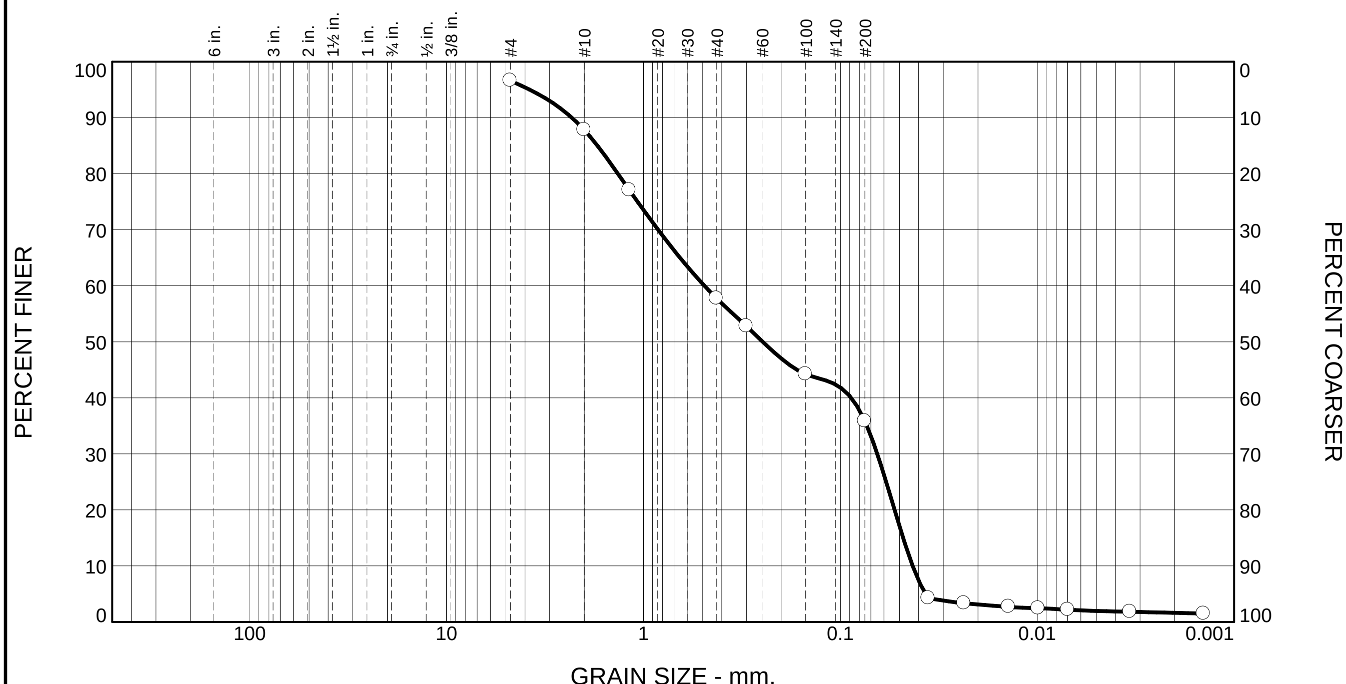
Date Received: **Date Tested:** 1/11/21
Tested By: GK
Checked By: GK
Title:

Location: B2@30'
Sample Number: B2@30 **Depth:** 30'
Date Sampled: 11/13/20

EnGEN Corporation

Client: Saddleback & Assc.
Project: Saddleback LAKE ELSINORE
Project No: 4626 GFS **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
			8.8	30.1	21.9	33.8	2.0

TEST RESULTS

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
#4	96.6		
#10	87.8		
#16	77.1		
#40	57.7		
#50	52.8		
#100	44.2		
#200	35.8		
0.0357 mm.	4.2		
0.0235 mm.	3.3		
0.0140 mm.	2.7		
0.0099 mm.	2.5		
0.0070 mm.	2.2		
0.0034 mm.	1.8		
0.0014 mm.	1.5		

* (no specification provided)

Material Description

Silty fine sand, light brown (5YR 5/6)

Atterberg Limits (ASTM D 4318)

PL= **LL=** **PI=**

Classification

USCS (D 2487)= **AASHTO (M 145)=**

Coefficients

$D_{90}=$ 2.3106 $D_{85}=$ 1.7137 $D_{60}=$ 0.4905
$D_{50}=$ 0.2480 $D_{30}=$ 0.0649 $D_{15}=$ 0.0479
$D_{10}=$ 0.0429 $C_u=$ 11.44 $C_c=$ 0.20

Remarks

F.M.=1.76

Date Received: **Date Tested:** 1/11/21

Tested By: GK

Checked By: GK

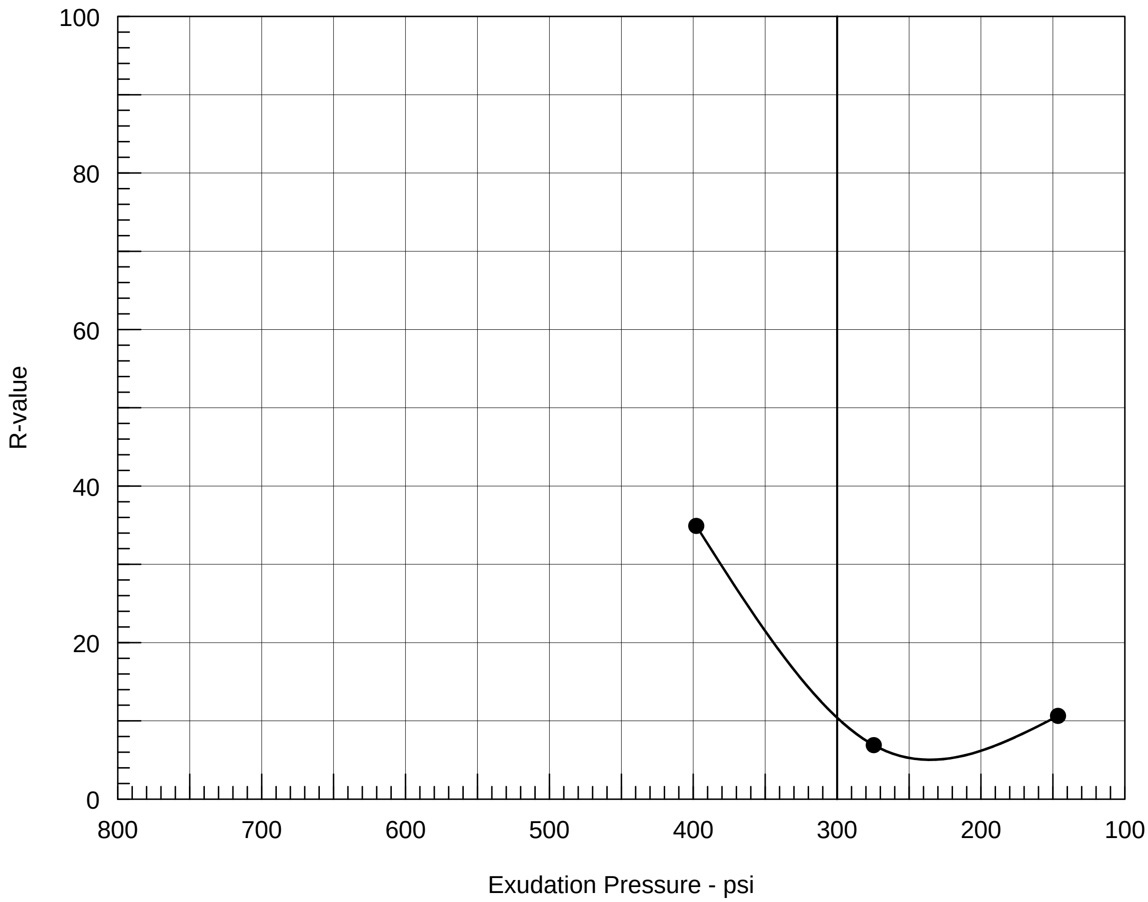
Title:

Location: B2@7.5'
Sample Number: B2@7.5 **Depth:** 7.5'
Date Sampled: 11/13/20

EnGEN Corporation

Client: Saddleback & Assc.
Project: Saddleback LAKE ELSINORE
Project No: 4626 GFS **Figure**

R-VALUE TEST REPORT



Resistance R-Value and Expansion Pressure - Cal Test 301

No.	Compact. Pressure psi	Density pcf	Moist. %	Expansion Pressure psi	Horizontal Press. psi @ 160 psi	Sample Height in.	Exud. Pressure psi	R Value	R Value Corr.
1	50	127.3	12.6	0.00	117	2.58	146	10.1	10.6
2	200	128.9	10.9	0.00	132	2.50	275	6.9	6.9
3	350	126.1	9.8	0.00	74	2.58	398	33.1	34.9

Test Results	Material Description
R-value at 300 psi exudation pressure = 10.4	Silty fine sand

Project No.: 4626GFS

Project: SADDLEBACK LAKE ELSINORE

Source of Sample: R-VALUES **Depth:** 0'-9'

