EEI
Geotechnical & Environmental Solutions

DUE DILIGENCE LEVEL PRELIMINARY GEOTECHNICAL EVALUATION

Proposed Residential Development
NWC Ironwood Avenue and Oliver Street,
Moreno Valley, Riverside County, California

EEI Project No.: GLO-71982.4

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Distribution: (3) Addressee one via electronic copy

1.0 INTRODUCTION

1.1 Purpose

The purpose of this evaluation was to provide due diligence level preliminary geotechnical information to Anderson Consulting Engineers, Inc. ("Client"), regarding the subject property in the City of Moreno Valley, Riverside County, California. The information developed in this evaluation is intended to provide the Client with an understanding of the physical conditions of site-specific subsurface soils, groundwater, and the regional geologic setting which could affect the cost or design of the proposed development at the subject property (Site Vicinity Map-**Figure 1**, Aerial Site Map-**Figure 2**).

This Due Diligence Preliminary Geotechnical Evaluation has been conducted in general accordance with the accepted geotechnical engineering principles and in general conformance with the approved revised proposal and cost estimate for the project by EEI, dated September 23, 2014.

EEI conducted an onsite field exploration on October 16 and 17, 2014 that included excavation and sampling of four (4) exploratory backhoe trenches and drilling and sampling of four (4) hollow stem auger geotechnical borings for the proposed development at the subject property. Also, three (3) additional shallow borings were drilled to depths of 3 feet or less below the existing ground surface in the areas of proposed detention basins in order to perform field percolation testing. Three (3) additional hollow stem auger geotechnical borings were drilled and sampled for the proposed offsite sewer alignment located approximately ¼ mile south of the main subject property. This Due Diligence Preliminary Geotechnical Evaluation has been prepared for the sole use of Anderson Consulting Engineers, Inc. Other parties, without the express written consent of EEI and the Client should not rely upon this due diligence level preliminary geotechnical evaluation.

1.2 Project Description

We understand that the Client is considering purchasing the subject property for a residential project. Based on a Project Exhibit provided to EEI by Anderson Consulting Engineers, Inc., it appears that the subject property will be developed into approximately 146 residential building pads and associated streets and other improvements. A future offsite sewer extension is proposed for the right-of-way near the southern terminus of Oliver Street approximately ¼ mile south of the subject property. The approximate depth of the proposed sewer is about 25 feet below the existing ground surface within the Eastern Municipal Water District (EMWD) water main right-of-way. No further information is known at this time.

1.3 Scope of Services

The scope of our services included:

- A review of readily available data pertinent to the subject property, including published and unpublished geologic reports/maps, aerial photographs, local groundwater information, and soils data for the area (**References**).
- Conduct a geotechnical reconnaissance of the subject property and nearby vicinity.
- Coordinate with Underground Service Alert to identify the presence of underground utilities for clearance of proposed boring and test pit locations.

- The drilling and logging of four (4) hollow stem auger (HSA) borings (B-1 through B-4) throughout the subject property, ranging from 16 to 50.5 feet below existing grade elevations (bgs). One of the HSA exploratory borings was extended to 50.5 feet bgs for preliminary evaluation of settlement.

- The drilling and logging of three (3) additional hollow stem (HSA) auger borings (B-5 through B-7) in readily accessible areas along the proposed offsite sewer extension near the southern terminus of Oliver Street. The three (3) borings were extended to depths of approximately 26.5 feet below the existing ground surface along the approximate proposed sewer alignment

- Perform field percolation testing at three (3) locations (P-1 through P-3) at depths of approximately 3 feet below the existing ground surface at the locations of proposed detention basins as shown on the Project Exhibit. Testing was performed in general accordance with the County of Riverside guidelines for percolation test methods for preliminary percolation/infiltration information. Percolation testing results are presented in **Table 4**.

- Excavate four (4) exploratory trenches (T-1 through T-4) utilizing a backhoe in readily accessible but widely separated areas of the subject property at depths from approximately 6.5 to 9 feet below the existing ground surface.

- The locations of each of the offsite exploratory borings (for the proposed offsite sewer) are presented on **Figure 2** (Aerial Site Map/Offsite Boring Location Plan). The locations of the exploratory borings, exploratory trenches and percolation test pits on the subject property are presented on **Figure 3** (Field Exploration Plan).

- An evaluation of seismicity and geologic hazards to include an evaluation of faulting.

- Completion of laboratory testing of representative earth materials encountered onsite to ascertain their pertinent soils engineering properties, including corrosion potential (**Appendix B**).

- The preparation of this report which presents our preliminary findings, conclusions, and recommendations.

2.0 BACKGROUND

2.1 Subject Property Description

Based on the information provided by Anderson Consulting Engineers, Inc. ("Client"), and a review of the Google Earth© online database, the subject property consists of approximately 80-acres of undeveloped vacant land located at the northwest corner of the intersection of Oliver Street and Ironwood Avenue, in the City of Moreno Valley, Riverside County, California. Nason Street forms the majority of the western boundary; Ironwood borders the south; and Oliver Street forms the eastern boundary. Vacant land is present to the north. In general, the area is characterized by rural residential and vacant land. Proposed development is for multi or single-family residential.

We understand that an offsite sewer alignment is proposed which is located approximately ¼ miles south of the main subject property area. The sewer alignment is proposed to be extended approximately 900 feet to the south from the existing terminus on Oliver Street near the intersection with Carol Place.

The subject property is approximately situated at 33.9448° north latitude and 117.1871° west longitude (GoogleEarth®, 2013).

2.2 Site Topography

A review of the Sunnymead, California 7.5-minute topographic quadrangle (USGS, 2012) and Project Exhibit/topographic map prepared by Anderson Consulting Engineers, Inc. indicates that the subject property elevation varies from approximately 1,840 to 1,980 feet above mean sea level (amsl). From east-west across the property is a series of north-south oriented ridges and alternating drainage gullies in the lower, southern portion of the property. The intervening ridges are generally about 5 to 10 feet higher in elevation than the adjacent drainage gullies. Rounded granitic outcrops are exposed in the northwestern and northeastern sections of the subject property. The site topography can be generally described as a relatively well-dissected alluvial fan descending from the eroded hills to the north. The overall surface gradient across the property is gently to moderately south or south-southeast.

2.3 Geologic Setting

The subject property and vicinity lies within the Peninsular Ranges Geomorphic Province of California (CGS, 2002). The Peninsular Ranges Geomorphic Province extends from the Transverse Ranges Geomorphic Province and the Los Angeles Basin, south to Baja California. This province varies in width from about 30- to 100-miles. It is bounded on the west by the Pacific Ocean, on the south by the Gulf of California and on the east by the Colorado Desert Province. The Transverse Ranges Geomorphic Province bounds the Peninsular Ranges on the north. The Peninsular Ranges are essentially a series of northwest-southeast oriented fault blocks. Major fault zones and subordinate fault zones found in the Peninsular Ranges Province typically trend in a northwest-southeast direction.

Regional geologic maps of the subject property vicinity (Morton et al., 2004) indicate the property is underlain by Cretaceous-age plutonic rocks composed of tonalite and Holocene age Younger Alluvial Fan deposits. Outcroppings of the weathered tonalite bedrock are exposed in the northwestern and northeastern portions of the property. Over the remainder of the property, the tonalite bedrock was found to be weathering into a soil with a "decomposed granite" or "dg" texture at depth in the exploratory borings and trenches and in general is covered with several feet of alluvial and colluvial (younger alluvial fan) soils also derived from the weathered tonalite. The alluvial and colluvial soils are generally comprised of relatively loose to dense silty sand. The property is relatively undeveloped and artificial fill was not encountered during our field exploration at the property.

Due to the proximity of the subject property area to several nearby active faults, strong ground shaking could occur at the property as a result of an earthquake on any one of the faults. Our review indicates that there are no known active faults crossing the property and the property is not within a State of California Earthquake Fault Zone (Jennings, 1994; Hart and Bryant, 1997, CDMG, 1974; 1998). Due to the presence of shallow bedrock and the lack of shallow groundwater at the property, the property is considered as having a low susceptibility to liquefaction.

2.4 Regional Groundwater

A seismic hazard zone map and report have not been completed for the Sunnymead Quadrangle, therefore, the depth to the historic high groundwater at the subject property is not known. Due to the presence of relatively shallow granitic bedrock at the property, static groundwater is not expected and groundwater was not encountered in any of the exploratory borings or trenches excavated at the property to a maximum explored depth of 50.5 feet below the existing ground surface. Within the lower drainage gullies, up to 30 feet in thickness of silty sand alluvium was encountered. Although not encountered within the alluvium during our field excavation, during times of heavy precipitation or runoff, a localized perched groundwater condition could exist. A review of topographic maps of the general vicinity of the subject property indicates regional topographic relief slopes gently towards the south or south-southeast.

This information suggests that regional groundwater in the property vicinity could be inferred to flow in the same general topographic direction.

3.0 FAULTING AND SEISMICITY

The portion of Southern California that includes the subject property is considered to be seismically active. Due to the proximity of the property area to several nearby active faults, strong ground shaking could occur at the property as a result of an earthquake on any one of the faults. Our review indicates that there are no known active faults crossing the property (Blake, 2000) and the property is not within an Earthquake Fault Zone (Hart and Bryant, 1997). It is our opinion, therefore, that the likelihood of surface fault rupture at the property is low. **Table 1** lists the major active faults within 25 miles that are likely to affect the property.

TABLE 1
Summary of Major Active Faults

	Fault Name	Approximate Distance From Site miles (kilometers)	Maximum Moment Magnitude
1	San Jacinto-San Jacinto Valley	1.5 (2.4)	6.9
2	San Jacinto-San Bernardino	5.8 (9.3)	6.7
3	San Andreas – San Bernardino M-1	12.1 (19.5)	7.5
4	San Andreas – SB-Coach. M-2b	12.1 (19.5)	7.7
5	San Andreas – SB-Coach. M-1b-2	12.1 (19.5)	7.7
6	San Andreas – Whole M-1a	12.1 (19.5)	8.0
7	North Frontal Fault Zone (West)	19.4 (31.2)	7.2
8	San Jacinto – Anza	21.0 (33.8)	7.2
9	Elsinore (Glen Ivy)	21.7 (35.0)	6.8
10	Cucamonga	21.9 (35.2)	6.9
11	Elsinore (Temecula)	22.8 (36.7)	6.8
12	Cleghorn	23.1 (37.2)	6.5
13	Chino-Central Ave. (Elsinore)	23.3 (37.5)	6.7

3.1 Seismic Parameters and Peak Ground Acceleration

Maximum considered ground motion maps provided in the California Building Code (CBC, 2013) were utilized with coordinates of 33.9448° north latitude and 117.1871° west longitude, to determine the subject property seismic parameters. EEI utilized seismic design criteria provided in the CBC (2013).

In accordance with the guidelines of the CBC (2013), the spectral parameters for the subject property (based on a Site Class B soil) are estimated to be $S_s = 2.166g$ and $S_1 = 0.982g$. Review of the geotechnical data obtained during our subsurface exploration, however, indicates that the property should be classified as Class D per the CBC (Table 1613.5.2). Consequently, Site Coefficients $F_a = 1.000$ and $F_v = 1.500$ appear to be appropriate for the property. Based on this information, the adjusted maximum considered earthquake spectral response parameters $S_{MS} = 2.166g$ and $S_{M1} = 1.472g$ are recommended for seismic design of the project. Assuming an occupancy category of II (Table 1604A.5), an S_{DS} value of 1.444g and an S_{D1} value of 0.982g, the proposed building at the property can be assigned a seismic design category of D [Table 1613.5.6 (1) and (2)]. Final selection of the appropriate seismic design coefficients should be made by the structural consultant based on the local laws and ordinances, expected building response, and desired level of conservatism.

Seismic Hazard Response Parameters are listed in **Table 2**.

TABLE 2
Seismic Hazard Response Parameters and Design Parameters CBC (2013)

Latitude: 33.9448° - Longitude: -117.1871° Seismic Parameter	Period (Sec)		Value
Mapped Spectral Acceleration Value, Soil Class B	0.2	S_s	2.166g
Mapped Spectral Acceleration Value, Soil Class B	1.0	S_1	0.982g
Site Coefficient, Subject Site Soil Classification D per 2013 CBC Table 1613.5.2	--	F_a	1.000
Site Coefficient, Subject Site Soil Classification D per 2013 CBC Table 1613.5.2	--	F_v	1.500
Adjusted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Site Class D	0.2	S_{MS}	2.166g
Adjusted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Site Class D	1.0	S_{M1}	1.472g
Design Spectral Response Acceleration Occupancy Category II per 2013 CBC Table1604.5	0.2	S_{DS}	1.444g
Design Spectral Response Acceleration Occupancy Category II per 2013 CBC Table1604.5	1.0	S_{D1}	0.982g
Peak Ground Acceleration Adjusted For Site Class Effects.		PGA_M	0.837g
Building Assigned Seismic Design Category per Table 1613.5.6 (1) and (2)	--	--	D

3.2 Ground Lurching or Shallow Ground Rupture

Based on the geography, topography and site-specific geotechnical conditions encountered during our preliminary geotechnical evaluation at the subject property, we consider the potential for ground lurching or shallow ground rupture at the property to be low; however, due to the active seismicity of California, this possibility cannot be completely ruled out. In light of this, the unlikely hazard of lurching or ground-rupture should not preclude consideration of "flexible" design for onsite utility lines and connections.

3.3 Liquefaction

Liquefaction is a phenomenon in which the strength and stiffness of a soil is reduced by earthquake shaking or other rapid loading. Liquefaction and related phenomena have been responsible for substantial structural damage in historical earthquakes, and are a design concern under certain conditions. Liquefaction occurs in saturated soils that are soils in which the space between individual particles is completely filled with water. This pore water exerts a pressure on the soil particles that influences how tightly the particles themselves are pressed together.

Prior to an earthquake, pore water pressure is typically low; however, earthquake motion can cause the pore water pressure to increase to the point where the soil particles can readily move with respect to each other. When liquefaction occurs; the strength of the soil decreases and the ability of a soil deposit to support structural loads are reduced.

A seismic hazard zone map and report for the Sunnymead Quadrangle has not been issued by the California Geological Survey (CGS) so the subject property is, therefore, not situated within a mapped Liquefaction Zone. The majority of the property is underlain by generally loose to medium dense alluvial and colluvial deposits that overlie relatively shallow granitic bedrock. The alluvial and colluvial soils are subject to removal and recompaction during site grading for the proposed residential development. It appears that liquefaction is not a significant geotechnical concern at the property.

Cyclic mobility is a liquefaction phenomenon, triggered by cyclic loading, occurring in soil deposits with static shear stresses lower than the soil strength. Deformations due to cyclic mobility develop incrementally because of static and dynamic stresses that exist during an earthquake. Lateral spreading, a common result of cyclic mobility, can occur on gently sloping and on flat ground close to rivers and lakes. These conditions do exist within the subject property, however, based on the conceptual site plan, the property should be relatively level following rough grading with a lack of free channel faces and cyclic mobility should not be an issue post-grading.

3.4 Seismic Induced Settlement

Seismically induced settlement can occur due to reorientation of soil particles during strong shaking of unsaturated sands, as well as in response to liquefaction of saturated loose granular soils.

Based on our evaluation and the geotechnical data obtained from our exploratory borings and trenches, we estimate the total seismic-induced settlement to be less than 1-inch. Differential earthquake induced settlements are estimated to be less than 0.5-inches over a 50-foot span.

4.0 FIELD EXPLORATION AND LABORATORY TESTING

4.1 Field Exploration

Field work for our geotechnical evaluation was conducted on October 16 and 17, 2014. A total of four (4) hollow stem auger borings were drilled on the subject property. Boring depths ranged from 11 feet to 50.5 feet below the existing ground surface. Three (3) additional hollow stem auger borings were drilled offsite in the area of the proposed sewer line extension. All three borings were drilled to an approximate depth of 26.5 feet below existing grade. In addition to the hollow stem auger borings, four (4) exploratory backhoe trenches were excavated on the subject property to depths ranging from 6.5 to 9 feet below existing grade. All exploratory borings and trenches were logged under the supervision of a Registered Professional Engineer and/or Certified Engineering Geologist at EEI. Boring and trench locations were adjusted as necessary due to existing utilities and improvements.

Blow count (N) values were determined utilizing a 140 pound automatic hammer, falling 30-inches onto a Standard Penetration Test (SPT) split-spoon sampler and a Modified California split-tube sampler. A truck mounted hollow-stem auger (HSA) drill rig and rubber-tired backhoe were used during field work. The blows per foot (N value) required to advance the 18-inch long SPT and 12-inch long Modified California split-tube samplers was measured at various initial depths followed by 5-foot intervals, recorded on the boring logs. The boring logs and trench logs for the field exploration are presented in **Appendix A**-Soil Classification Chart and Boring & Trench Logs. Relatively "undisturbed" samples were collected in a 2.42-inch (inside diameter) California Modified split-tube sampler for visual examination and laboratory testing in the exploratory borings. The soils were classified in accordance with the Unified Soil Classification System (ASTM, 2008). Representative bulk samples were also collected from both the exploratory borings and trenches for appropriate laboratory testing.

4.2 Subsurface Conditions

The results of our geotechnical exploration indicate that the proposed residential development is underlain by weathered Cretaceous-age plutonic rocks composed of tonalite. This material was observed to extend beyond the maximum depth of our exploratory borings and test pits (approximately 50.5 feet below existing grades). Alluvial soils up to 30 feet thick were observed to mantle the weathered tonalite bedrock within the lower lying channel/drainage areas.

On the higher, elevated ridge areas on the subject property, colluvial soils were observed to mantle the weathered tonalite bedrock with a thickness varying between 3 and 14 feet. The weathered tonalite bedrock can generally be described as gray, white or black speckled or orange to dark grayish-orange (depending on degree of weathering) with a "granitic" or phaneritic texture and was generally unweathered to highly weathered. Outcroppings of the weathered tonalite bedrock are exposed in the northwestern and northeastern portions of the site. Over the remainder of the property, the tonalite bedrock was found to be weathering into a medium dense to very dense silty sand soil with a "decomposed granite" or "dg" texture at depth in the exploratory borings and test pits. The alluvial and colluvial (younger alluvial fan) soils are also derived from the weathered tonalite. The alluvial and colluvial soils are generally comprised of orange-brown or red-brown, medium brown or light gray brown, fine to coarse, damp to moist, loose to dense silty sand. The property is relatively undeveloped and artificial fill was not encountered during our field exploration at the property.

Our exploratory excavations were performed utilizing light-duty equipment which can provide general excavation characteristics of the onsite materials. Large granitic (tonalite) bedrock outcrops are present on the northeast and northwest portions of the property, along with some isolated rock outcrops within the areas of the proposed development area and generally on the higher elevations of the property. Boulders were present at the surface in these areas, some localized "core rock or floaters" should be anticipated at variable depths in these areas. Based on observed subsurface conditions in the exploratory trench excavations and borings, the "decomposed granite" or "dg" is moderately to highly weathered and/or fractured and was relatively easy to excavate to the depths indicated with a light-duty backhoe and a drill rig equipped with flight auger equipment. No refusal was encountered in any of the exploratory excavations during our field exploration.

In general, the ease of rock excavation or rippability depends on various factors such as rock type, rock hardness and density, the amount of weathering, and the existence and characteristics of discontinuities such as joint spacing, foliation, or fractures.

Due to the relatively dense character of the granitic bedrock encountered onsite, it is likely that oversized rock materials will be created during grading operations. Native earth materials appear to be suitable for use as structural fill provided they are moisture conditioned (as needed), meet EEI's recommendations for size (Section 4.2.2 Fills), and are properly compacted. Dependent upon the grading plan, some of the oversized materials may be re-used in landscape areas.

For the proposed offsite sewer line location, weathered tonalite overlain by 15 feet of alluvial soils was encountered in boring B-5. In boring B-6, colluvial soils at depth were mantled by approximately 10 feet of artificial fill. For boring B-7, colluvial soils were encountered from the surface to the total explored depth. All three offsite borings were advanced to a total depth of 26.5 feet below the existing ground surface. The weathered tonalite bedrock and alluvial soils are generally unchanged from the materials encountered on the subject property. The offsite colluvial soils can generally be described as orange brown to brown, fine to coarse, moist, medium dense to dense silty sand and sandy silt. The artificial fill soils encountered can be described as light brown, fine to coarse, and damp, medium dense silty sand. Refusal was not encountered within any of our exploratory borings or test pits. Detailed descriptions of the encountered soils are provided on the boring logs and test pit logs included as **Appendix A**.

Due to the presence of relatively shallow granitic bedrock at subject property, static groundwater is not expected and groundwater was not encountered in any of the exploratory borings or trenches excavated at the property to a maximum explored depth of 50.5 feet below the existing ground surface. Our review of ground water monitoring data from nearby wells suggests that the groundwater level may fluctuate seasonally and yearly.

It should be noted that fluctuations in the ground water level could also occur due to variations in ground surface topography, subsurface stratifications, precipitation, irrigation, and other factors which may not have been evident at the time of our exploration.

4.3 Laboratory Testing and Classification

Representative samples were selected for laboratory testing to confirm their field classification(s). Field descriptions and classifications were visually classified according to the American Society for Testing and Materials (ASTM) D2488 which classifies soils under the Unified Soil Classification System (USCS). Representative soil samples were tested in the lab for grain size distribution, liquid limits, and plastic limits to determine actual classifications by ASTM D2487-Standard Practice for Classification of Soils for Engineering Purposes in accordance to the USCS. Final classifications of soils can be found on the boring logs in **Appendix A** and the laboratory test data in **Appendix B**.

4.3.1 Moisture Content and Dry Density

The in-situ moisture content and dry density of soils were determined for soil samples obtained from the borings. Moisture contents and dry densities of soils help to determine engineering design parameters for foundations, retaining walls, and other engineered structures. Moisture content on soil samples was conducted in general accordance with ASTM D2216, and was recorded as a percentage. In-situ moisture content and dry density information for soil samples retrieved from the field can be found on the boring logs located in **Appendix A**.

4.3.2 Grain Size Distribution

To help check field classifications of soils, the grain size distribution of representative soil samples was determined. In order to find the percentages of different sized particles in a particular soil stratum, soils were tested in general accordance with ASTM D422-Standard Test Method for Particle-Size Analysis of Soils. Grain size distribution curves and gradation results are presented in **Appendix B.**

4.3.3 Maximum Dry Density and Optimum Moisture Content

The maximum dry density and optimum moisture content were determined from a bulk sample obtained from boring B-1 at depths between 0 and 5 feet below existing grade. Our testing was performed in general accordance with ASTM D1557, Method A. Results of our testing are presented in **Appendix B**.

4.3.4 Direct Shear

Direct shear testing was conducted on three representative samples of the upper soils. One sample was remolded to 90 percent of their maximum dry density (based on ASTM D1557), and the other two samples in its natural state, to measure its shear strength characteristics for engineering purposes. The samples were inundated for at least 18 hours. The samples were placed in a shear box and a normal load was applied (10, 20, and 40 kilogram weights were used). The samples were then sheared at a controlled strain rate in a direct shear apparatus that measures horizontal displacement and shear resistance. Shear testing was run in general accordance with ASTM D3080. The results of our testing are presented in **Appendix B**.

4.3.5 Expansion Index

A soil sample from boring B-1 within the upper 5 feet of existing grade was tested for its expansion potential. Our expansion index testing was conducted in general accordance with ASTM D4829. The results of our expansion index testing are presented in **Appendix B.**

4.3.6 Sulfate/Corrosion

One representative sample of onsite earth material was collected for analysis at Clarkson Laboratory and Supply, Inc. located in Chula Vista, California for corrosion/soluble sulfate potential. This corrosion testing included soil minimum resistivity and pH by California Test 643, sulfate by California Test 417, and chloride by California Test 422. Results of these tests are presented in **Appendix B**.

It should be understood that the results provided in **Appendix B** are based upon pre-development conditions. Verification testing is recommended at the conclusion of grading on samples collected at or near finish grade.

5.0 CONCLUSIONS

Based on our field exploration, laboratory testing and engineering and geologic analysis, it is our opinion that the subject property is suitable for the proposed new development and associated improvements from a geotechnical engineering and geologic viewpoint; however, there are existing geotechnical conditions associated with the property that will warrant mitigation and/or consideration during planning stages. If site plans and/or the location of the proposed residential buildings or proposed offsite sewer line are revised, additional field studies may be warranted to address proposed site-specific conditions. As a result, EEI is providing the following conclusions:

- A total of four (4) hollow stem auger borings were drilled on the subject property. Boring depths ranged from 11 feet to 50.5 feet below the existing ground surface. Three (3) additional hollow stem auger borings were drilled offsite in the area of the proposed sewer line extension. All three borings were drilled to an approximate depth of 26.5 feet below existing grade. In addition to the hollow stem auger borings, four (4) exploratory backhoe trenches were excavated on the subject property to depths ranging from 6.5 to 9 feet below existing grade. The subject property is underlain by weathered Cretaceous-age plutonic rocks composed of tonalite. This material was observed to extend beyond the maximum depth of our exploratory borings and test pits (approximately 50.5 feet below existing grades). Alluvial soils up to 30 feet thick were observed to mantle the weathered tonalite bedrock within the lower lying channel/drainage areas. On the higher, elevated ridge areas on the subject property, colluvial soils were observed to mantle the weathered tonalite bedrock with a thickness varying between 3 and 14 feet. The weathered tonalite bedrock was can generally be described as gray, white or black speckled or orange to dark grayish-orange (depending on degree of weathering) with a "granitic" or phaneritic texture and was generally unweathered to highly weathered and very soft to moderately hard. The alluvial and colluvial (younger alluvial fan) soils are also derived from the weathered tonalite. The alluvial and colluvial soils are generally comprised of orange-brown or red-brown, medium brown or light gray brown, fine to coarse, damp to moist loose to dense silty sand.

- On the subject property, the weathered tonalite bedrock was observed to be mantled by up to approximately a 30-foot thick layer of alluvium in the lower drainage/wash areas while a relatively thin layer of colluvial soils, also mainly silty sand was observed in the exploratory borings and test pits on the higher "ridges".

- For the proposed offsite sewer line location, weathered tonalite overlain by 15 feet of alluvial soils was encountered in boring B-5. In boring B-6, colluvial soils at depth were mantled by approximately 10 feet of artificial fill. For boring B-7, colluvial soils were encountered from the surface to the total explored depth. All three offsite borings were advanced to a total depth of 26.5 feet below the existing ground surface. The weathered tonalite bedrock and alluvial soils are generally unchanged from the materials encountered on the subject property. The offsite colluvial soils can generally be described as orange brown to brown, fine to coarse, moist, loose to medium dense silty sand and sandy silt. The artificial fill soils encountered can be described as light brown, fine to coarse, and damp, loose to medium dense silty sand. No refusal was encountered in any of the onsite or offsite exploratory boring or test pit locations.

- We understand that the Client is considering purchasing the subject property for a residential project. Based on a Project Exhibit provided to EEI by Anderson Consulting Engineers, Inc., it appears that the subject property will be developed into approximately 146 residential building pads and associated streets and other improvements. A future offsite sewer extension is proposed for the right-of-way near the southern terminus of Oliver Street approximately ¼ south of the subject property. The approximate depth of the proposed sewer is about 25 feet below the existing ground surface within the Eastern Municipal Water District (EMWD) water main right-of-way. No further information is known at this time.

- Groundwater was not encountered in any of our onsite or offsite exploratory borings or exploratory test pits to the depths explored (approximately 50.5 feet below existing grades). It should be noted that variations in groundwater may result from fluctuations in the ground surface topography, subsurface stratification, precipitation, irrigation and other factors that may not have been evident at the time of our subsurface exploration.

- Laboratory test results indicate that the near surface materials are near neutral (pH = 7.1) and are moderately corrosive to ferrous metals with a minimum resistivity value of 5,200 ohm-cm. Laboratory testing of the upper soils yielded a soluble sulfate concentration of 0.005 percent and a chloride concentration of 0.007 percent, indicating a negligible corrosion potential to reinforced concrete.

- The results of our laboratory Expansion Index (EI) testing indicate an expansion index of 0, for the tested soils which represents a very low expansion potential.

- The subject property is located within an area of Southern California recognized as having a number of active and potentially-active faults. Our review indicates that there are no known active faults crossing the property and the property is not located within an Earthquake Fault Zone. The nearest active faults that could affect the property are the San Jacinto Valley segment of the San Jacinto Fault Zone, located approximately 1.5 miles from the property, the San Bernardino segment of the San Jacinto Fault Zone, located approximately 5.8 miles from the area of study, the San Bernardino M-1 segment of the San Andreas Fault Zone, the San Bernardino-Coachella Valley M-2b segment of the San Andreas Fault Zone, the San Bernardino-Coachella Valley M-1b-2 segment of the San Andreas Fault Zone and the San Bernardino-Coachella Valley M-1a segment of the San Andreas Fault Zone, all located approximately 12.1 miles from the area of study. Each of these active faults is capable of generating severe ground shaking at the property. A list of active faults within an approximate 25 mile radius is presented in **Table 1**.

- Based on EEI's evaluation, earth materials underlying the subject property are not considered susceptible to liquefaction or significant amounts of seismic settlement. Based on EEI's evaluation, the earth materials underlying the subject property of the proposed development appear to be susceptible to some seismically induced settlement on the order of less than one-inch with differential settlements of less than 0.5-inches over a 50-foot span. Liquefaction-induced lateral spreading does not appear to be a concern at the subject property due to the lack of shallow groundwater, the lack of nearby open face channels and the relatively shallow depth to bedrock.

- At this time and for the purposes of this Due Diligence Level Preliminary Evaluation, we cannot present specific footing recommendations that can be incorporated in the structural design, given that we do not have a scope of the proposed project, no grading or foundation plans were available at the time and no information was provided to us other than the Client is considering purchasing the property for future development.

6.0 RECOMMENDATIONS

The recommendations presented herein should be considered as preliminary for the purpose of characterizing the geotechnical and geologic conditions at the subject property prior to purchasing the property, and for preliminary information to aid the initial planning and design phases of development. Guidelines for site preparation, earthwork, and onsite improvements are provided in the following sections based on a limited number of widely spaced exploratory borings and test pits, and the assumption that the planned onsite development will consist of single-family, wood-frame, slab-on-grade 1- to 2-story residential structures and the planned offsite construction for the proposed sewer line. For more detailed and specific recommendations for the design and planning of the proposed structures at the property, we recommend to supplement this Due Diligence Level Preliminary Geotechnical Evaluation with an additional Geotechnical Evaluation. This additional Geotechnical Evaluation would include a supplementary subsurface evaluation, incorporating additional hollow stem auger borings (HSA), to identify more specifically and in areas not covered by this Due Diligence Level Preliminary Geotechnical Evaluation, the subject property's subsurface conditions and other zones potentially susceptible to seismically induced settlement to the depths explored.

6.1 General

Grading should conform to the guidelines presented in the 2013 California Building Code (CBC, 2013), as well as the requirements of the City of Moreno Valley and the County of Riverside. Additionally, general Earthwork and Grading Guidelines are provided herein as **Appendix C**.

During earthwork construction, removals and reprocessing of fill materials, as well as general grading procedures of the contractor should be observed and the fill placed selectively tested by representatives of the geotechnical engineer, EEI. If any unusual or unexpected conditions are exposed in the field, they should be reviewed by the Geotechnical Engineer and if warranted, modified and/or additional remedial recommendations will be offered. Specific guidelines and comments pertinent to the planned development are provided herein.

The recommendations presented herein have been completed using the preliminary information provided to us regarding site development. If information concerning the proposed development is revised, or any changes in the design and location of the proposed property improvements are made, the preliminary conclusions and recommendations contained in this report should not be considered applicable unless the changes are reviewed and conclusions of this report modified or approved in writing by this office.

6.2 Site Preparation and Grading

Debris and other deleterious material, such as organic soils and/or environmentally impacted earth materials should be removed from the subject property prior to the start of grading. Areas to receive fill should be properly benched in accordance with current industry standards of practice and guidelines specified in the CBC and the requirements of the local jurisdiction.

Existing utilities should be removed within the proposed building envelope. Abandoned trenches should be properly backfilled and tested. If unanticipated subsurface improvements (utility lines, septic systems, wells, utilities, etc.) are encountered during earthwork construction, the geotechnical engineer should be informed and appropriate remedial recommendations would then be provided.

6.3 Remedial Earthwork

The encountered portions of the existing surficial soils including the upper portions of the alluvial and colluvial soils were observed to be somewhat loose and variable in moisture content and relative density. As such, they are considered potentially compressible and unsuitable for the support of settlement-sensitive structures or engineered fill in their current condition. Therefore, where not already removed by the proposed site grading, the existing materials should be completely removed and recompacted in the areas to receive the proposed building improvement and other settlement-sensitive improvements. Based on the results of our subsurface exploration, we recommend that these removals extend to approximate depths on the order of a minimum of 4 feet to a maximum of 30 feet below the existing ground surface, or 24-inches below the bottoms of the proposed foundations. At the approximate location of boring B-1, 30 feet of relatively loose to medium dense silty sand alluvium was encountered above the tonalite bedrock and should be removed to a depth of 30 feet. A similar situation likely exists in other, lower elevation portions of the site within the drainage channels.

Following removal of the upper soils, the bottom of the resulting excavation(s) should be observed by a representative of EEI to check that unsuitable materials have been sufficiently removed. It should be understood that based on the observations of our field representative, localized deeper removals may be recommended. The base of the removal area should be level to avoid differential fill thicknesses under proposed improvements. This remedial earthwork should extend at least 5 feet outside the proposed building limits and/or 5 feet beyond the area to receive fill. Note that vertical sides exceeding 5 feet in depth may be prone to sloughing and may require laying back to an inclination of 1:1 (horizontal to vertical).

After removal of the upper soils and observation of the excavation bottoms, the over-excavated areas should be scarified to a minimum depth of 6-inches, moisture conditioned as needed to achieve at least optimum moisture content and re-compacted to at least 90 percent of the maximum dry density (based on ASTM D1557). The over-excavated areas should then be backfilled with onsite and/or imported soils that are placed and compacted as recommended herein until design finish grades are reached.

6.4 Yielding Subgrade Conditions

The soils encountered at the subject property can often exhibit "pumping" or yielding once they become saturated. This can often occur in response to periods of significant precipitation, such as during the winter rainy season, or if the bottom of an excavation is situated relatively close to the groundwater level. In order to help stabilize the yielding subgrade soils within the bottom of the removal areas, the contractor can consider the placement of uniform sized, ¾- to 2-inch crushed rock within areas exhibiting the "pumping" conditions. The crushed rock should be properly tracked into the underlying soils such that it is adequately intruded into and interlocks with the soils. We expect that a 6- to 12-inch thick section of the crushed rock will be required.

Following the placement and tracking of the gravel layer into the underlying "pumping" soils, it is recommended that Mirafi 600X stabilization fabric (or approved equivalent) then be placed upon the gravel layer. Fill soils, which should be placed and compacted in accordance with the recommendations presented herein, should then be placed upon the fabric until design finish grades are reached. The gravel and stabilization fabric should extend at least 5 feet laterally beyond the limits of the "pumping" areas. These operations should be performed under the observation and testing of a representative of EEI in order to evaluate the effectiveness of these measures and to provide additional recommendations for mitigative measures, as warranted.

6.5 Fill Placement

The soils encountered at the subject property can often exhibit "pumping" or yielding once they become saturated. This can often occur in response to periods of significant precipitation, such as during the winter rainy season, or if the bottom of an excavation is situated relatively close to the groundwater level. In order to help stabilize the yielding subgrade soils within the bottom of the removal areas, the contractor can consider the placement of uniform sized, ¾- to 2-inch crushed rock within areas exhibiting the "pumping" conditions. The crushed rock should be properly tracked into the underlying soils such that it is adequately intruded into and interlocks with the soils. We expect that a 6- to 12-inch thick section of the crushed rock will be required. Following the placement and tracking of the gravel layer into the underlying "pumping" soils, it is recommended that Mirafi 600X stabilization fabric (or approved equivalent) then be placed upon the gravel layer. Fill soils, which should be placed and compacted in accordance with the recommendations presented herein, should then be placed upon the fabric until design finish grades are reached. The gravel and stabilization fabric should extend at least 5 feet laterally beyond the limits of the "pumping" areas. These operations should be performed under the observation and testing of a representative of EEI in order to evaluate the effectiveness of these measures and to provide additional recommendations for mitigative measures, as warranted.

If import soils are needed, the earthwork contractor should ensure that all proposed fill materials are approved by the Geotechnical Engineer prior to use. Representative soil samples should be made available for testing at least ten working days prior to hauling to the subject property to allow for laboratory tests.

Fill materials should be placed in 6- to 8-inch loose lifts, moisture conditioned as necessary to at least optimum moisture and compacted to a minimum of 90 percent maximum density according to ASTM D1557. The upper 12-inches of pavement subgrade should be moisture conditioned to at least optimum moisture and compacted to at least 95 percent of the maximum dry density as determined by ASTM D1557. Suitable heavy grading equipment should be utilized to properly mix, spread, moisture condition or dry, and compact each fill lift.

Those areas to receive fill (including over-excavated areas) or surface improvements should be scarified at least 6-inches, moisture conditioned to at least one percent over optimum moisture content and recompacted to at least 90 percent of the maximum dry density (based on ASTM D1557).

6.6 Shrinkage and Bulking

Several factors will impact earthwork balancing on the subject property, including shrinkage, bulking, subsidence, trench spoils from utilities and footing excavations, and final pavement section thickness as well as the accuracy of topography.

Shrinkage, bulking and subsidence are primarily dependent upon the degree of compactive effort achieved during construction. For planning purposes, the shrinkage factor is estimated to be on the order of 10 to 15 percent for the onsite natural soils to be utilized as fill. This shrinkage factor may vary with methods employed by the contractor. Subsidence is estimated to be on the order of 0.1 feet. For preliminary planning purposes, bulking of the granitic bedrock derived materials is estimated to be 0 to 10 percent. Losses from site clearing and removal of existing site improvements as well as generation of oversize material may affect earthwork quantity calculation and should be considered.

The previous estimates are intended as an aid for the project engineers in estimating earthwork quantities. It is recommended that the site development be planned to include an area that could be raised or lowered to accommodate final site balancing.

7.0 PRELIMNARY FOUNDATION RECOMMENDATIONS

7.1 General

The conclusions and recommendations presented herein, are based on the assumption that the planned development will consist of two- to four-story wood frame residential structures with slab-on-grade. It is our understanding that these conceptual plans are also part of the due diligence phase of the project and may or may not be the final design. As such, the conclusions and recommendations contained in this Due Diligence Preliminary Geotechnical Evaluation report should be considered preliminary, and should be reviewed, revised and/or approved in writing by EEI at the time the project design is finalized.

Be advised that as part of the foundation design election process, there is always a cost/benefit evaluation. Although we are providing alternatives for foundation design we have not accomplished the cost/benefit evaluation.

7.2 Preliminary Foundation Design

Lightly loaded wood-frame, two- to four-story residential buildings with a slab-on-grade, can be supported on conventional continuous or isolated spread footings bearing upon at least 24-inches of properly compacted fill materials. In preparation for foundation construction, the earthwork contractor should ensure that the site has been prepared as recommended, and that field density tests have been performed to adequately document the relative compaction of the structural fill.

Conventional foundations can be designed to impose dead plus long term live load bearing pressures of 2,000 pounds per square foot (psf). The allowable foundation bearing pressure is for footings having a minimum width of 15-inches and a minimum depth of 18-inches embedment below the lowest adjacent finish grade for one or two story buildings and 18-inches wide and a minimum 24-inches embedment below lowest adjacent finish grade for three or four-story buildings. The allowable soil bearing pressure can be increased by one-third when considering transient loads of short duration, such as wind or earthquake loads. Based on the prevailing geotechnical conditions encountered during our subsurface exploration, we recommend that foundations be reinforced with at least two No. 4 bars placed at the top of the footing and two placed at the bottom.

Horizontal loads acting on foundations and stem walls cast in open excavations against undisturbed native soil or against properly placed and compacted fill will be resisted by friction acting along the base of the footing and by passive earth pressures against the side of the footing and stem wall. The frictional resistance acting along the base of footings founded on suitable foundation soils may be computed using a coefficient of friction equal to 0.30 with the normal dead load.

Passive earth pressures acting against the side of footings and stem walls may be assumed to be equivalent to a fluid weighing 250 pounds per cubic foot. Passive pressure in the upper 1.0-foot should be neglected unless confined by concrete slabs-on-grade or asphalt concrete pavement. The values given above may be increased by one-third for transient wind or seismic loads.

7.3 Footing Setbacks

All footings should maintain a minimum 7-foot horizontal setback from the base of the footing to any descending slope (if existing onsite). This distance is measured from the outside footing face at the bearing elevation. Footings should maintain a minimum horizontal setback of H/3 (H=slope height) from the base of the footing to the descending slope face and no less than seven feet, or greater than 40 feet.

Footings adjacent to unlined drainage swales or underground utilities (if any) should be deepened to a minimum of 6-inches below the invert of the adjacent unlined swale or utilities. This distance is measured from the footing face at the bearing elevation. Footings for structures adjacent to retaining walls should be deepened so as to extend below a 1:1 projection from the heel of the wall. Alternatively, walls may be designed to accommodate structural loads from buildings or appurtenances.