Sample Number: TP2@0-9 FEET

Date: 2/4/2021

Tested by: JP

Checked by: HWB

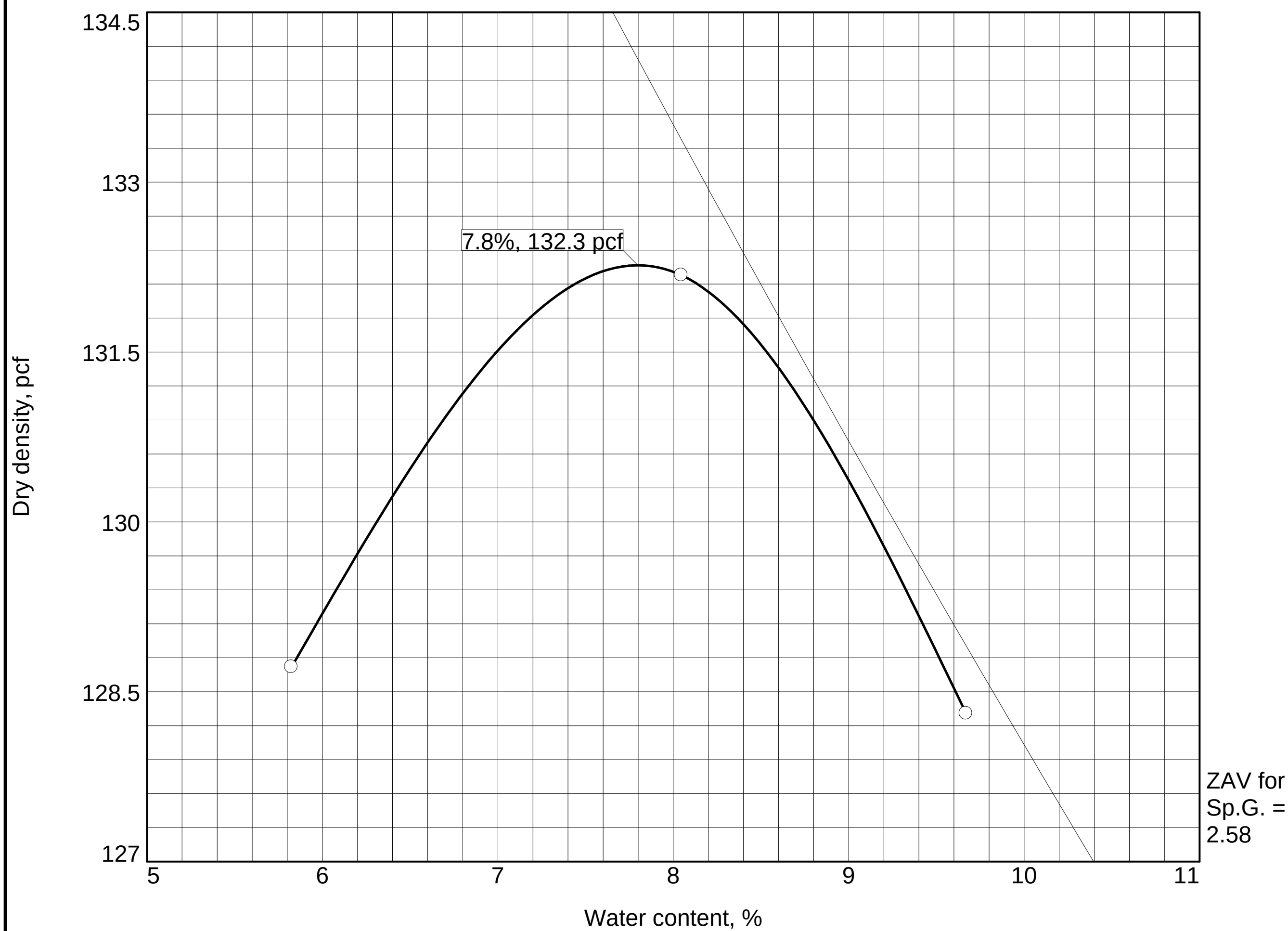
Remarks:

R-VALUE TEST REPORT

EnGEN Corporation

Figure ______

MAXIMUM DENSITY - OPTIMUM MOISTURE REPORT



Test specification: ASTM D 1557-00 Method A Modified

Elev/ Depth	Classification		Nat. Moist.	Sp.G.	LL	PI	% > #4	% < No.200
	USCS	AASHTO						
0-5'								

TEST RESULTS	MATERIAL DESCRIPTION
Maximum dry density = 132.3 pcf Optimum moisture = 7.8 %	Silty Sand

Project No. 4626GFS **Client:** Saddleback
Project: Saddleback, Lake Elsinore

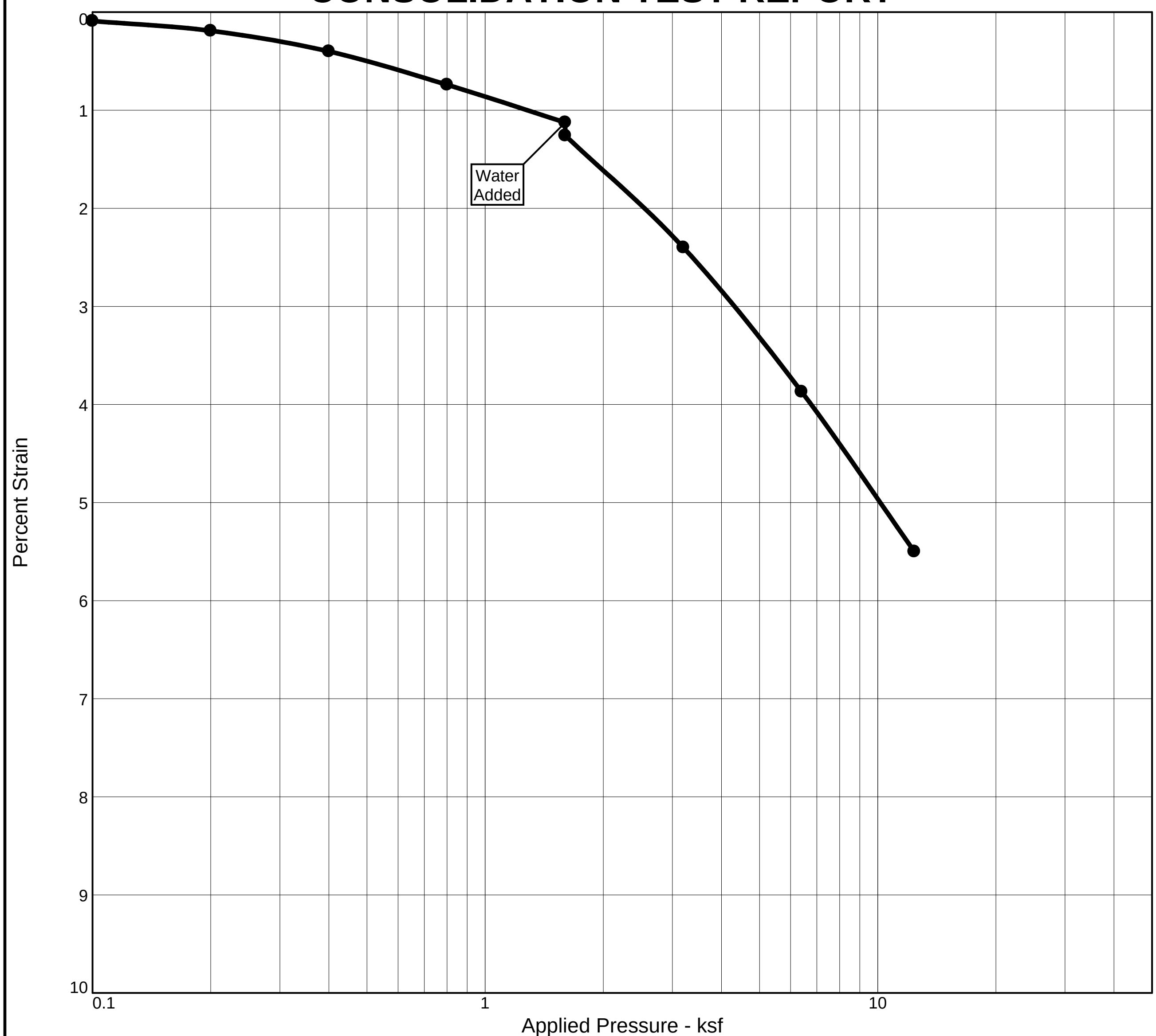
○**Location:** BH1 **Sample Number:** B1

Remarks:

EnGEN Corporation

Figure

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (ksf)	e_o	Swell Press. (ksf)	Clpse. %	C_r
Sat.	Moist.									
24.2 %	4.8 %	108.1			2.65		0.530		0.1	

MATERIAL DESCRIPTION	USCS	AASHTO
Silty fine sand, medium yellowish brown (5/YR 5/4)	SM	

Project No. 4626GFS **Client:** Saddleback & Assc.