7.4 Construction

The foundation construction considerations contained herein are presented as minimum preliminary recommendations from a soils engineering standpoint. Laboratory test results indicate the onsite soils' swell (expansion) potential is very low. During grading of the site, we recommend that no soil possessing an Expansion Index of more than 20 be placed within 18-inches of finish grade, if possible. As such, design parameters provided herein assume that finish grade soil materials will have a low expansion potential.

Recommendations by the project's design-structural engineer or architect, which may exceed the soils engineer's recommendations, should take precedence over the following minimum preliminary considerations. Final foundation design should be provided based on the expansion potential of the near surface soils encountered during grading.

7.5 Concrete Slab-on-Grade

Interior slabs can be grade supported by structural fill whose placement/compaction is documented by the project soils engineer/engineer geologist as recommended herein. The thickness of the slab should be in accordance with the structural engineer's design. However, based on geotechnical considerations, we recommend that concrete slabs be a minimum of 4-inches in thickness. Concrete slabs should be underlain by at least 2-inches of clean sand with a Sand Equivalent (SE) of at least 30. Where moisture condensation is undesirable, concrete slabs should be underlain with a moisture/vapor retarder consisting of a minimum 10-mil, visqueen membrane, with all laps sealed. The membrane should be underlain by a 2-inch layer of clean sand with the aforementioned sand layer placed over the visqueen to aid concrete curing. To reduce the potential for buildup of hydrostatic pressures, the free draining material under the slabs should have positive drainage with no low lying areas (i.e., depressions) created.

Floor slabs should be suitably reinforced and jointed (in accordance with Structural Engineer's recommendations) so that a small amount of independent movement can occur without causing damage. Based on the encountered geotechnical conditions, we recommend that floor slabs be reinforced with No. 4 bars spaced on 18-inch centers (each way)

The contractor should take the appropriate precautions to make sure that the reinforcement is placed and maintained within the middle one-third of the slab. Exterior slabs, such as walkways and driveways, can be adequately supported on documented structural fill that is at minimum 12-inches in thickness, and placed and compacted in accordance with the recommendations contained herein.

In preparation for slab or flatwork construction, the earthwork contractor should ensure that the onsite soils have been prepared as recommended and that field density tests have been performed to adequately document the relative compaction of the structural fill. Preparation of the native soils should be documented prior to placement of aggregate, structural components and/or fill.

Some minor cracking of slabs can be expected due to shrinkage. The potential for this slab cracking can be reduced by careful control of water/cement ratios in the concrete. The contractor should take appropriate curing precautions during the pouring of concrete in hot or windy weather to reduce the potential for cracking of slabs. We recommend that a slipsheet (or equivalent) be utilized if grouted fill, tile, or other crack-sensitive floor covering is planned directly on concrete slabs. All slabs should be designed in accordance with structural considerations.

All dedicated exterior flatwork should conform to standards provided by the governing agency including section composition, supporting material thickness and any requirements for reinforcing steel. Concrete mix proportions and construction techniques, including the addition of water and improper curing, can adversely affect the finished quality of the concrete and result in cracking and spalling of the slab. We recommend that all placement and curing be performed in accordance with procedures outlined by the American Concrete Institute and/or Portland Cement Association. Special consideration should be given to concrete placed and cured during hot or cold weather conditions. Proper control joints should be provided to reduce the potential for damage resulting from shrinkage.

Laboratory test results indicate that the upper soils contain soluble sulfate concentrations of 0.005 percent and chloride concentrations of 0.007 percent. The results of these analyses indicate a negligible corrosion potential to concrete. As such, Type II cement can be used in concrete elements that will be in contact with the upper soils.

8.0 PAVEMENT DESIGN RECOMMENDATIONS

Deleterious material, excessively wet or dry pockets, concentrated zones of oversized rock fragments, and any other unsuitable yielding materials encountered during grading should be removed. Once compacted fill and/or native soils are brought to the proposed pavement subgrade elevations, the subgrade should be proof-rolled in order to check for a uniform firm and unyielding surface. Representatives of the project geotechnical engineer should observe all grading and fill placement.

The upper 12-inches of pavement subgrade soils should be scarified; moisture conditioned to at least optimum moisture content and compacted to at least 95 percent of the laboratory standard (ASTM D1557). If loose or yielding materials are encountered during subgrade preparation, evaluation should be performed by EEI. Aggregate base materials should be properly prepared (i.e., processed and moisture conditioned) and compacted to at least 95 percent of the maximum dry density as determined by ASTM D1557. Aggregate base materials should conform to Caltrans specifications for Class 2 aggregate base.

All pavement section changes should be properly transitioned. Although not anticipated, if adverse conditions are encountered during the preparation of subgrade materials, special construction methods may need to be employed. A representative of the project geotechnical engineer should be present for the preparation of subgrade and aggregate base.

For design purposes we have assumed a Traffic Index (TI) of 4.5 for the proposed parking areas and 6.0 for drive areas at the subject property. These assumed TI's should be verified as necessary by the Civil Engineer or Traffic Engineer. For preliminary design purposes, we have conservatively assumed a preliminary R-Value of 20 for the materials likely to be exposed at subgrade. The modulus of subgrade reaction (K-Value) was estimated at 70 pounds per square inch per inch (psi/in) for an R-Value of 20 (Caltrans, 1974).

TABLE 3
Preliminary Pavement Design Recommendations

Traffic Index (TI)	Pavement Surface	Aggregate Base Material (1)
4.5 – Parking Stalls	3.0-inches Asphalt Concrete	6.0-inches
6.0 – Main Drive Areas	4.0-inches Asphalt Concrete	8.0-inches
Trash Area and Concrete Pavement	5.5-inches Portland Cement Concrete (2)	Optional

(1) R-Value of 78 for Caltrans Class 2 aggregate base
(2) Reinforcement and control joints placed in accordance with the structural engineer's requirements

The recommended pavement sections provided above are intended as a preliminary minimum guideline. If thinner or highly variable pavement sections are constructed, increased maintenance and repair could be expected. If the ADT (average daily traffic) or ADTT (average daily truck traffic) increases beyond that intended, as reflected by the assumed traffic index used for design, increased maintenance and repair could be required for the pavement section. Final pavement design should be verified by testing of soils exposed at subgrade after grading has been completed. Thicker pavement sections could result if R-Value testing indicates lower values.

9.0 DEVELOPMENT RECOMMENDATIONS

9.1 Landscape Maintenance and Planting

Water is known to decrease the physical strength of earth materials, significantly reducing stability by high moisture conditions. Surface drainage away from foundations and graded slopes should be maintained. Only the volume and frequency of irrigation necessary to sustain plant life should be applied.

Consideration should be given to selecting lightweight, deep rooted types of landscape vegetation which require low irrigation that are capable of surviving the local climate. From a soils engineering viewpoint, "leaching" of the onsite soils is not recommended for establishing landscaping. If landscape soils are processed for the addition of amendments, the processed soils should be re-compacted to at least 90 percent relative compaction (based on ASTM D1557).

9.2 Site Drainage

Positive site drainage should be maintained at all times. Drainage should not flow uncontrolled over slopes or the subject parcel. Runoff should be channeled away from slopes and structures and not allowed to pond and/or seep uncontrolled into the ground. Pad drainage should be directed toward an acceptable outlet. Although not required, roof gutters and down spouts may be considered to control roof drainage, discharging a minimum of 10 feet from the proposed structures, or into a subsurface drainage system. Consideration should be given to eliminating open bottom planters directly adjacent to proposed structures for a minimum distance of ten feet. As an alternative, closed-bottom type planters could be utilized, with a properly designed drain outlet placed in the bottom of the planter.

9.3 Stormwater Disposal Systems

EEI understands that current plans call for runoff generated from the facility to be disposed of in engineered subsurface features onsite.

9.3.1 Percolation Testing

During our subsurface exploration at the subject property, EEI conducted percolation testing in three widely separated locations (P1, P2 and P3) near the southern property boundary at the locations for proposed detention basins as shown on the conceptual site plan. Our testing was performed at an approximate depth of approximately 3 feet below the existing ground surface. A minimum 2-inch layer of ½-inch diameter crushed gravel was placed at the bottom of the excavation prior to testing. The approximate locations of our percolation test borings are provided on **Figure 3**.

Percolation testing was conducted by one of EEI's field geologists under the guidance of a Registered Engineering Geologist and Registered Civil Engineer with EEI. In general accordance with the County of Riverside guidelines for percolation testing, the percolation test locations were pre-soaked by pouring at least 5 gallons of water into the excavation. Testing was started after the hole was allowed to pre-soak for at least one hour. During testing, a minimum of 12-inches of water was placed in the excavation and the rate of the water drop was recorded at approximately 10 minute intervals. This procedure was repeated for the test hole until rates varied generally less than 10 percent for the test hole. We note that a soil profile's percolation rate is not the same as its infiltration rate. Therefore, the measured/calculated field percolation rate was converted to an estimated infiltration rate utilizing a reduction factor known as the Porchet method (Ritzema, 1974). Upon conclusion of testing, the perforated pipe was removed and the test excavation was backfilled. Results of percolation testing are presented in the following table, **Table 4**.

TABLE 4
Percolation Test Results

Test		Depth of Test (feet below existing grade)	Stabilized Percolation Rate (in/hr)	Infiltration Rate (in/hr)
P1		3	3.84	0.82
P2		3	3.60	0.50
P3		3	3.60	0.45

9.3.2 Summary of Findings

Based on the results of our percolation testing, it appears that a preliminary tested infiltration rate of 0.45-inches per hour can be used in preliminary design of subsurface stormwater retention/disposal devices at the property.

9.3.3 Structural Setback from Retention Devices

It is recommended that retention/disposal devices be situated at least three times their depth, or a minimum of 15 feet (whichever is greater), from the outside bottom edge of structural foundations. Structural foundations include (but are not limited to) buildings, loading docks, retaining walls, and screen walls.

9.4 Additional Site Improvements

Recommendations for additional grading, exterior concrete flatwork design and construction can be provided upon request. If in the future, additional property improvements are planned for the subject property, recommendations concerning the design and construction of improvements would be provided upon request.

9.5 Trenching

All temporary excavations for grading purposes and installation of underground utilities should be constructed in accordance with OSHA guidelines and local safety codes. Temporary excavations over 5 feet in height should be evaluated by the project engineer, and could require shoring, sloping, or a combination thereof. Temporary excavations within the onsite materials should be stable at 1:1 inclinations for cuts less than 10 feet in height.

Footing trench excavations for structures and walls should be observed and approved by a representative of the project soils engineer prior to placing reinforcement. Footing trench spoil and excess soils generated from utility trench excavations should be compacted to a minimum relative compaction of 90 percent (based on ASTM D1557) if not removed from the subject property. All excavations should conform to OSHA and local safety codes.

9.6 Utility Backfill

Fill around the pipe should be placed in accordance with details shown on the drawings, and should be placed in layers not to exceed 8-inches loose (unless otherwise approved by the geotechnical engineer) and compacted to at least 90 percent of the maximum dry density as determined in accordance with ASTM D1557 (Modified Proctor). The geotechnical engineer should approve all backfill material. Select material should be used when called for on the drawings, or when recommended by the geotechnical engineer. Care should be taken during backfill and compaction operations to maintain alignment and prevent damage to the joints. The backfill should be kept free from oversized material, chunks of highly plastic clay, or other objectionable material. Backfill soils should be non-expansive, non-corrosive, and compatible with native earth materials. Backfill materials and testing should be in accordance with the CBC (2013), and the requirements of the local governing jurisdiction.

Pipe backfill areas should be graded and maintained in such a condition that erosion or saturation will not damage the pipe bed or backfill. Flooding trench backfill is not recommended. Heavy equipment should not be operated over any pipe until it has been properly backfilled with a minimum of 2 to 3 feet of cover. The utility trench should be systematically backfilled to allow maximum time for natural settlement. Backfill should not occur over porous, wet, or spongy subgrade surfaces. Should these conditions exist, the areas should be removed, replaced and recompacted.

10.0 PLAN REVIEW

Once detailed site and grading plans are available, they should be submitted to this office for review and comment, to reduce the potential for discrepancies between plans and the preliminary recommendations presented herein. If conditions are found to differ substantially from those stated, appropriate recommendations would be provided. Additional field studies may be warranted.

11.0 LIMITATIONS

This Due Diligence Level Preliminary Geotechnical Evaluation has been conducted in accordance with generally accepted geotechnical engineering principles and practices. Findings provided herein have been derived in accordance with current standards of practice, and no warranty is expressed or implied. Standards of practice are subject to change with time. This Preliminary Evaluation report has been prepared for the sole use of Anderson Consulting Engineers, Inc. (Client), within a reasonable time from its authorization. Site conditions, land use (both onsite and offsite), or other factors may change as a result of man-made influences, and additional work may be required with the passage of time.

This Due Diligence Level Preliminary Geotechnical Evaluation should not be relied upon by other parties without the express written consent of EEI and the Client; therefore, any use or reliance upon this geotechnical evaluation by a party other than the Client should be solely at the risk of such third party and without legal recourse against EEI, its employees, officers, or directors, regardless of whether the action in which recovery of damages is brought or based upon contract, tort, statue, or otherwise. The Client has the responsibility to see that all parties to the project, including the designer, contractor, subcontractor, and building official, etc. are aware of this report in its complete form. This report contains information that may be used in the preparation of contract specifications; however, the report is not designed as a specification document, and may not contain sufficient information for use without additional assessment. EEI assumes no responsibility or liability for work or testing performed by others. In addition, this report may be subject to review by the controlling authorities.

12.0 REFERENCES

American Society of Civil Engineers (ASCE), 2005, Minimum Design Loads for Buildings and Other Structures, ASCE Document ASCE/SEI 7-05.

American Society for Testing and Materials (ASTM), 2008, Annual Book of ASTM Standards, Volume 04.08, Construction: Soil and Rock (I), Standards D 420 - D 5876.

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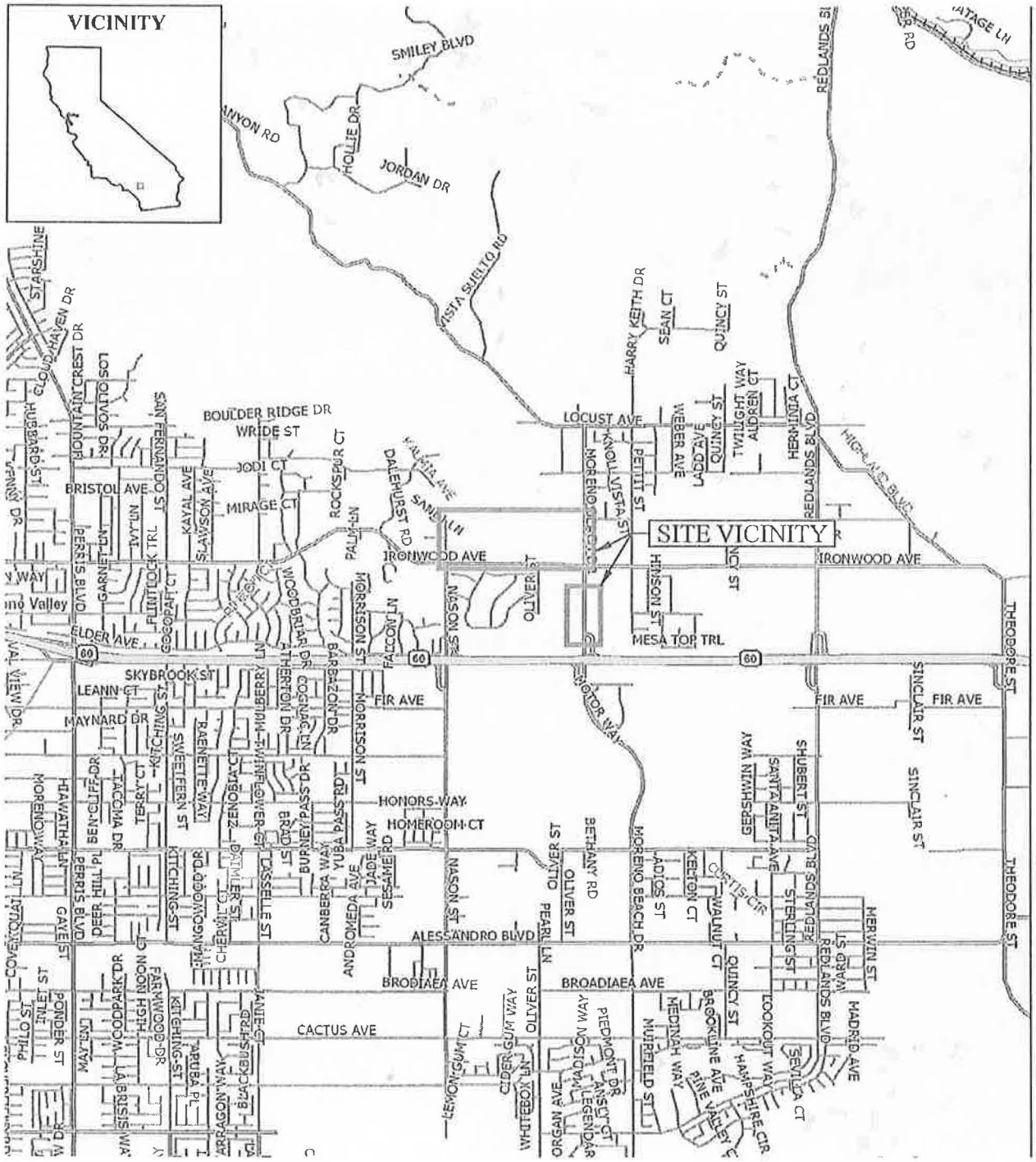
FIGURES



Map Source: © 2007 DeLorme. Topo USA ® 7.0 West Region

Scale: 1" = 4000'

0 FT 2400 FT 4000 FT 8000 FT

Note: All locations are approximate

SITE VICINITY MAP

BELLACAP, LLC

North of Ironwood Avenue Between Oliver Street & Nason Street

Moreno Valley, California 92555

EEI Project No. GLO-71982.4

Created October 2014

EEI Geotechnical & Environmental Solutions

CREATED BY:	JAB
REVISION DATE:	-
REVISION NO.:	-

FIGURE 1

Map Source: © Google Earth 2014, Imagery Date 11/06/2012

Sewerline Boring Locations
B7

Scale: 1" = 600'

0 FT 360 FT 600 FT 1200 FT

Note: All locations are approximate

N

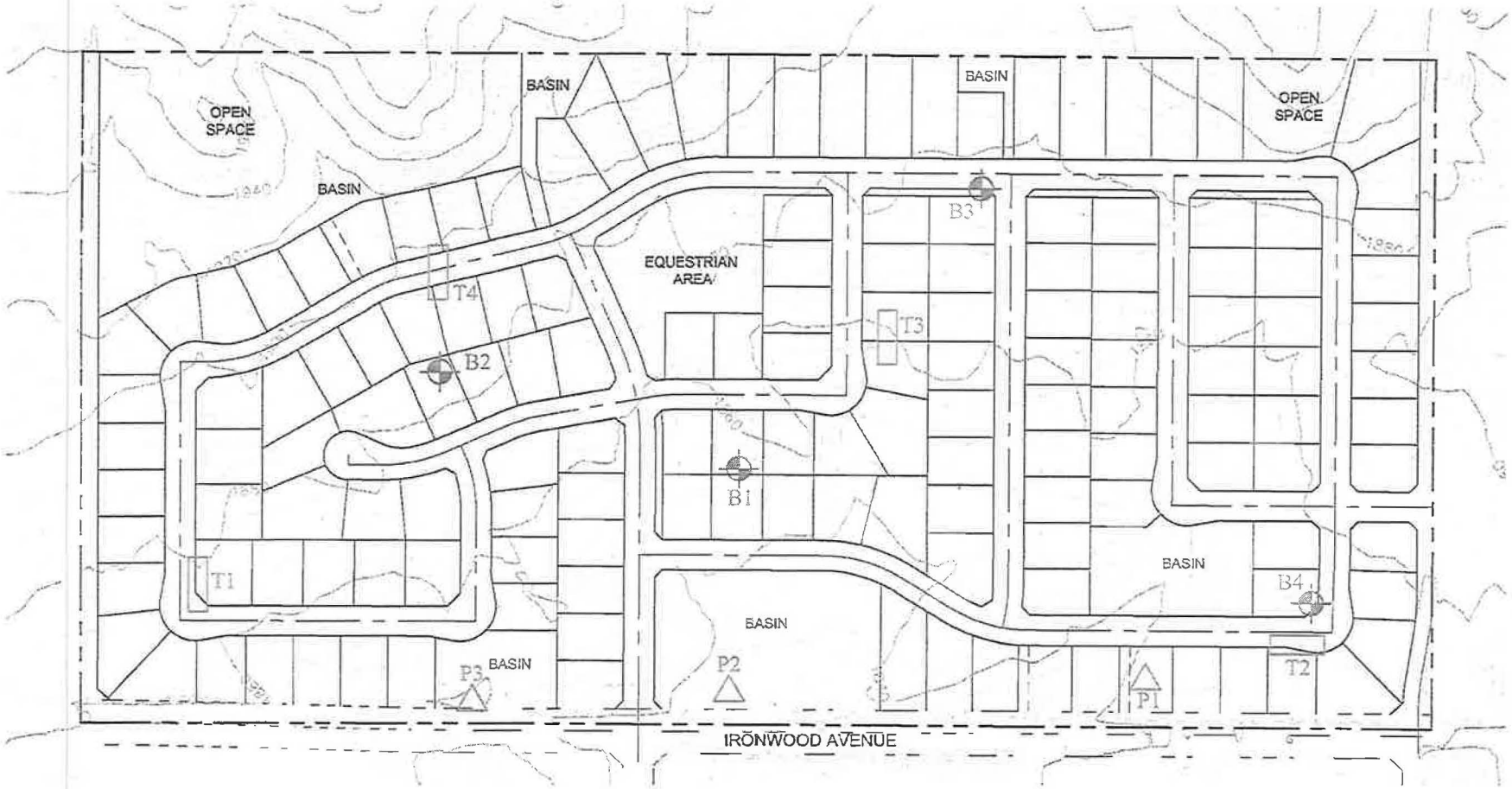
AERIAL SITE MAP & OFFSITE BORING LOCATION

BELLACAP, LLC
North of Ironwood Avenue Between Oliver Street & Nason Street
Moreno Valley, California 92555
EEI Project No. GLO-71982.4
Created October 2014

EEI Geotechnical & Environmental Solutions

CREATED BY:	JAB
REVISION DATE:	-
REVISION NO.:	-

FIGURE 2



Map Source: Anderson Consulting Engineers, Inc., Ironwood, Project Exhibit, dated Sept. 16, 2014

LEGEND

T1 Test Pit

P3 Percolation Test

B4 Boring

NOTE: All Locations are Approximate

Scale: 1" = 300'

0 FT 180 FT 300 FT 600 FT

Note: All locations are approximate

N

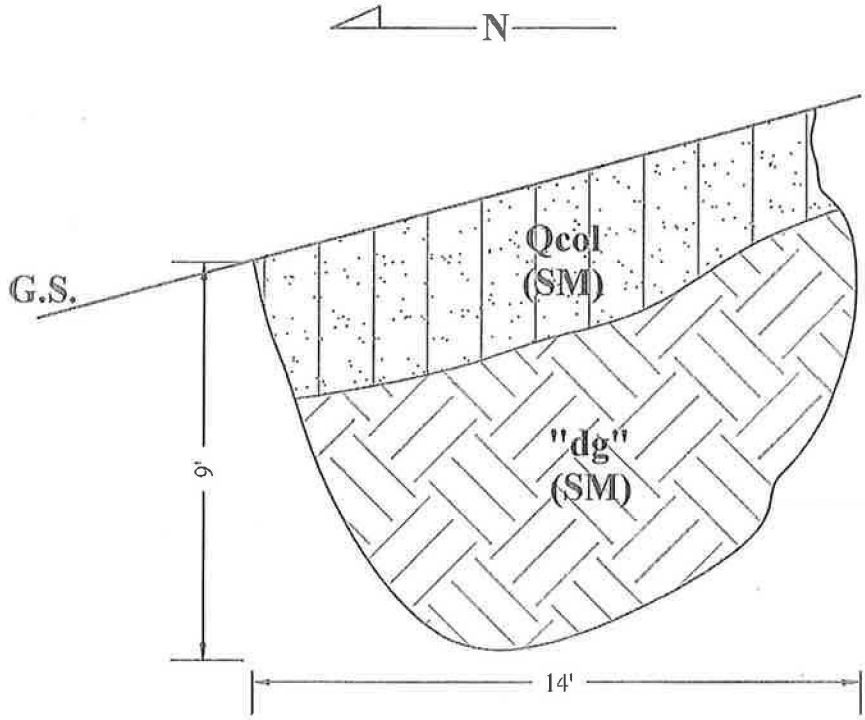
FIELD EXPLORATION PLAN

BELLACAP, LLC
North of Ironwood Avenue Between Oliver Street & Nason Street
Moreno Valley, California 92555
EEI Project No. GLO-71982.4
Created October 2014

EEI Geotechnical & Environmental Solutions

CREATED BY:	JAB
REVISION DATE:	-
REVISION NO.:	-

FIGURE 3



@ 0-3' COLLUVIUM (Qcol)
Silty-Sand (SM) - red-orange, medium to coarse, slightly moist, medium dense to dense, trace clay, weathered "dg" texture, oxidized.

@ 3'-9' WEATHERED BEDROCK: Tonalite (Kt)
Decomposed Granitics ("dg") - yellow-brown to gray-brown mottled/speckled, medium to coarse, slightly moist to moist, very soft to moderately hard, moderately to heavily weathered, minor clay, "dg" texture, less weathered with depth, weathers to silty sand (SM).

Total Depth = 9'
Bulk Sample @ 0-9'

Scale: 1" = 4'

0 FT 2.4 FT 4 FT 8 FT

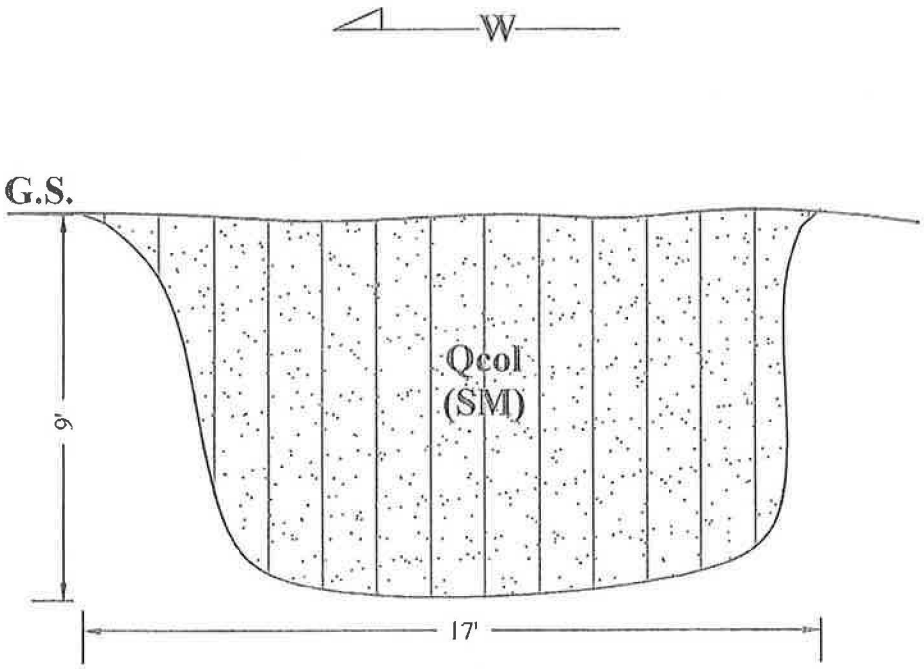
Note: All locations are approximate

TRENCH LOG
ANDERSON CONSULTING ENGINEERS
Preliminary Geotechnical Evaluation
Ironwood
Moreno Valley, California
EEI Project No. GLO-71982.4
Created November 2014

EEI Geotechnical & Environmental Services

CREATED BY:	JAB
REVISION DATE:	-
REVISION NO.:	-

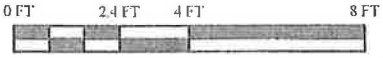
FIGURE T1



@ 0-9' COLLUVIUM (Qcol)
Silty-Sand (SM) - red-brown, medium to coarse, slightly moist to moist, medium dense to dense, trace clay, weathered "dg" texture, oxidized.

Total Depth = 9'
Bulk Sample @ 0-6'

Scale: 1" = 4'

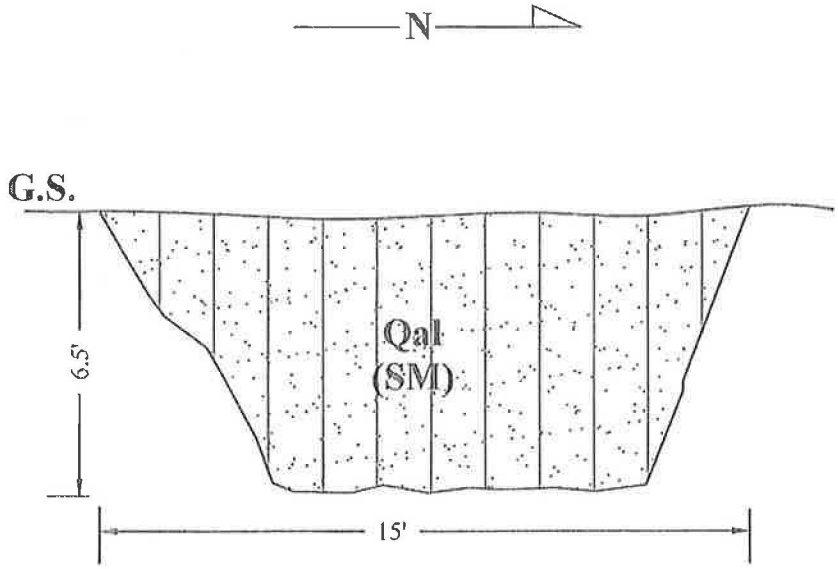


Note: All locations are approximate

TRENCH LOG
ANDERSON CONSULTING ENGINEERS
Preliminary Geotechnical Evaluation
Ironwood
Moreno Valley, California
EEI Project No. GLO-71982.4
Created November 2014

CREATED BY:	
JAB	
REVISION DATE:	
-	
REVISION NO.:	
-	

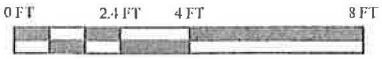
FIGURE T2



@ 0-6.5' ALLUVIUM (Qal)
Silty-Sand (SM) - medium-brown, fine to coarse, slightly moist, medium dense, trace clay, minor pinhole root porosity, trace gravel.

Total Depth = 6.5'
No Sampling

Scale: 1" = 4'

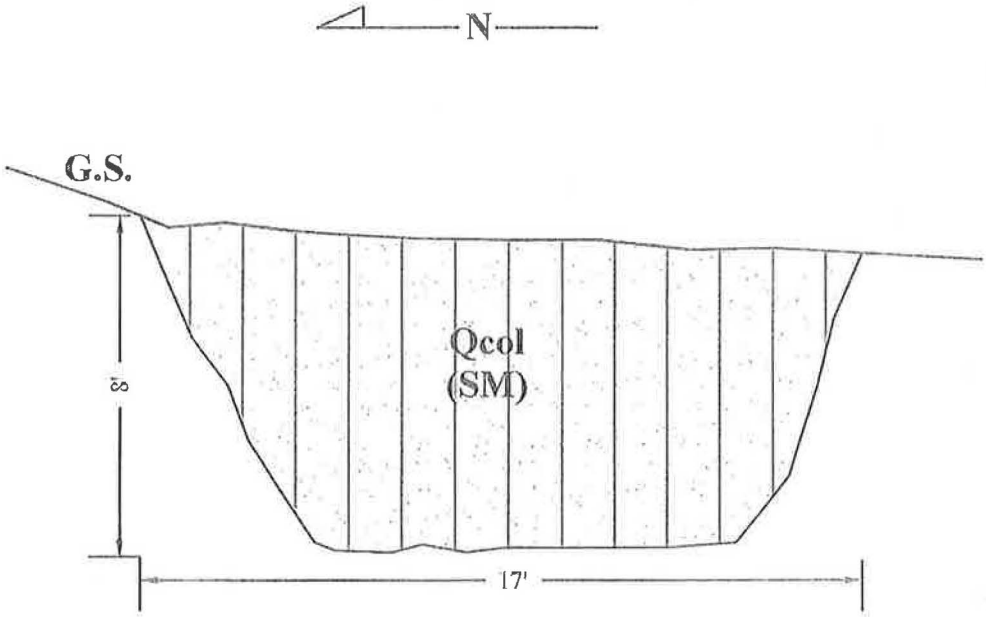


Note: All locations are approximale

TRENCH LOG
ANDERSON CONSULTING ENGINEERS
Preliminary Geotechnical Evaluation
Ironwood
Moreno Valley, California
EEI Project No. GLO-71982.4
Created November 2014

CREATED BY:	
JAB	
REVISION DATE:	
-	
REVISION NO.:	
-	

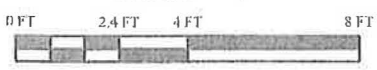
FIGURE T3



@ 0-8' COLLUVIUM (Qcol)
Silty-Sand (SM) - light gray-brown mottled, fine to medium, trace coarse, slightly moist to moist, medium dense to dense, trace clay, weathered "dg" texture.

Total Depth = 8'
No Sampling

Scale: 1" = 4'



Note: All locations are approximate

TRENCH LOG
ANDERSON CONSULTING ENGINEERS
Preliminary Geotechnical Evaluation
Ironwood
Moreno Valley, California
EEI Project No. GLO-71982.4
Created November 2014

EEI	CREATED BY: JAB REVISION DATE: . REVISION NO.: .	**FIGURE T4**

APPENDIX A
SOIL CLASSIFICATION CHART, BORING LOGS AND TEST PIT LOGS

SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVEL AND GRAVELLY SOILS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
				GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
		GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	CLEAN SANDS (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
				SP	POORLY-GRADED SANDS, GRAVELLY SAND, LITTLE OR NO FINES
		SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	SILTY SANDS, SAND - SILT MIXTURES
				SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY
				OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
CLAYSTONE				CL	CLAYSTONE, Pliocene Fernando Formation/late Miocene Puente Formation

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

BOREHOLE LOG

EEI Geotechnical & Environmental Solutions	Number: B-1	
	Client: Global - Anderson Consulting Engineers	Sheet: 1 of 2
Date Started: 10/17/2014	Date Finished: 10/17/2014	Location: Ironwood - Moreno Valley, Moreno Valley, Riverside County, Ca
EEI Rep: BM	Project No.: GLO-71982.4	Drill Rig/Sampling Method: Truck Mounted CME-75 / 140 lb Auto Hammer
		Borehole Diameter: 6-inch

SAMPLE LOG								BOREHOLE LOG
Bulk	Sample Type	Blows Per 6"	Dry Unit Wt. (pcf)	Moisture (%)	Depth In Feet	USCS Symbol	Graphic Log	Geologic Description (SoilType, Color, Grain, Minor Soil Component, Moisture, Density, Odor, Etc.)
					1			ALLUVIUM SILTY-SAND, orange-brown mottled, fine to coarse grained, damp, loose
	MC	3 4 5	105	3	2, 3, 4			
	MC	4 6 8	108	3	5, 6, 7			
	MC	3 3 4	138	3	8, 9			
	MC	4 5 6	104	7	10, 11, 12, 13, 14			@ 10' Becomes fine to medium grained, moist, loose; micaceous
	SPT	3 3 3		9	15, 16, 17, 18, 19	SM		
	MC	6 11 16	122	8	20, 21, 22, 23, 24			@ 20' becomes medium dense
	SPT	6 8 8		7	25, 26, 27, 28, 29			
	MC	8 10 11	124	7	30, 31	SM		WEATHERED BEDROCK: TONALITE (Kt) DECOMPOSED GRANITICS ("DG"), gray-white to black speckled, damp, dense; weathered, friable, phaneritic texture, moderate to heavily weathered

BOREHOLE LOG

EEI Geotechnical & Environmental Solutions	BOREHOLE LOG	Number: B-1	
	Client: Global - Anderson Consulting Engineers	Sheet: 2 of 2	
Date Started: 10/17/2014	Date Finished: 10/17/2014	Location: Ironwood - Moreno Valley, Moreno Valley, Riverside County, Ca	
EEI Rep: BM	Project No.: GLO-71982.4	Drill Rig/Sampling Method: Truck Mounted CME-75 / 140 lb Auto Hammer	Borehole Diameter: 6-inch

SAMPLE LOG								BOREHOLE LOG
Bulk	Sample Type	Blows Per 6"	Dry Unit Wt. (pcf)	Moisture (%)	Depth In Feet	USCS Symbol	Graphic Log	Geologic Description (SoilType, Color, Grain, Minor Soil Component, Moisture, Density, Odor, Etc.)
					33			
					34			
	SPT	50-4"		4	35			@ 35' Becomes very dense
					36			
					37			
					38			
					39			
	MC	50-3"	103	7	40			
					41	SM		
					42			
					43			
					44			
	SPT	50-5"		7	45			@ 45' DECOMPOSED GRANITICS ("DG"), dark gray, orange and white speckled, damp, medium dense to dense; moderately weathered
					46			
					47			
					48			
					49			
	NR	50-3"			50			@ 50' No sample recovery
					51			
					52			Total depth: 50.5-feet No groundwater encountered
					53			Hole backfilled on 10/17/2014 with drilled cuttings
					54			
					55			
					56			
					57			
					58			
					59			
					60			
					61			
					62			
					63			

BOREHOLE LOG

Number:	B-2
Client:	Global - Anderson Consulting Engineers
Sheet:	1 of 1
Location:	Ironwood - Moreno Valley Moreno Valley, Riverside County, Ca
Date Started:	10/17/2014
Date Finished:	10/17/2014
EEI Rep:	BM
Project No.:	GLO-71982.4
Drill Rig/Sampling Method:	Truck Mounted CME-75 / 140 lb Auto Hammer
Borehole Diameter:	6-inch

SAMPLE LOG | **BOREHOLE LOG**

Bulk	Sample Type	Blows Per 6"	Dry Unit Wt. (pcf)	Moisture (%)	Depth In Feet	USCS Symbol	Graphic Log	Geologic Description (SoilType, Color, Grain, Minor Soil Component, Moisture, Density, Odor, Etc.)
					1	SM		**COLLUVIUM** SILTY-SAND, orange-brown mottled, fine to coarse grained, damp, loose
	MC	4 6 8	111	2	2, 3, 4			
	MC	7 8 10	137	3	5, 6			
	MC	5 6 9	115	4	7, 8, 9			
	MC	5 31 50-3"	126	3	10, 11			
					12	SM		**WEATHERED BEDROCK: TONALITE (Kt)** DECOMPOSED GRANITICS ("DG"), white, orange and black speckled, damp, dense to very dense; unweathered to moderately weathered
	SPT	23 50-3"		4	13, 14, 15			
					16			Total depth: 15.75-feet No groundwater encountered Hole backfilled on 10/17/2014 with drilled cuttings
					17, 18, 19, 20, 21, 22, 23, 24, 25, 26, 27, 28, 29, 30, 31			

BOREHOLE LOG

EEI Geotechnical & Environmental Solutions	BOREHOLE LOG		Number: B-3
	Client: Global - Anderson Consulting Engineers		Sheet: 1 of 1
Date Started: 10/17/2014	Date Finished: 10/17/2014	Location: Ironwood - Moreno Valley, Moreno Valley, Riverside County, Ca	
EEI Rep: BM	Project No.: GLO-71982.4	Drill Rig/Sampling Method: Truck Mounted CME-75 / 140 lb Auto Hammer	Borehole Diameter: 6-inch

SAMPLE LOG

Bulk	Sample Type	Blows Per 6"	Dry Unit Wt. (pcf)	Moisture (%)	Depth In Feet	USCS Symbol	Graphic Log	Geologic Description (SoilType, Color, Grain, Minor Soil Component, Moisture, Density, Odor, Etc.)
					1	SM		**COLLUVIUM** SILTY-SAND, red-brown, fine to coarse grained with occasional gravels, dry to damp, dense
	MC	50-3"	85	5	2, 3			
	MC	20 22 26	119	3	4, 5, 6			@ 5' Becomes medium dense
	MC	23 31 34	123	4	7, 8, 9			@ 8' Becomes dense
	MC	9 13 18	113	2	10, 11	SM		**WEATHERED BEDROCK: TONALITE (Kt)** DECOMPOSED GRANITICS ("DG"), orange, brown and black speckled, damp, medium dense to dense; occasional unweathered to moderately weathered
	SPT	8 9 10		4	12, 13, 14, 15, 16			
	MC	23 44 50-5"	128	9	17, 18, 19, 20, 21			@ 19' Becomes very dense
					22, 23, 24, 25, 26, 27, 28, 29, 30, 31			Total depth: 21.5-feet No groundwater encountered Hole backfilled on 10/17/2014 with drilled cuttings

BOREHOLE LOG

EEI Geotechnical & Environmental Solutions	**Number:** B-4	
Client: Global - Anderson Consulting Engineers	**Sheet:** 1 of 1	
Location: Ironwood - Moreno Valley, Moreno Valley, Riverside County, Ca		
Date Started: 10/17/2014	**Date Finished:** 10/17/2014	
EEI Rep: BM	**Project No.:** GLO-71982.4	
Drill Rig/Sampling Method: Truck Mounted CME-75 / 140 lb Auto Hammer	**Borehole Diameter:** 6-inch	

SAMPLE LOG								BOREHOLE LOG
Bulk	Sample Type	Blows Per 6"	Dry Unit Wt. (pcf)	Moisture (%)	Depth In Feet	USCS Symbol	Graphic Log	Geologic Description (SoilType, Color, Grain, Minor Soil Component, Moisture, Density, Odor, Etc.)
					1			**COLLUVIUM** SILTY-SAND, red-brown, fine to coarse grained with occasional gravels, moist, medium dense
					2			
	MC	13 13 19	111	4	3			
					4			
					5			
	MC	6 7 7	111	3	6			@ 5' Becomes loose
					7	SM		
	MC	7 21 50-5"	115	5	8			@ 8' Becomes dense to very dense
					9			
					10			
	MC	39 50-4"	116	8	11			
					12			
					13			
					14			**WEATHERED BEDROCK: TONALITE (Kt)** DECOMPOSED GRANITICS ("DG"), orange, brown and black speckled, moist, medium dense; weathered
	SPT	6 9 9		4	15	SM		
					16			
					17			Total depth: 16.5-feet No groundwater encountered Hole backfilled on 10/17/2014 with drilled cuttings
					18			
					19			
					20			
					21			
					22			
					23			
					24			
					25			
					26			
					27			
					28			
					29			
					30			
					31			

BOREHOLE LOG

Number: B-5		
Client: Global - Anderson Consulting Engineers	Sheet: 1 of 1	
Location: Ironwood - Moreno Valley, Moreno Valley, Riverside County, Ca		
Date Started: 10/17/2014	Date Finished: 10/17/2014	
EEI Rep: BM	Project No.: GLO-71982.4	
Drill Rig/Sampling Method: Truck Mounted CME-75 / 140 lb Auto Hammer	Borehole Diameter: 6-inch	

SAMPLE LOG

Bulk	Sample Type	Blows Per 6"	Dry Unit Wt. (pcf)	Moisture (%)	Depth In Feet	USCS Symbol	Graphic Log	Geologic Description (SoilType, Color, Grain, Minor Soil Component, Moisture, Density, Odor, Etc.)
					1–14	SM		**ALLUVIUM** SILTY-SAND, Orange-brown mottled, fine to coarse grained, poorly sorted, damp, loose to medium dense
	MC	3 5 6	106	3	5–7	SM		
	MC	5 9 15	123	5	10–14	SM		
	SPT	10 10 15		6	15–19	SM		**WEATHERED BEDROCK: TONALITE (Kt)** DEOMPOSED GRANITICS ("DG"), orange, brown and black speckled, moist, medium dense; weathered
	MC	10 20 25	124	12	20–24	SM		
	SPT	10 12 18		11	25–26	SM		
					27–31			Total depth: 26.5-feet No groundwater encountered Hole backfilled on 10/17/2014 with drilled cuttings

BOREHOLE LOG VES-71982.4.GPJ EEI.GDT 11/25/14

BOREHOLE LOG

EEI Geotechnical & Environmental Solutions	Number: B-6	
	Client: Global - Anderson Consulting Engineers	Sheet: 1 of 1
Date Started: 10/17/2014	Date Finished: 10/17/2014	Location: Ironwood - Moreno Valley, Moreno Valley, Riverside County, Ca
EEI Rep: BM	Project No.: GLO-71982.4	Drill Rig/Sampling Method: Truck Mounted CME-75 / 140 lb Auto Hammer
		Borehole Diameter: 6-inch

SAMPLE LOG								BOREHOLE LOG
Bulk	Sample Type	Blows Per 6"	Dry Unit Wt. (pcf)	Moisture (%)	Depth In Feet	USCS Symbol	Graphic Log	Geologic Description (SoilType, Color, Grain, Minor Soil Component, Moisture, Density, Odor, Etc.)
	MC	5 7 8	116	2	1–9	SM		FILL SILTY-SAND, light brown, fine to coarse grained, damp, loose
	MC	8 11 13	111	7	10–26	ML		COLLUVIUM SANDY-SILT, orange-brown, fine to medium grained, moderately sorted, moist, medium dense
	SPT	5 6 6		5	15–17			
	MC	8 13 16	118	3	20–24			
	SPT	8 10 12		6	25–26			
					27–31			Total depth: 26.5-feet No groundwater encountered Hole backfilled on 10/17/2014 with drilled cuttings

BOREHOLE LOG

Number:	B-7
Client:	Global - Anderson Consulting Engineers
Sheet:	1 of 1
Location:	Ironwood - Moreno Valley Moreno Valley, Riverside County, Ca
Date Started:	10/17/2014
Date Finished:	10/17/2014
EEI Rep:	BM
Project No.:	GLO-71982.4
Drill Rig/Sampling Method:	Truck Mounted CME-75 / 140 lb Auto Hammer
Borehole Diameter:	6-inch

SAMPLE LOG | **BOREHOLE LOG**

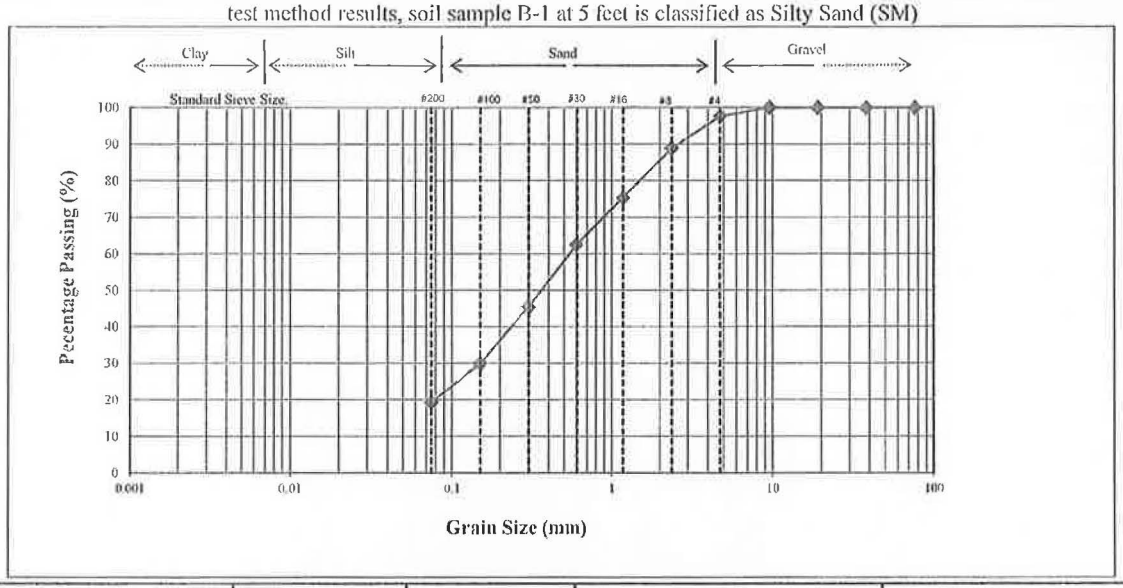
Bulk	Sample Type	Blows Per 6"	Dry Unit Wt. (pcf)	Moisture (%)	Depth In Feet	USCS Symbol	Graphic Log	Geologic Description (Soil Type, Color, Grain, Minor Soil Component, Moisture, Density, Odor, Etc.)
					1	SM		COLLUVIUM SILTY-SAND, brown to orange-brown, fine to coarse grained, moist, medium dense
					2			
					3			
					4			
	MC	41 24 26	115	3	5			
					6			
					7			
					8			
					9			
	MC	8 14 21	112	3	10			
					11			
					12			
					13			
					14			
	SPT	6 10 10		8	15	ML		@ 15' SANDY-SILT, brown, fine to coarse, moist, medium dense, micaceous
					16			
					17			
					18			
					19			
	MC	8 13 19	122	7	20			
					21			
					22			
					23			
					24			
	SPT	8 10 12		5	25			
					26			
					27			Total depth: 26.5-feet No groundwater encountered Hole backfilled on 10/17/2014 with drilled cuttings
					28			
					29			
					30			
					31			

APPENDIX B
LABORATORY TEST DATA

PARTICLE-SIZE ANALYSIS OF SOILS
ASTM METHOD D 422 (SIEVE ANALYSIS)

Sample :	B-1 @ 5 ft.		D_{10} (mm)	N/A
Total Weight (g)	129.2		D_{30} (mm)	0.15
Dry Weight (g)	125.8		D_{60} (mm)	0.53
Wet Sieve Weight (g)	101.5		Cu	N/A
Initial Moisture (%)	2.7		Cc	N/A

According to ASTM D 2487 Unified Soil Classification System (USCS) and ASTM D 422 (Standard Test Method for Particle-Size Analysis) test method results, soil sample B-1 at 5 feet is classified as Silty Sand (SM)



Sieve Size (in)	Sieve Size (mm)	Cumulative Weight of dry soil (gm)	Percent Retained (%)	Percent Passing (%)
3"	76.2		0.0	100.0
1.5"	38.1		0.0	100.0
3/4"	19.05		0.0	100.0
3/8"	9.53		0.0	100.0
#4	4.75	3.0	2.4	97.6
#8	2.36	14.0	11.1	88.9
#16	1.18	31.0	24.6	75.4
#30	0.6	47.2	37.5	62.5
#50	0.3	68.7	54.6	45.4
#100	0.15	88.2	70.1	29.9
#200	0.075	101.5	80.7	19.3

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2195 Faraday Avenue, Suite K, Carlsbad CA 92008

Client: Anderson Consulting Engineers

Project Name: Ironwood

Job Number: GLO-71982.4

Date: 10/24/14

Boring Number: B-1

Depth: 5 ft.

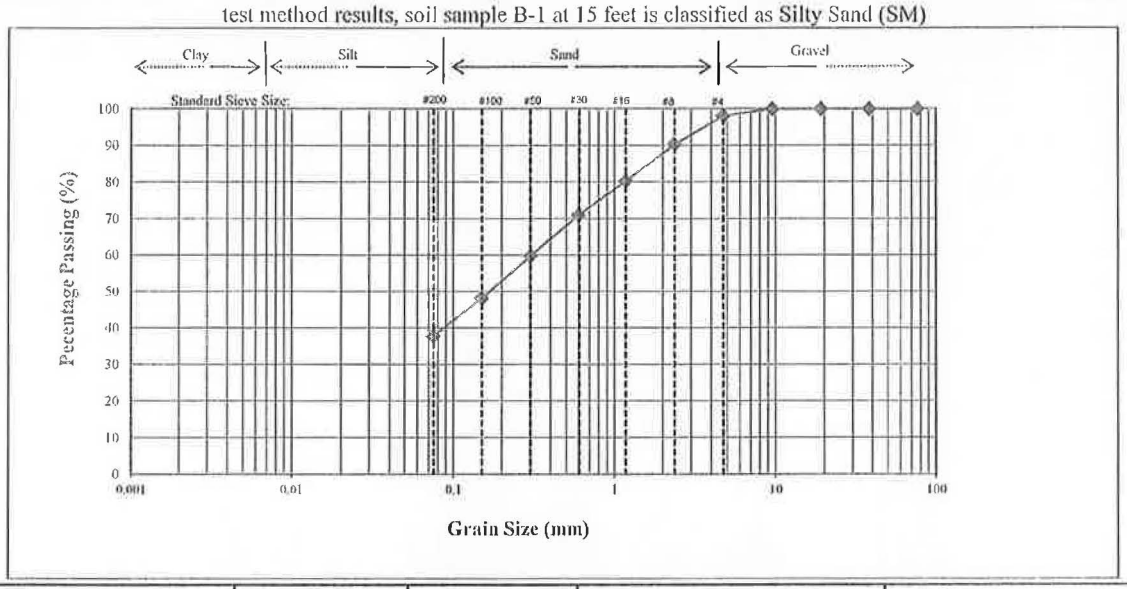
Soil Description: Brown Mottled Silty Sand SM

Tested by: B D

PARTICLE-SIZE ANALYSIS OF SOILS
ASTM METHOD D 422 (SIEVE ANALYSIS)

Sample :	B-1 @15 ft.	D10 (mm)	N/A
Total Weight (g)	117.5	D30 (mm)	N/A
Dry Weight (g)	108.1	D60 (mm)	0.30
Wet Sieve Weight (g)	67.3	Cu	N/A
Initial Moisture (%)	8.7	Cc	N/A

According to ASTM D 2487 Unified Soil Classification System (USCS) and ASTM D 422 (Standard Test Method for Particle-Size Analysis) test method results, soil sample B-1 at 15 feet is classified as Silty Sand (SM)



Sieve Size (in)	Sieve Size (mm)	Cumulative Weight of dry soil (gm)	Percent Retained (%)	Percent Passing (%)
3"	76.2		0.0	100.0
1.5"	38.1		0.0	100.0
3/4"	19.05		0.0	100.0
3/8"	9.53		0.0	100.0
#4	4.75	2.1	1.9	98.1
#8	2.36	10.6	9.8	90.2
#16	1.18	21.4	19.8	80.2
#30	0.6	31.4	29.0	71.0
#50	0.3	43.6	40.3	59.7
#100	0.15	56.1	51.9	48.1
#200	0.075	67.3	62.3	37.7

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2195 Faraday Avenue, Suite K, Carlsbad CA 92008

Client: Anderson Consulting Engineers

Project Name: Ironwood

Job Number: GLO-71982.4

Date: 10/24/14

Boring Number: B-1

Depth: 15 ft.

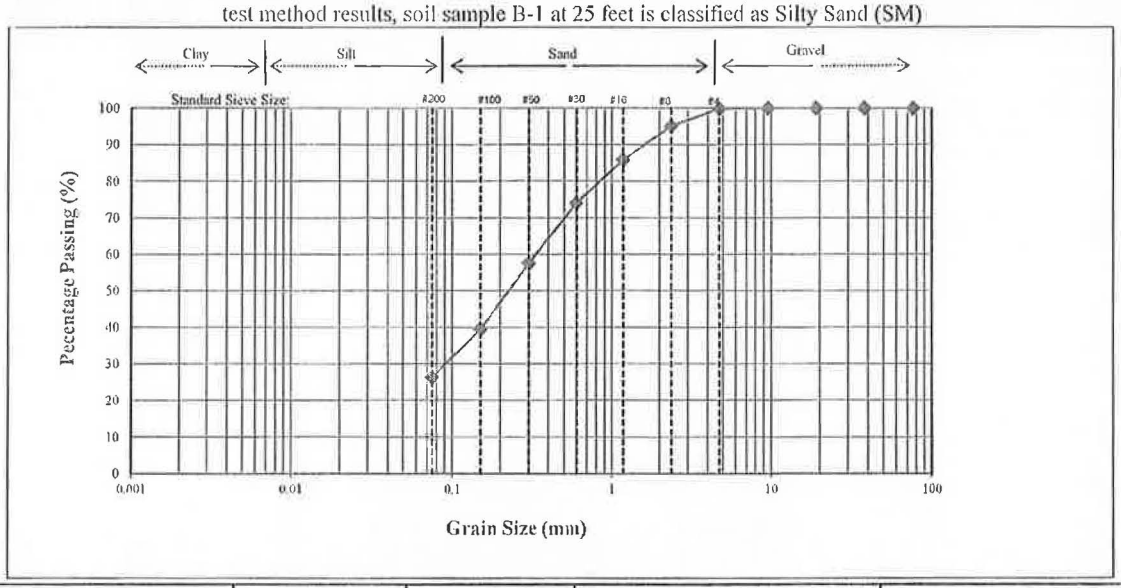
Soil Description: Brown Mottled Silty Sand SM

Tested by: B D

PARTICLE-SIZE ANALYSIS OF SOILS
ASTM METHOD D 422 (SIEVE ANALYSIS)

Sample :	B-1 @ 25 ft.	D10 (mm)	N/A
Total Weight (g)	123.6	D30 (mm)	0.08
Dry Weight (g)	115.8	D60 (mm)	0.33
Wet Sieve Weight (g)	85.3	Cu	N/A
Initial Moisture (%)	6.7	Cc	N/A

According to ASTM D 2487 Unified Soil Classification System (USCS) and ASTM D 422 (Standard Test Method for Particle-Size Analysis) test method results, soil sample B-1 at 25 feet is classified as Silty Sand (SM)



Sieve Size (in)	Sieve Size (mm)	Cumulative Weight of dry soil (gm)	Percent Retained (%)	Percent Passing (%)
3"	76.2		0.0	100.0
1.5"	38.1		0.0	100.0
3/4"	19.05		0.0	100.0
3/8"	9.53		0.0	100.0
#4	4.75	0.2	0.2	99.8
#8	2.36	5.7	4.9	95.1
#16	1.18	16.4	14.2	85.8
#30	0.6	30.1	26.0	74.0
#50	0.3	49.2	42.5	57.5
#100	0.15	70.1	60.5	39.5
#200	0.075	85.3	73.7	26.3

Client: Anderson Consulting Engineers

Project Name: Ironwood

Job Number: GLO-71982.4

Date: 10/24/14

Boring Number: B-1

Depth: 25 ft.

Soil Description: Brown Mottled Silty Sand SM

Tested by: B D

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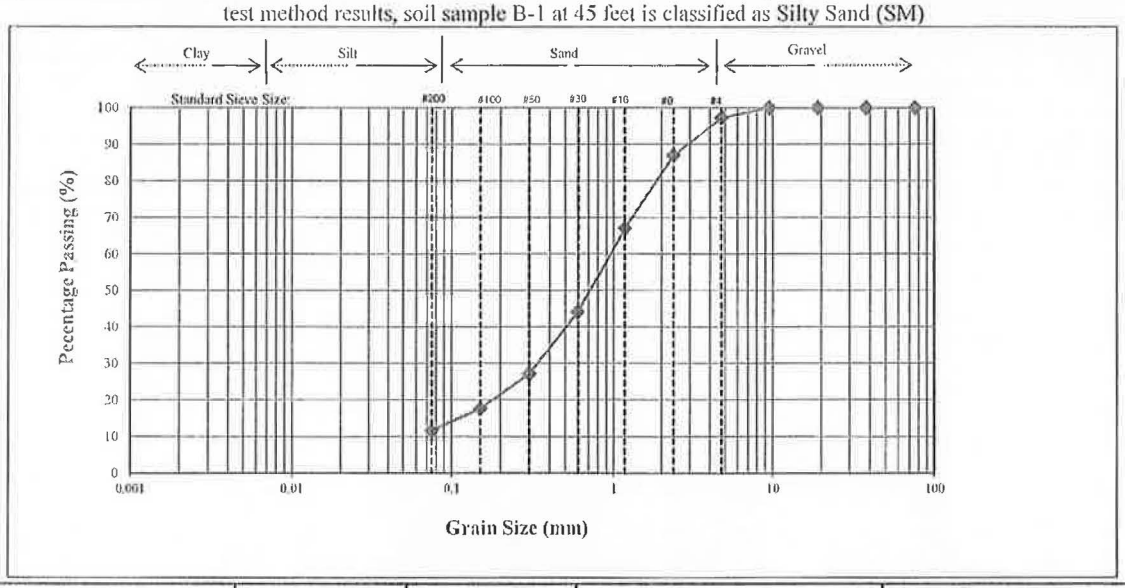
PARTICLE-SIZE ANALYSIS OF SOILS
ASTM METHOD D 422 (SIEVE ANALYSIS)

Sample :	B-1 @ 35 ft.	D10 (mm)	N/A
Total Weight (g)	126.5	D30 (mm)	0.30
Dry Weight (g)	122.1	D60 (mm)	0.90
Wet Sieve Weight (g)	108.0	Cu	N/A
Initial Moisture (%)	3.6	Cc	N/A

According to ASTM D 2487 Unified Soil Classification System (USCS) and ASTM D 422 (Standard Test Method for Particle-Size Analysis) test method results, soil sample B-1 at 45 feet is classified as Silty Sand (SM)



Sieve Size (in)	Sieve Size (mm)	Cumulative Weight of dry soil (gm)	Percent Retained (%)	Percent Passing (%)
3"	76.2		0.0	100.0
1.5"	38.1		0.0	100.0
3/4"	19.05		0.0	100.0
3/8"	9.53		0.0	100.0
#4	4.75	3.3	2.7	97.3
#8	2.36	15.9	13.0	87.0
#16	1.18	40.3	33.0	67.0
#30	0.6	68.2	55.9	44.1
#50	0.3	89.0	72.9	27.1
#100	0.15	100.5	82.3	17.7
#200	0.075	108.0	88.5	11.5

Client: Anderson Consulting Engineers

Project Name: Ironwood

Job Number: GLO-71982.4

Date: 10/24/14

Boring Number: B-1

Depth: 35 ft.

Soil Description: Gray-Green Mottled Silty Sand SM

Tested by: B D

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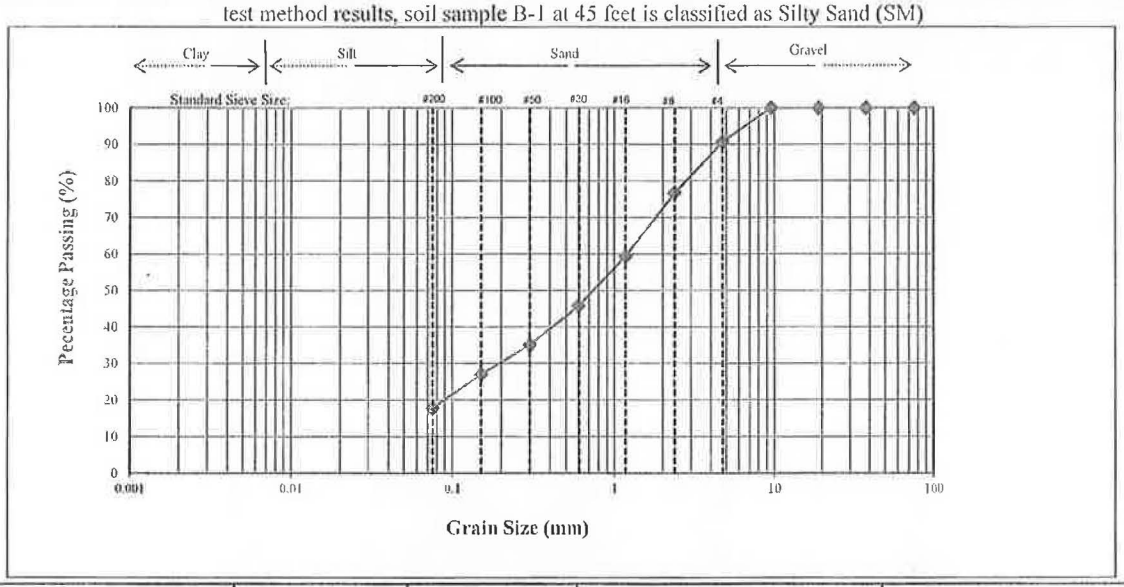
PARTICLE-SIZE ANALYSIS OF SOILS
ASTM METHOD D 422 (SIEVE ANALYSIS)

Sample :	B-1 @ 45 ft.	D10 (mm)	N/A
Total Weight (g)	120.2	D30 (mm)	0.20
Dry Weight (g)	112.2	D60 (mm)	1.18
Wet Sieve Weight (g)	92.3	Cu	N/A
Initial Moisture (%)	7.1	Cc	N/A

According to ASTM D 2487 Unified Soil Classification System (USCS) and ASTM D 422 (Standard Test Method for Particle-Size Analysis) test method results, soil sample B-1 at 45 feet is classified as Silty Sand (SM)



Sieve Size (in)	Sieve Size (mm)	Cumulative Weight of dry soil (gm)	Percent Retained (%)	Percent Passing (%)
3"	76.2		0.0	100.0
1.5"	38.1		0.0	100.0
3/4"	19.05		0.0	100.0
3/8"	9.53		0.0	100.0
#4	4.75	10.4	9.3	90.7
#8	2.36	26.1	23.3	76.7
#16	1.18	45.6	40.6	59.4
#30	0.6	60.9	54.3	45.7
#50	0.3	72.8	64.9	35.1
#100	0.15	81.9	73.0	27.0
#200	0.075	92.3	82.3	17.7

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Client: Anderson Consulting Engineers

Project Name: Ironwood

Job Number: GLO-71982.4

Date: 10/24/14

Boring Number: B-1

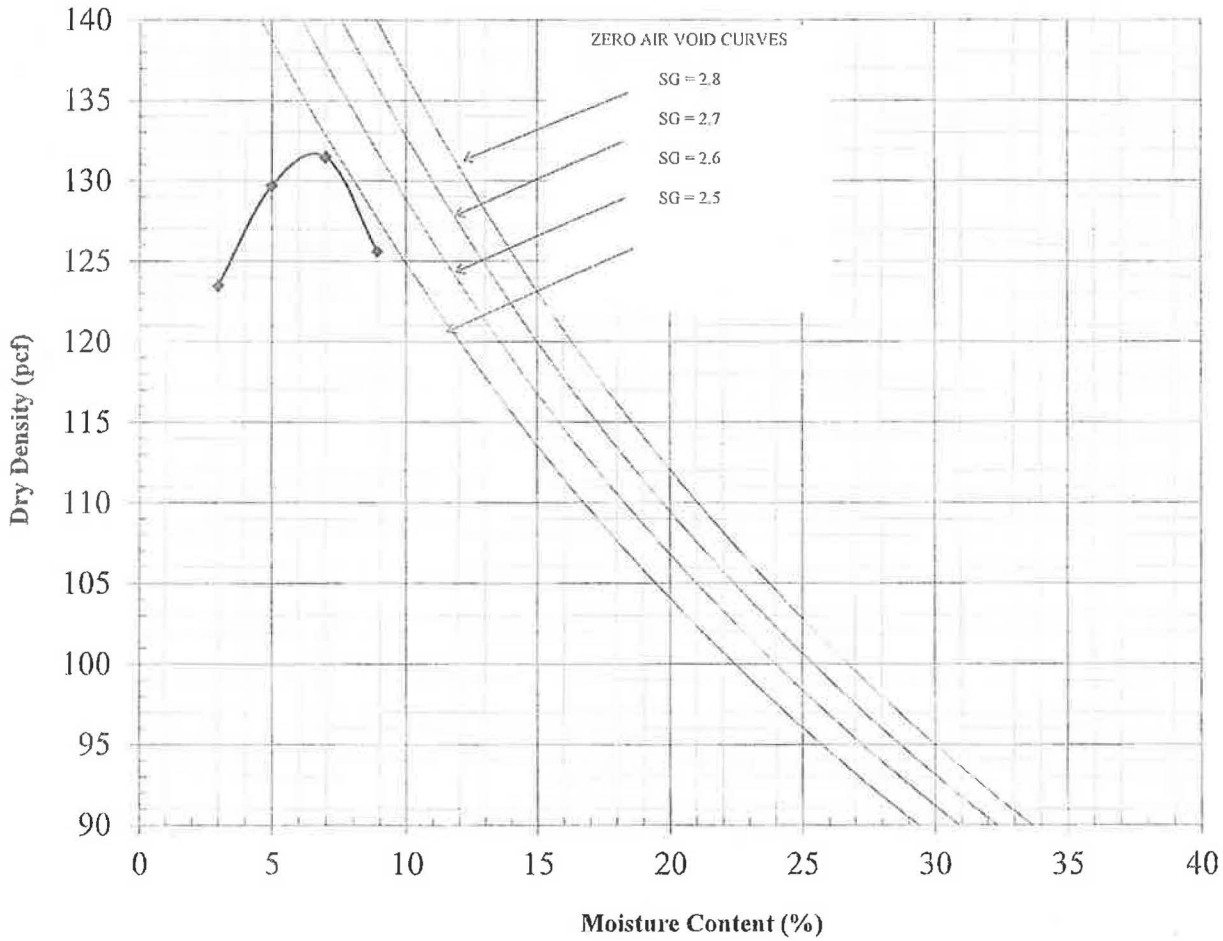
Depth: 45 ft.

Soil Description: Orange-Gray Mottled Silty Sand (SM)

Tested by: B D

LABORATORY COMPACTION ASTM D 1557

Sample	1	2	3	4
Mold and wet soil (lbs.)	8.550	8.850	9.000	8.870
Mold (lbs.)	4.310	4.310	4.310	4.310
Wet Soil (lbs.)	4.240	4.540	4.690	4.560
Wet Density (pcf)	127.20	136.20	140.70	136.80
Moisture (%)	3.0	5.0	7.0	8.9
Dry Density (pcf)	123.5	129.7	131.5	125.6



Maximum density 132.0 pcf @ 6.5% moisture

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2195 Faraday, Suite K, Carlsbad, CA 92008

Client: Anderson Consulting Engineers

Project Name: Ironwood

Procedure: Method A

Job Number: GLO-71982.4

Date: 10/21/14

Boring Number: B-1

Location: 0-5 ft.

Soil Description: Brown Mottled Silty Sand SM

Tested by: B D

EXPANSION INDEX TEST

ASTM METHOD D 4829

B-1 @ 1-5 ft.

Moisture Content of Initial Sample		% Saturation of Re-molded Sample		Moisture Content of Final Sample	
Tare No. -	S6	Wt. of Soil and Ring (g) -	615.2	Wt. of Soil and Ring (g) -	641.3
Wet Weight and Tare (g) -	134.0	Ring Weight (g) -	199.0	Ring Weight (g) -	199.0
Dry Weight and Tare (g) -	128.8	Wet Weight of Soil (g) -	416.2	Wet Weight of Soil (g) -	442.3
Tare Weight (g) -	51.2	Dry Weight of Soil (g) -	390.1	Dry Weight of Soil (g) -	390.1
Water Loss (g) -	5.2	Volume of Ring (ft^3) -	0.0073	Weight of Water (g) -	52.2
Dry Weight (g) -	77.6	Dry Density (pcf) -	117.8	*Final Moisture (%)*	*13.4*
Initial Moisture (%) -	*6.7*	*Initital Saturation (%) -*	*42.1*	*Final Saturation (%) -*	*84.0*

Expansion Test - UBC (144 PSF)				
	Date	Time	Reading	
Add Weight	10/21/2014	10:20	0.000	
10 Minutes		10:30	0.000	Initial Reading
Add Water		11:30	0.002	
		3:00	0.002	
	10/22/2014	6:17	0.002	Final Reading

EImeasured	=	2
EI_{50}	=	0

Expansion Index, EI_{50}	Potential Expansion
0-20	Very Low
21-50	Low
51-90	Medium
91-130	High
>130	Very High

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2195 Faraday Avenue, Suite K, Carlsbad, CA 92008

Client: Anderson Consulting Engineers

Job Name; Ironwood

Job Number: GLO-71982.4

Date: 10/21/14

Boring Number: B-1

Depth: 0-5 ft.

Soil Description: Brown Mottled Silty Sand SM

Tested by: BD

DIRECT SHEAR TEST ASTM D 3080

Job Data
Job No.: GLO-71982.4
Client: Anderson Consulting Engineers
Date: 10/22/14
Sample Data
Sample: B-1 @ 0 - 5 ft.
Remolded: 90 % Relative Compaction
Remarks: Soaked Before Placing in Shear Box
Soil Description: Brown Silty Sand (SM)

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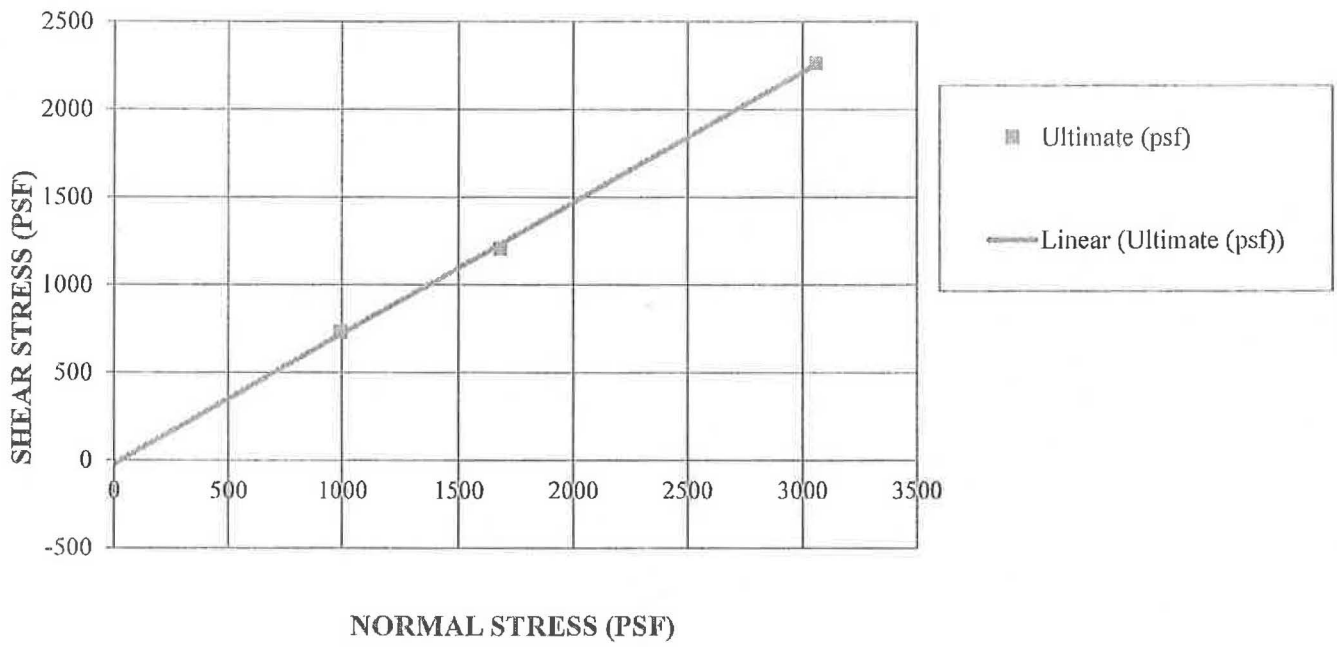
SHEAR TEST DIAGRAM



Test Results

	Phi		Cohesion	
Ultimate (psf)	37	degrees	-20	psf

Average Initial Moisture	6.5%
Average Dry Density	118.7 pcf
Average Final Moisture	12.9%

DIRECT SHEAR TEST ASTM D 3080

Job Data
Job No.: GLO-71982.4
Client: Anderson Consulting Engineers
Date: 10/24/14
Sample Data
Sample: B-1 @ 5 ft.
Remolded: 90 % Relative Compaction
Remarks: Soaked Before Placing in Shear Box
Soil Description: Brown Silty Sand (SM)

EEI Geotechnical & Environmental Solutions

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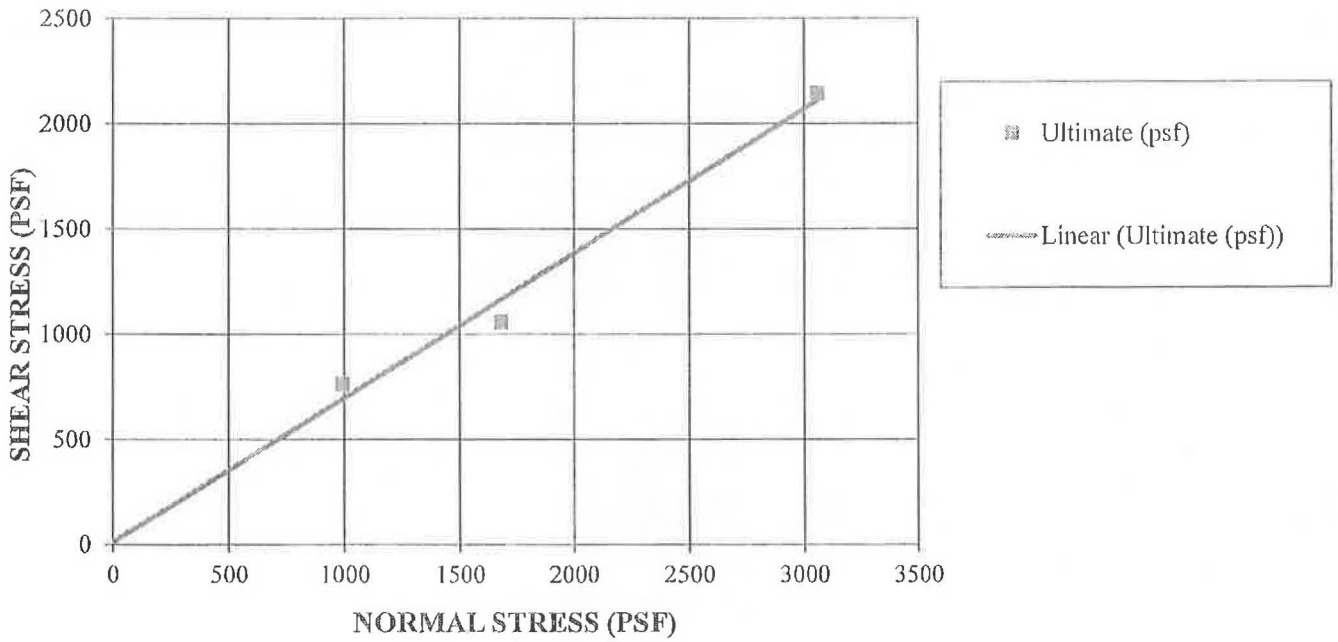
SHEAR TEST DIAGRAM



Test Results

	Phi		Cohesion	
Ultimate (psf)	34	degrees	12	psf

Average Initial Moisture	2.7%
Average Dry Density	109.7 pcf
Average Final Moisture	15.4%

DIRECT SHEAR TEST ASTM D 3080

Job Data	
Job No.:	GLO-71982.4
Client:	Anderson Consulting Engineers
Date:	10/24/14
Sample Data	
Sample:	B-2 @ 5 ft.
Remolded:	90 % Relative Compaction
Remarks:	Soaked Before Placing in Shear Box
Soil Description:	Orange-Brown Silty Sand (SM)

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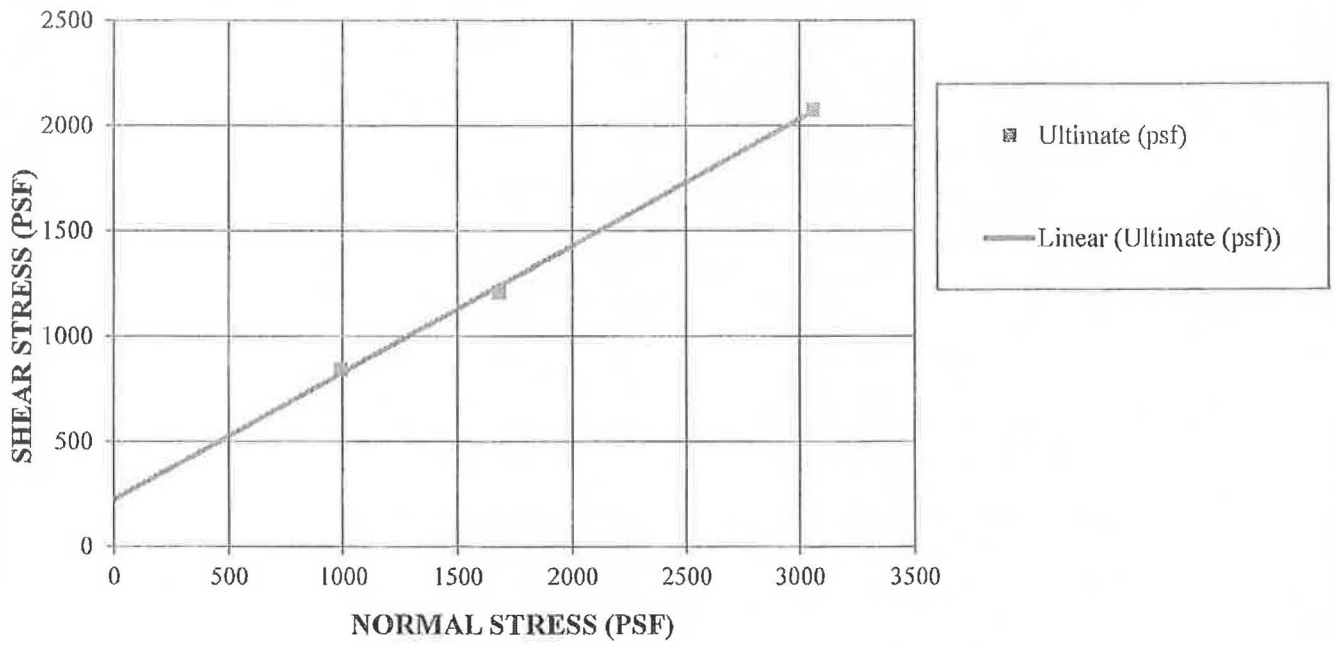
2195 Faraday Avenue, Suite K, Carlsbad, CA 92008

SHEAR TEST DIAGRAM



Test Results

	Phi		Cohesion	
Ultimate (psf)	31	degrees	225	psf

Average Initial Moisture	3.0%
Average Dry Density	112.9 pcf
Average Final Moisture	15.7%

L A B O R A T O R Y R E P O R T

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A N A L Y T I C A L A N D C O N S U L T I N G C H E M I S T S

Date: October 29, 2014
Purchase Order Number: GLO-71982-4
Sales Order Number: 24454
Account Number: EEI
To:

EEI Environmental Equalizers Inc
2195 Faraday Avenue Suite K
Carlsbad, CA 92008
Attention: Hector Estrella/Jeff Blake

Laboratory Number: S05463 Customers Phone: 760-431-3747

Sample Designation:

One soil sample received on 10/23/14 at 3:00pm, from Global - Ironwood Project# GLO-70982-4, marked as B-1 @ 0'-5' SM.