Project: SADDLEBACK LAKE ELSINORE

Source of Sample: CONSOLS **Depth:** 20-FEET **Sample Number:** B1@20'

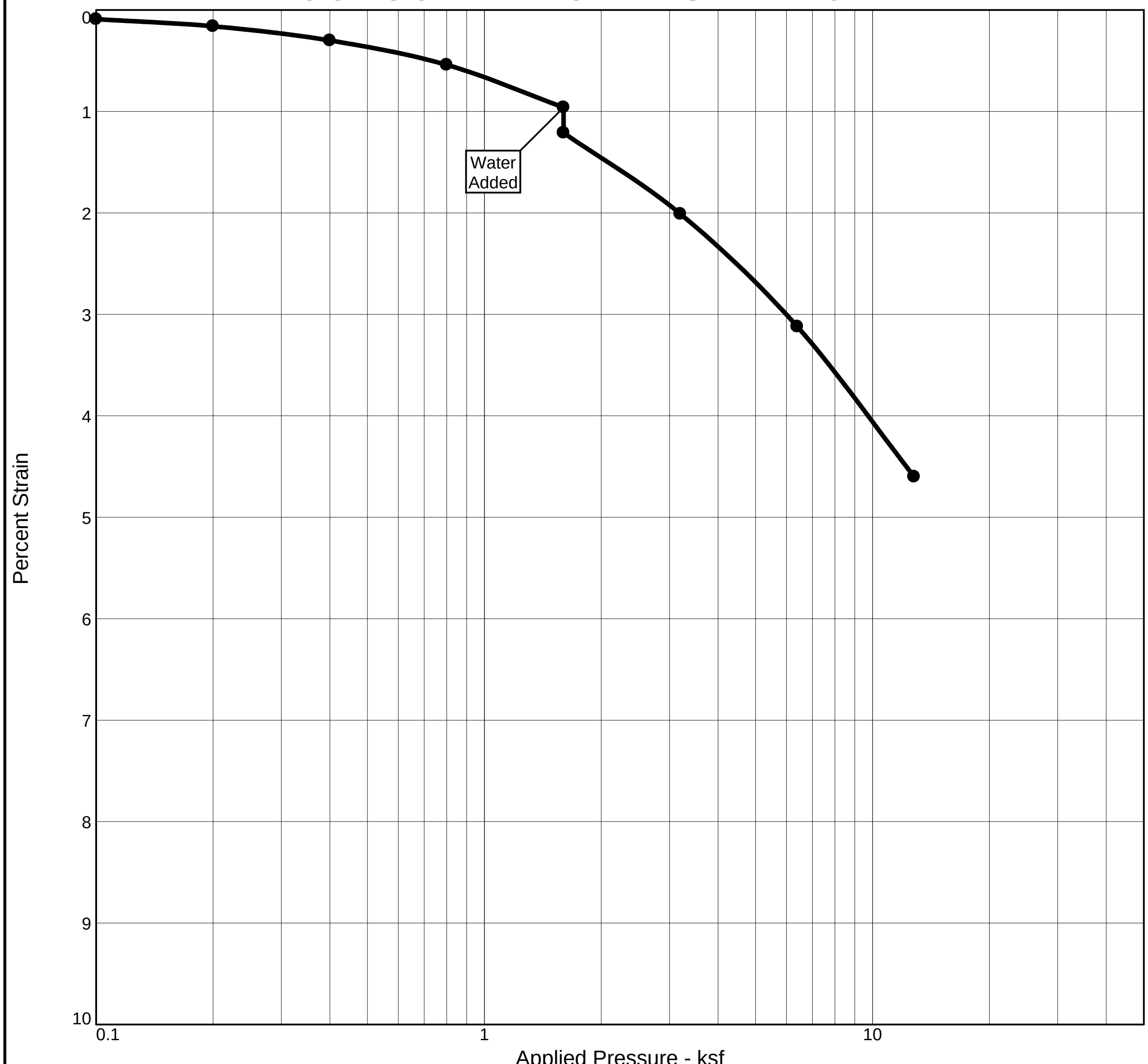
EnGEN Corporation

Remarks:
SAMPLE B1 @ 20'
COLLECTED BY CM
COLLECTED ON 11-13-2020

Figure

Tested By: GK **Checked By:** HWB

CONSOLIDATION TEST REPORT



Natural Sat.	Natural Moist.	Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (ksf)	e_o	Swell Press. (ksf)	Clpse. %	C_r
26.6 %	5.8 %	104.7			2.65		0.580		0.2	

MATERIAL DESCRIPTION	USCS	AASHTO
Silty to clayey fine to medium sand, light brown (5YR 5/6)	SM-SP	

Project No. 4626GFS **Client:** Saddleback & Assc.

Project: SADDLEBACK LAKE ELSINORE

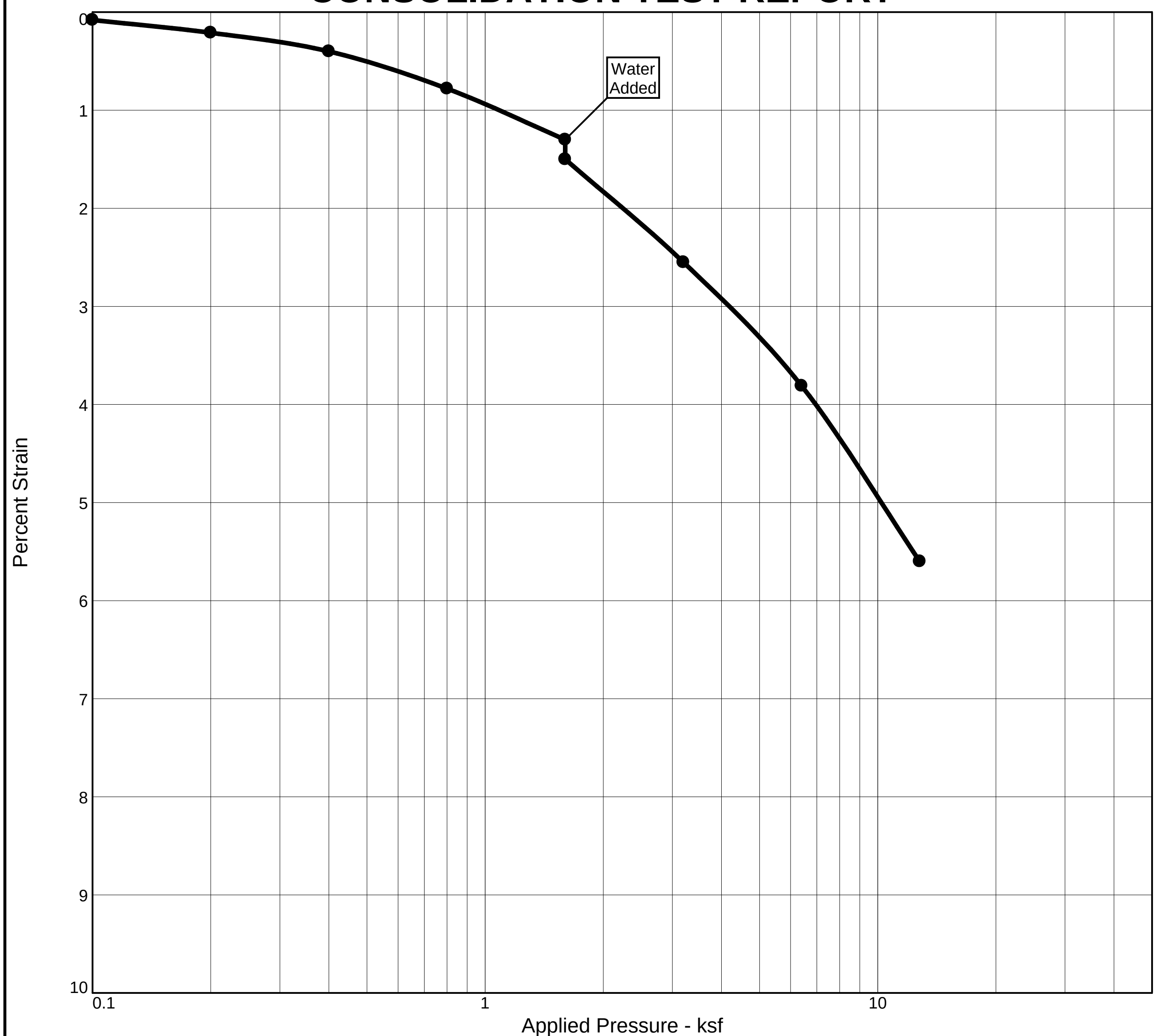
Source of Sample: CONSOLS **Depth:** 25-FEET **Sample Number:** B1@25'

Remarks:
SAMPLE B1 @ 25'
COLLECTED BY CM
COLLECTED ON 11-13-2020

EnGEN Corporation

Figure

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (ksf)	e_o	Swell Press. (ksf)	Clpse. %	C_r
Sat.	Moist.									
111.3 %	18.0 %	115.7			2.65		0.429		0.2	

MATERIAL DESCRIPTION	USCS	AASHTO
Silty to clayey fine sand, light brown (5YR 5/6)	SM	

Project No. 4626GFS **Client:** Saddleback & Assc.

Project: SADDLEBACK LAKE ELSINORE

Source of Sample: CONSOLS **Depth:** 30-FEET **Sample Number:** B1@30'