Analysis By California Test 643, 1999, Department of Transportation Division of Construction, Method for Estimating the Service Life of Steel Culverts.

pH 7.1

Water Added (ml)	Resistivity (ohm-cm)
10	13000
5	9500
5	7800
5	7300
5	6400
5	5800
5	5200
5	5500
5	5800

35 years to perforation for a 16 gauge metal culvert.
46 years to perforation for a 14 gauge metal culvert.
63 years to perforation for a 12 gauge metal culvert.
81 years to perforation for a 10 gauge metal culvert.
98 years to perforation for a 8 gauge metal culvert.

Water Soluble Sulfate Calif. Test 417 0.005%

Water Soluble Chloride Calif. Test 422 0.007%

Laura Torres

Laura Torres
LT/dbb

APPENDIX C
EARTHWORK AND GRADING GUIDELINES

EARTHWORK AND GRADING GUIDELINES

GENERAL

These guidelines present general procedures and recommendations for earthwork and grading as required on the approved grading plans, including preparation of areas to be filled, placement of fill and installation of subdrains and excavations. The recommendations contained in the geotechnical report are applicable to each specific project, are part of the earthwork and grading guidelines and would supersede the provisions contained hereafter in the case of conflict. Observations and/or testing performed by the consultant during the course of grading may result in revised recommendations which could supersede these guidelines or the recommendations contained in the geotechnical report. Figures A through O are provided at the back of this appendix, exhibiting generalized cross sections relating to these guidelines.

The contractor is responsible for the satisfactory completion of all earthworks in accordance with provisions of the project plans and specifications. The project soil engineer and engineering geologist (geotechnical consultant) or their representatives should provide observation and testing services, and geotechnical consultation throughout the duration of the project.

EARTHWORK OBSERVATIONS AND TESTING

Geotechnical Consultant

Prior to the commencement of grading, a qualified geotechnical consultant (a soil engineer and engineering geologist) should be employed for the purpose of observing earthwork procedures and testing the fills for conformance with the recommendations of the geotechnical report, the approved grading plans, and applicable grading codes and ordinances.

The geotechnical consultant should provide testing and observation so that determination may be made that the work is being completed as specified. It is the responsibility of the contractor to assist the consultant and keep them aware of work schedules and predicted changes, so that the consultant may schedule their personnel accordingly.

All removals, prepared ground to receive fill, key excavations, and subdrains should be observed and documented by the project engineering geologist and/or soil engineer prior to placing any fill. It is the contractor's responsibility to notify the engineering geologist and soil engineer when such areas are ready for observation.

Laboratory and Field Tests

Maximum dry density tests to determine the degree of compaction should be performed in accordance with American Standard Testing Materials test method ASTM designation D-1557-78. Random field compaction tests should be performed in accordance with test method ASTM designations D-1556-82, D-2937 or D-2922 & D-3017, at intervals of approximately two (2) feet of fill height per 10,000 sq. ft. or every one thousand cubic yards of fill placed. These criteria would vary depending on the soil conditions and the size of the project. The location and frequency of testing would be at the discretion of the geotechnical consultant

Contractor's Responsibility

All clearing, site preparation, and earthwork performed on the project should be conducted by the contractor, with observation by geotechnical consultants and staged approval by the appropriate governing agencies. It is the contractor's responsibility to prepare the ground surface to receive the fill to the satisfaction of the soil engineer, and to place, spread, moisture condition, mix and compact the fill in accordance with the recommendations of the soil engineer. The contractor should also remove all major deleterious material considered unsatisfactory by the soil engineer.

It is the sole responsibility of the contractor to provide adequate equipment and methods to accomplish the earthwork in accordance with applicable grading guidelines, codes or agency ordinances, and approved grading plans. Sufficient watering apparatus and compaction equipment should be provided by the contractor with due consideration for the fill material, rate of placement, and climatic conditions. If, in the opinion of the geotechnical consultant, unsatisfactory conditions such as questionable weather, excessive oversized rock, deleterious material or insufficient support equipment are resulting in a quality of work that is not acceptable, the consultant will inform the contractor, and the contractor is expected to rectify the conditions, and if necessary, stop work until conditions are satisfactory.

The contractor will properly grade all surfaces to maintain good drainage and prevent ponding of water. The contractor will take action to control surface water and to prevent erosion control measures that have been installed.

SITE PREPARATION

All vegetation including brush, trees, thick grasses, organic debris, and other deleterious material should be removed and disposed of offsite, and must be concluded prior to placing fill. Existing fill, soil, alluvium, colluvium, or rock materials determined by the soil engineer or engineering geologist as unsuitable for structural in-place support should be removed prior to fill placement. Depending upon the soil conditions, these materials may be reused as compacted fills. Any materials incorporated as part of the compacted fills should be approved by the soil engineer.

Any underground structures such as cesspools, cisterns, mining shafts, tunnels, septic tanks, wells, pipelines, or other structures not located prior to grading are to be removed or treated in a manner recommended by the soil engineer. Soft, dry, spongy, highly fractured, or otherwise unsuitable ground extending to such a depth that surface processing cannot adequately improve the condition should be over excavated down to firm ground and approved by the soil engineer before compaction and filling operations continue. Over excavated and processed soils which have been properly mixed and moisture-conditioned should be recompacted to the minimum relative compaction as specified in these guidelines.

Existing ground which is determined to be satisfactory for support of the fills should be scarified to a minimum depth of six (6) inches, or as directed by the soil engineer. After the scarified ground is brought to optimum moisture (or greater) and mixed, the materials should be compacted as specified herein. If the scarified zone is greater than 6 inches in depth, it may be necessary to remove the excess and place the material in lifts restricted to six (6) inches in compacted thickness.

Existing grind which is not satisfactory to support compacted fill should be over excavated as required in the geotechnical report or by the onsite soils engineer and/or engineering geologists. Scarification, discing, or other acceptable form of mixing should continue until the soils are broken down and free of large fragments or clods, until the working surface is reasonably uniform and free from ruts, hollows, hummocks, or other uneven features which would inhibit compaction as described above.

Where fills are to be placed on ground with slopes steeper than 5:1 (horizontal to vertical) gradient, the ground should be benched. The lowest bench, which will act as a key, should be a minimum of 12 feet wide and should be at least two (2) feet deep into competent material, approved by the soil engineer and/or engineering geologist. In fill over cut slope conditions, the recommended minimum width of the lowest bench or key is at least 15 feet with the key excavated on competent material, as designated by the Geotechnical Consultant. As a general rule, unless superseded by the Soil Engineer, the minimum width of fill keys should be approximately equal to one-half (½) the height of the slope.

Standard benching is typically four feet (minimum) vertically, exposing competent material. Benching may be used to remove unsuitable materials, although it is understood that the vertical height of the bench may exceed four feet. Pre stripping may be considered for removal of unsuitable materials in excess of four feet in thickness.

All areas to receive fill, including processed areas, removal areas, and toe of fill benches should be observed and approved by the soil engineer and/or engineering geologist prior to placement of fill. Fills may then be properly placed and compacted until design grades are attained.

COMPACTED FILLS

Earth materials imported or excavated on the property may be utilized as fill provided that each soil type has been accepted by the soil engineer. These materials should be free of roots, tree branches, other organic matter or other deleterious materials. All unsuitable materials should be removed from the fill as directed by the soil engineer. Soils of poor gradation, undesirable expansion potential, or substandard strength characteristics may be designated unsuitable by the consultant and may require mixing with other earth materials to serve as a satisfactory fill material.

Fill materials generated from benching operations should be dispersed throughout the fill area. Benching operations should not result in the benched material being placed only within a single equipment width away from the fill/bedrock contact.

Oversized materials, defined as rock or other irreducible materials with a maximum size exceeding 12 inches in one dimension, should not be buried or placed in fills unless the location of materials and disposal methods are specifically approved by the soil engineer. Oversized material should be taken offsite or placed in accordance with recommendations of the soil engineer in areas designated as suitable for rock disposal. Oversized material should not be placed vertically within 10 feet of finish grade or horizontally within 20 feet of slope faces.

To facilitate trenching, rock should not be placed within the range of foundation excavations or future utilities unless specifically approved by the soil engineer and/or the representative developers.

If import fill material is required for grading, representative samples of the material should be analyzed in the laboratory by the soil engineer to determine its physical properties. If any material other than that previously analyzed is imported to the fill or encountered during grading, analysis of this material should be conducted by the soil engineer as soon as practical.

Fill material should be placed in areas prepared to receive fill in near-horizontal layers that should not exceed six (6) inches compacted in thickness. The soil engineer may approve thicker lifts if testing indicates the grading procedures are such that adequate compaction is being achieved. Each layer should be spread evenly and mixed to attain uniformity of material and moisture suitable for compaction.

Fill materials at moisture content less than optimum should be watered and mixed, and "wet" fill materials should be aerated by scarification, or should be mixed with drier material. Moisture conditioning and mixing of fill materials should continue until the fill materials have uniform moisture content at or above optimum moisture.

After each layer has been evenly spread, moisture-conditioned and mixed, it should be uniformly compacted to a minimum of 90 percent of maximum density as determined by ASTM test designation, D 1557-78, or as otherwise recommended by the soil engineer. Compaction equipment should be adequately sized and should be reliable to efficiently achieve the required degree of compaction.

Where tests indicate that the density of any layer of fill, or portion thereof, is below the required relative compaction or improper moisture content, the particular layer or portion will be reworked until the required density and/or moisture content has been attained. No additional fill will be placed in an area until the last placed lift of fill has been tested and found to meet the density and moisture requirements, and is approved by the soil engineer.

Compaction of slopes should be accomplished by over-building the outside edge a minimum of three (3) feet horizontally, and subsequently trimming back to the finish design slope configuration. Testing will be performed as the fill is horizontally placed to evaluate compaction as the fill core is being developed. Special efforts may be necessary to attain the specified compaction in the fill slope zone. Final slope shaping should be performed by trimming and removing loose materials with appropriate equipment. A final determination of fill slope compaction should be based on observation and/or testing of the finished slope face.

If an alternative to over-building and cutting back the compacted fill slope is selected, then additional efforts should be made to achieve the required compaction in the outer 10 feet of each lift of fill by undertaking the following:

- Equipment consisting of a heavy short-shanked sheepsfoot should be used to roll (horizontal) parallel to the slopes continuously as fill is placed. The sheepsfoot roller should also be used to roll perpendicular to the slopes, and extend out over the slope to provide adequate compaction to the face slope.

- Loose fill should not be spilled out over the face of the slope as each lift is compacted. Any loose fill spilled over a previously completed slope face should be trimmed off or be subject to re-rolling.

- Field compaction tests will be made in the outer two (2) to five (5) feet of the slope at two (2) to three (3) foot vertical intervals, subsequent to compaction operations.

- After completion of the slope, the slope face should be shaped with a small dozer and then re-rolled with a sheepsfoot to achieve compaction to near the slope face. Subsequent to testing to verify compaction, the slopes should be grid-rolled to achieve adequate compaction to the slope face. Final testing should be used to confirm compaction after grid rolling.

- Where testing indicates less than adequate compaction, the contractor will be responsible to process, moisture condition, mix and recompact the slope materials as necessary to achieve compaction. Additional testing should be performed to verify compaction.

- Erosion control and drainage devices should be designed by the project civil engineer in compliance with the ordinances of the controlling governmental agencies, and/or in accordance with the recommendations of the soil engineer or engineering geologist.

EXCAVATIONS

Excavations and cut slopes should be observed and mapped during grading by the engineering geologist. If directed by the engineering geologist, further excavations or over-excavation and refilling of cut areas should be performed. When fills over cut slopes are to be graded, the cut portion of the slope should be observed by the engineering geologist prior to placement of the overlying fill portion of the slope. The engineering geologist should observe all cut slopes and should be notified by the contractor when cut slopes are started.

If, during the course of grading, unanticipated adverse or potentially adverse geologic conditions are encountered, the engineering geologist and soil engineer should investigate, evaluate and make recommendations to mitigate (or limit) these conditions. The need for cut slope buttressing or stabilizing should be based on as-grading evaluations by the engineering geologist, whether anticipated previously or not.

Unless otherwise specified in soil and geological reports, no cut slopes should be excavated higher or steeper than that allowed by the ordinances of controlling governmental agencies. Additionally, short-term stability of temporary cut slopes is the contractor's responsibility.

Erosion control and drainage devices should be designed by the project civil engineer and should be constructed in compliance with the ordinances of the controlling governmental agencies, and/or in accordance with the recommendations of the soil engineer or engineering geologist.

SUBDRAIN INSTALLATION

Subdrains should be installed in accordance with the approved embedment material, alignment and details indicated by the geotechnical consultant. Subdrain locations or construction materials should not be changed or modified without approval of the geotechnical consultant. The soil engineer and/or engineering geologist may recommend and direct changes in subdrain line, grade and drain material in the field, pending exposed conditions. The location of constructed subdrains should be recorded by the project civil engineer.

COMPLETION

Consultation, observation and testing by the geotechnical consultant should be completed during grading operations in order to state an opinion that all cut and filled areas are graded in accordance with the approved project specifications.

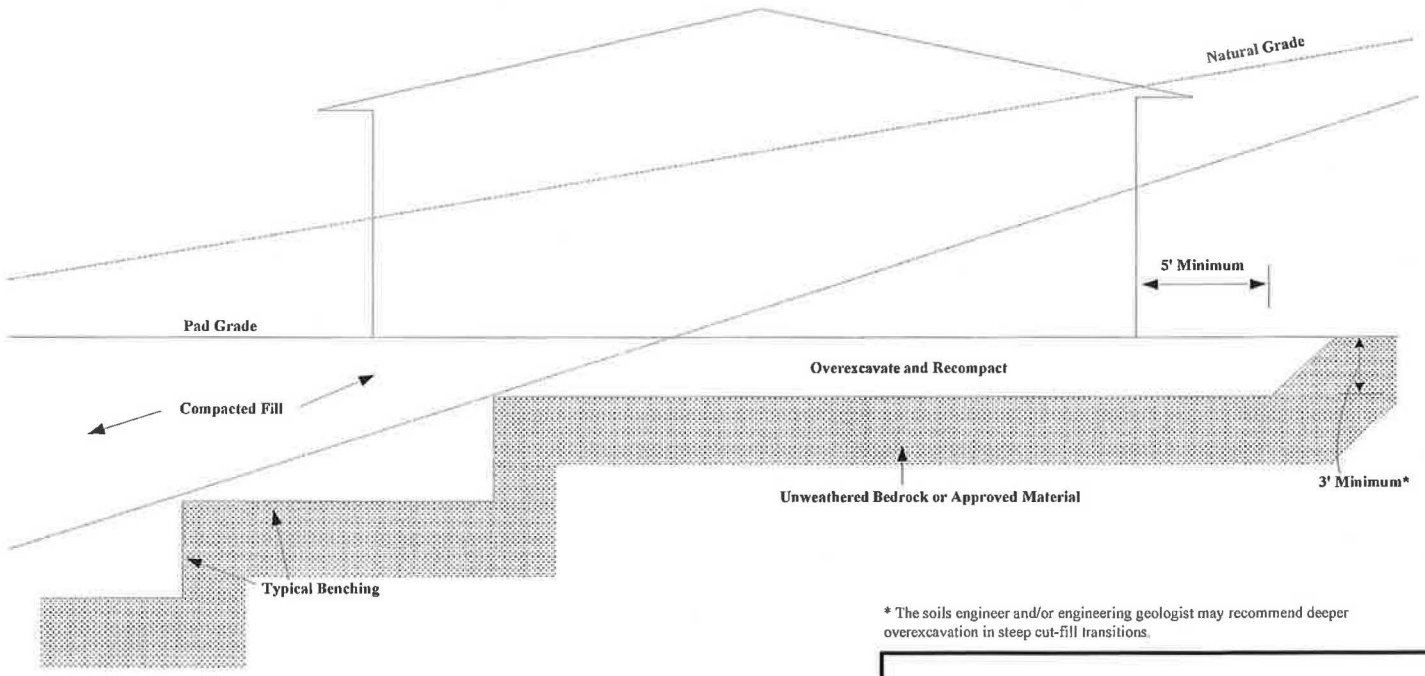
After completion of grading and after the soil engineer and engineering geologist have finished their observations, final reports should be submitted subject to review by the controlling governmental agencies. No additional grading should be undertaken without prior notification of the soil engineer and/or engineering geologist.

All finished cut and fill slopes should be protected from erosion, including but not limited to planting in accordance with the plan design specifications and/or as recommended by a landscape architect. Such protection and/or planning should be undertaken as soon as possible after completion of grading.

ATTACHMENTS

Figure A – Transition Lot Detail Cut Lot
Figure B – Transition Lot Detail Cut - Fill
Figure C – Rock Disposal Pits
Figure D – Detail for Fill Slope Toeing out on a Flat Alluviated Canyon
Figure E – Removal Adjacent to Existing Fill
Figure F – Daylight Cut Lot Detail
Figure G -- Skin Fill of Natural Ground
Figure H – Typical Stabilization Buttress Fill Design
Figure I – Stabilization Fill for Unstable Material Exposed in Portion of Cut Slope
Figure J – Fill Over Cut Detail
Figure K – Fill Over Natural Detail
Figure L – Oversize Rock Disposal
Figure M – Canyon Subdrain Detail
Figure N – Canyon Subdrain Alternate Details
Figure O – Typical Stabilization Buttress Subdrain Detail
Figure P – Retaining Wall Backfill

TRANSITION LOT DETAIL
CUT LOT – MATERIAL TYPE TRANSITION



* The soils engineer and/or engineering geologist may recommend deeper overexcavation in steep cut-fill transitions.

Note: Figure not to scale

EARTHWORK AND GRADING GUIDELINES
TRANSITION LOT DETAIL
CUT LOT – MATERIAL TYPE TRANSITION

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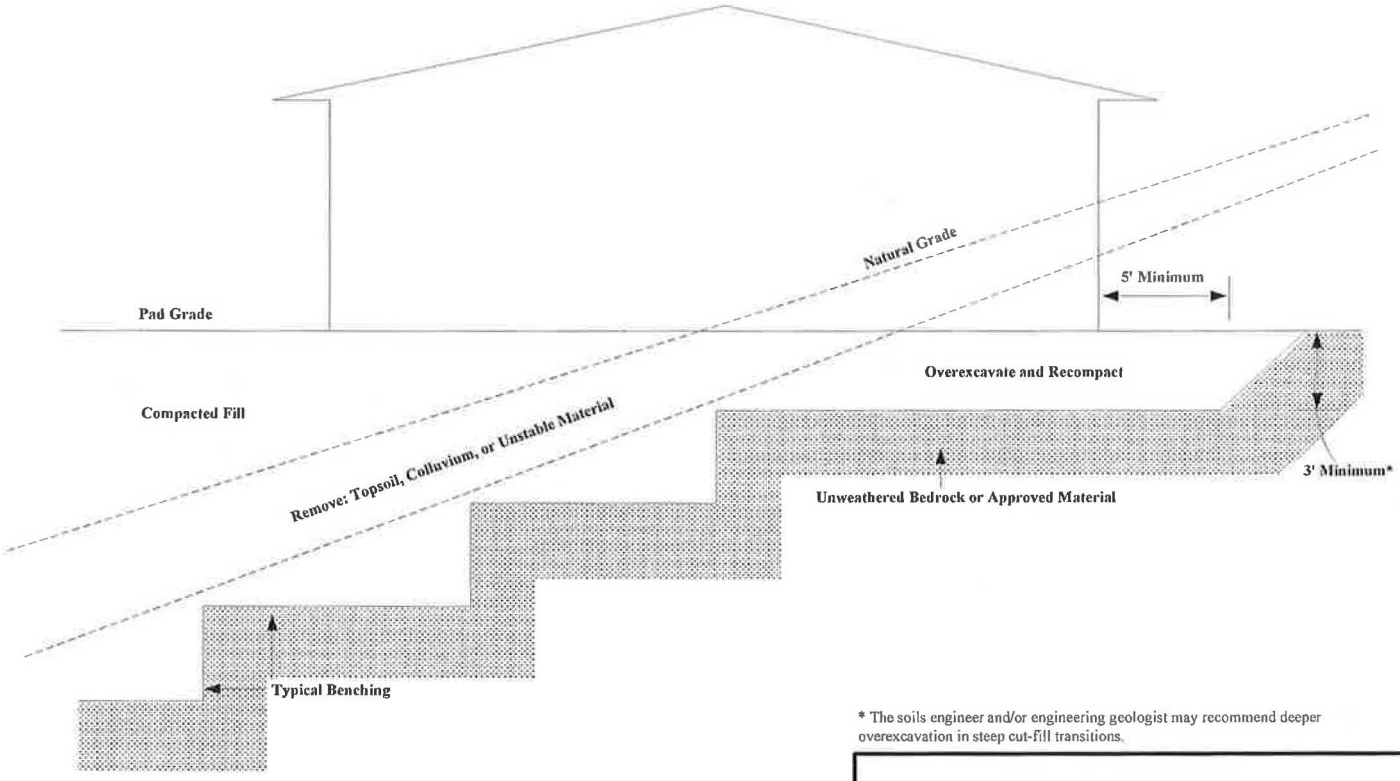
FIGURE A

TRANSITION LOT DETAIL
CUT – FILL – DAYLIGHT TRANSITION



* The soils engineer and/or engineering geologist may recommend deeper overexcavation in steep cut-fill transitions.

Note: Figure not to scale

EARTHWORK AND GRADING GUIDELINES
TRANSITION LOT DETAIL
CUT – FILL – DAYLIGHT TRANSITION

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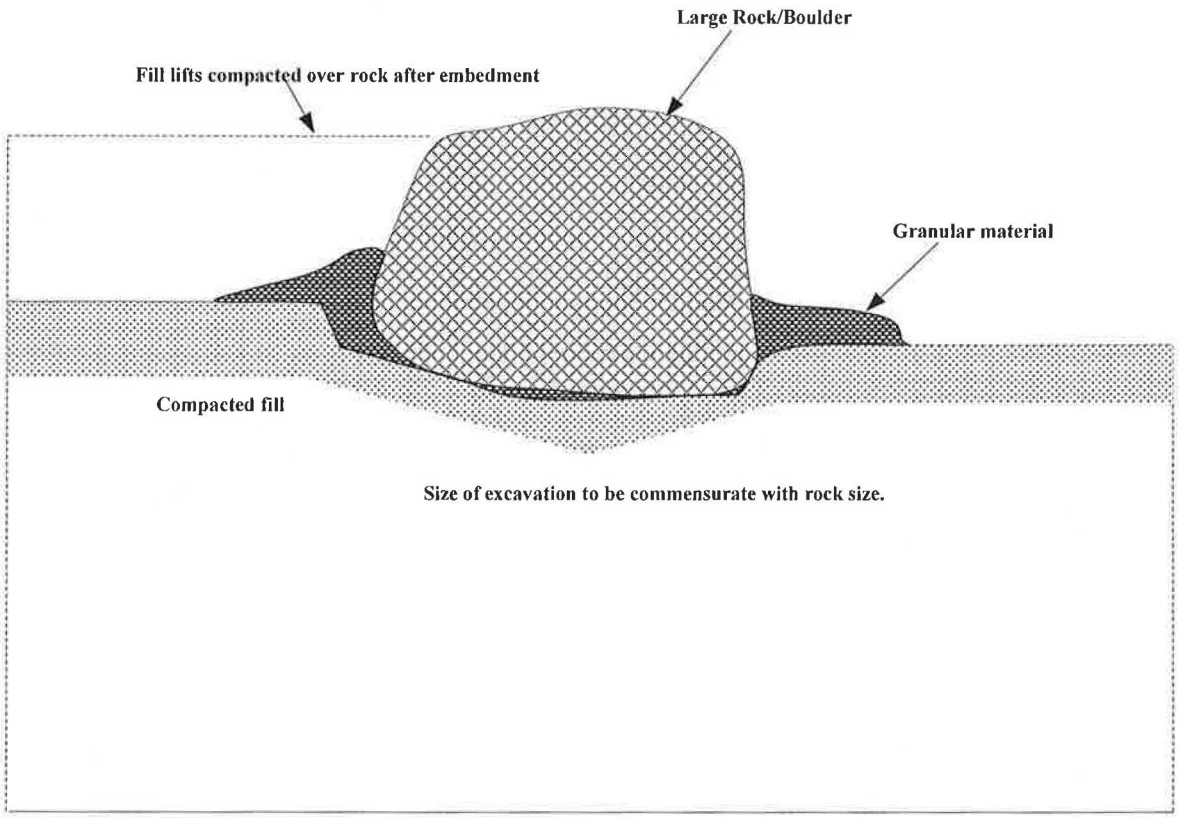
FIGURE B

ROCK DISPOSAL PITS



Note:
(1) Large rock is defined as having a diameter larger than 3 feet in maximum size.
(2) Pit shall be excavated into compacted fill to a depth equal to half of the rock size.
(3) Granular soil shall be pushed into the pit and then flooded around the rock using a sheepsfoot to help with compaction.
(4) A minimum of 3 feet of compacted fill should be laid over each pit.
(5) Pits shall have at least 15 feet of separation between one another, horizontally.
(6) Pits shall be placed at least 20 feet from any fill slope.
(7) Pits shall be used only in deep fill areas.

Note: Figure not to scale

EARTHWORK AND GRADING GUIDELINES
ROCK DISPOSAL PITS

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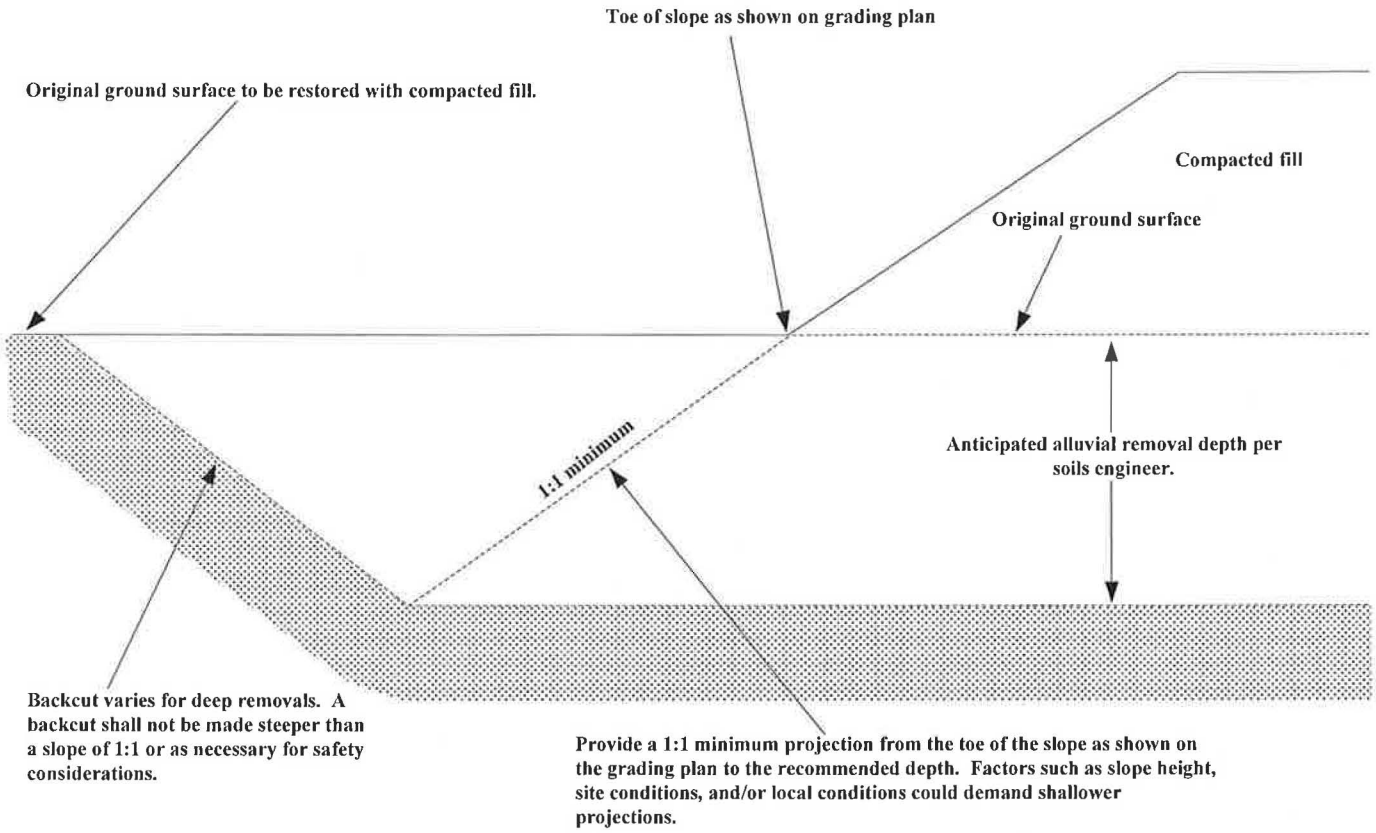
FIGURE C

DETAIL FOR FILL SLOPE TOEING OUT ON FLAT ALLUVIATED CANYON



Note: Figure not to scale

EARTHWORK AND GRADING GUIDELINES
DETAIL FOR FILL SLOPE TOEING OUT ON A FLAT ALLUVIATED CANYON

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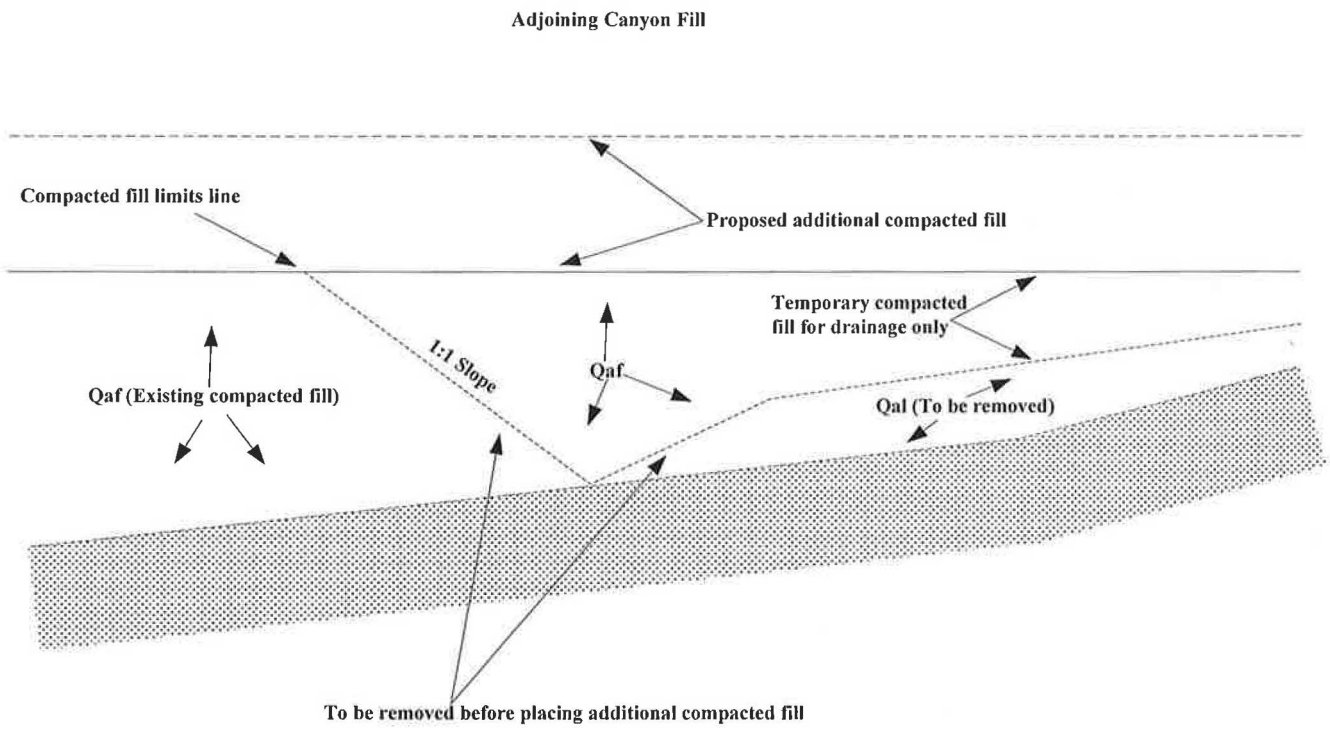
FIGURE D

REMOVAL ADJACENT TO EXISTING FILL



Legend		
Qaf -	Artificial Fill	
Qal -	Alluvium	

Note: Figure not to scale

EARTHWORK AND GRADING GUIDELINES
REMOVAL ADJACENT TO EXISTING FILL

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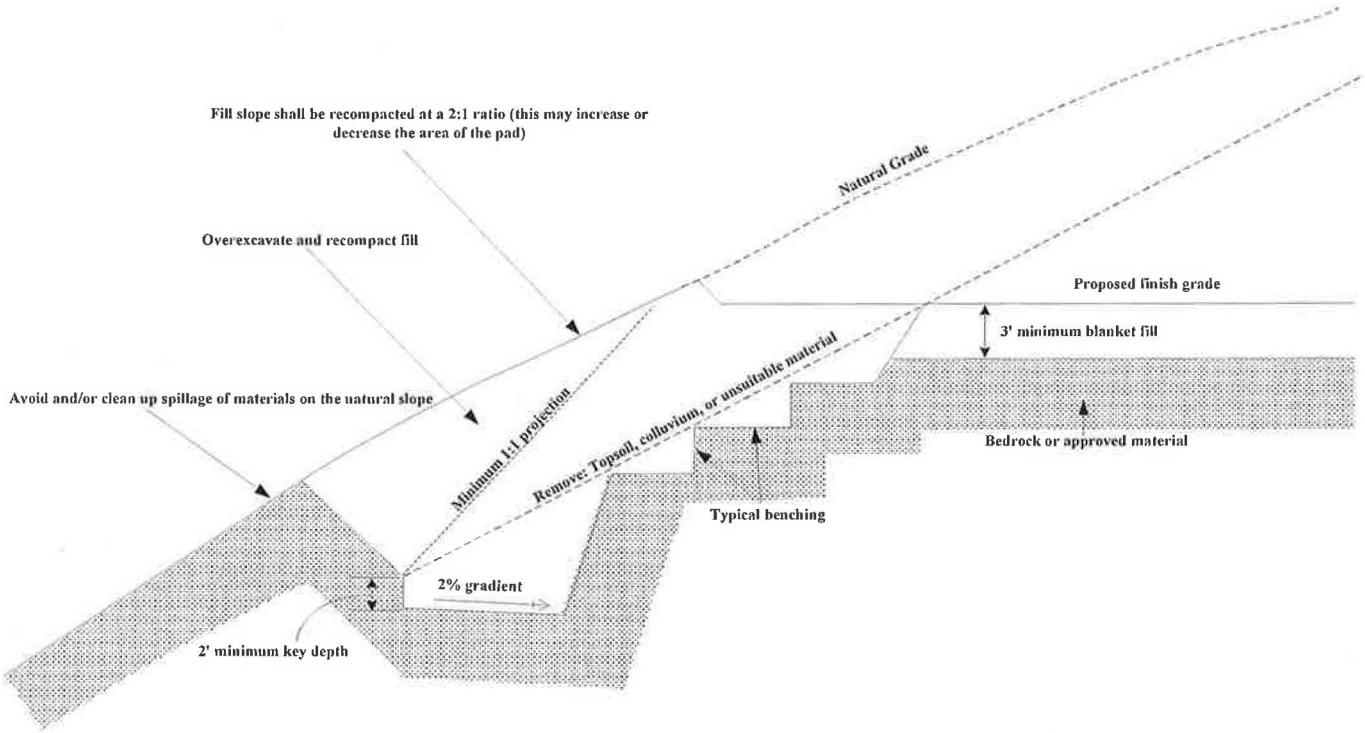
FIGURE E

DAYLIGHT CUT LOT DETAIL



Note: (1) Subdrain and key width requirements shall be determined based on exposed subsurface conditions and the thickness of overburden.

(2) Pad overexcavation and recompaction shall be completed if determined as necessary by the soils engineer and/or engineering geologist.

Note: Figure not to scale

EARTHWORK AND GRADING GUIDELINES
DAYLIGHT CUT LOT DETAIL

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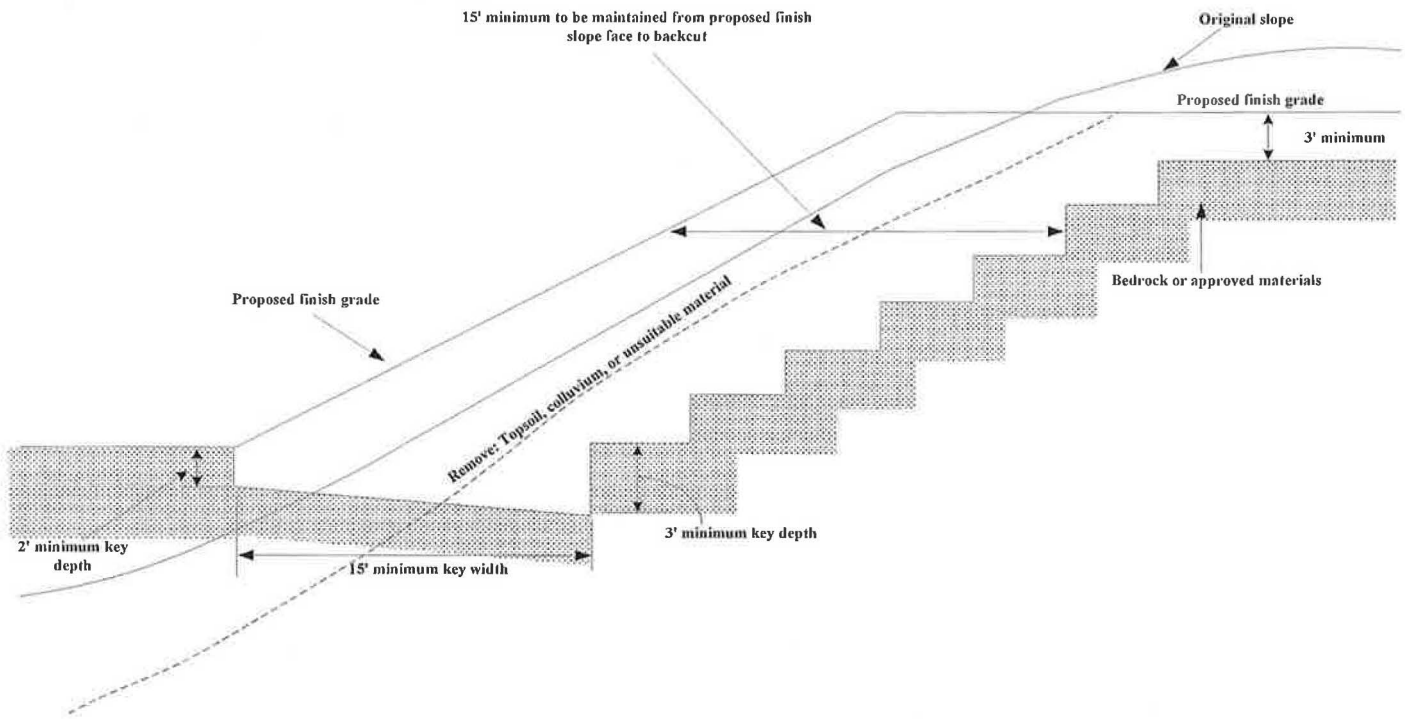
FIGURE F

SKIN FILL OF NATURAL GROUND



Note: (1) The need and disposition of drains will be determined by the soils engineer and/or engineering geologist based on site conditions.

(2) Pad overexcavation and recompaction shall be completed if determined as necessary by the soils engineer and/or engineering geologist.