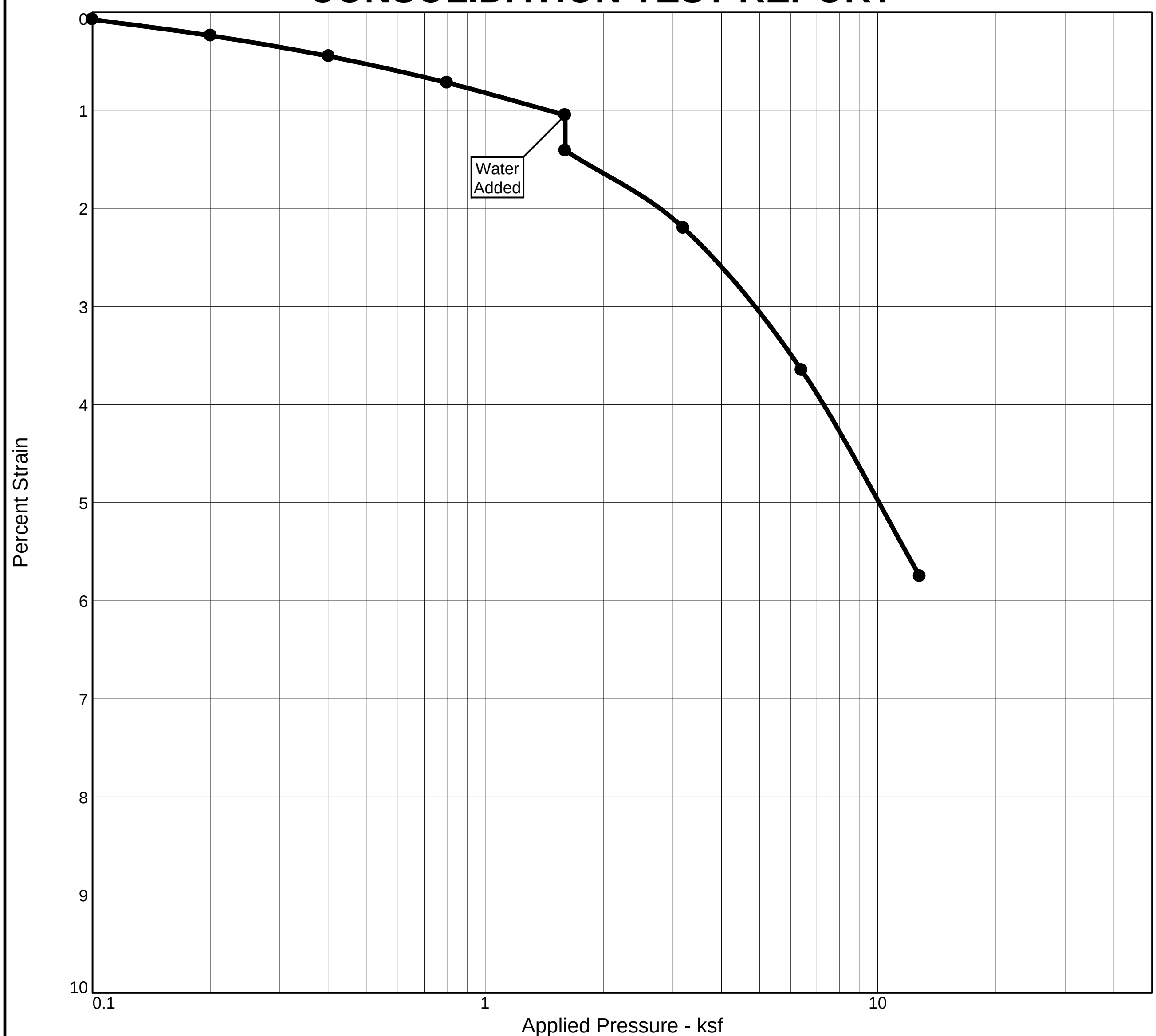
EnGEN Corporation

Remarks:
SAMPLE B1
B1 @ 30'
COLLECTED BY CM
COLLECTED ON 11-13-2020

Figure

Tested By: GK **Checked By:** HWB

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (ksf)	e_o	Swell Press. (ksf)	Clpse. %	C_r
Sat.	Moist.									
85.7 %	15.5 %	111.8			2.65		0.480		0.4	

MATERIAL DESCRIPTION	USCS	AASHTO
Silty fine sand, light brown (5YR 5/6)	SM-SP	

Project No. 4626GFS **Client:** Saddleback & Assc.

Project: SADDLEBACK LAKE ELSINORE

Source of Sample: CONSOLS **Depth:** 15-feet **Sample Number:** B4@15'

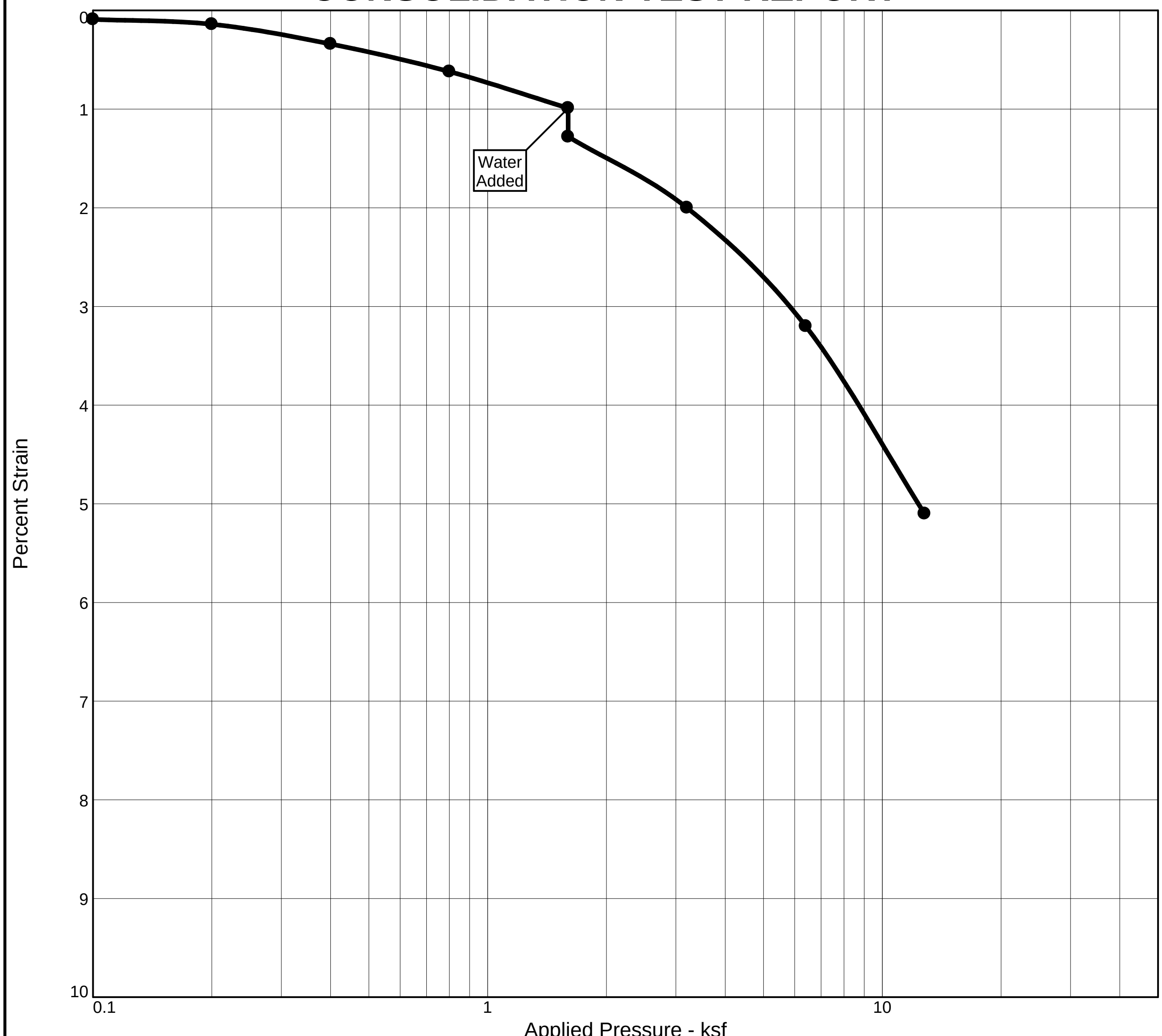
Remarks:
SAMPLE B4 @ 15'
COLLECTED BY CM
COLLECTED ON 11-13-2020

EnGEN Corporation

Figure

Tested By: GK **Checked By:** HWB

CONSOLIDATION TEST REPORT



Natural Sat.	Natural Moist.	Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (ksf)	e_o	Swell Press. (ksf)	Clpse. %	C_r
90.9 %	14.3 %	116.6			2.65		0.418		0.3	

MATERIAL DESCRIPTION	USCS	AASHTO
Silty fine to medium sand, with trace clay, medium yellowish brown (5YR 5/4)	SM	

Project No. 4626GFS **Client:** Saddleback & Assc.

Project: SADDLEBACK LAKE ELSINORE

Source of Sample: CONSOLS **Depth:** 20-FEET **Sample Number:** B4@20'