Note: Figure not to scale

EARTHWORK AND GRADING GUIDELINES
SKIN FILL OF NATURAL GROUND

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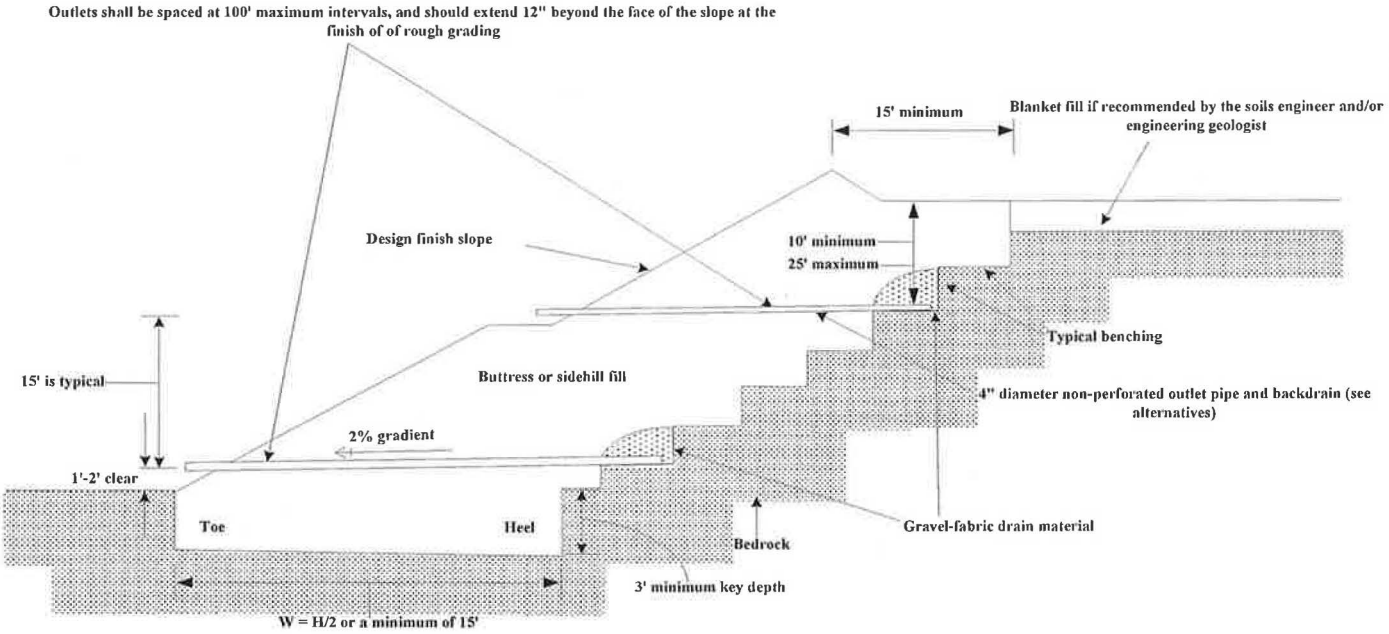
FIGURE G

TYPICAL STABILIZATION BUTTRESS FILL DESIGN



EARTHWORK AND GRADING GUIDELINES
TYPICAL STABILIZATION BUTTRESS FILL DESIGN

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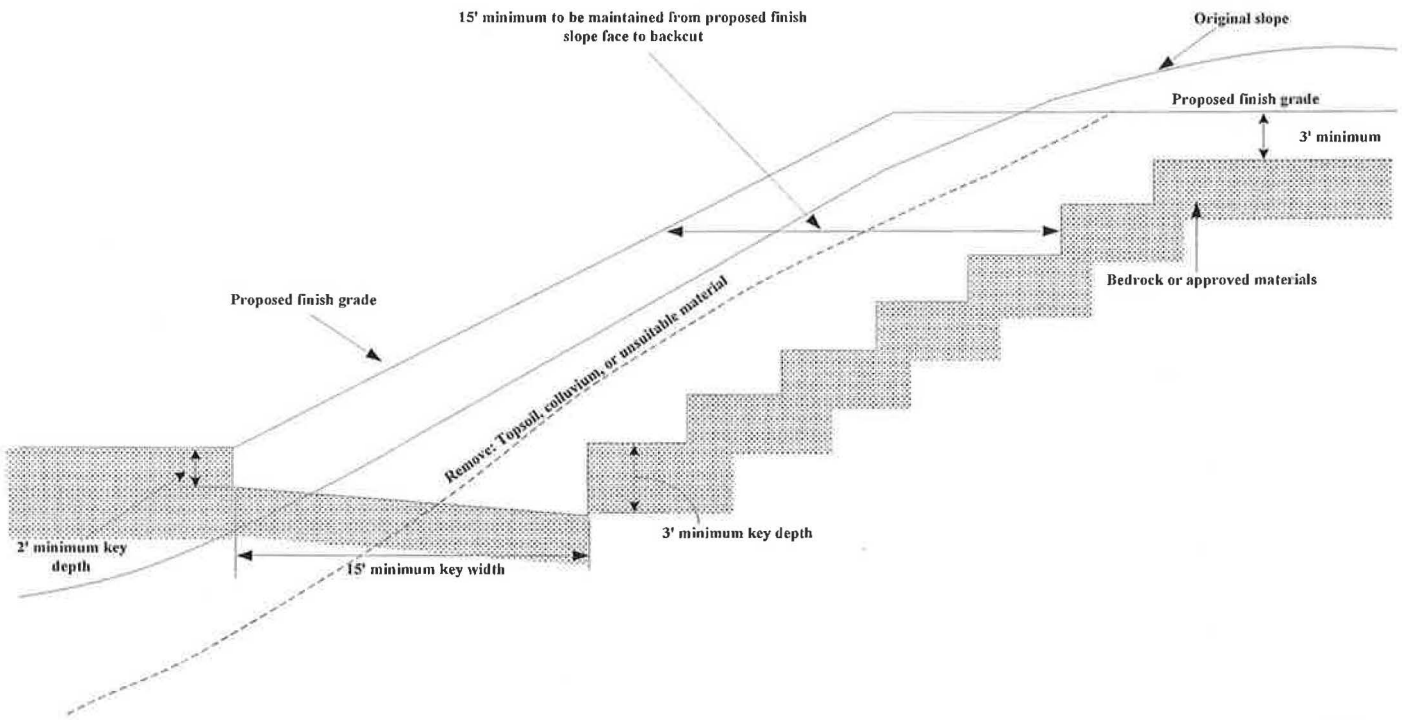
FIGURE H

Note: Figure not to scale

SKIN FILL OF NATURAL GROUND



Note: (1) The need and disposition of drains will be determined by the soils engineer and/or engineering geologist based on site conditions.

(2) Pad overexcavation and recompaction shall be completed if determined as necessary by the soils engineer and/or engineering geologist.

Note: Figure not to scale

EARTHWORK AND GRADING GUIDELINES
SKIN FILL OF NATURAL GROUND

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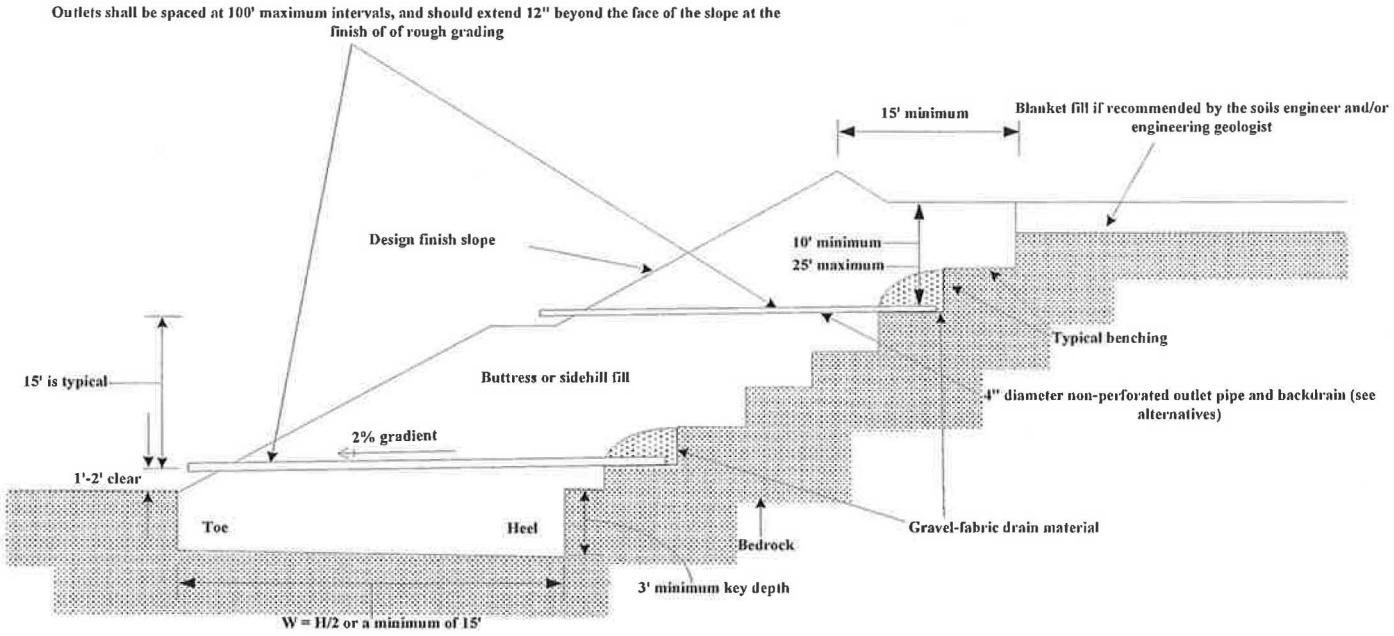
FIGURE G

TYPICAL STABILIZATION BUTTRESS FILL DESIGN



Note: Figure not to scale

EARTHWORK AND GRADING GUIDELINES
TYPICAL STABILIZATION BUTTRESS FILL DESIGN

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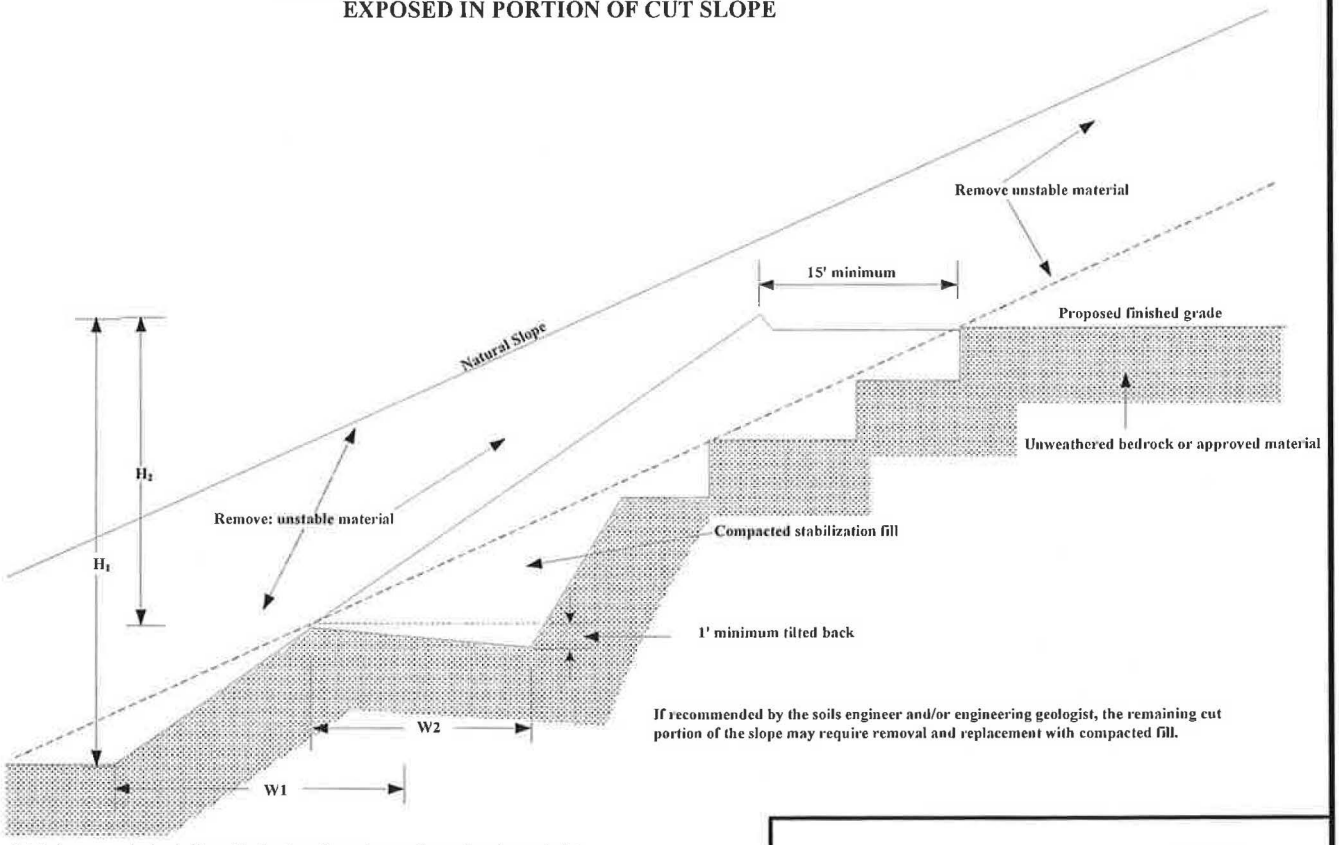
FIGURE H

STABILIZATION FILL FOR UNSTABLE MATERIAL EXPOSED IN PORTION OF CUT SLOPE



Note: (1) Subdrains are required only if specified by the soils engineer and/or engineering geologist.
(2) "W" shall be the equipment width (15') for slope heights less than 25 feet. For slopes greater than 25 feet "W" shall be determined by the project soils engineer and/or the engineering geologist. "W" shall never be less than H/2.

Note: Figure not to scale

EARTHWORK AND GRADING GUIDELINES
STABILIZATION FILL FOR UNSTABLE MATERIAL EXPOSED IN PORTION OF CUT SLOPE

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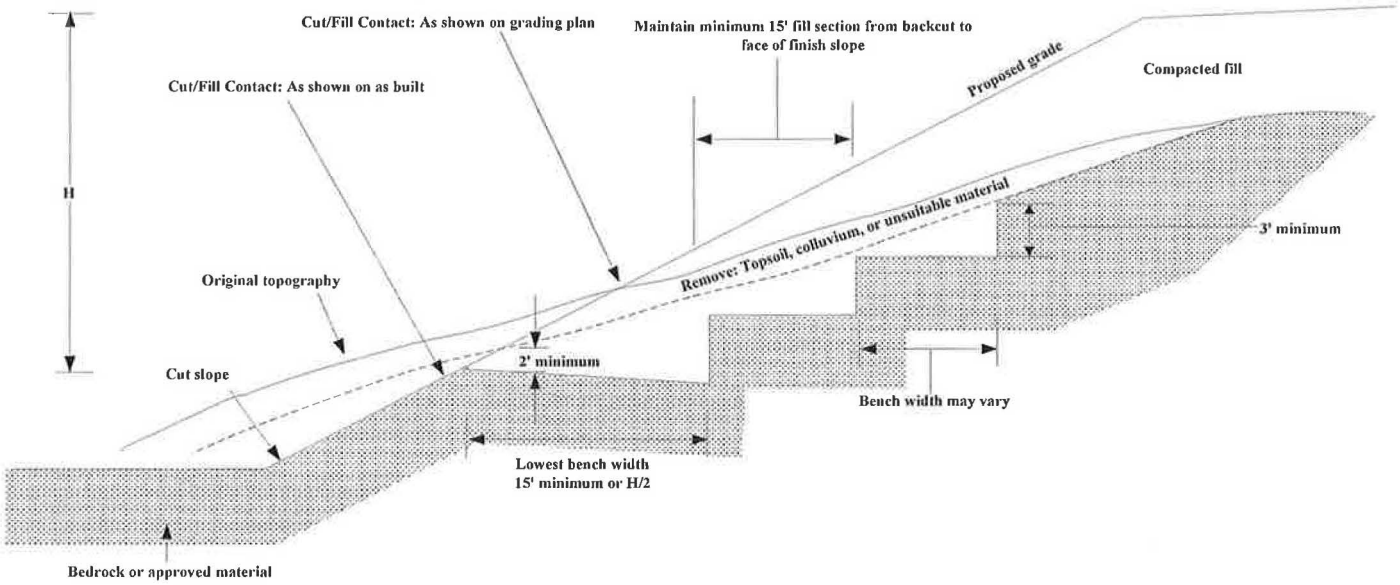
FIGURE I

FILL OVER CUT DETAIL



Note: The cut sectioin shall be excavated and evaluated by the soils engineer/engineering geologist prior to constructing the fill portion.

Note: Figure not to scale

EARTHWORK AND GRADING GUIDELINES
FILL OVER CUT DETAIL

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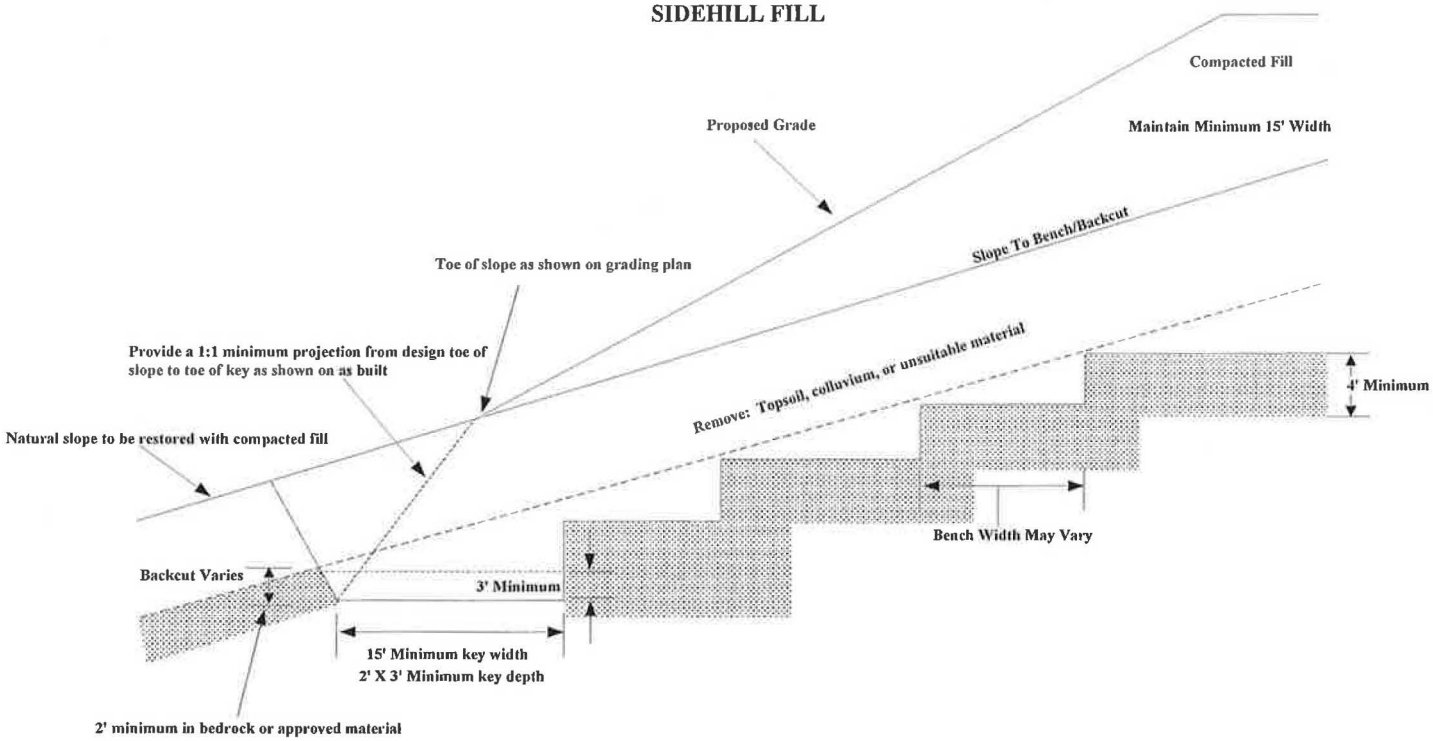
FIGURE J

FILL OVER NATURAL DETAIL
SIDEHILL FILL



Note: (1) Special recommendations shall be provided by the soils engineer/engineering geologist where the natural slope approaches or exceeds the design slope ratio.
(2) The need for and disposition of drains would be determined by the soils engineer/engineering geologist based upon exposed conditions.

Note: Figures not to scale

EARTHWORK AND GRADING GUIDELINES
FILL OVER NATURAL DETAIL
SIDEHILL FILL

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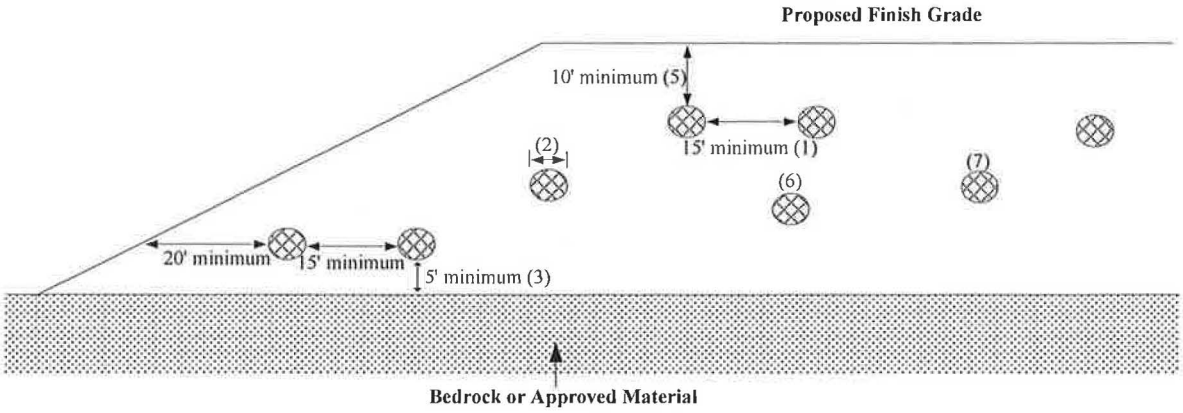
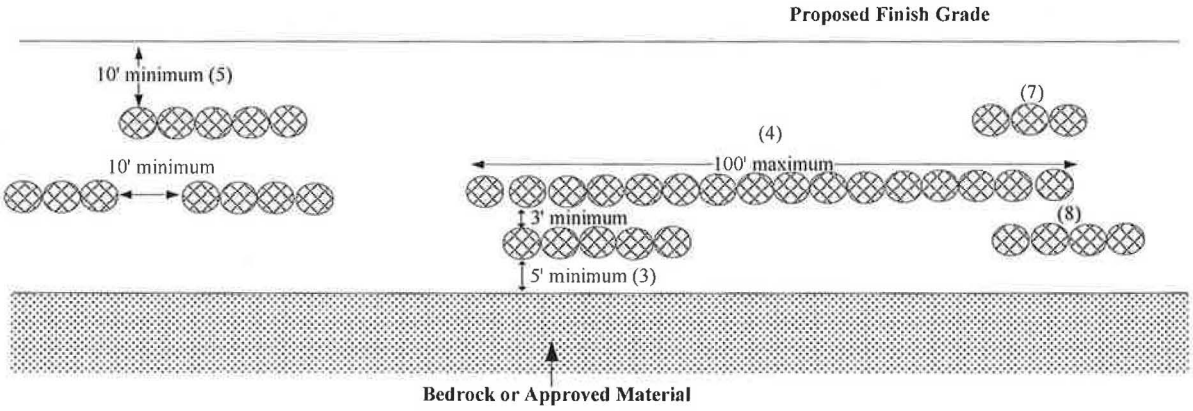
FIGURE K

OVERSIZE ROCK DISPOSAL

View Normal to Slope Face



View Parallel to Slope Face



Note:
(1) One Equipment width or a minimum of 15 feet.
(2) Height and width may vary depending on rock size and type of equipment used. Length of windrow shall be no greater than 100 feet maximum.
(3) If approved by the soils engineer and/or engineering geologist.
(4) Orientation of windrows may vary but shall be as recommended by the soils engineer and/or engineering geologist. Unless recommended staggering of windrows is not necessary.
(5) Areas shall be cleared for utility trenches, foundations, and swimming pools.
(6) Voids in windrows shall be filled by flooding granular soil into place. Granular soil shall be any soil which has a unified soil classification system (Universal Building Code (UBC) 29-1). Designation of SM, SP, SW, GP, or GW.
(7) After fill between windrows is placed and compacted with the lift of fill covering windrow, windrow shall be proof rolled with a D-9 dozer or equivalent.
(8) Oversized rock is defined as larger than 12", and less than 4 feet in size.

Approximate Scale: 1" = 30'

Note: All distances are approximate

EARTHWORK AND GRADING GUIDELINES
OVERSIZE ROCK DISPOSAL

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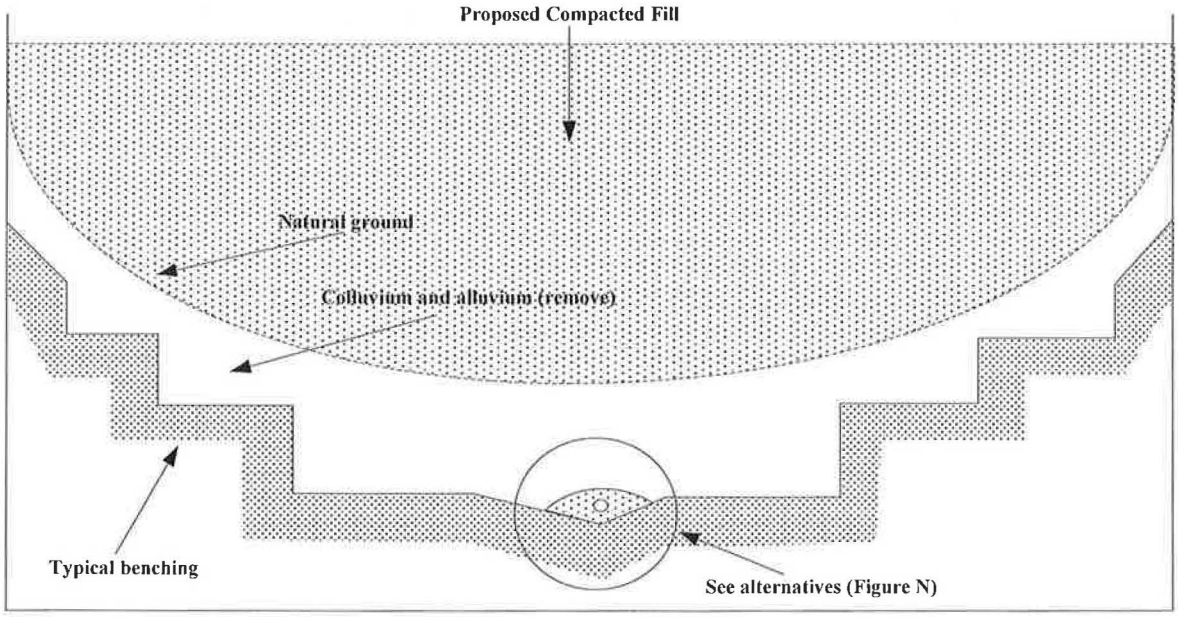
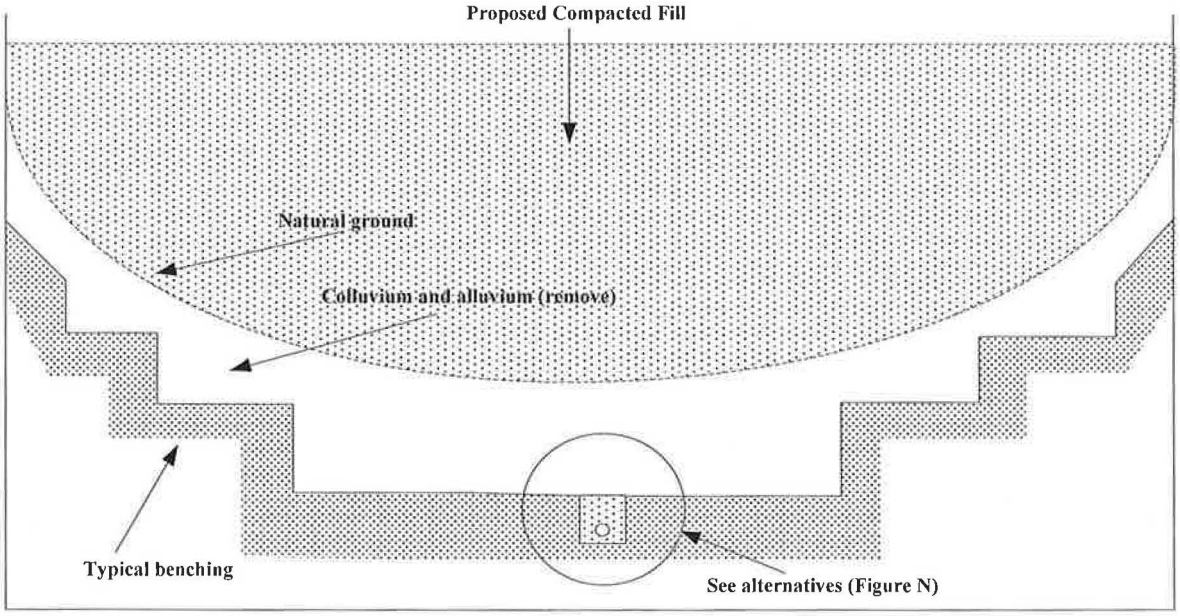
FIGURE L

CANYON SUBDRAIN DETAIL

Type A



Type B



Note: Alternatives, locations, and extent of subdrains should be determined by the soils engineer and/or engineering geologist during actual grading.

EARTHWORK AND GRADING GUIDELINES
CANYON SUBDRAIN DETAIL

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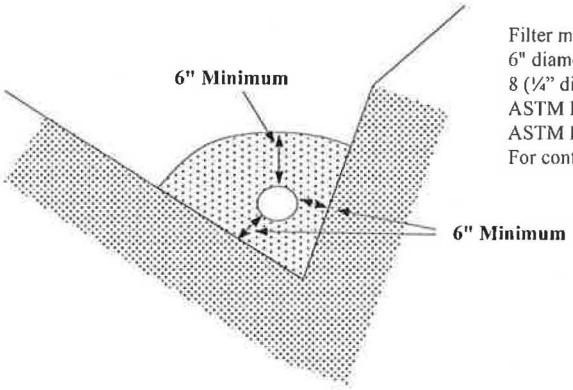
FIGURE M

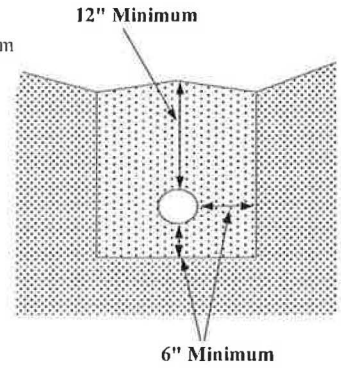
Note: Figures not to scale

CANYON SUBDRAIN ALTERNATE DETAILS

Alternate 1: Perforated Pipe and Filter Material



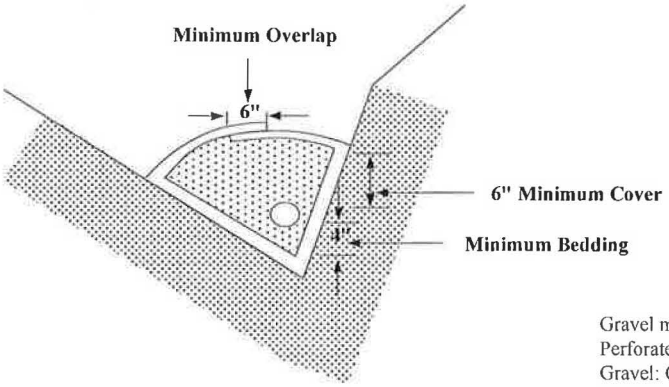
Filter material: Minimum volume of 9 feet^3/linear foot.
6" diameter ABS or PVC pipe or approved substitute with minimum 8 (¼" diameter) perforations per linear foot in bottom half of pipe.
ASTM D 2751, SDR 35 or ASTM D 1527, Schedule 40.
ASTM D 3034, SDR 35 or ASTM D 1785, Schedule 40.
For continuous run in excess of 500 feet use 8" diameter pipe.

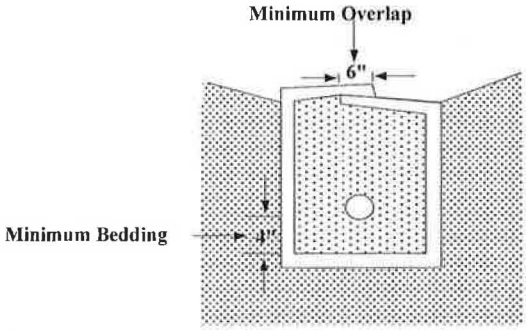


Filter Material

Sieve Size	Percent Passing
1"	100
¾"	90-100
3/8"	40-100
No. 4	25-40
No. 8	18-33
No. 30	5-15
No. 50	0-7
No. 200	0-3

Alternate 2: Perforated Pipe, Gravel and Filter Fabric





Gravel material 9 feet^3/linear foot.
Perforated pipe: see alternate 1.
Gravel: Clean ¾" rock or approved substitute.
Filter Fabric: Mirafi 140 or approved substitute.

Note: Figures not to scale

EARTHWORK AND GRADING GUIDELINES
CANYON SUBDRAIN ALTERNATE DETAILS

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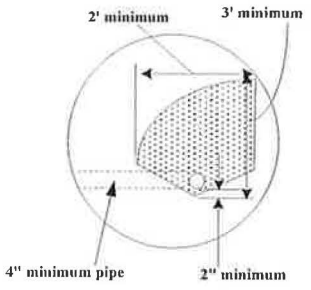
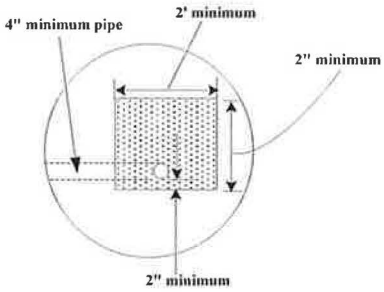
FIGURE N

TYPICAL STABILIZATION BUTTRESS SUBDRAIN DETAIL





Filter Material: Minimum of 5 ft^3/linear foot of pipe or 4 ft^3/linear foot of pipe when placed in square cut trench.

Alternative In Lieu Of Filter Material: Gravel may be encased in approved filter fabric. Filter fabric shall be mirafi 140 or equivalent. Filter fabric shall be lapped a minimum of 12" on all joints.

Minimum 4" Diameter Pipe: ABS-ASTM D-2751, SDR 35 or ASTM D-1527 schedule 40 PVC-ASTM D-3034, SDR 35 or ASTM D-1785 schedule 40 with a crushing strength of 1,000 pounds minimum, and a minimum of 8 uniformly spaced perforations per foot of pipe installed with perforations at bottom of pipe. Provide cap at upstream end of pipe. Slope at 2% to outlet pipe. Outlet pipe shall be connected to the subdrain pipe with tee or elbow.

Note:
(1) Trench for outlet pipes shall be backfilled with onsite soil.
(2) Backdrains and lateral drains shall be located at the elevation of every bench drain. First drain shall be located at the elevation just above the lower lot grade. Additional drains may be required at the discretion of the soils engineer and/or engineering geologist.

Filter Material -- Shall be of the following specification or an approved equivalent:

Filter Material

Sieve Size	Percent Passing
1"	100
¾"	90-100
3/8"	40-100
No. 4	25-40
No. 8	18-33
No. 30	5-15
No. 50	0-7
No. 200	0-3

Gravel - Shall be of the following specification or an approved equivalent:

Filter Material

Sieve Size	Percent Passing
1½"	100
No. 4	50
No. 200	8

Sand equivalent: Minimum of 50

Note: Figures not to scale

EARTHWORK AND GRADING GUIDELINES
TYPICAL STABILIZATION BUTTRESS SUBDRAIN DETAIL

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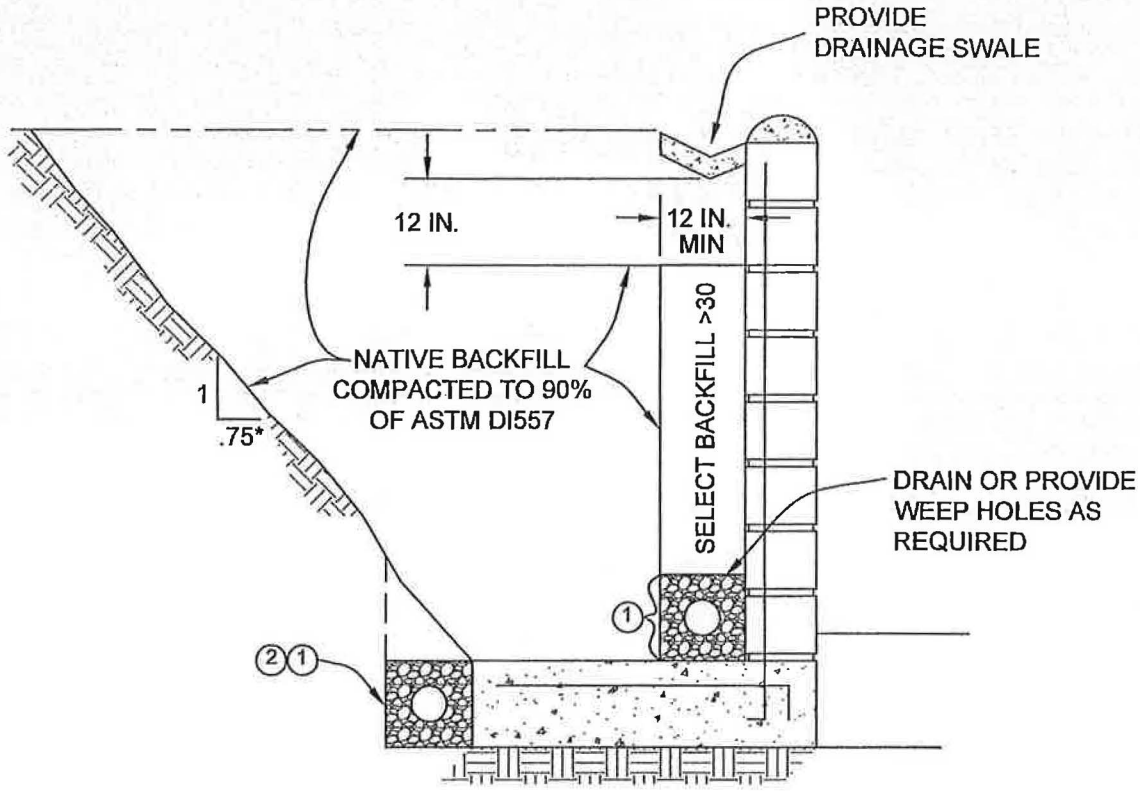
FIGURE O



* OR AS REQUIRED FOR SAFETY

NOTES

1. 4-INCH PERFORATED PVC SCHEDULE 40 OR APPROVED ALTERNATE. PLACE PERFORATION DOWN AND SURROUND WITH A MINIMUM OF 1 CUBIC FOOT PER LINEAL FOOT (1 FT. /FT.) OF 3/4 INCH ROCK OR APPROVED ALTERNATE AND WRAPPED IN FILTER FABRIC.

2. PLACE DRAIN AS SHOWN WHERE MOISTURE MIGRATION THROUGH THE WALL IS UNDESIRABLE.

NOTE: FIGURE NOT TO SCALE

EARTHWORK & GRADING GUIDELINES

TYPICAL RETAINING WALL BACKFILL

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FIGURE P

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Geotechnical & Environmental Solutions

SUPPLEMENTAL GEOTECHNICAL EVALUATION

Proposed Residential Development
NWC Ironwood Avenue and Oliver Street &
Tract No. 31556 Offsite Sewer
Oliver Street Extension/60 Freeway Undercrossing
Moreno Valley, Riverside County, California

May 18, 2015

EEI Project No. GLO-71982.4a

Corporate Office: 2195 Faraday Avenue • Suite K • Carlsbad, California 92008-7207 • Ph: 760-431-3747 Fax: 760-431-3748
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SUPPLEMENTAL GEOTECHNICAL EVALUATION

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Project Site Location:

NWC Ironwood Avenue and Oliver Street &
Oliver Street Extension/60 Freeway Undercrossing
Moreno Valley, Riverside County, California

Prepared by:

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Senior Geotechnical Engineer

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Senior Project Geologist

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2195 Faraday Avenue, Suite K
Carlsbad, California 92008-7207

EEI Project No. GLO-71982.4a

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1.2 Project Description 1
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Distribution: (2) Addressee (one via electronic copy and one hard copy)

1.0 INTRODUCTION

1.1 Purpose

The purpose of this evaluation was two-fold. The first purpose was to provide supplemental geotechnical information to Anderson Consulting Engineers, Inc. ("Client"), regarding the undeveloped subject property at the northwest corner of Ironwood Avenue and Oliver Street (Tract No. 31556) in the City of Moreno Valley, Riverside County, California. The information developed in this Supplemental Geotechnical Evaluation is intended to provide the Client with additional/supplemental subsurface information for portions of the property that were not investigated during EEI's Feasibility-Level Geotechnical Investigation performed in 2014 and also to provide additional/supplemental conclusions and recommendations for the proposed property development.

The second purpose of this Supplemental Geotechnical Evaluation was to provide the Client with geotechnical information regarding the subsurface conditions at the proposed bore and receiving pit locations for the 60 Freeway undercrossing of Tract No. 31556 offsite sewer to determine general subsurface conditions and to provide conclusions/recommendations based on the ease/difficulty of excavations at the pit locations (Site Vicinity Map-**Figure 1**, Aerial Site Map-**Figure 2**).

This Supplemental Geotechnical Evaluation has been conducted in general accordance with the accepted geotechnical engineering principles and in general conformance with the proposed and optional scope of services presented in the approved proposal and cost estimate for the project by EEI, dated March 10, 2015.

EEI conducted an onsite field exploration on April 16 and 17, 2015 that included drilling and sampling of eight (8) hollow stem auger geotechnical borings (borings B-8 through B-15) for the proposed development at the undeveloped subject property. Two (2) additional hollow stem auger geotechnical borings (borings B-16 and B-17) were performed at the proposed bore and receiving pit locations at the 60 freeway undercrossing for Tract No. 31556 offsite sewer, located approximately 3,500 feet south of the undeveloped subject property at the northwest corner of the Ironwood Avenue/Oliver Street intersection. This Supplemental Geotechnical Evaluation has been prepared for the sole use of Anderson Consulting Engineers, Inc. Other parties, without the express written consent of EEI and the Client should not rely upon this Supplementa; Geotechnical Evaluation.