EnGEN Corporation

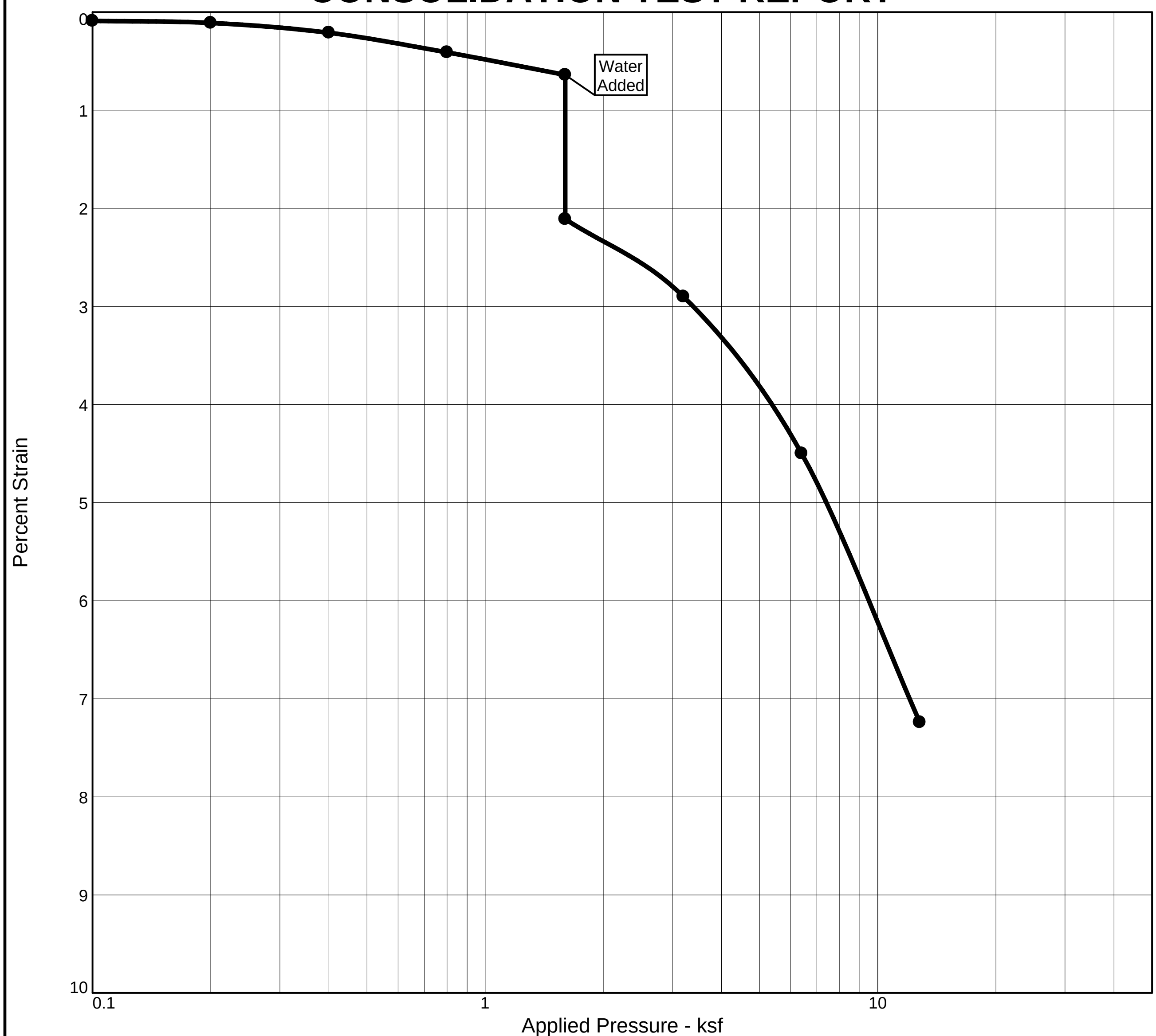
Remarks:

SAMPLE B4 @ 20'
COLLECTED BY CM
COLLECTED ON 11-13-2020

Figure

Tested By: GK **Checked By:** HWB

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (ksf)	e_o	Swell Press. (ksf)	Clpse. %	C_r
Sat.	Moist.									
69.2 %	13.1 %	110.3			2.65		0.500		1.5	

MATERIAL DESCRIPTION	USCS	AASHTO
Silty fine to medium sand, light brown (5YR 5/6)	SM	

Project No. 4626GFS **Client:** Saddleback & Assc.

Project: SADDLEBACK LAKE ELSINORE

Source of Sample: CONSOLS **Depth:** 25-FEET **Sample Number:** B4@25'

EnGEN Corporation

Remarks:

SAMPLE B4 @ 25'
COLLECTED BY CM
COLLECTED ON 11-13-2020

Figure

Tested By: GK **Checked By:** HWB

APPENDIX 3 - SUBSURFACE EXPLORATORY LOGS

GEOTECHNICAL BORING LOG

Project Number: 4626GS

Boring Number: B-1

Date: 11/13/20

Project: Saddleback Business Park

Surface Elevation: 1265

Logged By: CM

Elevation	Soil Graphic	Description	Sampler	Sample Depth	USCS	Blow Count	Dry Density	In-Situ Moisture Content	% Collapse	% Passing #200
1265		Undocumented Fill(Afu), silty fine sand, light brown (5YR 6/4) dry, loose		0	SM					
		Alluvium(Qal), silty fine sand, light brown (5YR 5/6), dry, medium dense, slightly moist, loose, pinhole pores, root hairs			SM	20-18-22				
1260		Silty fine sand, light brown (5YR 5/6), moist, loose		5	SM	5-3-3	107.4	5.5		
		Silty fine sand, light brown (5YR 5/6), moist, loose			SM	7-5-6	107.4	9.5		
1255		Silty fine to medium sand, light brown (5YR 5/6), very moist, loose		10	SM	2-2-2	107.2	11.8		
1250		Silty fine sand, medium yellowish brown (5YR 5/4), wet, loose		15	SM	2-1-5	108.2	17.5		
1245		Silty to fine to medium sand, light brown (5YR 5/6), wet, loose, heaving, partially disturbed sample		20	SM	6-5-4	107.1	5.2	0.1	
1240		Silty to fine to medium sand, light brown (5YR 5/6), wet, loose		25	SM	2-2-3	103.7	19.6	0.2	
1235		Medium yellowish brown (5YR 5/4), wet, very loose		30	SM	4-2-2	101.4	25.3	.02	
1230		Silty fine sand, medium yellowish brown (10 YR 5/4), overlying medium sand, light olive gray		35	SP-SM	3-9-18	102.2	17.4		

Notes:

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GEOTECHNICAL BORING LOG SUMMARY

Boring Number: B-1
Project Number: 4626GS
Project: Saddleback Business Park
Surface Elevation: 1265
Date: 11/13/20
Logged By: CM

Elevation	Soil Graphic	Description	Sampler	Sample Depth	USCS	Blow Count	Dry Density	In-Situ Moisture Content	% Collapse	% Passing #200
		(5YR 4/1), wet, medium dense			GP-SP					
		Santiago Formation(Tsi)								
1225		Gravelly sand to sandy gravel light olive gray (5YR 4/1), wet, very dense, gravel up to 2" in diameter		40	GP/SP	15-25-50 for 5"	n/a			
1220		Silty medium sand, light olive gray (5YR 4/1), moist, very dense		45	SM	50 for 5"	n/a			
		REFUSAL @ 45.5 FEET GROUNDWATER @ 12 FEET								
1215				50						
1210				55						
1205				60						
1200				65						
1195				70						

Notes:

GEOTECHNICAL BORING LOG

Project Number: 4626GS | **Project:** Saddleback Business Park

Boring Number: B-2 | **Surface Elevation:** 1263

Date: 11/13/20 | **Logged By:** CM

Elevation	Soil Graphic	Description	Sampler	Sample Depth	USCS	Blow Count	Dry Density	In-Situ Moisture Content	% Collapse	% Passing #200
		Undocumented Fill(Afu), silty fine sand, light brown (5YR 5/6), dry, medium dense, dry, loose		0	SM					
1260		Alluvium(Qal), silty fine sand, light brown (5YR 5/6), dry, medium dense, slightly moist, loose			SM	6-9-8				44.8
		Slightly moist, loose		5	SM	3-3-2				42.1
1255		Moist, very loose			SM	1-1-push				35.8
		Silty to clayey, fine to medium sand, light brown (5YR 5/6), wet, very loose		10	SM	1-push				38.0
1250										
		Silty fine to medium sand, light brown (5YR 5/6), wet, very loose		15	SM	1-1-1				42.5
1245										
		Silty fine to medium sand, light brown (5YR 5/6), wet, very loose		20	SM	2-1-1				37.5
1240										
		Medium sand, medium yelloish brown (10YR 5/4), wet, very loose		25	SP	1-1-3				17.7
1235										
		Silty fine sand, dark grayish brown (10YR 4/2), wet, loose		30	SM	1-3-5				23.6
		Santiago Formation (Tsi)			SP					
1230										
		Recovered as medium to coarse sand, light olive gray (5YR 6/1), wet, dense		35	SP	3-10-27				

Notes:

GEOTECHNICAL BORING LOG SUMMARY

Boring Number: B-2
Project: Saddleback Business Park
Date: 11/13/20
Project Number: 4626GS
Surface Elevation: 1263
Logged By: CM

Elevation	Soil Graphic	Description	Sampler	Sample Depth	USCS	Blow Count	Dry Density	In-Situ Moisture Content	% Collapse	% Passing #200
1225										
		Medium to course sand to fine gravel, light olive gray (5YR 6/1), wet, very dense		40	GP/SP	7-22-31				
1220										
		Medium to coarse sand, light olive gray (5YR 6/1), wet, very dense		45	SP	7-17-36				
1215		REFUSAL @ 48 FEET GROUNDWATER @ 9 FEET								
				50						
1210										
				55						
1205										
				60						
1200										
				65						
1195										
				70						
1190										

Notes:

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GEOTECHNICAL BORING LOG

Project Number: 4626GS

Project: Saddleback Business Park

Boring Number: B-3

Surface Elevation: 1267

Date: 11/13/20

Logged By: CM

Elevation	Soil Graphic	Description	Sampler	Sample Depth	USCS	Blow Count	Dry Density	In-Situ Moisture Content	% Collapse	% Passing #200
		Undocumented Fill(Afu), silty fine sand, medium yellowish brown(10YR 5/4), dry, medium dense		0	SM					
1265					SM	7-14-20				
		1.5" Rock in sampler		5	SM	10-10-16				
1260		Alluvium(Qal), silty fine to medium sand, light brown (5YR 5/6), moist, loose, pinhole pores			SM	5-5-7				
		Silty fine to medium sand, light brown (5YR 5/6), moist, loose, pinhole pores		10	SC-SM	3-3-5				
1255										
		Silty to clayey fine sand, light brown (5YR 5/6), moist, loose		15	SP-SM	3-6-7				
1250										
		Silty to clayey fine sand, light brown (5YR 5/6), wet, very loose		20	SC-SM	2-2-2				
1245										
		Silty to clayey fine sand, light brown (5YR 5/6), wet, very loose		25	SM	5-6-7				
1240										
		Silty fine sand, light brown (5YR 5/6), wet, medium dense		30	SM	7-12-19				
1235		TOTAL DEPTH 31.5 FEET GROUNDWATER @ 17 FEET								
				35						