1.2 Project Description

The Client has requested that EEI perform a Supplemental Geotechnical Evaluation of the subject property (Tract No. 31556) consisting of approximately 80-acres of undeveloped vacant land located at the northwest corner of the intersection of Oliver Street and Ironwood Avenue, in the City of Moreno Valley, Riverside County, California. EEI has previously performed a Due Diligence Level Feasibility Geotechnical Evaluation for the property (EEI Project No. GLO-71982.4, Dated November 25, 2014) consisting of the excavation of four (4) small diameter borings (borings B-1 through B-4), 4 (four) backhoe test pits (T-1 through T-4) and three (3) percolation test pits (P-1 through P-3) on the property for the proposed residential development. Nason Street forms the majority of the western boundary; Ironwood borders the south; and Oliver Street forms the eastern boundary. Vacant land is present to the north. In general, the area is characterized by rural residential and vacant land. Proposed development is for 146 multi- or single-family residential lots.

Based on a review of the sewer plans provided by Anderson Consulting Engineers ("Client") and a review of the GoogleEarth© online database, the proposed project consists of the construction of an offsite sewer to service future Tract No. 31556.

The proposed offsite sewer alignment appears to begin at the northwest corner of the intersection of Ironwood Avenue and Oliver Street (which is the southeast corner of Tract No. 31556) and extends south along the existing alignment of Oliver Street which terminates at Carol Place and continues south along the alignment of the proposed future extension of Oliver Street to the 60 Freeway (Moreno Valley Freeway). The proposed offsite sewer would then extend under the 60 Freeway and continue southward along a proposed Eastern Municipal Water District (EMWD) easement to Eucalyptus Avenue where the proposed sewer would be tied-in to the existing sewer. The proposed offsite sewer extends approximately 3,500 feet from the Ironwood Avenue/Oliver Street intersection to Eucalyptus Avenue, including the 60 Freeway undercrossing. Based on a review of the sewer plans, it appears that the proposed offsite sewer will be 12-inches in diameter and composed of either PVC plastic or vitrified clay pipe (VCP).

For the proposed undercrossing of the 60 Freeway by the offsite sewer alignment, the bore and jack method will be utilized for the construction. Along the proposed sewer alignment immediately north of the 60 Freeway, a proposed 40-by 20-foot boring pit will be excavated to a depth of approximately 15 feet in order to accommodate horizontal drilling equipment. An approximately 15- by 20-foot receiving pit will be excavated to a depth of approximately 20 feet immediately adjacent to the south side of the 60 Freeway for the removal of drilling spoils. Once the pits are excavated and the drilling equipment is in place, a near horizontal boring will be drilled between the boring and receiving pits underneath the 60 Freeway and then shored with approximately 161 feet of 30-inch diameter steel casing. 189 feet of sewer pipe will then be placed through the steel casing and tied into the pipe sections on either side of the 60 Freeway. It appears that the depth from the surface to the top of the steel casing will vary from approximately 11 to 15 feet below existing grade. The drilling equipment will then be removed after pipe installation and the pits backfilled to complete the operation. The Client has requested that EEI conduct a Preliminary Geotechnical Evaluation of the subsurface conditions at the locations of the boring pit and receiving pit locations to determine the general subsurface conditions and the ease/difficulty of excavation at the pit locations.

1.3 Scope of Services

- Research of aerial photographs, readily available literature regarding local and regional geologic conditions, groundwater depth, fault rupture hazard zones and known fault locations, and other reasonably available geotechnical data or subsurface information relevant to the subject project(s).

- Conduct a geotechnical reconnaissance/site visit of the subject property(s) and nearby vicinity to mark boring locations for Underground Service Alert and to verify drilling rig access.

- Coordinate with Underground Service Alert to identify the presence of underground utilities for clearance of proposed boring and test pit locations.

- The drilling and logging of eight (8) small diameter exploratory borings (B-8 through B-15) in readily accessible areas of the subject property (Tract No. 31556) not previously investigated by EEI, ranging in depth from 15.5 to 42 feet below existing grade elevations (bgs).

- The drilling and logging of two (2) small diameter exploratory borings (B-16 and B-17) adjacent to each side of the proposed 60 Freeway undercrossing at the proposed bore and receiving pit locations for the Tract No. 31556 offsite sewer. Both borings were advanced to a depth of 21.5 feet below the existing grade elevations (bgs).

- The locations of the exploratory borings on the subject property (Tract No. 31556) are presented on **Figure 3** (Field Exploration Plan).

- The locations of each of the offsite exploratory borings (for the proposed offsite sewer boring and receiving pits) are presented on **Figure 4** (Offiste Sewer Boring Locations).
- An evaluation of seismicity and geologic hazards to include an evaluation of faulting.
- Completion of laboratory testing of representative earth materials encountered onsite to ascertain their pertinent soils engineering properties, including corrosion potential and excavation characteristics (**Appendix B**).
- The preparation of this report which presents our preliminary findings, conclusions, and recommendations for both the proposed Tract No. 31556 residential development and offsite sewer bore and receiving pit locations for the 60 Freeway undercrossing.

2.0 BACKGROUND

2.1 Subject Property Description

Based on the information provided by Anderson Consulting Engineers, Inc. ("Client"), and a review of the GoogleEarth© online database, and EEI's previous due diligence geotechnical evaluation, the subject property consists of approximately 80-acres of undeveloped vacant land (Tract No. 31556) located at the northwest corner of the intersection of Oliver Street and Ironwood Avenue, in the City of Moreno Valley, Riverside County, California. Nason Street forms the majority of the western boundary; Ironwood borders the south; and Oliver Street forms the eastern boundary. Vacant land is present to the north. In general, the area is characterized by rural residential and vacant land. Proposed development is for multi or single-family residential.

We understand that the proposed offsite sewer extends approximately 3,500 feet from the Ironwood Avenue/Oliver Street intersection (Tract No. 31556) to Eucalyptus Avenue, including the 60 Freeway undercrossing. Based on a review of the sewer plans, it appears that the proposed offsite sewer will be 12-inches in diameter and composed of either PVC plastic or vitrified clay pipe (VCP).

The subject property (Tract 31556) is approximately situated at 33.9448° north latitude and 117.1871° west longitude (GoogleEarth®, 2013).

2.2 Site Topography

A review of the Sunnymead, California 7.5-minute topographic quadrangle (USGS, 2012) and Project Exhibit/topographic map prepared by Anderson Consulting Engineers, Inc. indicates that the subject property (Tract No. 31556) elevation varies from approximately 1,840 to 1,980 feet above mean sea level (amsl). From east-west across the property is a series of north-south oriented ridges and alternating drainage gullies in the lower, southern portion of the property. The intervening ridges are generally about 5 to 10 feet higher in elevation than the adjacent drainage gullies. Rounded granitic outcrops are exposed in the northwestern and northeastern sections of the property. The property topography can be generally described as a relatively well-dissected alluvial fan descending from the eroded hills to the north. The overall surface gradient across the property is gently to moderately south or south-southeast.

2.3 Geologic Setting

The subject property and vicinity lies within the Peninsular Ranges Geomorphic Province of California (CGS, 2002). The Peninsular Ranges Geomorphic Province extends from the Transverse Ranges Geomorphic Province and the Los Angeles Basin, south to Baja California.

This province varies in width from about 30- to 100-miles. It is bounded on the west by the Pacific Ocean, on the south by the Gulf of California and on the east by the Colorado Desert Province. The Transverse Ranges Geomorphic Province bounds the Peninsular Ranges on the north. The Peninsular Ranges are essentially a series of northwest-southeast oriented fault blocks. Major fault zones and subordinate fault zones found in the Peninsular Ranges Province typically trend in a northwest-southeast direction.

Regional geologic maps of the subject property and vicinity (Morton et al., 2004) indicate the property is underlain by Cretaceous-age plutonic rocks composed of tonalite and Holocene age Younger Alluvial Fan deposits. Outcroppings of the weathered tonalite bedrock are exposed in the northwestern and northeastern portions of the property. Over the remainder of the property, the tonalite bedrock was found to be weathering into a soil with a "decomposed granite" or "dg" texture at depth in the exploratory borings in general is covered with several feet of alluvial and colluvial (younger alluvial fan) soils also derived from the weathered tonalite. The alluvial and colluvial soils are generally comprised of relatively loose to dense silty sand. The property is relatively undeveloped and artificial fill was not encountered during our field exploration at the property.

At the offsite sewer bore and receiving pit locations, several feet of artificial fill soils, likely associated with the construction of the 60 Freeway, were encountered at the surface and were underlain by alluvial soils to the maximum explored depth during our geotechnical evaluation.

Due to the proximity of the subject property area to several nearby active faults, strong ground shaking could occur at the property as a result of an earthquake on any one of the faults. Our review indicates that there are no known active faults crossing the property and the property is not within a State of California Earthquake Fault Zone (Jennings, 1994; Hart and Bryant, 1997, CDMG, 1974; 1998). Due to the presence of shallow bedrock and the lack of shallow groundwater at the property, the property is considered as having a low susceptibility to liquefaction.

2.4 Regional Groundwater

A seismic hazard zone map and report have not been completed for the Sunnymead Quadrangle, therefore, the depth to the historic high groundwater at the subject property is not known. Due to the presence of relatively shallow granitic bedrock at the property, static groundwater is not expected and groundwater was not encountered during our previous due diligence geotechnical evaluation or during our Supplementa; Geotechnical Evaluation to a maximum explored depth of 42 feet below the existing ground surface. Within the lower drainage gullies, up to 30 feet in thickness of silty sand alluvium was encountered. Although not encountered within the alluvium during our field excavation, during times of heavy precipitation or runoff, a localized perched groundwater condition could exist. A review of topographic maps of the general vicinity of the property indicates regional topographic relief slopes gently towards the south or south-southeast. This information suggests that regional groundwater in the property vicinity could be inferred to flow in the same general topographic direction.

3.0 FAULTING AND SEISMICITY

The portion of Southern California that includes the subject property is considered to be seismically active. Due to the proximity of the property area to several nearby active faults, strong ground shaking could occur at the property as a result of an earthquake on any one of the faults. Our review indicates that there are no known active faults crossing the property (Blake, 2000) and the property is not within an Earthquake Fault Zone (Hart and Bryant, 1997). It is our opinion, therefore, that the likelihood of surface fault rupture at the property is low. **Table 1** lists the major active faults within 25 miles that are likely to affect the property.

TABLE 1
Summary of Major Active Faults

	Fault Name	Approximate Distance From Site miles (kilometers)	Maximum Moment Magnitude
1	San Jacinto-San Jacinto Valley	1.5 (2.4)	6.9
2	San Jacinto-San Bernardino	5.8 (9.3)	6.7
3	San Andreas – San Bernardino M-1	12.1 (19.5)	7.5
4	San Andreas – SB-Coach. M-2b	12.1 (19.5)	7.7
5	San Andreas – SB-Coach. M-1b-2	12.1 (19.5)	7.7
6	San Andreas – Whole M-1a	12.1 (19.5)	8.0
7	North Frontal Fault Zone (West)	19.4 (31.2)	7.2
8	San Jacinto – Anza	21.0 (33.8)	7.2
9	Elsinore (Glen Ivy)	21.7 (35.0)	6.8
10	Cucamonga	21.9 (35.2)	6.9
11	Elsinore (Temecula)	22.8 (36.7)	6.8
12	Cleghorn	23.1 (37.2)	6.5
13	Chino-Central Ave. (Elsinore)	23.3 (37.5)	6.7

3.1 Seismic Parameters and Peak Ground Acceleration

Maximum considered ground motion maps provided in the California Building Code (CBC, 2013) were utilized with coordinates of 33.9448° north latitude and 117.1871° west longitude, to determine the subject property seismic parameters. EEI utilized seismic design criteria provided in the CBC (2013).

In accordance with the guidelines of the CBC (2013), the spectral parameters for the subject property (based on a Site Class B soil) are estimated to be $S_s = 2.166g$ and $S_1 = 0.982g$. Review of the geotechnical data obtained during our subsurface exploration, however, indicates that the property should be classified as Class D per the CBC (Table 1613.5.2). Consequently, Site Coefficients $F_a = 1.000$ and $F_v = 1.500$ appear to be appropriate for the property. Based on this information, the adjusted maximum considered earthquake spectral response parameters $S_{MS} = 2.166g$ and $S_{M1} = 1.472g$ are recommended for seismic design of the project. Assuming an occupancy category of II (Table 1604A.5), an S_{DS} value of 1.444g and an S_{D1} value of 0.982g, the proposed building at the property can be assigned a seismic design category of D [Table 1613.5.6 (1) and (2)]. Final selection of the appropriate seismic design coefficients should be made by the structural consultant based on the local laws and ordinances, expected building response, and desired level of conservatism.

Seismic Hazard Response Parameters are listed in **Table 2**.

TABLE 2
Seismic Hazard Response Parameters and Design Parameters CBC (2013)

Latitude: 33.9448° - Longitude: -117.1871° Seismic Parameter	Period (Sec)		Value
Mapped Spectral Acceleration Value, Soil Class B	0.2	S_s	2.166g
Mapped Spectral Acceleration Value, Soil Class B	1.0	S_1	0.982g
Site Coefficient, Subject Site Soil Classification D per 2013 CBC Table 1613.5.2	--	F_a	1.000
Site Coefficient, Subject Site Soil Classification D per 2013 CBC Table 1613.5.2	--	F_v	1.500
Adjusted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Site Class D	0.2	S_{MS}	2.166g
Adjusted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Site Class D	1.0	S_{M1}	1.472g
Design Spectral Response Acceleration Occupancy Category II per 2013 CBC Table1604.5	0.2	S_{DS}	1.444g
Design Spectral Response Acceleration Occupancy Category II per 2013 CBC Table1604.5	1.0	S_{D1}	0.982g
Peak Ground Acceleration Adjusted For Site Class Effects.		PGA_M	0.837g
Building Assigned Seismic Design Category per Table 1613.5.6 (1) and (2)	--	--	D

3.2 Ground Lurching or Shallow Ground Rupture

Based on the geography, topography and site-specific geotechnical conditions encountered during our preliminary and Supplementa; Geotechnical Evaluations at the subject property, we consider the potential for ground lurching or shallow ground rupture at the property to be low; however, due to the active seismicity of California, this possibility cannot be completely ruled out. In light of this, the unlikely hazard of lurching or ground-rupture should not preclude consideration of "flexible" design for onsite utility lines and connections.

3.3 Liquefaction

Liquefaction is a phenomenon in which the strength and stiffness of a soil is reduced by earthquake shaking or other rapid loading. Liquefaction and related phenomena have been responsible for substantial structural damage in historical earthquakes, and are a design concern under certain conditions. Liquefaction occurs in saturated soils that are soils in which the space between individual particles is completely filled with water. This pore water exerts a pressure on the soil particles that influences how tightly the particles themselves are pressed together.

Prior to an earthquake, pore water pressure is typically low; however, earthquake motion can cause the pore water pressure to increase to the point where the soil particles can readily move with respect to each other. When liquefaction occurs; the strength of the soil decreases and the ability of a soil deposit to support structural loads are reduced.

A seismic hazard zone map and report for the Sunnymead Quadrangle has not been issued by the California Geological Survey (CGS) so the subject property is, therefore, not situated within a mapped Liquefaction Zone. The lower elevation portions of the property is generally underlain by generally loose to medium dense alluvial and colluvial deposits that overlie relatively shallow granitic bedrock while the

upper elevation portions of the site are generally underlain by dense to very dense alluvial and colluvial deposits over granitic bedrock. The alluvial and colluvial soils are subject to partial and/or complete removal and recompaction during site grading for the proposed residential development. It appears that liquefaction is not a significant geotechnical concern at the property.

Cyclic mobility is a liquefaction phenomenon, triggered by cyclic loading, occurring in soil deposits with static shear stresses lower than the soil strength. Deformations due to cyclic mobility develop incrementally because of static and dynamic stresses that exist during an earthquake. Lateral spreading, a common result of cyclic mobility, can occur on gently sloping and on flat ground close to rivers and lakes. These conditions do exist within the subject property, however, based on the conceptual site plan, the property should be relatively level following rough grading with a lack of free channel faces and cyclic mobility should not be an issue post-grading.

3.4 Seismic Induced Settlement

Seismically induced settlement can occur due to reorientation of soil particles during strong shaking of unsaturated sands, as well as in response to liquefaction of saturated loose granular soils.

Based on our evaluation and the geotechnical data obtained from our exploratory borings, we estimate the total seismic-induced settlement to be less than 1-inch. Differential earthquake induced settlements are estimated to be less than 0.5-inches over a 50-foot span.

4.0 FIELD EXPLORATION AND LABORATORY TESTING

4.1 Field Exploration

Fieldwork for our geotechnical evaluation was conducted on April 16 and 17, 2015. The drilling and logging of eight (8) small diameter exploratory borings (B-8 through B-15) in readily accessible areas of the subject property (Tract No. 31556) not previously investigated by EEI, ranging in depth from 15.5 to 42 feet below existing grade elevations (bgs). Additionally, drilling and logging of two (2) small diameter exploratory borings (B-16 and B-17) was completed adjacent to each side of the proposed 60 Freeway undercrossing at the proposed bore and receiving pit locations for Tract No. 31556 offsite sewer. Both borings were advanced to a depth of 21.5 feet below the existing grade elevations (bgs). All exploratory borings were logged under the supervision of a Registered Professional Engineer and/or Certified Engineering Geologist at EEI. Boring locations were adjusted as necessary due to existing utilities and improvements.

Blow count (N) values were determined utilizing a 140-pound automatic hammer, falling 30-inches onto a Standard Penetration Test (SPT) split-spoon sampler and a Modified California split-tube sampler. A truck mounted hollow-stem auger (HSA) drill rig was used during fieldwork. The blows per foot (N value) required to advance the 18-inch long SPT and 12-inch long Modified California split-tube samplers was measured at various initial depths followed by 5-foot intervals, recorded on the boring logs. The boring logs for the field exploration are presented in **Appendix A**-Soil Classification Chart and Boring & Trench Logs. Relatively "undisturbed" samples were collected in a 2.42-inch (inside diameter) California Modified split-tube sampler for visual examination and laboratory testing in the exploratory borings. The soils were classified in accordance with the Unified Soil Classification System (ASTM, 2008). Representative bulk samples were also collected from the exploratory borings for appropriate laboratory testing.

4.2 Subsurface Conditions

The results of our geotechnical exploration indicate that the proposed residential development is underlain by weathered Cretaceous-age plutonic rocks composed of tonalite. This material was observed to extend beyond the maximum depth of our exploratory borings (approximately 42 feet below existing grades). Alluvial soils up to 30 feet thick were observed to mantle the weathered tonalite bedrock within the lower lying channel/drainage areas during our previous geotechnical site evaluation and were not penetrated during our maximum explored depth during our supplemental evaluation.

On the higher, elevated ridge areas on the subject property, colluvial soils were observed to mantle the weathered tonalite bedrock with a thickness varying between 2 and 21.5 feet (similar to the 3 to 14 feet encountered during due diligence evaluation). The weathered tonalite bedrock can generally be described as gray, white or black speckled or orange to dark grayish-orange (depending on degree of weathering) with a "granitic" or phaneritic texture and was generally unweathered to highly weathered. Outcroppings of the weathered tonalite bedrock are exposed in the northwestern and northeastern portions of the site. Over the remainder of the property, the tonalite bedrock was found to be weathering into a medium dense to very dense silty sand soil with a "decomposed granite" or "dg" texture at depth in the exploratory borings and test pits. The alluvial and colluvial (younger alluvial fan) soils are also derived from the weathered tonalite.

The alluvial and colluvial soils are generally comprised of orange-brown mottled or light brown, fine to coarse, damp to moist, loose to dense silty sand and gravelly sand. The property is relatively undeveloped and artificial fill was not encountered during our field exploration at the property.

Our exploratory excavations were performed utilizing light-duty equipment, which can provide general excavation characteristics of the onsite materials. Large granitic (tonalite) bedrock outcrops are present on the northeast and northwest portions of the property, along with some isolated rock outcrops within the areas of the proposed development area and generally on the higher elevations of the property. Boulders were present at the surface in these areas, some localized "core rock or floaters" should be anticipated at variable depths in these areas. Based on observed subsurface conditions in the exploratory borings, the "decomposed granite" or "dg" is moderately to highly weathered and/or fractured and was relatively easy to excavate to the depths indicated with a drill rig equipped with flight auger equipment. Refusal was encountered at a depth of 42 feet bgs in exploratory boring B-8 during our field exploration.

In general, the ease of rock excavation or rippability depends on various factors such as rock type, rock hardness and density, the amount of weathering, and the existence and characteristics of discontinuities such as joint spacing, foliation, or fractures.

Due to the relatively dense character of the granitic bedrock encountered onsite, it is likely that oversized rock materials will be created during grading operations. Native earth materials appear to be suitable for use as structural fill provided they are moisture conditioned (as needed), meet EEI's recommendations for size and are properly compacted. Dependent upon the grading plan, some of the oversized materials may be re-used in landscape areas.

For the proposed offsite sewer bore pit and receiving pit locations, alluvial soils at depth were overlain by artificial fill soils from the surface to a depth of approximately 11 feet bgs in exploratory boring B-16 and a depth of approximately 16 feet bgs in exploratory boring B-17. Alluvial soils were then encountered below the artificial fill to the maximum explored depth of 21.5 feet bgs. The offsite alluvial soils can

generally be described as light brown to brown, fine to medium, moist, medium dense silty sand. The artificial fill soils encountered can be described as brown, red-brown or dark orange-brown, fine to coarse, damp to moist, loose to medium dense silty sand and sandy silt. Refusal was not encountered within either of our offsite exploratory borings. Detailed descriptions of the encountered soils are provided on the boring logs and test pit logs included as **Appendix A**.

Due to the presence of relatively shallow granitic bedrock at subject property, static groundwater is not expected and groundwater was not encountered in any of the exploratory borings excavated at the property to a maximum explored depth of 42 feet below the existing ground surface. Our review of ground water monitoring data from nearby wells suggests that the groundwater level may fluctuate seasonally and yearly.

It should be noted that fluctuations in the ground water level could also occur due to variations in ground surface topography, subsurface stratifications, precipitation, irrigation, and other factors which may not have been evident at the time of our exploration.

4.3 Laboratory Testing and Classification

Representative samples were selected for laboratory testing to confirm their field classification(s). Field descriptions and classifications were visually classified according to the American Society for Testing and Materials (ASTM) D2488 which classifies soils under the Unified Soil Classification System (USCS). Representative soil samples were tested in the lab for grain size distribution, liquid limits, and plastic limits to determine actual classifications by ASTM D2487-Standard Practice for Classification of Soils for Engineering Purposes in accordance to the USCS. Final classifications of soils can be found on the boring logs in **Appendix A** and the laboratory test data in **Appendix B**.

4.3.1 Moisture Content and Dry Density

The in-situ moisture content and dry density of soils were determined for soil samples obtained from the borings. Moisture contents and dry densities of soils help to determine engineering design parameters for foundations, retaining walls, and other engineered structures. Moisture content on soil samples was conducted in general accordance with ASTM D2216, and was recorded as a percentage. In-situ moisture content and dry density information for soil samples retrieved from the field can be found on the boring logs located in **Appendix A**.

4.3.2 Grain Size Distribution

To help check field classifications of soils, the grain size distribution of representative soil samples was determined. In order to find the percentages of different sized particles in a particular soil stratum, soils were tested in general accordance with ASTM D422-Standard Test Method for Particle-Size Analysis of Soils. Grain size distribution curves and gradation results are presented in **Appendix B.**

4.3.3 Maximum Dry Density and Optimum Moisture Content

The maximum dry density and optimum moisture content were determined from a bulk sample obtained from boring B-8 at depths between 0 and 5 feet below existing grade. Our testing was performed in general accordance with ASTM D1557, Method A. Results of our testing are presented in **Appendix B**.

4.3.4 Expansion Index

A soil sample from boring B-8 within the upper 5 feet of existing grade was tested for its expansion potential. Our expansion index testing was conducted in general accordance with ASTM D4829. The results of our expansion index testing are presented in **Appendix B.**

4.3.5 Sulfate/Corrosion

One representative sample of onsite earth material was collected for analysis at Clarkson Laboratory and Supply, Inc. located in Chula Vista, California for corrosion/soluble sulfate potential. This corrosion testing included soil minimum resistivity and pH by California Test 643, sulfate by California Test 417, and chloride by California Test 422. Results of these tests are presented in **Appendix B**.

It should be understood that the results provided in **Appendix B** are based upon pre-development conditions. Verification testing is recommended at the conclusion of grading on samples collected at or near finish grade.

5.0 CONCLUSIONS

Based on our field exploration, laboratory testing and engineering and geologic analysis, it is our opinion that the subject property is suitable for the proposed new development and associated improvements from a geotechnical engineering and geologic viewpoint; however, there are existing geotechnical conditions associated with the property that will warrant mitigation and/or consideration during planning stages. If site plans and/or the location of the proposed residential buildings or proposed offsite sewer bore and receiving pit locations are revised, additional field studies may be warranted to address proposed site-specific conditions.

In general, based on the results of EEI's supplemental investigation, conclusions are generally unchanged from those presented in our previous due diligence geotechnical report, dated November 25, 2014. As a result, EEI is providing the following conclusions:

- A total of eight (8) hollow stem auger borings were drilled on the subject property (Tract No. 31556). Boring depths ranged from 15.5 to 42 feet below the existing ground surface. Two (2) additional hollow stem auger borings were drilled offsite in the area of the proposed Tract No. 31556 offsite sewer line undercrossing of Freeway 60. Both of these borings were drilled to an approximate depth of 21.5 feet below existing grade. The results of our geotechnical exploration indicate that the subject property development is underlain by weathered Cretaceous-age plutonic rocks composed of tonalite. This material was observed to extend beyond the maximum depth of our exploratory borings (approximately 42 feet below existing grades). Alluvial soils up to 30 feet thick were observed to mantle the weathered tonalite bedrock within the lower lying channel/drainage areas during our previous geotechnical site evaluation and were not penetrated during our maximum explored depth during our supplemental evaluation.

- On the higher, elevated ridge areas on the subject property, colluvial soils were observed to mantle the weathered tonalite bedrock with a thickness varying between 2 and 21.5 feet (similar to the 3 to 14 feet encountered during due diligence evaluation). The weathered tonalite bedrock can generally be described as gray, white or black speckled or orange to dark grayish-orange (depending on degree of weathering) with a "granitic" or phaneritic texture and was generally

unweathered to highly weathered. Outcroppings of the weathered tonalite bedrock are exposed in the northwestern and northeastern portions of the property. Over the remainder of the property, the tonalite bedrock was found to be weathering into a medium dense to very dense silty sand soil with a "decomposed granite" or "dg" texture at depth in the exploratory borings and test pits. The alluvial and colluvial (younger alluvial fan) soils are also derived from the weathered tonalite. The alluvial and colluvial soils are generally comprised of orange-brown mottled or light brown, fine to coarse, damp to moist, loose to dense silty sand and gravelly sand. Refusal was encountered at a depth of 42 feet bgs in exploratory boring B-8 during our field exploration.

- For the proposed offsite sewer bore pit and receiving pit locations, alluvial soils at depth were overlain by artificial fill soils from the surface to a depth of approximately 11 feet bgs in exploratory boring B-16 and a depth of approximately 16 feet bgs in exploratory boring B-17. Alluvial soils were then encountered below the artificial fill to the maximum explored depth of 21.5 feet bgs. The offsite alluvial soils can generally be described as light brown to brown, fine to medium, moist, medium dense silty sand. The artificial fill soils encountered can be described as brown, red-brown or dark orange-brown, fine to coarse, damp to moist, loose to medium dense silty sand and sandy silt. Refusal was not encountered within either of our offsite exploratory borings.

- Groundwater was not encountered in any of our onsite or offsite exploratory borings to the depths explored (approximately 42 feet below existing grades). It should be noted that variations in groundwater may result from fluctuations in the ground surface topography, subsurface stratification, precipitation, irrigation and other factors that may not have been evident at the time of our subsurface exploration.

- Laboratory test results indicate that the near surface materials are mildly alkaline (pH = 7.4) and are corrosive to ferrous metals with a minimum resistivity value of 3,500 ohm-cm. Laboratory testing of the upper soils yielded a soluble sulfate concentration of 0.003 percent and a chloride concentration of 0.001 percent, indicating a negligible corrosion potential to reinforced concrete.

- The results of our laboratory Expansion Index (EI) testing indicate an Expansion Index of 2, for the tested soils which represents a very low expansion potential.

- The subject property is located within an area of Southern California recognized as having a number of active and potentially active faults. Our review indicates that there are no known active faults crossing the property and the property is not located within an Earthquake Fault Zone. The nearest active faults that could affect the property are the San Jacinto Valley segment of the San Jacinto Fault Zone, located approximately 1.5 miles from the property, the San Bernardino segment of the San Jacinto Fault Zone, located approximately 5.8 miles from the area of study, the San Bernardino M-1 segment of the San Andreas Fault Zone, the San Bernardino-Coachella Valley M-2b segment of the San Andreas Fault Zone, the San Bernardino-Coachella Valley M-1b-2 segment of the San Andreas Fault Zone and the San Bernardino-Coachella Valley M-1a segment of the San Andreas Fault Zone, all located approximately 12.1 miles from the area of study. Each of these active faults is capable of generating severe ground shaking at the property. A list of active faults within an approximate 25-mile radius is presented in **Table 1**.

- Based on EEI's evaluation, earth materials underlying the subject property are not considered susceptible to liquefaction or significant amounts of seismic settlement. Based on EEI's evaluation, the earth materials underlying the property of the proposed development appear to be susceptible to some seismically induced settlement on the order of less than one-inch with

differential settlements of less than 0.5-inches over a 50-foot span. Liquefaction-induced lateral spreading does not appear to be a concern at the property due to the lack of shallow groundwater, the lack of nearby open face channels and the relatively shallow depth to bedrock.

- At this time we cannot present specific footing recommendations that can be incorporated in the structural design, given that we do not have a scope of the proposed project, no grading or foundation plans were available at the time and no information was provided to us other than the Client is considering purchasing the property for future development.

6.0 RECOMMENDATIONS

The recommendations presented herein should be considered as preliminary for the purpose of characterizing the geotechnical and geologic conditions at the subject property prior to purchasing the property, and for preliminary information to aid the initial planning and design phases of development. Guidelines for site preparation, earthwork, and onsite improvements were previously provided in the Due Diligence Level Geotechnical Evaluation report by EEI dated November 25, 2014. Based on the results of our Supplemental Geotechnical Evaluation of the subject property, the following recommendations as presented are essentially unchanged from the 2014 EEI report, except where specifically discussed. The subsurface conditions encountered at the additional boring locations verified the subsurface conditions encountered in the 2014 exploratory borings and test pits and therefore, has verified the recommendations and conclusions of the earlier referenced EEI report, except where specifically discussed due to a change in subsurface conditions from the previous EEI report.

Our assumption is also that the onsite development will still consist of single-family, wood-frame, slab-on-grade 1- to 2-story residential structures and the planned offsite construction for the proposed sewer line including the bore-and-jack method for the 60 Freeway undercrossing of the sewer line.

6.1 General

Grading should conform to the guidelines presented in the 2013 California Building Code (CBC, 2013), as well as the requirements of the City of Moreno Valley and the County of Riverside. Additionally, general Earthwork and Grading Guidelines are provided herein as **Appendix C**.

During earthwork construction, removals and reprocessing of fill materials, as well as general grading procedures of the contractor should be observed and the fill placed selectively tested by representatives of the geotechnical engineer, EEI. If any unusual or unexpected conditions are exposed in the field, they should be reviewed by the Geotechnical Engineer and if warranted, modified and/or additional remedial recommendations will be offered. Specific guidelines and comments pertinent to the planned development are provided herein.

The recommendations presented herein have been completed using the preliminary information provided to us regarding site development. If information concerning the proposed development is revised, or any changes in the design and location of the proposed property improvements are made, the preliminary conclusions and recommendations contained in this report should not be considered applicable unless the changes are reviewed and conclusions of this report modified or approved in writing by this office.

6.2 Site Preparation and Grading

Debris and other deleterious material, such as organic soils and/or environmentally impacted earth materials should be removed from the subject property prior to the start of grading. Areas to receive fill should be properly benched in accordance with current industry standards of practice and guidelines specified in the CBC and the requirements of the local jurisdiction.

Existing utilities should be removed within the proposed building envelope. Abandoned trenches should be properly backfilled and tested. If unanticipated subsurface improvements (utility lines, septic systems, wells, utilities, etc.) are encountered during earthwork construction, the geotechnical engineer should be informed and appropriate remedial recommendations would then be provided.

6.3 Remedial Earthwork

The encountered portions of the existing surficial soils including the upper portions of the alluvial and colluvial soils were observed to be somewhat loose and variable in moisture content and relative density. As such, they are considered potentially compressible and unsuitable for the support of settlement-sensitive structures or engineered fill in their current condition. Therefore, where not already removed by the proposed site grading, the existing native materials should be completely removed and recompacted in the areas to receive the proposed building improvement and other settlement-sensitive improvements within some of the lower elevation portions of the property. Based on the results of our subsurface exploration, we recommend that these removals extend to approximate depths on the order of a minimum of 4 feet to a maximum of 30 feet below the existing ground surface, or 24-inches below the bottoms of the proposed foundations. At the approximate location of boring B-1, 30 feet of relatively loose to medium dense silty sand alluvium was encountered above the tonalite bedrock and should be removed to a depth of 30 feet. A similar situation likely exists in other, lower elevation portions of the property within the drainage channels. In the upper elevation portions of the property, alluvial and colluvial soils tend to be dense to very dense and relatively well cemented, will likely require a minimum depth of removal and recompaction, and will generally not require complete removal and recompaction to the granitic bedrock. Once grading plans become available to EEI or review, the estimated removal depth of the native materials at the property can be approximately determined and provided.

Following removal of the upper soils, the bottom of the resulting excavation(s) should be observed by a representative of EEI to check that unsuitable materials have been sufficiently removed. It should be understood that based on the observations of our field representative, localized deeper removals may be recommended. The base of the removal area should be level to avoid differential fill thicknesses under proposed improvements. This remedial earthwork should extend at least 5 feet outside the proposed building limits and/or 5 feet beyond the area to receive fill. Note that vertical sides exceeding 5 feet in depth may be prone to sloughing and may require laying back to an inclination of 1:1 (horizontal to vertical).

After removal of the upper soils and observation of the excavation bottoms, the over-excavated areas should be scarified to a minimum depth of 6-inches, moisture conditioned as needed to achieve at least optimum moisture content and re-compacted to at least 90 percent of the maximum dry density (based on ASTM D1557). The over-excavated areas should then be backfilled with onsite and/or imported soils that are placed and compacted as recommended herein until design finish grades are reached.

6.4 Fill Placement

Fill material should possess a very low expansion potential (expansion index of less than 21 as determined by ASTM D4829), be free of organic matter (less than 3 percent organics by weight) and other deleterious material. Much of the onsite materials appear to be suitable for re-use as fill, provided they do not contain rocks greater than 6-inches in maximum dimension, organic debris and other deleterious materials. Rock fragments exceeding 6-inches in one dimension should be segregated and exported from the subject property, or utilized for landscaping.

If import soils are needed, the earthwork contractor should ensure that all proposed fill materials are approved by the geotechnical engineer prior to use. Representative soil samples should be made available for testing at least ten working days prior to hauling to the subject property to allow for laboratory tests.

Fill materials should be placed in 6- to 8-inch loose lifts, moisture conditioned as necessary to at least optimum moisture and compacted to a minimum of 90 percent maximum dry density according to ASTM D1557. The upper 12-inches of pavement subgrade should be moisture conditioned to at least optimum moisture and compacted to at least 95 percent of the maximum dry density as determined by ASTM D1557. Suitable heavy grading equipment should be utilized to properly mix, spread, moisture condition or dry, and compact each fill lift.

Earthwork may be affected by the existing soil moisture content exceeding optimum. Moist to very moist earth materials may be difficult to mix and compact in their native condition, and drying or mixing with drier soils may be warranted to achieve the recommended relative compaction.

Those areas to receive fill (including over-excavated areas) or surface improvements should be scarified at least 12-inches, moisture conditioned to at least optimum moisture content and recompacted to at least 90 percent of the maximum dry density (based on ASTM D1557).

6.5 Yielding Subgrade Conditions

The soils encountered at the subject property can often exhibit "pumping" or yielding once they become saturated. This can often occur in response to periods of significant precipitation, such as during the winter rainy season. If this occurs and in order to help stabilize the yielding subgrade soils within the bottom of the removal areas, the contractor can consider as an option, the placement of uniform sized, ¾- to 2-inch crushed rock within areas exhibiting the "pumping" conditions. The crushed rock should be properly tracked into the underlying soils such that it is adequately intruded into and interlocks with the soils. We expect that a 6- to 12-inch thick section of the crushed rock will be required.

Following the placement and tracking of the gravel layer into the underlying "pumping" soils, it is recommended that Mirafi 600X stabilization fabric (or approved equivalent) then be placed upon the gravel layer. Fill soils, which should be placed and compacted in accordance with the recommendations presented herein, should then be placed upon the fabric until design finish grades are reached. The gravel and stabilization fabric should extend at least 5 feet laterally beyond the limits of the "pumping" areas. These operations should be performed under the observation and testing of a representative of EEI in order to evaluate the effectiveness of these measures and to provide additional recommendations for mitigative measures, as warranted.

6.6 Shrinkage and Bulking

Several factors will affect earthwork balancing on the subject property, including shrinkage, bulking, subsidence, trench spoils from utilities and footing excavations, and final pavement section thickness as well as the accuracy of topography.

Shrinkage, bulking and subsidence are primarily dependent upon the degree of compactive effort achieved during construction. For planning purposes, the shrinkage factor is estimated to be about 10 to 15 percent for the onsite natural soils to be utilized as fill. This shrinkage factor may vary with methods employed by the contractor. Subsidence is estimated to be about 0.1 feet. For preliminary planning purposes, bulking of the granitic bedrock derived materials is estimated to be 0 to 10 percent. Losses from site clearing and removal of existing site improvements as well as generation of oversize material may affect earthwork quantity calculation and should be considered.

The previous estimates are intended as an aid for the project engineers in estimating earthwork quantities. It is recommended that the site development be planned to include an area that could be raised or lowered to accommodate final site balancing.

Several factors will affect earthwork balancing on the subject property, including shrinkage, bulking, subsidence, trench spoils from utilities and footing excavations, and final pavement section thickness as well as the accuracy of topography

7.0 PRELIMNARY FOUNDATION RECOMMENDATIONS

7.1 General

The conclusions and recommendations presented herein are based on the assumption that the planned development will consist of two- to four-story wood frame residential structures with slab-on-grade. It is our understanding that these conceptual plans are also part of the due diligence phase of the project and may or may not be the final design. As such, the conclusions and recommendations contained in this Supplemental Geotechnical Evaluation report should be considered preliminary, and should be reviewed, revised and/or approved in writing by EEI at the time the project design is finalized.

Be advised that as part of the foundation design election process, there is always a cost/benefit evaluation. Although we are providing alternatives for foundation design, we have not accomplished the cost/benefit evaluation.

7.2 Preliminary Foundation Design

Lightly loaded wood-frame, two- to four-story residential buildings with a slab-on-grade, can be supported on conventional continuous or isolated spread footings bearing upon at least 24-inches of properly compacted fill materials.

In preparation for foundation construction, the earthwork contractor should ensure that the site has been prepared as recommended, and that field density tests have been performed to adequately document the relative compaction of the structural fill.

Conventional foundations can be designed to impose dead plus long term live load bearing pressures of 2,000 pounds per square foot (psf). The allowable foundation bearing pressure is for footings having a minimum width of 15-inches and a minimum depth of 18-inches embedment below the lowest adjacent finish grade for one or two story buildings and 18-inches wide and a minimum 24-inches embedment below lowest adjacent finish grade for three or four-story buildings. The allowable soil bearing pressure can be increased by one-third when considering transient loads of short duration, such as wind or earthquake loads. Based on the prevailing geotechnical conditions encountered during our subsurface exploration, we recommend that foundations be reinforced with at least two No. 4 bars placed at the top of the footing and two placed at the bottom.

Horizontal loads acting on foundations and stem walls cast in open excavations against undisturbed native soil or against properly placed and compacted fill will be resisted by friction acting along the base of the footing and by passive earth pressures against the side of the footing and stem wall. The frictional resistance acting along the base of footings founded on suitable foundation soils may be computed using a coefficient of friction equal to 0.30 with the normal dead load.

Passive earth pressures acting against the side of footings and stem walls may be assumed to be equivalent to a fluid weighing 250 pounds per cubic foot. Passive pressure in the upper 1.0-foot should be neglected unless confined by concrete slabs-on-grade or asphalt concrete pavement. The values given above may be increased by one-third for transient wind or seismic loads.

7.3 Footing Setbacks

All footings should maintain a minimum 7-foot horizontal setback from the base of the footing to any descending slope (if existing onsite). This distance is measured from the outside footing face at the bearing elevation. Footings should maintain a minimum horizontal setback of H/3 (H=slope height) from the base of the footing to the descending slope face and no less than 7 feet, or greater than 40 feet.