Notes:

GEOTECHNICAL BORING LOG

Project Number: 4626GS
Boring Number: B-4
Date: 11-30-20

Project: Saddleback Business Park
Surface Elevation: 1271
Logged By: CM

Elevation	Description	Sample Depth	USCS	Blow Count	Dry Density	In-Situ Moisture Content	% Collapse	% Passing #200
1270	Alluvium(Qal), silty fine to medium sand, light brown (5YR 5/6), slightly moist, loose	0	SM					
			SM	3-4-6				
1265	Highly porous, pores are pinholes to 1/8" diameter	5	SM	4-5-5				
	Highly porous, pinhole pores		SM	3-4-5				
1260	Medium dense, highly porous, pinhole pores	10	SM	6-9-11				
1255	Silty to fine sand, light brown (5YR 5 6), moist, loose	15	SM	4-5-5			0.4	
1250	Silty fine to medium sand, light brown (5YR 5/6), wet, loose	20	SM	3-5-5			0.3	
1245	Very loose	25	SM	2-2-2			1.5	
1240	Silty fine sand overlying medium sand, light brown (5YR 5/6), wet, medium dense	30	SP-SM	2-4-6				
	TOTAL DEPTH 31.5 FEET GROUNDWATER @ 18 FEET							
1235		35						

Notes:

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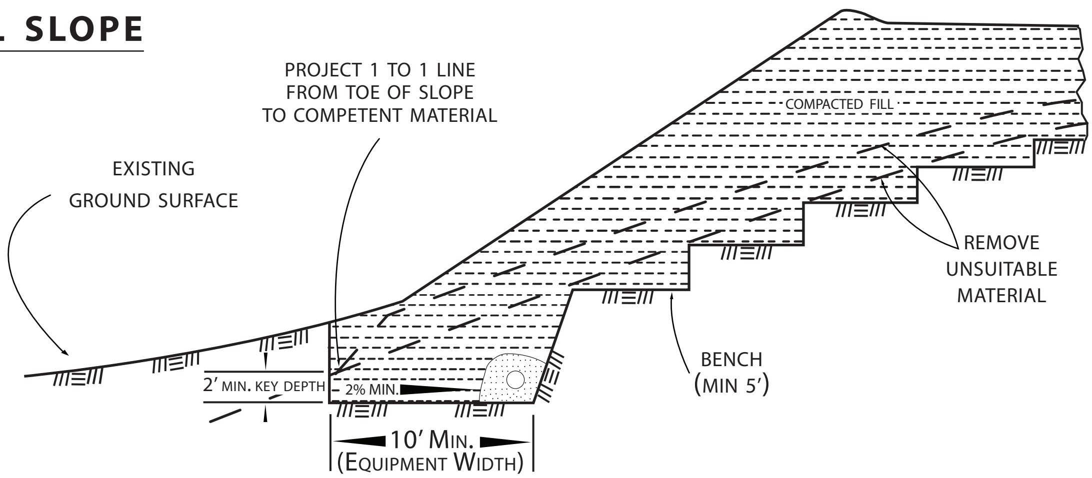
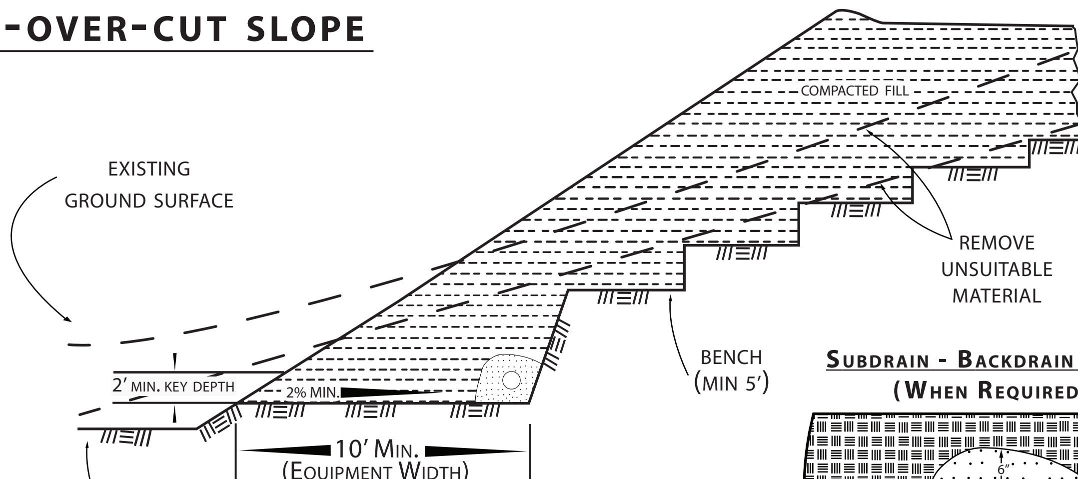
APPENDIX 4 - TYPICAL GRADING DETAILS

Key and Benching Detail

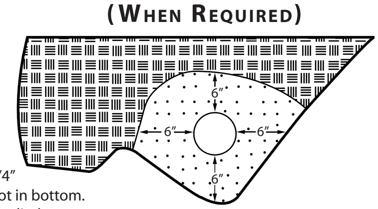
FILL SLOPE



FILL-OVER-CUT SLOPE



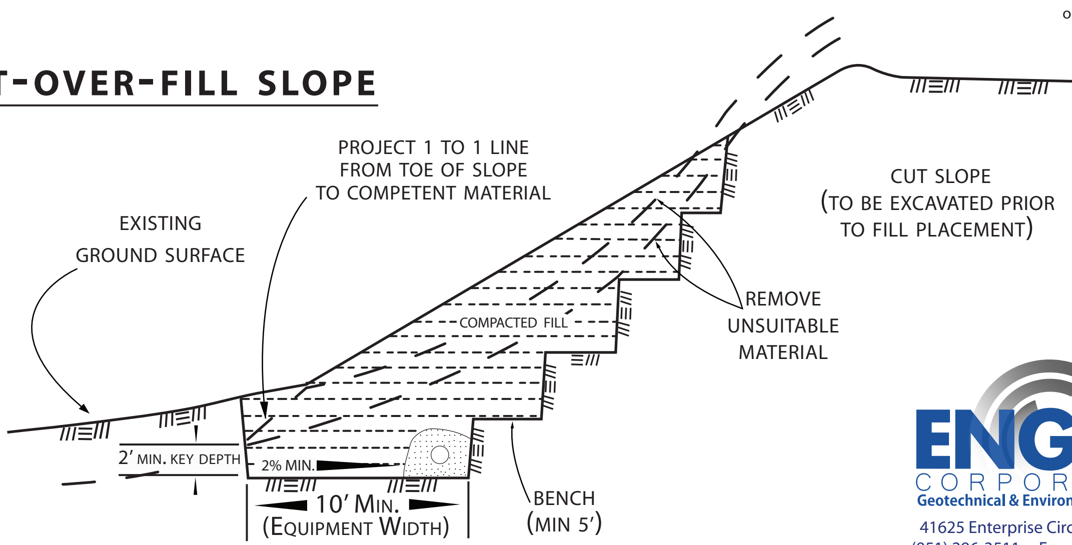
Subdrain - Backdrain Detail
(When Required)



4" Minmum Diameter ABS OR PVC Pipe or Approved substitute with minimum 8" seperation between 1/4" diameter perforations, per linear foot in bottom. Subdrain should daylight to suitable discharge facility per geotechnical engineer's approval.

Note: An approved filter fabric (Burrieto) may be wrapped around 3/4" crushed rock or pea gravel.

CUT-OVER-FILL SLOPE



Note: Back drain may be recommended by the geotechnical consultant based on actual field conditions encountered. Bench dimension recommendations may also be altered on field conditions encountered.