Footings adjacent to unlined drainage swales or underground utilities (if any) should be deepened to a minimum of 6-inches below the invert of the adjacent unlined swale or utilities. This distance is measured from the footing face at the bearing elevation. Footings for structures adjacent to retaining walls should be deepened so as to extend below a 1:1 projection from the heel of the wall. Alternatively, walls may be designed to accommodate structural loads from buildings or appurtenances.

7.4 Construction

The foundation construction considerations contained herein are presented as minimum preliminary recommendations from a soils engineering standpoint. Laboratory test results indicate the onsite soils' swell (expansion) potential is very low. During grading of the site, we recommend that no soil possessing an Expansion Index of more than 20 be placed within 18-inches of finish grade, if possible. As such, design parameters provided herein assume that finish grade soil materials will have a low expansion potential.

Recommendations by the project's design-structural engineer or architect, which may exceed the soils engineer's recommendations, should take precedence over the following minimum preliminary considerations. Final foundation design should be provided based on the expansion potential of the near surface soils encountered during grading.

7.5 Concrete Slab-on-Grade

Interior slabs can be grade supported by structural fill whose placement/compaction is documented by the project soils engineer/engineer geologist as recommended herein. The thickness of the slab should be in accordance with the structural engineer's design. However, based on geotechnical considerations, we recommend that concrete slabs be a minimum of 4-inches in thickness. Concrete slabs should be underlain by at least 2-inches of clean sand with a Sand Equivalent (SE) of at least 30. Where moisture condensation is undesirable, concrete slabs should be underlain with a moisture/vapor retarder consisting of a minimum 10-mil, visqueen membrane, with all laps sealed. The membrane should be underlain by a 2-inch layer of clean sand with the aforementioned sand layer placed over the visqueen to aid concrete curing. To reduce the potential for buildup of hydrostatic pressures, the free draining material under the slabs should have positive drainage with no low lying areas (i.e., depressions) created.

Floor slabs should be suitably reinforced and jointed (in accordance with Structural Engineer's recommendations) so that a small amount of independent movement can occur without causing damage. Based on the encountered geotechnical conditions, we recommend that floor slabs be reinforced with No. 4 bars spaced on 18-inch centers (each way)

The contractor should take the appropriate precautions to make sure that the reinforcement is placed and maintained within the middle one-third of the slab. Exterior slabs, such as walkways and driveways, can be adequately supported on documented structural fill that is at minimum 12-inches in thickness, and placed and compacted in accordance with the recommendations contained herein.

In preparation for slab or flatwork construction, the earthwork contractor should ensure that the onsite soils have been prepared as recommended and that field density tests have been performed to adequately document the relative compaction of the structural fill. Preparation of the native soils should be documented prior to placement of aggregate, structural components and/or fill.

Some minor cracking of slabs can be expected due to shrinkage. The potential for this slab cracking can be reduced by careful control of water/cement ratios in the concrete. The contractor should take appropriate curing precautions during the pouring of concrete in hot or windy weather to reduce the potential for cracking of slabs. We recommend that a slipsheet (or equivalent) be utilized if grouted fill, tile, or other crack-sensitive floor covering is planned directly on concrete slabs. All slabs should be designed in accordance with structural considerations.

All dedicated exterior flatwork should conform to standards provided by the governing agency including section composition, supporting material thickness and any requirements for reinforcing steel. Concrete mix proportions and construction techniques, including the addition of water and improper curing, can adversely affect the finished quality of the concrete and result in cracking and spalling of the slab. We recommend that all placement and curing be performed in accordance with procedures outlined by the American Concrete Institute and/or Portland Cement Association. Special consideration should be given to concrete placed and cured during hot or cold weather conditions. Proper control joints should be provided to reduce the potential for damage resulting from shrinkage.

Laboratory test results indicate that the upper soils contain soluble sulfate concentrations of 0.003 percent and chloride concentrations of 0.001 percent. The results of these analyses indicate a negligible corrosion potential to concrete. As such, Type II cement can be used in concrete elements that will be in contact with the upper soils.

8.0 PAVEMENT DESIGN RECOMMENDATIONS

Deleterious material, excessively wet or dry pockets, concentrated zones of oversized rock fragments, and any other unsuitable yielding materials encountered during grading should be removed. Once compacted fill and/or native soils are brought to the proposed pavement subgrade elevations, the subgrade should be proof-rolled in order to check for a uniform firm and unyielding surface. Representatives of the project geotechnical engineer should observe all grading and fill placement.

The upper 12-inches of pavement subgrade soils should be scarified; moisture conditioned to at least optimum moisture content and compacted to at least 95 percent of the laboratory standard (ASTM D1557). If loose or yielding materials are encountered during subgrade preparation, evaluation should be performed by EEI. Aggregate base materials should be properly prepared (i.e., processed and moisture conditioned) and compacted to at least 95 percent of the maximum dry density as determined by ASTM D1557. Aggregate base materials should conform to Caltrans specifications for Class 2 aggregate base.

All pavement section changes should be properly transitioned. Although not anticipated, if adverse conditions are encountered during the preparation of subgrade materials, special construction methods may need to be employed. A representative of the project geotechnical engineer should be present for the preparation of subgrade and aggregate base.

For design purposes, we have assumed a Traffic Index (TI) of 4.5 for the proposed parking areas and 6.0 for drive areas at the subject property. These assumed TI's should be verified as necessary by the Civil Engineer or Traffic Engineer. For preliminary design purposes, we have conservatively assumed a preliminary R-Value of 20 for the materials likely to be exposed at subgrade. The modulus of subgrade reaction (K-Value) was estimated at 70 pounds per square inch per inch (psi/in) for an R-Value of 20 (Caltrans, 1974).

TABLE 3
Preliminary Pavement Design Recommendations

Traffic Index (TI)	Pavement Surface	Aggregate Base Material (1)
4.5 – Parking Stalls	3.0-inches Asphalt Concrete	6.0-inches
6.0 – Main Drive Areas	4.0-inches Asphalt Concrete	8.0-inches
Trash Area and Concrete Pavement	5.5-inches Portland Cement Concrete (2)	Optional

(1) R-Value of 78 for Caltrans Class 2 aggregate base
(2) Reinforcement and control joints placed in accordance with the structural engineer's requirements

The recommended pavement sections provided above are intended as a preliminary minimum guideline. If thinner or highly variable pavement sections are constructed, increased maintenance and repair could be expected. If the ADT (average daily traffic) or ADTT (average daily truck traffic) increases beyond that intended, as reflected by the assumed traffic index used for design, increased maintenance and repair could be required for the pavement section. Final pavement design should be verified by testing of soils exposed at subgrade after grading has been completed. Thicker pavement sections could result if R-Value testing indicates lower values.

9.0 DEVELOPMENT RECOMMENDATIONS

9.1 Landscape Maintenance and Planting

Water is known to decrease the physical strength of earth materials, significantly reducing stability by high moisture conditions. Surface drainage away from foundations and graded slopes should be maintained. Only the volume and frequency of irrigation necessary to sustain plant life should be applied.

Consideration should be given to selecting lightweight, deep rooted types of landscape vegetation which require low irrigation that are capable of surviving the local climate. From a soils engineering viewpoint, "leaching" of the onsite soils is not recommended for establishing landscaping. If landscape soils are processed for the addition of amendments, the processed soils should be re-compacted to at least 90 percent relative compaction (based on ASTM D1557).

9.2 Site Drainage

Positive site drainage should be maintained at all times. Drainage should not flow uncontrolled over slopes or the subject property. Runoff should be channeled away from slopes and structures and not allowed to pond and/or seep uncontrolled into the ground. Pad drainage should be directed toward an acceptable outlet. Although not required, roof gutters and down spouts may be considered to control roof drainage, discharging a minimum of 10 feet from the proposed structures, or into a subsurface drainage system. Consideration should be given to eliminating open bottom planters directly adjacent to proposed structures for a minimum distance of 10 feet. As an alternative, closed-bottom type planters could be utilized, with a properly designed drain outlet placed in the bottom of the planter.

9.3 Stormwater Disposal Systems

EEI understands that current plans call for runoff generated from the facility to be disposed of in engineered subsurface features onsite. Percolation testing was conducted at three widely separated locations for proposed onsite detention basins during our Due Diligence Level Geotechnical Evaluation performed in 2014 and the percolation testing results are presented in the referenced report by EEI dated November 25, 2014.

9.4 Additional Site Improvements

Recommendations for additional grading, exterior concrete flatwork design and construction can be provided upon request. If in the future, additional property improvements are planned for the subject property, recommendations concerning the design and construction of improvements would be provided upon request.

9.5 Trenching

All temporary excavations for grading purposes and installation of underground utilities should be constructed in accordance with OSHA guidelines and local safety codes. Temporary excavations over 5 feet in height should be evaluated by the project engineer, and could require shoring, sloping, or a combination thereof. Temporary excavations within the onsite materials should be stable at 1:1 inclinations for cuts less than 10 feet in height.

Footing trench excavations for structures and walls should be observed and approved by a representative of the project soils engineer prior to placing reinforcement. Footing trench spoil and excess soils generated from utility trench excavations should be compacted to a minimum relative compaction of 90 percent (based on ASTM D1557) if not removed from the subject property. All excavations should conform to OSHA and local safety codes.

9.6 Utility Backfill

Fill around the pipe should be placed in accordance with details shown on the drawings, and should be placed in layers not to exceed 8-inches loose (unless otherwise approved by the geotechnical engineer) and compacted to at least 90 percent of the maximum dry density as determined in accordance with ASTM D1557 (Modified Proctor). The geotechnical engineer should approve all backfill material. Select material should be used when called for on the drawings, or when recommended by the geotechnical

engineer. Care should be taken during backfill and compaction operations to maintain alignment and prevent damage to the joints. The backfill should be kept free from oversized material, chunks of highly plastic clay, or other objectionable material. Backfill soils should be non-expansive, non-corrosive, and compatible with native earth materials. Backfill materials and testing should be in accordance with the CBC (2013), and the requirements of the local governing jurisdiction.

Pipe backfill areas should be graded and maintained in such a condition that erosion or saturation will not damage the pipe bed or backfill. Flooding trench backfill is not recommended. Heavy equipment should not be operated over any pipe until it has been properly backfilled with a minimum of 2 to 3 feet of cover. The utility trench should be systematically backfilled to allow maximum time for natural settlement. Backfill should not occur over porous, wet, or spongy subgrade surfaces. Should these conditions exist, the areas should be removed, replaced and recompacted.

10.0 BORE & JACK RECOMMENDATIONS

Based on the results of our subsurface exploration, it is our opinion that the proposed trenchless (bore-and-jack) operations are feasible and that sufficient ground cover exists to provide stable conditions adjacent to the 60 Freeway for the proposed Tract No. 31556 offsite sewer freeway undercrossing, provided such operations are adequately performed by a qualified contractor(s) who is experienced in bore-and-jack pipe installation methods in the local area. Geotechnical data, obtained from our subsurface exploration, indicates that potential settlement and heave problems are not likely at or adjacent to the proposed bore-and-jack locations, assuming adequate procedures are performed.

It is the contractor's responsibility to design and select the appropriate tunnel construction method, support system and to follow the requirements of the health and safety rules of the State of California pertaining to bore-and-jack/tunnel construction and permit requirements of local agencies such as the City of Moreno Valley or County of Riverside. The contractor should develop an appropriate mitigation plan for deficient soil conditions, as well as a contingency plan for potential emergency conditions, such as loss of ground, excessive settlement and other construction problems. Provisions for controlling running sand and groundwater at localized areas should be provided during the boring operation to reduce the potential for ground loss and ground subsidence. The following geotechnical recommendations are provided for bore-and-jack operations at the offsite sewer alignment location undercrossing, based on our subsurface exploration.

10.1 Utilities and Subsurface Obstructions

The EEI exploratory borings completed during the Supplemental Geotechnical Evaluation (borings B-16 and B-17) were both advanced to a depth of approximately 21.5 feet below the existing ground surface with boring B-16 located immediately north of the 60 Freeway at the proposed "boring" pit location and boring B-17 being located immediately south of the 60 Freeway at the proposed "receiving" pit location. Artificial fill soils were encountered from the surface to depths of 11 feet and 16 feet below existing grade, respectively. The artificial fill soils were generally composed of loose to medium dense, damp to moist, fine to coarse silty sand and sandy silt. The artificial fill soils were underlain by generally medium dense, moist, fine to medium silty sand.

It is possible that underground utilities and other subsurface obstructions will be encountered within the proposed bore-and-jack location undercrossing the 60 Freeway right-of-way, but not likely. However, we recommend that the contractor perform a comprehensive evaluation of the locations of buried utilities and other structures within the proposed bore-and-jack location. Such an evaluation can include, but is not necessarily limited to, advancing a series of potholes and using geophysical methods to help identify the locations of the utilities/structures. It should be understood that if utilities are present within the proposed pipeline alignment, the nature of the utilities' backfill materials might differ from the materials encountered during our subsurface exploration.

10.2 Tunnelman's Ground Classification

Attempts to classify soil characteristics for tunneling operations include the Rock Quality Designation (RQD) by Deere, Rock Mass Rating (RMR) by Bieniawski, Rock Tunneling Quality Index (Q) by Barton, and Tunnelman's Ground Classification by Terzaghi. The RQD, RMR, and Q designations characterize rock masses while the Tunnelman's Ground Classification was derived to describe soil conditions for tunneling through soft sediments. The Tunnelman's Ground Classification **(Table 4)** categorizes predictive soil behaviors for saturated and unsaturated conditions.

Based on the geotechnical conditions observed during our subsurface exploration, we consider the encountered portions of the artificial fill and alluvial soils are comprised of loose to medium dense running soils.

10.3 Jacking and Receiving Pits

Bore-and-jack operations involve the initial construction of a jacking/tunneling pit and a receiving pit at each end of the pipe segment to be jacked. The jacking and receiving pits should be shored as required. The shoring system should be designed as recommended below. The jacking equipment should not impose a reaction of more than 4,000 psf on the soils within the jacking pit. Pipes for use with microtunnelling systems must be designed to withstand the high axial jacking forces, and this is likely to be a far more significant design parameter than any post installation loading. Jacking operations and tunneling operations should be performed in accordance with the Standard Specifications for Public Works Construction, Sections 306-2 and 306-3 of the latest edition. Grouting through the pipe casing after jacking is recommended to fill any possible voids created by the jacking operation.

TABLE 4
TUNNELMAN'S GROUND CLASSIFICATION FOR SOIL

GROUND CLASSIFICATIONS	GROUND BEHAVIOR	TYPICAL SOIL TYPES
Hard	Tunnel heading may be advanced without roof support.	Cemented sand and gravel and over consolidated clay above the groundwater table.
Firm	Ground in which a roof section of a tunnel can be left unsupported for several days without inducing a perceptible movement of the ground.	Loess above the water table, hard clay, marl, cemented sand and gravel when not highly overstressed.
Raveling	Chunks or flakes of soil begin to drop out of roof at some point during the ground-movement period.	Residual soils or soils with clay binder may be fast raveling below the groundwater table and slow raveling above the groundwater table. Stiff fissured clays may be slow raveling or fast raveling depending on the degree of overstress.

TABLE 4
TUNNELMAN'S GROUND CLASSIFICATION FOR SOIL

GROUND CLASSIFICATIONS	GROUND BEHAVIOR	TYPICAL SOIL TYPES
Slow Raveling	The time required to excavate 5-feet of tunnel and install a rib set and lagging in a small tunnel is about 6-hours. Therefore, if the stand-up time of raveling ground is more than 6-hours, using ribs and lagging, such a soil would be classed as slow raveling.	
Fast Raveling	If the stand-up time is less than 6-hours, using ribs and lagging, such a soil would be classed as fast raveling.	
Squeezing	Ground slowly advances into tunnel without any signs of fracturing. The loss of ground caused by squeeze and the resulting settlement of the ground surface can be substantial	Ground with low frictional strength. Rate of squeeze depends on degree of overstress. Stiff to hard clay under high cover may move in combination of raveling at execution surface and squeezing at depths.
Swelling	Ground slowly advances into the tunnel partly or chiefly because of an increase in the volume of the ground. The volume increase is in response to an increase of water content. In every other respect, swelling ground in a tunnel behaves like a stiff non-squeezing, or slowly squeezing, non-swelling clay.	Highly preconsolidated clay with plasticity index greater than about 30, generally containing significant percentages of montmorillonite clay
Running	The removal of lateral support on any surface rising at an angle more than 34° (to the horizontal) is immediately followed by a running movement of the soil particles.	Clean, dry granular materials.
Cohesive Running	If the running ground has a trace of cohesion, then the run is preceded by a brief period of progressive raveling.	Apparent cohesion in moist sand or weak cementation in any granular soil may allow the material to stand for a brief period of raveling before it breaks down and runs. Such behavior is cohesive running.
Very Soft Squeezing	Ground advances rapidly into tunnel in a plastic flow.	
Flowing	Ground supporting a tunnel cannot be classified as flowing ground unless water flows or seeps through it toward the tunnel. For this reason, a flowing condition is encountered only in free air tunnels below the water table or under compressed air when the pressure is not high enough in the tunnel to dry the bottom.	Only occurs in inorganic silt, fine silty sand, clean sand or gravel, or sand-and-gravel with some clay binder. Organic silt may behave either as a flowing or as a very soft, squeezing ground.

10.4 Shoring

The vertical face of the jacking pit and receiving pit may be shored with sheet piles and/or soldier piles and lagging. The face of the shaft also can be supported by ribs and lagging. The details of sheet piling, soldier beam and lagging system may be designed according to the recommendations provided in the following sections of this report. Frequent contact grouting may be necessary to backpack the support during construction to reduce the potential for settlement. While groundwater was not encountered during our subsurface exploration, the shoring contractor should also consider the possibility of localized perched groundwater in the design and installation procedures of the shoring system.

10.5 Lateral Earth Pressures

The shoring system should be designed to resist the pressure exerted by the retained soils and any additional lateral forces due to loads applied near the top of the excavations. A cantilevered shoring system should be designed for an equivalent fluid weight of 35 pcf. Braced shoring walls supporting a level ground surface should be designed for a uniform lateral pressure of 20H psf, where H is the height of the retained earth in feet.

For surcharge loads due to traffic, the shoring should be designed for an additional uniform horizontal pressure of 75 psf for passenger car traffic and 150 psf for heavy truck traffic. For other surcharge loads, the wall should be designed for a uniform horizontal pressure equal to one-third the anticipated surcharge pressure. These parameters all assume a level ground surface and that temporary shoring will not be subject to hydrostatic pressures. The shoring system should be properly embedded beneath the toe of the excavation to provide adequate structural stability.

10.6 Passive Resistance

It is recommended that the design of the shoring system incorporates a passive equivalent fluid weight of 350 pcf for the shoring embedded within relatively competent older alluvium. If utilized, soldier piles should be spaced no closer than 3 diameters on center. The soldier piles should be drilled and backfilled with concrete to the full depth of the passive resistance zone. The area providing the passive resistance can be assumed to have a width equal to twice the concrete pile diameter. The recommended passive pressure for the shoring assumes a horizontal surface for the soil mass extending at least 10 feet in front of the face of the shoring, or three times the height of the surface generating passive pressure, whichever is greater. The shoring system should be embedded a sufficient depth beneath the toe of the excavation to provide structural stability. We recommend that a factor of safety of at least 1.2 be applied to the calculated embedment depth and that the passive pressure be limited to 2,500 psf. The assumed geotechnical conditions should be verified as necessary during shoring construction by a representative of this firm.

10.7 Lagging

If soldier piles are used as shoring, timber lagging may be used between the soldier piles to help support the exposed soils. If lagging is to remain after construction, treated lumber should be used. Lagging should be designed for the full lateral pressure recommended on the previous page. Voids between the soil and lagging should be grouted or slurried to reduce the potential for the voids to propagate to the surface.

10.8 Utility Backfill

Fill around the pipe should be placed in accordance with details shown on the drawings, and should be placed in layers not to exceed 8 inches loose (unless otherwise approved by the geotechnical engineer) and compacted to at least 90 percent of the maximum dry density as determined in accordance with ASTM D 1557 (Modified Proctor). The geotechnical engineer should approve all backfill material. Select material should be used when called for on the drawings, or when required by the geotechnical engineer. Care should be taken during backfill and compaction operations to maintain alignment and prevent damage to the joints. The backfill should be kept free from stones, chunks of highly plastic clay, or other objectionable material. Backfill soils should be non-expansive, non-corrosive, and compatible with native earth materials. Backfill materials and testing should be in accordance with the 2013 CBC, and/or the City of Moreno Valley specifications.

All pipe backfill areas should be graded and maintained in such a condition that erosion or saturation will not damage the pipe bed or backfill. Flooding trench backfill is not recommended. Heavy equipment should not be operated over any pipe until it has been properly backfilled with a minimum two to three feet of cover. The utility trench should be systematically backfilled to allow maximum time for natural settlement. Backfill should not occur over porous, wet, or spongy subgrade surfaces. Should these conditions exist, the areas should be removed, replaced, and recompacted.

10.9 Pre-Construction Survey and Monitoring

Consideration should be given to performing a pre-condition survey of improvements in the area prior to construction, to document any current existing distress. Settlement points should be considered to monitor any settlement of adjacent facilities during construction.

10.10 Other Design Considerations

Excavation, shoring and dewatering systems should be properly designed and installed to minimize the effects of aerial settlement, during construction. The preparation of plans for the excavations, shoring, sheeting, and dewatering systems is normally the responsibility of the contractor. Consideration should be given to characterize the material as the bore-and-jack operations proceed; and based on the behavior of the soil; the support system and method of trenchless construction may be altered accordingly.

10.11 Plan Review

It is recommended that EEI be provided the opportunity to review the final project plans prior to construction. The purpose of this review is to assess the general compliance of the plans with the recommendations provided in this report and the incorporation of these recommendations into the project plans and specifications.

10.12 Observation and Testing During Construction

It is recommended that EEI be retained to observe the bore-and-jack operations as they progress; provide observation and testing services during placement and backfill of underground utilities, observe drilling of soldier piles (if warranted), and to observe final site drainage. This is to observe compliance with the design concepts, specifications and recommendations, and to allow for possible changes in the event that subsurface conditions differ from those anticipated prior to the start of construction

11.0 PLAN REVIEW

Once detailed site and grading plans are available, they should be submitted to this office for review and comment, to reduce the potential for discrepancies between plans and the preliminary recommendations presented herein. If conditions are found to differ substantially from those stated, appropriate recommendations would be provided. Additional field studies may be warranted.

12.0 LIMITATIONS

This Supplemental Geotechnical Evaluation has been conducted in accordance with generally accepted geotechnical engineering principles and practices. Findings provided herein have been derived in accordance with current standards of practice, and no warranty is expressed or implied. Standards of practice are subject to change with time. This Supplemental Geotechnical Evaluation report has been prepared for the sole use of Anderson Consulting Engineers, Inc. (Client), within a reasonable time from its authorization. Site conditions, land use (both onsite and offsite), or other factors may change as a result of man-made influences, and additional work may be required with the passage of time.

This Supplemental Geotechnical Evaluation should not be relied upon by other parties without the express written consent of EEI and the Client; therefore, any use or reliance upon this geotechnical evaluation by a party other than the Client should be solely at the risk of such third party and without legal recourse against EEI, its employees, officers, or directors, regardless of whether the action in which recovery of damages is brought or based upon contract, tort, statue, or otherwise.

The Client has the responsibility to see that all parties to the project, including the designer, contractor, subcontractor, and building official, etc. are aware of this report in its complete form. This report contains information that may be used in the preparation of contract specifications; however, the report is not designed as a specification document, and may not contain sufficient information for use without additional assessment. EEI assumes no responsibility or liability for work or testing performed by others. In addition, this report may be subject to review by the controlling authorities.

13.0 REFERENCES

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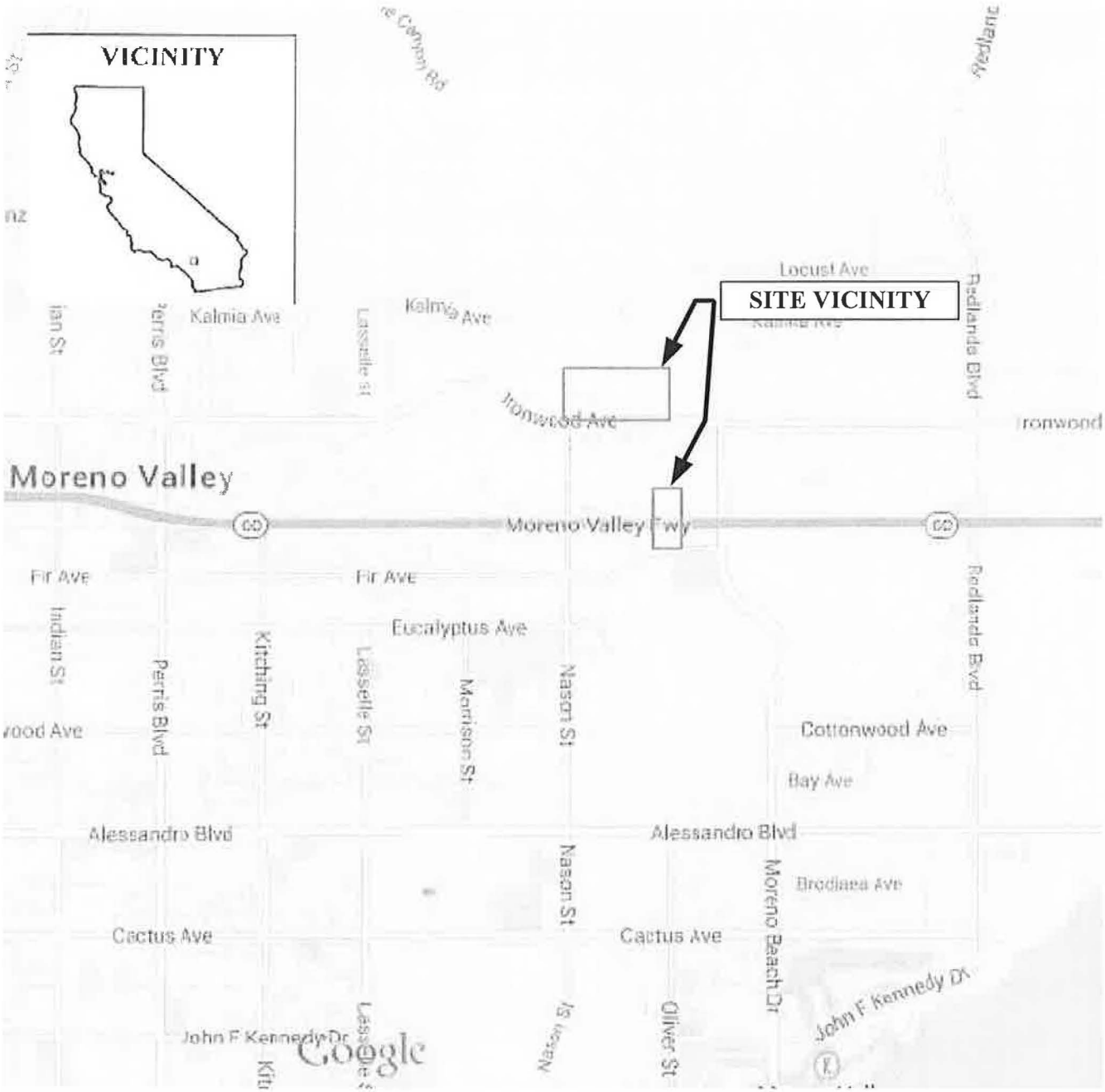
FIGURES



Map Source: Google Maps®; Accessed 2015

LEGEND

N

Scale: 1" = 1mi

0 1mi 2mi

Note: All Locations Are Approximate

SITE VICINITY MAP

BELLACAP, LLC

North of Ironwood Ave Between Oliver St & Nason St

Moreno Valley, California 92555

EEI Project No. GLO-71982.4a

Created April 2015

EEI

FIGURE 1

Source: Google Earth, 2015; Image Date: April 20, 2015

LEGEND

N

Scale: 1" = 900'

0 900 FT 1800 FT

Note: All Locations Are Approximate

AERIAL SITE MAP & OFFSITE BORINGS

BELLACAP, LLC

North of Ironwood Avenue Between Oliver St & Nason St

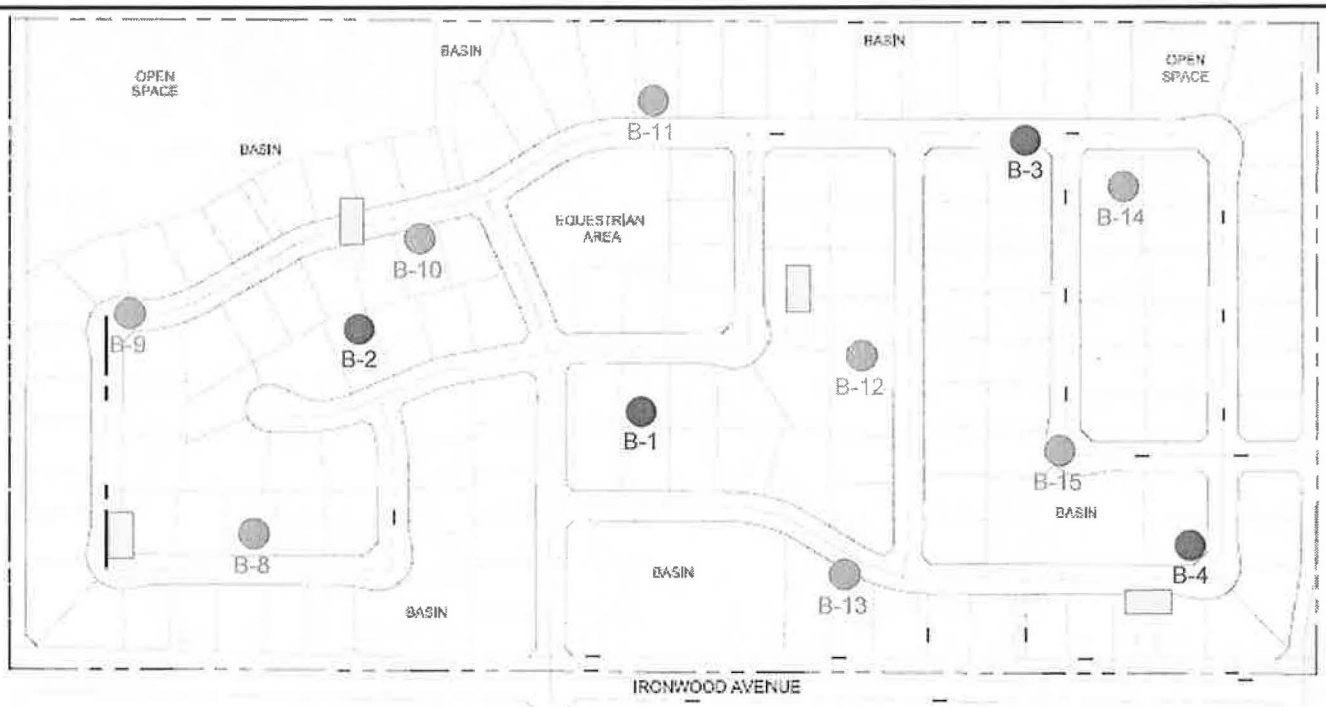
Moreno Valley, California

EEI Project No. GLO-71982.4a

Created April 2015

EEI

FIGURE 2



Source: Anderson Consulting Engineers, IronwoodProject Exhibit

LEGEND

Approximate Boring Locations (4/16-17/15)
B-15

Previous Boring Locations (10/17/14)
B-4

Previous Test Pit Locations (10/16/14)

Scale: 1" = 300'

0 FT 300 FT 600 FT

FIELD EXPLORATION PLAN
BELLACAP, LLC
Oliver Street North & South of Moreno Valley Fwy
Moreno Valley, California
EEI Project No. GLO-71982.4a
Created April 2015

FIGURE 3

OLIVER STREET

LEGEND

B-17 **Approximate Boring Location**

Scale: 1" = 35'

0 35 FT 70 FT

SCALE 1"=35'

North

OFFSITE SEWER BORING LOCATIONS
BELLACAP, LLC
Oliver Street North & South of Moreno Valley Fwy
Moreno Valley, California
EEI Project No. GLO-71982.4a
Created April 2015

EEI

FIGURE 4

APPENDIX A
SOIL CLASSIFICATION CHART, BORING LOGS

SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVEL AND GRAVELLY SOILS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVELS		GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
		(LITTLE OR NO FINES)		GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
		GRAVELS WITH FINES		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
		(APPRECIABLE AMOUNT OF FINES)		GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	CLEAN SANDS		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
		(LITTLE OR NO FINES)		SP	POORLY-GRADED SANDS, GRAVELLY SAND, LITTLE OR NO FINES
		SANDS WITH FINES		SM	SILTY SANDS, SAND - SILT MIXTURES
		(APPRECIABLE AMOUNT OF FINES)		SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY
				OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
CLAYSTONE				CL	CLAYSTONE, Pliocene Fernando Formation/late Miocene Puente Formation

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

BOREHOLE LOG

Number:	B-1
Client:	Global - Anderson Consulting Engineers
Sheet:	1 of 2
Location:	Ironwood - Moreno Valley Moreno Valley, Riverside County, Ca
Date Started:	10/17/2014
Date Finished:	10/17/2014
EEI Rep:	BM
Project No.:	GLO-71982.4
Drill Rig/Sampling Method:	Truck Mounted CME-75 / 140 lb Auto Hammer
Borehole Diameter:	6-inch

SAMPLE LOG								BOREHOLE LOG
Bulk	Sample Type	Blows Per 6"	Dry Unit Wt. (pcf)	Moisture (%)	Depth in Feet	USCS Symbol	Graphic Log	Geologic Description (Soil Type, Color, Grain, Minor Soil Component, Moisture, Density, Other Etc.)
					1			**ALLUVIUM** **SILTY-SAND**, orange-brown mottled, fine to coarse grained, damp, loose
					2			
	MC	3 4 5	105	3	3			
					4			
					5			
	MC	4 6 8	108	3	6			
					7			
	MC	3 3 4	138	3	8			
					9			
					10			
	MC	4 5 6	104	7	11			@ 10' Becomes fine to medium grained, moist, loose; micaceous
					12			
					13			
					14			
					15	SM		
	SPT	3 3 3		9	16			
					17			
					18			
					19			
					20			
	MC	9 11 16	122	8	21			@ 23' becomes medium dense
					22			
					23			
					24			
					25			
	SPT	6 8 8		7	26			
					27			
					28			
					29			
					30			
	MC	8 10 11	124	7	31	SM		**WEATHERED BEDROCK: TONALITE (Kt)** DECOMPOSED GRANITICS ("DG"), gray-white to black speckled, damp, dense; weathered, friable, phaneritic texture, moderate to heavily weathered

BOREHOLE LOG VES-71982.4.GPJ EEI.GDT 11/25/14

BOREHOLE LOG

EEI Geotechnical & Environmental Solutions	BOREHOLE LOG	Number: B-1	
	Client: Global - Anderson Consulting Engineers	Sheet: 2 of 2	
Date Started: 10/17/2014	Date Finished: 10/17/2014	Location: Ironwood - Moreno Valley, Moreno Valley, Riverside County, Ca	
EEI Rep: BM	Project No.: GLO-71982.4	Drill Rig/Sampling Method: Truck Mounted CME-75 / 140 lb Auto Hammer	Borehole Diameter: 6-inch

SAMPLE LOG								BOREHOLE LOG
Bulk	Sample Type	Blows Per 6"	Dry Unit Wt. (pcf)	Moisture (%)	Depth in Feet	USCS Symbol	Graphic Log	Geologic Description (Soil Type, Color, Grain, Minor Soil Component, Moisture, Density, Odor, Etc.)
					33			
					34			
	SPT	50-4"		4	35			@ 35' Becomes very dense
					36			
					37			
					38			
					39			
	MC	50-3"	103	7	40			
					41	SM		
					42			
					43			
					44			
	SPT	50-5"		7	45			@ 45' DECOMPOSED GRANITICS ("DG"), dark gray, orange and white speckled, damp, medium dense to dense; moderately weathered
					46			
					47			
					48			
					49			
	NR	50-3"			50			@ 50' No sample recovery
					51			
					52			Total depth: 50.5-feet No groundwater encountered
					53			Hole backfilled on 10/17/2014 with drilled cuttings
					54			
					55			
					56			
					57			
					58			
					59			
					60			
					61			
					62			
					63			

BOREHOLE LOG VES-71982.4.GPJ EEI.GDT 11/2/14

BOREHOLE LOG

EEI Geotechnical & Environmental Solutions	Number: B-2	
	Client: Global - Anderson Consulting Engineers	Sheet: 1 of 1
	Location: Ironwood - Moreno Valley, Moreno Valley, Riverside County, Ca	
Date Started: 10/17/2014	Date Finished: 10/17/2014	
EEI Rep: BM	Project No.: GLO-71982.4	Drill Rig/Sampling Method: Truck Mounted CME-75 / 140 lb Auto Hammer	Borehole Diameter: 6-inch

SAMPLE LOG								BOREHOLE LOG
Bulk	Sample Type	Blows Per 6"	Dry Unit Wt. (pcf)	Moisture (%)	Depth In Feet	USCS Symbol	Graphic Log	Geologic Description (Soil Type, Color, Grain, Minor Soil Component, Moisture, Density, Odor, Etc.)
					1			**COLLUVIUM**
					2			SILTY SAND, orange-brown mottled, fine to coarse grained, damp, loose
	MC	4 6 8	111	2	3			
					4			
	MC	7 8 15	137	3	5	SM		
					6			
					7			
	MC	5 6 9	115	4	8			
					9			
	MC	5 31 50-3"	126	3	10			
					11			**WEATHERED BEDROCK: TONALITE (Kt)**
					12			DECOMPOSED GRANITICS ("DG"), white, orange and black speckled, damp, dense to very dense; unweathered to moderately weathered
					13	SM		
					14			
	SPT	23 50-3"		4	15			
					16			
					17			Total depth: 15.75-feet
					18			No groundwater encountered
					19			Hole backfilled on 10/17/2014 with drilled cuttings
					20			
					21			
					22			
					23			
					24			
					25			
					26			
					27			
					28			
					29			
					30			
					31			

BOREHOLE LOG

Number:	B-3
Client:	Global - Anderson Consulting Engineers
Sheet:	1 of 1
Location:	Ironwood - Moreno Valley Moreno Valley, Riverside County, Ca
Date Started:	10/17/2014
Date Finished:	10/17/2014
EEI Rep:	BM
Project No.:	GLO-71982.4
Drill Rig/Sampling Method:	Truck Mounted CME-75 / 140 lb Auto Hammer
Borehole Diameter:	6-inch

SAMPLE LOG

Bulk	Sample Type	Blows Per 6"	Dry Unit Wt. (pcf)	Moisture (%)	Depth to Feet	USCS Symbol	Graphic Log	Geologic Description (Soil Type, Color, Grain, Minor Soil Component, Moisture, Density, Odor, Etc.)
					1	SM		**COLLUVIUM** SILTY-SAND, red-brown, fine to coarse grained with occasional gravels, dry to damp, dense
	MC	50-3"	85	5	2, 3			
	MC	20 22 26	119	2	5, 6			@ 5' Becomes medium dense
	MC	23 31 34	123	4	8, 9			@ 8' Becomes dense
	MC	9 13 18	113	2	10, 11	SM		**WEATHERED BEDROCK: TONALITE (Kt)** DECOMPOSED GRANITICS ("DG"), orange, brown and black speckled, damp, medium dense to dense; occasional unweathered to moderately weathered
	SPT	8 9 10		4	15, 16			
					19			@ 19' Becomes very dense
	MC	23 44 65-6"	128	8	20, 21			
					22			Total depth: 21.5-feet No groundwater encountered Hole backfilled on 10/17/2014 with drilled cuttings
					23, 24, 25, 26, 27, 28, 29, 30, 31			

BOREHOLE LOG VES-71982.4.GPJ EEI.GDT 11/25/14

BOREHOLE LOG

EEI Geotechnical & Environmental Solutions	Number: B-4	
	Client: Global - Anderson Consulting Engineers	Sheet: 1 of 1
	Location: Ironwood - Moreno Valley, Moreno Valley, Riverside County, Ca	
Date Started: 10/17/2014	Date Finished: 10/17/2014	
EEI Rep: BM	Project No.: GLO-71982.4	Drill Rig/Sampling Method: Truck Mounted CME-75 / 140 lb Auto Hammer	Borehole Diameter: 6-inch

SAMPLE LOG								BOREHOLE LOG
Bulk	Sample Type	Blows Per 6"	Dry Unit Wt. (pcf)	Moisture (%)	Depth in Feet	USCS Symbol	Graphic Log	Geologic Description (Soil Type, Color, Grain, Moist, Sol Component, Moisture, Density, Odor, Etc)
	MC	13 13 16	111	4	1–4	SM		**COLLUVIUM** SILTY-SAND, red-brown, fine to coarse grained with occasional gravels, moist, medium dense
	MC	6 7 7	111	3	5–6	SM		@ 5' Becomes loose
	MC	7 21 50-5"	115	5	7–9	SM		@ 8' Becomes dense to very dense
	MC	39 50-4"	118	8	10–13	SM		
	SPT	8 9 9		4	14–16	SM		**WEATHERED BEDROCK: TONALITE (Kt)** DECOMPOSED GRANITICS ("DG"), orange, brown and black speckled, moist, medium dense; weathered
					17–31			Total depth: 16.5 feet No groundwater encountered Hole backfilled on 10/17/2014 with drilled cuttings