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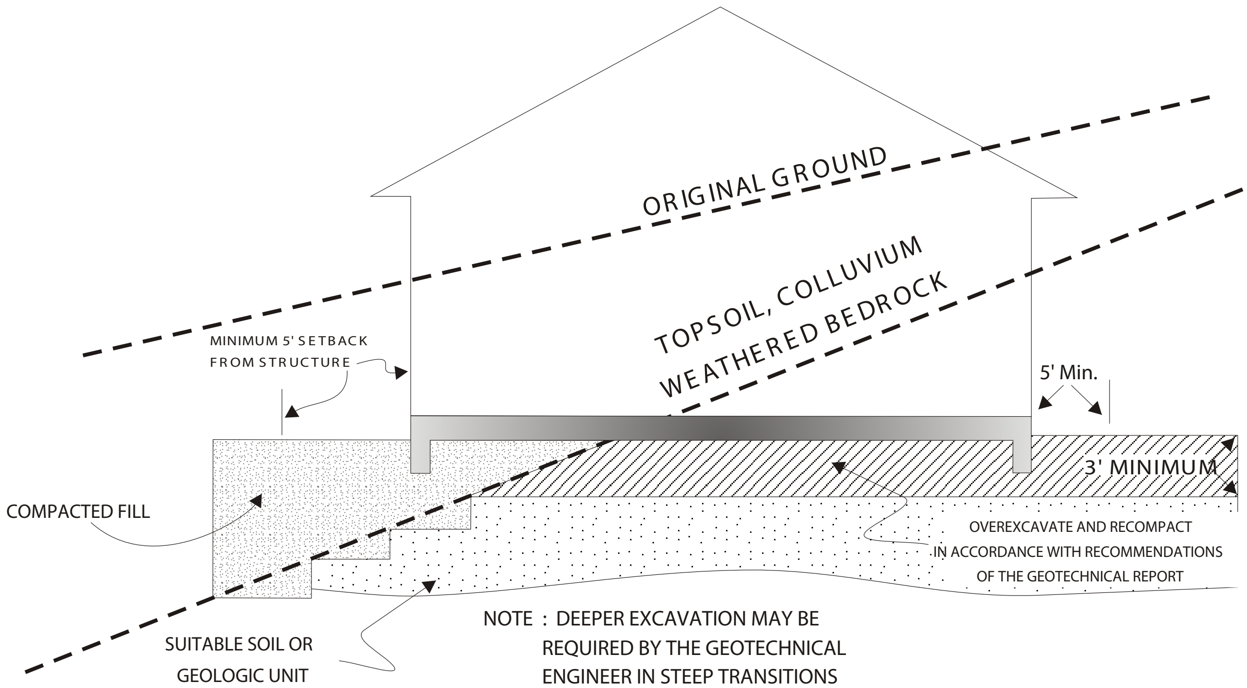
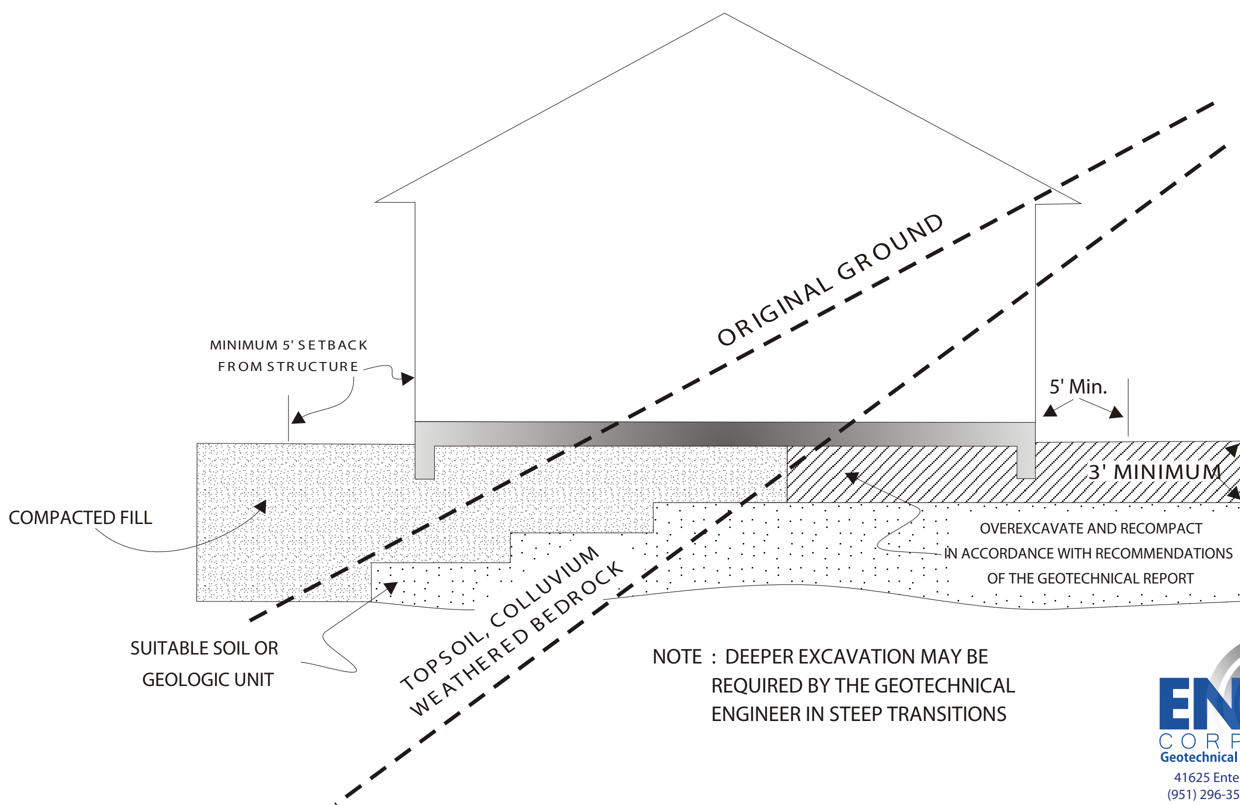
GENERAL GRADING RECOMMENDATIONS

CUT LOT



CUT-FILL LOT (TRANSITION)



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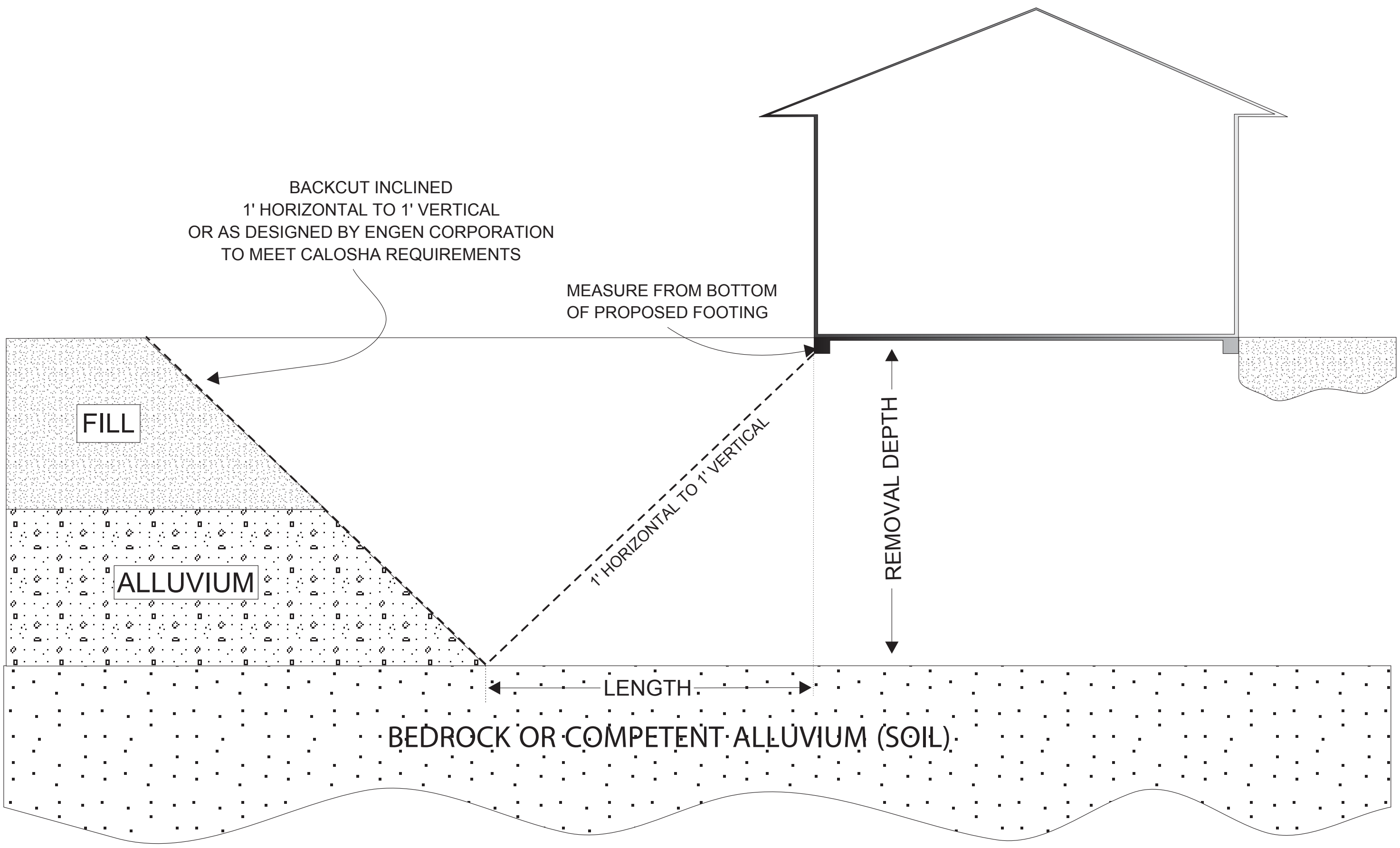
Minimum Horizontal Removal Limits



REMOVALS TO EXTEND TO THE HORIZONTAL DISTANCE OUTSIDE OF THE BUILDING LINES EQUAL TO DEPTH OF REMOVAL (LENGTH = HEIGHT)

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APPENDIX 5 – LIQUEFACTION ANALYSIS

LIQUEFACTION CALCULATIONS SUMMARY

Layer Depth Range (ft)	Field N	Soil Classification (USCS)	Estimated Fines %	Factor of Safety	Settlement (inches)	
0 to 9	17	SM	44.8	NA (groundwater)	0.00	
Groundwater at 9 ft						
9 to 13	1	SM	38.0	0.18	2.17	
13 to 18	2	SM	42.5	0.18	2.44	
18 to 23	2	SM	37.5	0.16	2.48	
23 to 28	4	SM	17.7	0.17	2.32	
28 to 32	8	SM	23.6	0.23	1.38	
					10.8	**Total**

Calculations based on:
"Soil Liqufaction During Earthquakes", EERI Monogram MNO-12, by I.M. Idriss and R.W. Boulanger, 2008.

Note:

Peak Ground Acceleration	0.945
Earthquake Magnitude	6.8
Water table depth	9 ft

APPENDIX 6 - PLATE 1 - GEOTECHNICAL FEASIBILITY STUDY PLAN

Geotchnical Study Report Site Plan

Project Name | Saddleback Business Park
Project No. | 4626GS Client | Saddleback & Assoc.
Site Plan | Plate No. | 1 Date | Feb, 2021
Scale : 1" = 80'

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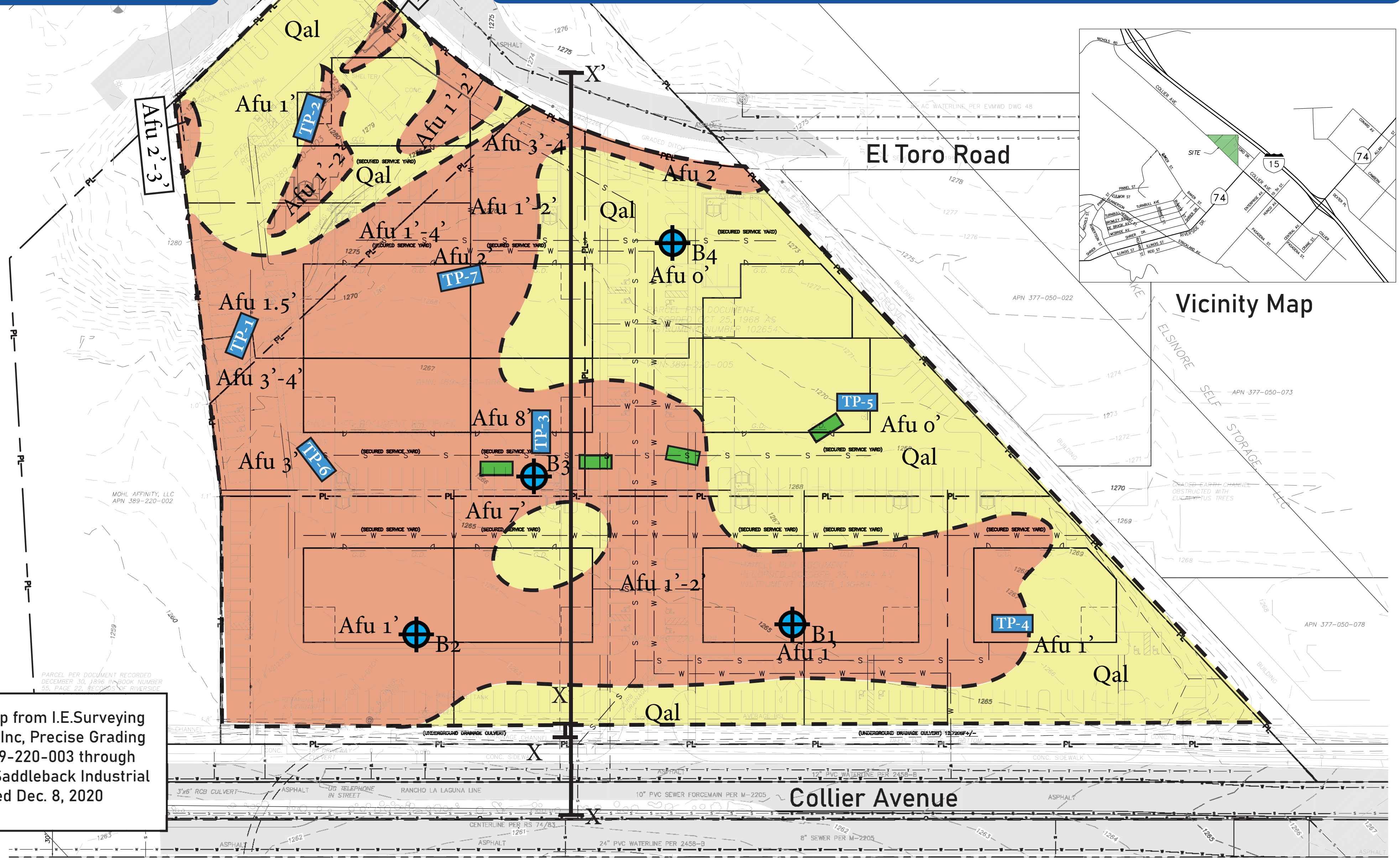
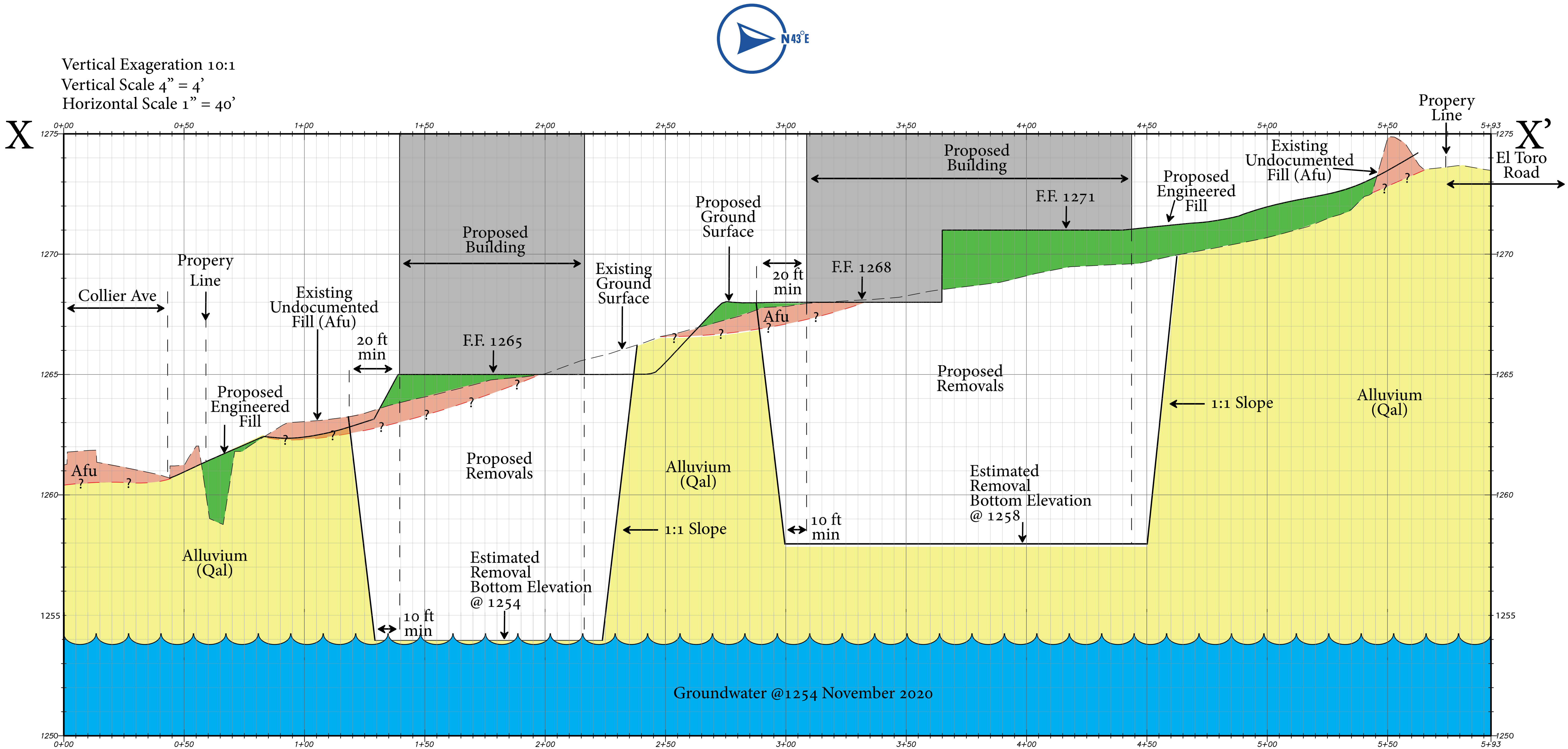
Legend

- Approximate Location Of Geologic Contact
- Afu — Undocumented Fill
- Qal — Alluvium
- B4 Afu 1' — Approximate Location Of Exploratory Boring with Esitmated Depth of Undocumented Fill
- Existing Pit, Purpose Unknown
- TP-7 Afu 1' — Approximate Location of Backhoe Test Pit with Esitmated Depth of Undocumented Fill
- X—X' — Approximate Location of Geotechnical Cross Section



Note: Base map from I.E.Surveying & Engineering Inc, Precise Grading Plan, APN 389-220-003 through 389-220-004, Saddleback Industrial plan dated Dec. 8, 2020

APPENDIX 7 - PLATE 2 - GEOTECHNICAL CROSS SECTION X-X'



Note: Base map from I.E.Surveying & Engineering Inc, Precise Grading Plan, APN 389-220-003 through 389-220-004, Saddleback Industrial plan dated Dec. 8, 2020

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Legend

	Approximate Location Of Geologic Contact		Proposed Engineered Fill
Afu	Undocumented Fill		Groundwater
Qal	Alluvium	X—X'	Approximate Location of Geotechnical Cross Section

Geotchnical Cross Section X - X'

Project Name | Saddleback Business Park

Project No. | 4626GS Client | Saddleback & Assoc.

Site Plan | Plate No. | 2 Date | Feb, 2021