BOREHOLE LOG VES-71982.4.GPJ EEI.GDT 11/5/14

BOREHOLE LOG

Number:	B-5
Client:	Global - Anderson Consulting Engineers
Sheet:	1 of 1
Location:	Ironweed - Moreno Valley Moreno Valley, Riverside County, Ca
Date Started:	10/17/2014
Date Finished:	10/17/2014
EEI Rep:	BM
Project No.:	GLO-71982.4
Drill Rig/Sampling Method:	Truck Mounted CME-75 / 140 lb Auto Hammer
Borehole Diameter:	6-inch

SAMPLE LOG / BOREHOLE LOG

Bulk	Sample Type	Blows Per 6"	Dry Unit Wt. (pcf)	Moisture (%)	Depth in Feet	USCS Symbol	Graphic Log	Geologic Description (Soil Type, Color, Grain, Minor Soil Component, Moisture, Density, Odor, Etc.)
					1	SM		**ALLUVIUM** SILTY-SAND, Orange-brown moist, fine to coarse grained, poorly sorted, damp, loose to medium dense
	MC	3 5 6	106	3	6			
	MC	5 9 16	123	5	10			
	SPT	10 10 15		6	15	SM		**WEATHERED BEDROCK: TONALITE (K0)** DECOMPOSED GRANITICS ("DG"), orange, brown and black speckled, moist, medium dense; weathered
	MC	18 20 25	124	12	20			
	SPT	10 12 18		11	25			
					27			Total depth: 26.5-feet No groundwater encountered Hole backfilled on 10/17/2014 with drilled cuttings
					30			

BOREHOLE LOG VER 7.1 BD & GPJ EEI.GDT 11/25/14

BOREHOLE LOG

Number:	B-6
Client:	Global - Anderson Consulting Engineers
Sheet:	1 of 1
Location:	Ironwood - Moreno Valley Moreno Valley, Riverside County, Ca
Date Started:	10/17/2014
Date Finished:	10/17/2014
EEI Rep:	BM
Project No.:	GLO-71982.4
Drill Rig/Sampling Method:	Truck Mounted CME-75 / 140 lb Auto Hammer
Borehole Diameter:	6-inch

SAMPLE LOG								BOREHOLE LOG
Bulk	Sample Type	Blows Per 6"	Dry Unit Wt. (pcf)	Moisture (%)	Depth in Feet	USCS Symbol	Graphic Log	Geologic Description (Soil Type, Color, Grain, Minor Soil Component, Moisture, Density, Odor, Etc.)
	MC	5 7 8	116	2	1–9	SM		**FILL** SILTY-SAND, light brown, fine to coarse grained, damp, loose
	MC	8 11 13	111	7	10–14	ML		**COLLUVIUM** SANDY-SILT, orange-brown, fine to medium grained, moderately sorted, moist, medium dense
	SPT	5 6 6		5	15–16			
	MC	8 13 16	118	3	17–21			
	SPT	8 10 12		6	22–26			
					27–31			Total depth: 26.5-feet No groundwater encountered Hole backfilled on 10/17/2014 with drilled cuttings

BOREHOLE LOG VES-71982.4.GPJ EEI.GDT 11/25/14

BOREHOLE LOG

Number:	B-7
Client:	Global - Anderson Consulting Engineers
Sheet:	1 of 1
Location:	Ironwood - Moreno Valley Moreno Valley, Riverside County, Ca
Date Started:	10/17/2014
Date Finished:	10/17/2014
EEI Rep:	BM
Project No.:	GLO-71982.4
Drill Rig/Sampling Method:	Truck Mounted CME-75 / 140 lb Auto Hammer
Borehole Diameter:	6-inch

SAMPLE LOG / BOREHOLE LOG

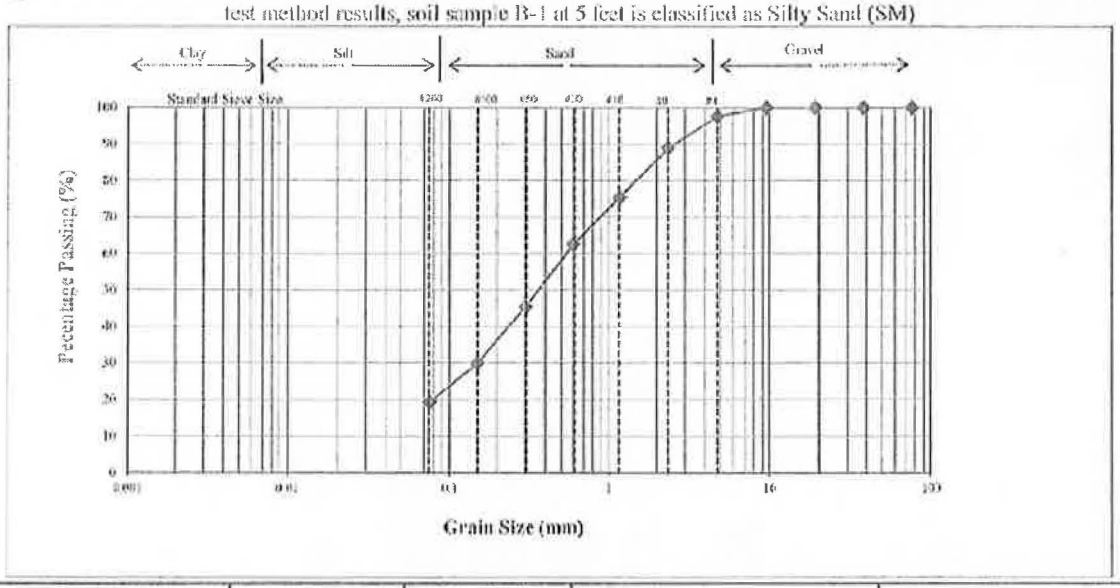
Bulk	Sample Type	Blows Per 6"	Dry Unit Wt. (pcf)	Moisture (%)	Depth in Feet	USCS Symbol	Graphic Log	Geologic Description (Soil Type, Color, Grain, Minor Soil Component, Moisture, Density, Odor, Etc.)
					1	SM		**COLLUVIUM** SILTY-SAND, brown to orange-brown, fine to coarse grained, moist, medium dense
	MC	11 24 26	115	3	5–6			
	MC	8 14 21	112	3	10–11			
	SPT	6 10 10		8	15–16	ML		@ 15' SANDY-SILT, brown, fine to coarse, moist, medium dense, micaceous
	MC	6 13 19	122	7	20–21			
	SPT	8 10 12		5	25–26			
					27–31			Total depth: 26.5-feet No groundwater encountered Hole backfilled on 10/17/2014 with drilled cuttings

APPENDIX B
LABORATORY TEST DATA

PARTICLE-SIZE ANALYSIS OF SOILS
ASTM METHOD D 422 (SIEVE ANALYSIS)

Sample :	B-1 @ 5 ft.		D10 (mm)	N/A
Total Weight (g)	129.2		D30 (mm)	0.15
Dry Weight (g)	125.8		D60 (mm)	0.53
Wet Sieve Weight (g)	101.5		Cu	N/A
Initial Moisture (%)	2.7		Cc	N/A

According to ASTM D 2487 Unified Soil Classification System (USCS) and ASTM D 422 (Standard Test Method for Particle-Size Analysis) test method results, soil sample B-1 at 5 feet is classified as Silty Sand (SM)



Sieve Size (in)	Sieve Size (mm)	Cumulative Weight of dry soil (gm)	Percent Retained (%)	Percent Passing (%)
3"	76.2		0.0	100.0
1.5"	38.1		0.0	100.0
3/4"	19.05		0.0	100.0
3/8"	9.53		0.0	100.0
#4	4.75	3.0	2.4	97.6
#8	2.36	14.0	11.1	88.9
#16	1.18	31.0	24.6	75.4
#30	0.6	47.2	37.5	62.5
#50	0.3	68.7	54.6	45.4
#100	0.15	88.2	70.1	29.9
#200	0.075	101.5	80.7	19.3

EEI
Geotechnical & Environmental Solutions

2195 Faraday Avenue, Suite K, Carlsbad CA 92008

Client: Anderson Consulting Engineers

Project Name: Ironwood

Job Number: GLO-71982.4

Date: 10/24/14

Boring Number: B-1

Depth: 5 ft.

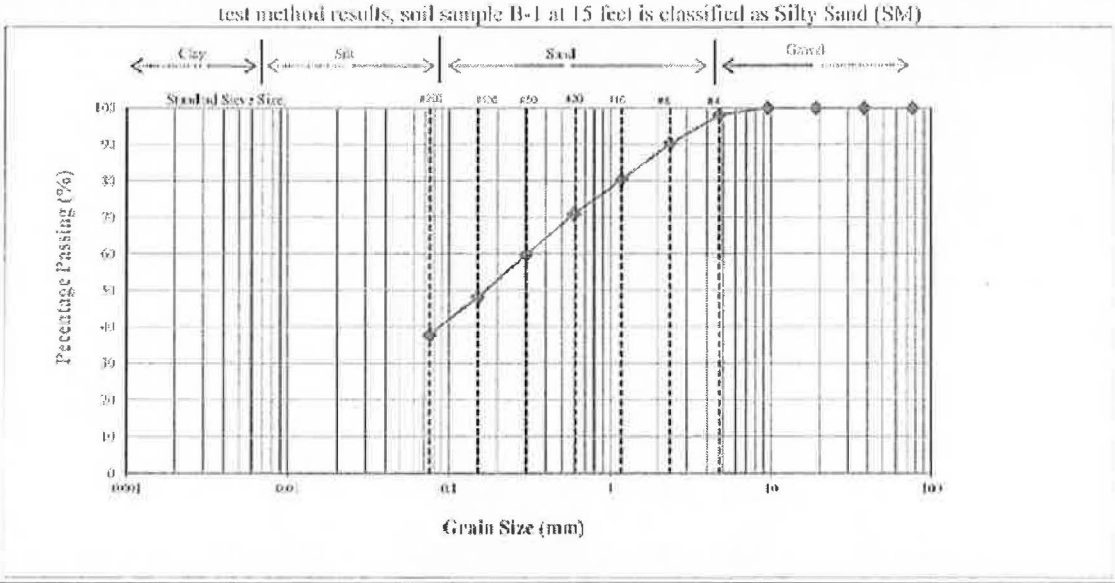
Soil Description: Brown Mottled Silty Sand SM

Tested by: B D

PARTICLE-SIZE ANALYSIS OF SOILS
ASTM METHOD D 422 (SIEVE ANALYSIS)

Sample :	B-1 @15 ft.		D10 (mm)	N/A
Total Weight (g)	117.5		D30 (mm)	N/A
Dry Weight (g)	108.1		D60 (mm)	0.30
Wet Sieve Weight (g)	67.3		Cu	N/A
Initial Moisture (%)	8.7		Cc	N/A

According to ASTM D 2487 Unified Soil Classification System (USCS) and ASTM D 422 (Standard Test Method for Particle-Size Analysis) test method results, soil sample B-1 at 15 feet is classified as Silty Sand (SM)



Sieve Size (in)	Sieve Size (mm)	Cumulative Weight of dry soil (gm)	Percent Retained (%)	Percent Passing (%)
3"	76.2		0.0	100.0
1.5"	38.1		0.0	100.0
3/4"	19.05		0.0	100.0
3/8"	9.53		0.0	100.0
#4	4.75	2.1	1.9	98.1
#8	2.36	10.6	9.8	90.2
#16	1.18	21.4	19.8	80.2
#30	0.6	31.4	29.0	71.0
#50	0.3	43.6	40.3	59.7
#100	0.15	56.1	51.9	48.1
#200	0.075	67.3	62.3	37.7

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2195 Faraday Avenue, Suite K, Carlsbad CA 92008

Client: Anderson Consulting Engineers

Project Name: Ironwood

Job Number: GLO-71982.4

Date: 10/24/14

Boring Number: B-1

Depth: 15 ft.

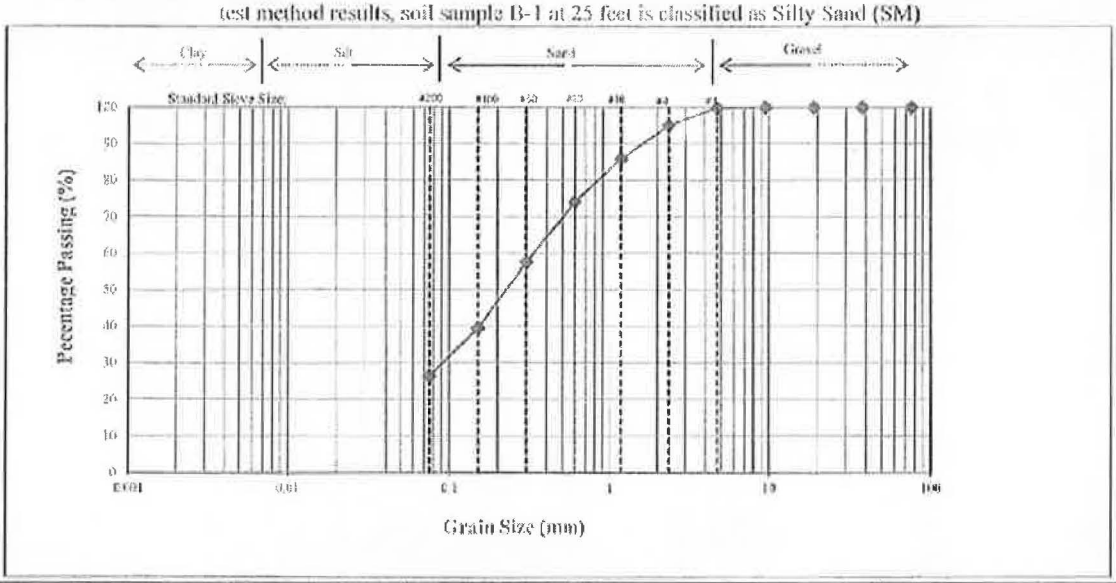
Soil Description: Brown Mottled Silty Sand SM

Tested by: B D

PARTICLE-SIZE ANALYSIS OF SOILS
ASTM METHOD D 422 (SIEVE ANALYSIS)

Sample :	B-1 @ 25 ft.	D10 (mm)	N/A
Total Weight (g)	123.6	D30 (mm)	0.08
Dry Weight (g)	115.8	D60 (mm)	0.33
Wet Sieve Weight (g)	85.3	Cu	N/A
Initial Moistnre (%)	6.7	Cc	N/A

According to ASTM D 2487 Unified Soil Classification System (USCS) and ASTM D 422 (Standard Test Method for Particle-Size Analysis) test method results, soil sample B-1 at 25 feet is classified as Silty Sand (SM)



Sieve Size (in)	Sieve Size (mm)	Cumulative Weight of dry soil (gm)	Percent Retained (%)	Percent Passing (%)
3"	76.2		0.0	100.0
1.5"	38.1		0.0	100.0
3/4"	19.05		0.0	100.0
3/8"	9.53		0.0	100.0
#4	4.75	0.2	0.2	99.8
#8	2.36	5.7	4.9	95.1
#16	1.18	16.4	14.2	85.8
#30	0.6	30.1	26.0	74.0
#50	0.3	49.2	42.5	57.5
#100	0.15	70.1	60.5	39.5
#200	0.075	85.3	73.7	26.3

Client: Anderson Consulting Engineers

Project Name: Ironwood

Job Number: GLO-71982.4

Date: 10/24/14

Boring Number: B-1

Depth: 25 ft.

Soil Description: Brown Mottled Silty Sand SM

Tested by: B D

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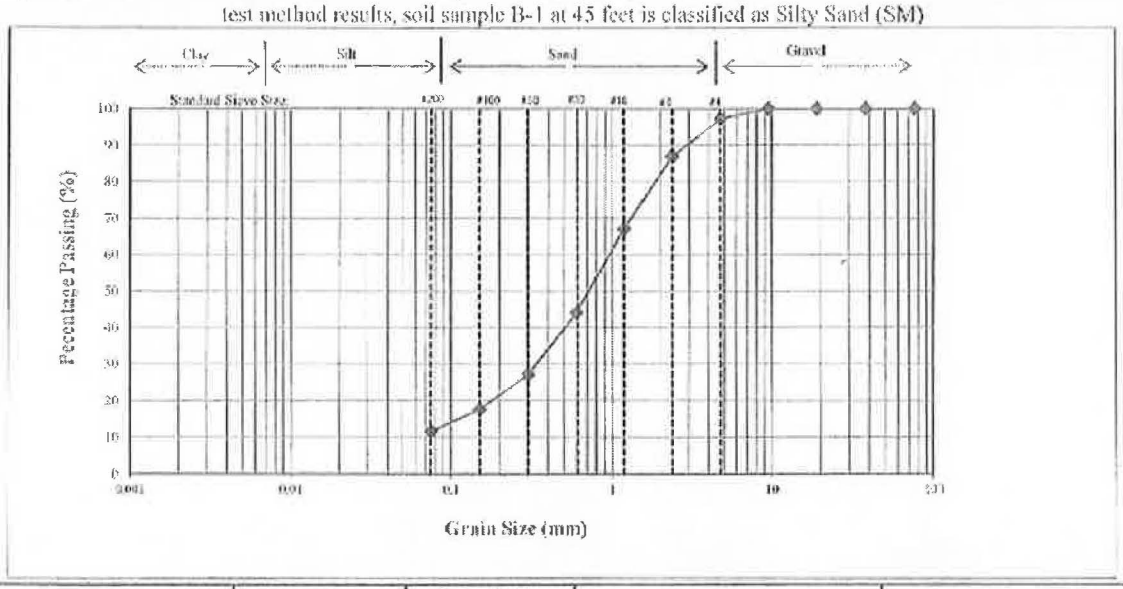
PARTICLE-SIZE ANALYSIS OF SOILS
ASTM METHOD D 422 (SIEVE ANALYSIS)

Sample :	B-1 @ 35 ft.		D10 (mm)	N/A
Total Weight (g)	126.5		D30 (mm)	0.30
Dry Weight (g)	122.1		D60 (mm)	0.90
Wet Sieve Weight (g)	108.0		Cu	N/A
Initial Moisture (%)	3.6		Cc	N/A

According to ASTM D 2487 Unified Soil Classification System (USCS) and ASTM D 422 (Standard Test Method for Particle-Size Analysis) test method results, soil sample B-1 at 45 feet is classified as Silty Sand (SM)



Sieve Size (in)	Sieve Size (mm)	Cumulative Weight of dry soil (gm)	Percent Retained (%)	Percent Passing (%)
3"	76.2		0.0	100.0
1.5"	38.1		0.0	100.0
3/4"	19.05		0.0	100.0
3/8"	9.53		0.0	100.0
#4	4.75	3.3	2.7	97.3
#8	2.36	15.9	13.0	87.0
#16	1.18	40.3	33.0	67.0
#30	0.6	68.2	55.9	44.1
#50	0.3	89.0	72.9	27.1
#100	0.15	100.5	82.3	17.7
#200	0.075	108.0	88.5	11.5

Client: Anderson Consulting Engineers

Project Name: Ironwood

Job Number: GLO-71982.4

Date: 10/24/14

Boring Number: B-1

Depth: 35 ft.

Soil Description: Gray-Green Mottled Silty Sand SM

Tested by: B D

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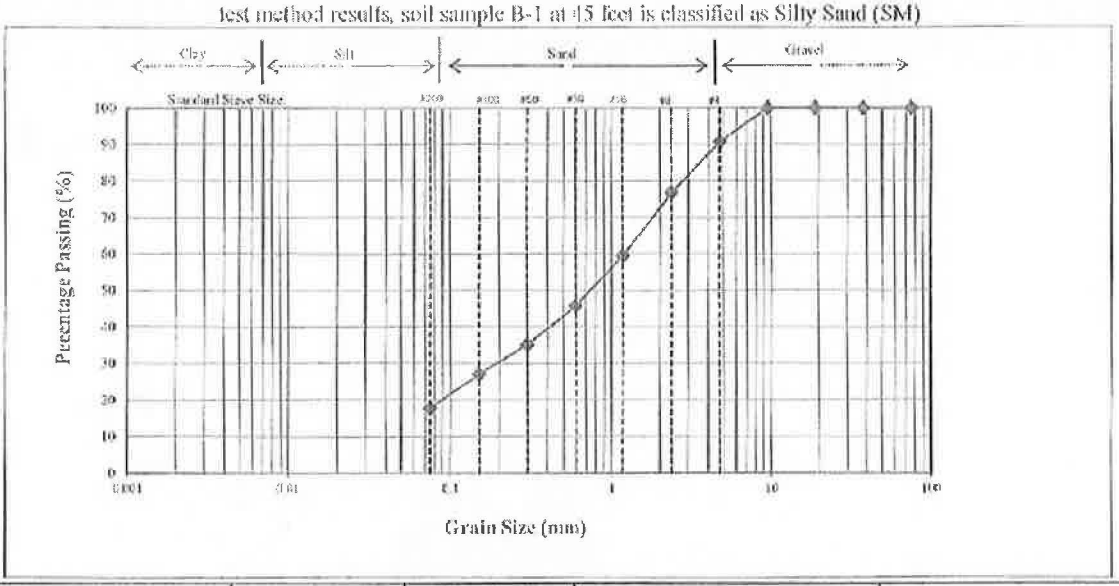
PARTICLE-SIZE ANALYSIS OF SOILS
ASTM METHOD D 422 (SIEVE ANALYSIS)

Sample :	B-1 @ 45 ft.	D10 (mm)	N/A
Total Weight (g)	120.2	D30 (mm)	0.20
Dry Weight (g)	112.2	D60 (mm)	1.18
Wet Sieve Weight (g)	92.3	Cu	N/A
Initial Moisture (%)	7.1	Cc	N/A

According to ASTM D 2487 Unified Soil Classification System (USCS) and ASTM D 422 (Standard Test Method for Particle-Size Analysis) test method results, soil sample B-1 at 45 feet is classified as **Silty Sand (SM)**



Sieve Size (in)	Sieve Size (mm)	Cumulative Weight of dry soil (gm)	Percent Retained (%)	Percent Passing (%)
3"	76.2		0.0	100.0
1.5"	38.1		0.0	100.0
3/4"	19.05		0.0	100.0
3/8"	9.53		0.0	100.0
#4	4.75	10.4	9.3	90.7
#8	2.36	26.1	23.3	76.7
#16	1.18	45.6	40.6	59.4
#30	0.6	60.9	54.3	45.7
#50	0.3	72.8	64.9	35.1
#100	0.15	81.9	73.0	27.0
#200	0.075	92.3	82.3	17.7

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Client: Anderson Consulting Engineers

Project Name: Ironwood

Job Number: GLO-71982.4

Date: 10/24/14

Boring Number: B-1

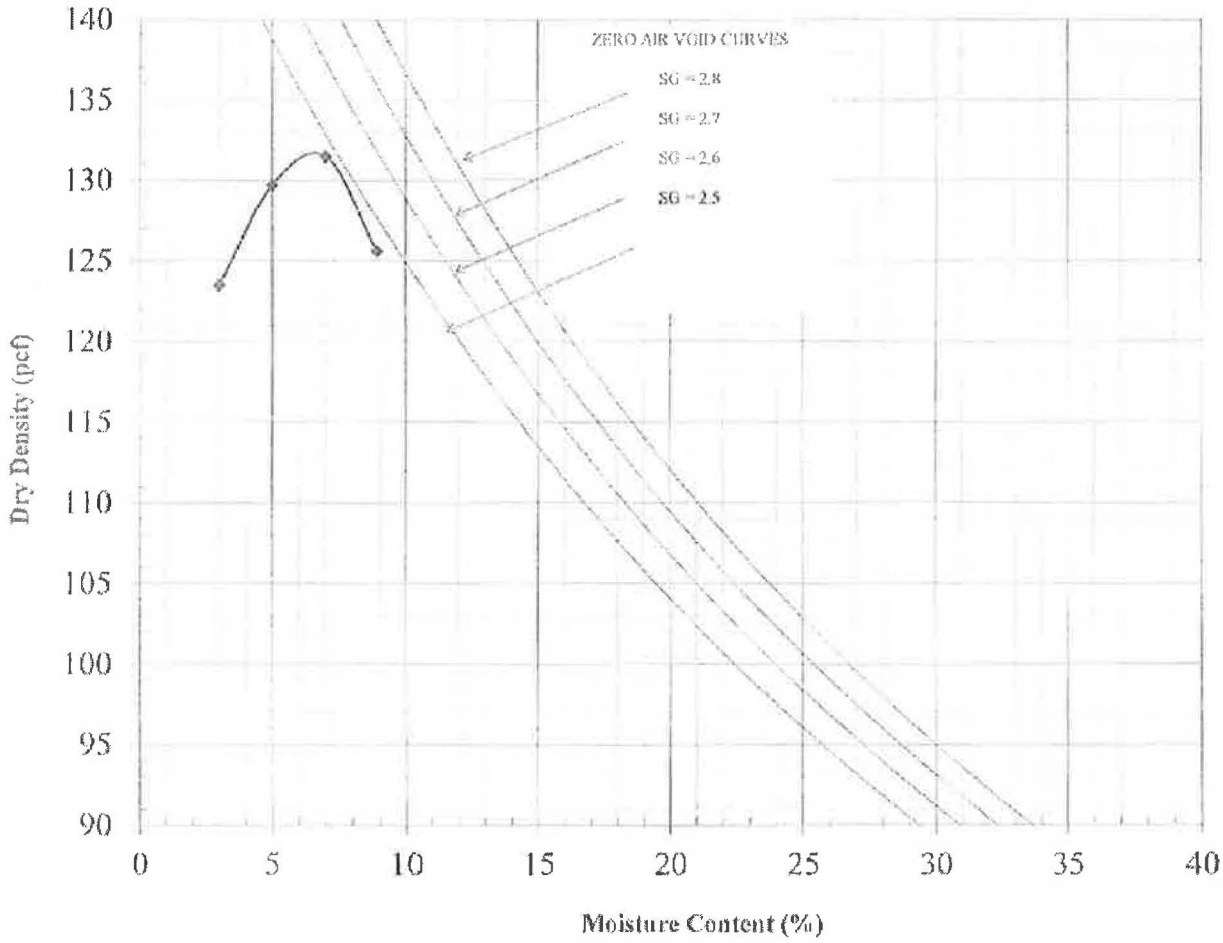
Depth: 45 ft.

Soil Description: Orange-Gray Mottled Silty Sand (SM)

Tested by: B D

LABORATORY COMPACTION ASTM D 1557

Sample	1	2	3	4
Mold and wet soil (lbs.)	8.550	8.850	9.000	8.870
Mold (lbs.)	4.310	4.310	4.310	4.310
Wet Soil (lbs.)	4.240	4.540	4.690	4.560
Wet Density (pcf)	127.20	136.20	140.70	136.80
Moisture (%)	3.0	5.0	7.0	8.9
Dry Density (pcf)	123.5	129.7	131.5	125.6



Maximum density 132.0 pcf @ 6.5% moisture

Geotechnical & Environmental Solutions

2195 Faraday, Suite K, Carlsbad, CA 92008

Client: Anderson Consulting Engineers

Project Name: Ironwood

Procedure: Method A

Job Number: GLO-71982.4

Date: 10/21/14

Boring Number: B-1

Location: 0-5 ft.

Soil Description: Brown Mottled Silty Sand SM

Tested by: B D

EXPANSION INDEX TEST

ASTM METHOD D 4829

B-1 @ 1-5 ft.

Moisture Content of Initial Sample		% Saturation of Re-molded Sample		Moisture Content of Final Sample	
Tare No. -	S6	Wt. of Soil and Ring (g) -	615.2	Wt. of Soil and Ring (g) -	641.3
Wet Weight and Tare (g) -	134.0	Ring Weight (g) -	199.0	Ring Weight (g) -	199.0
Dry Weight and Tare (g) -	128.8	Wet Weight of Soil (g) -	416.2	Wet Weight of Soil (g) -	442.3
Tare Weight (g) -	51.2	Dry Weight of Soil (g) -	390.1	Dry Weight of Soil (g) -	390.1
Water Loss (g) -	5.2	Volume of Ring (ft^3) -	0.0073	Weight of Water (g) -	52.2
Dry Weight (g) -	77.6	Dry Density (pcf) -	117.8	*Final Moisture (%)*	*13.4*
Initial Moisture (%) -	*6.7*	*Initital Saturation (%) -*	*42.1*	*Final Saturation (%) -*	*84.0*

Expansion Test - UBC (144 PSF)				
	Date	Time	Reading	
Add Weight	10/21/2014	10:20	0.000	
10 Minutes		10:30	0.000	Initial Reading
Add Water		11:30	0.002	
		3:00	0.002	
	10/22/2014	6:17	0.002	Final Reading

EImeasured	=	2
EI_{50}	=	0

Expansion Index, EI_{50}	Potential Expansion
0-20	Very Low
21-50	Low
51-90	Medium
91-130	High
>130	Very High

EEI Geotechnical & Environmental Solutions

2195 Faraday Avenue, Suite K, Carlsbad, CA 92008

Client: Anderson Consulting Engineers

Job Name: Ironwood

Job Number: GLO-71982.4

Date: 10/21/14

Boring Number: B-1

Depth: 0-5 ft.

Soil Description: Brown Mottled Silty Sand SM

Tested by: BD

DIRECT SHEAR TEST ASTM D 3080

Job Data	
Job No.:	GLO-71982.4
Client:	Anderson Consulting Engineers
Date:	10/22/14
Sample Data	
Sample:	B-1 @ 0 - 5 ft.
Remolded:	90 % Relative Compaction
Remarks:	Soaked Before Placing in Shear Box
Soil Description:	Brown Silty Sand (SM)

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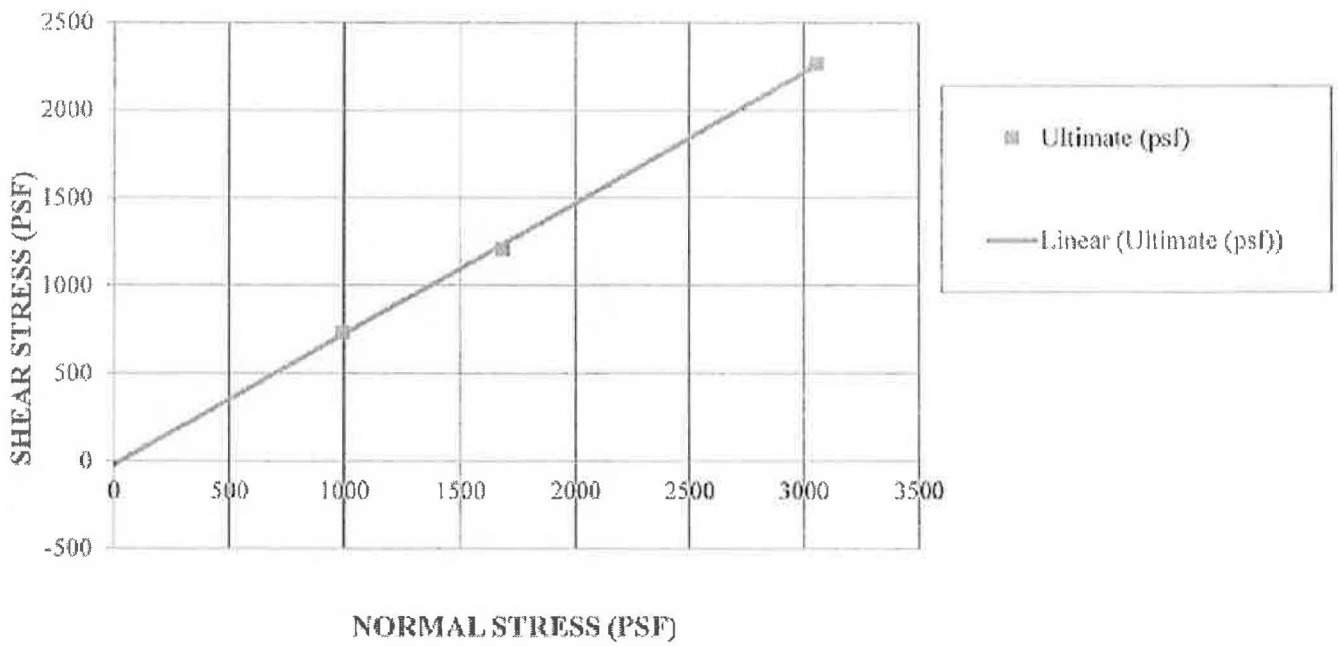
SHEAR TEST DIAGRAM



Test Results

	Phi		Cohesion	
Ultimate (psf)	37	degrees	-20	psf

Average Initial Moisture	6.5%
Average Dry Density	118.7 pcf
Average Final Moisture	12.9%

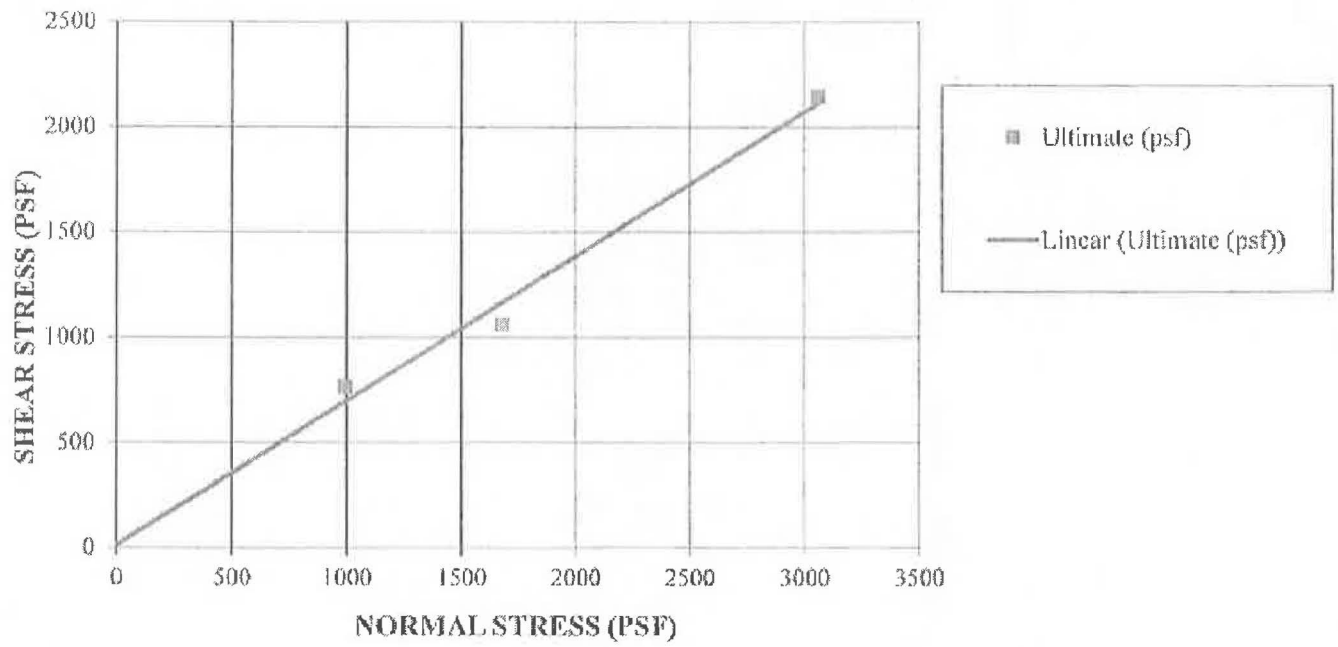
DIRECT SHEAR TEST ASTM D 3080

Job Data
Job No.: GLO-71982.4
Client: Anderson Consulting Engineers
Date: 10/24/14
Sample Data
Sample: B-1 @ 5 ft.
Remolded: 90 % Relative Compaction
Remarks: Soaked Before Placing in Shear Box
Soil Description: Brown Silty Sand (SM)

EEI Geotechnical & Environmental Solutions

2195 Faraday Avenue, Suite K, Carlsbad, CA 92008

SHEAR TEST DIAGRAM



Test Results

	Phi		Cohesion	
Ultimate (psf)	34	degrees	12	psf

Average Initial Moisture	2.7%
Average Dry Density	109.7 pcf
Average Final Moisture	15.4%

DIRECT SHEAR TEST ASTM D 3080

Job Data
Job No.: GLO-71982.4
Client: Anderson Consulting Engineers
Date: 10/24/14
Sample Data
Sample: B-2 @ 5 ft.
Remolded: 90 % Relative Compaction
Remarks: Soaked Before Placing in Shear Box
Soil Description: Orange-Brown Silty Sand (SM)

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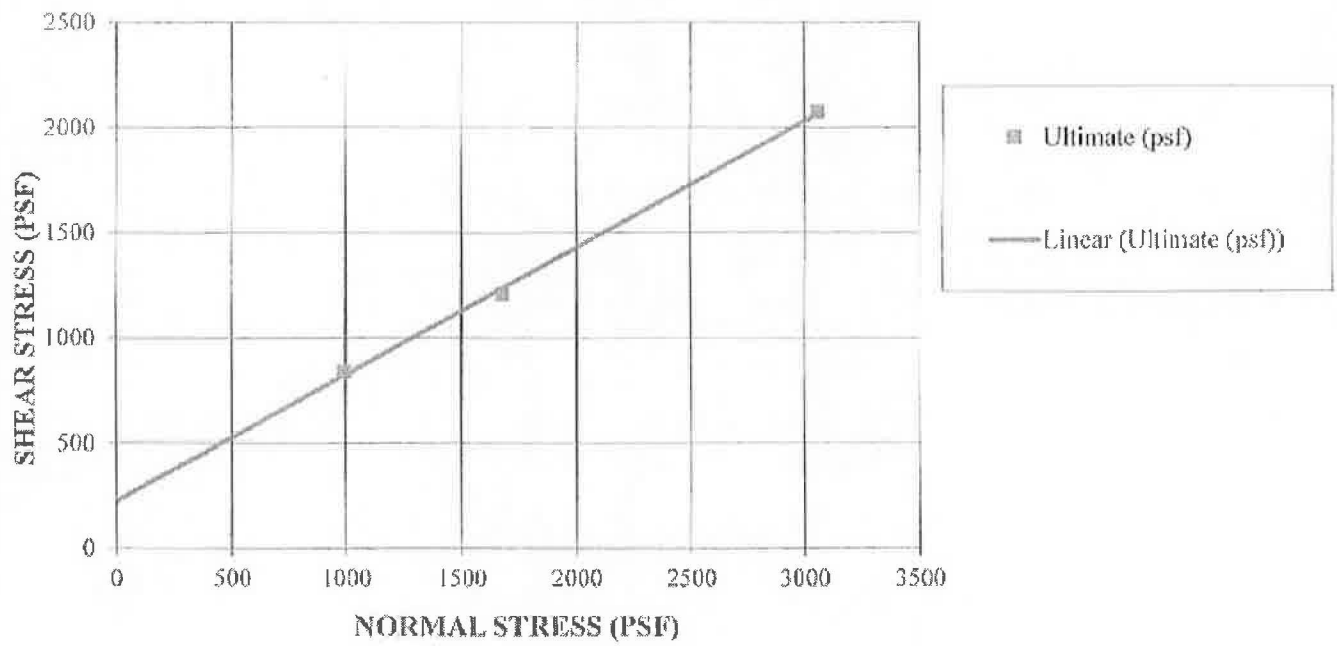
Geotechnical & Environmental Solutions

2195 Faraday Avenue, Suite K, Carlsbad, CA 92008

SHEAR TEST DIAGRAM



Test Results

	Phi		Cohesion	
Ultimate (psf)	31	degrees	225	psf

Average Initial Moisture	3.0%
Average Dry Density	112.9 pcf
Average Final Moisture	15.7%

L A B O R A T O R Y R E P O R T

Telephone (619) 425-1993 Fax 425-7917 Established 1928

C L A R K S O N L A B O R A T O R Y A N D S U P P L Y I N C.
350 Trousdale Dr. Chula Vista, Ca. 91910 www.clarksonlab.com
A N A L Y T I C A L A N D C O N S U L T I N G C H E M I S T S

Date: October 29, 2014
Purchase Order Number: GLO-71982-4
Sales Order Number: 24454
Account Number: EEI
To:

EEI Environmental Equalizers Inc
2195 Faraday Avenue Suite K
Carlsbad, CA 92008
Attention: Hector Estrella/Jeff Blake

Laboratory Number: S05463 Customers Phone: 760-431-3747

Sample Designation:

One soil sample received on 10/23/14 at 3:00pm, from Global - Ironwood Project# GLO-70982-4, marked as B-1 @ 0'-5' SM.

Analysis By California Test 643, 1999, Department of Transportation Division of Construction, Method for Estimating the Service Life of Steel Culverts.

pH 7.1

Water Added (ml)	Resistivity (ohm-cm)
10	13000
5	9500
5	7800
5	7300
5	6400
5	5800
5	5200
5	5500
5	5800

35 years to perforation for a 16 gauge metal culvert.
46 years to perforation for a 14 gauge metal culvert.
63 years to perforation for a 12 gauge metal culvert.
81 years to perforation for a 10 gauge metal culvert.
98 years to perforation for a 8 gauge metal culvert.

Water Soluble Sulfate	Calif. Test 417	0.005%
Water Soluble Chloride	Calif. Test 422	0.007%

Laura Torres

Laura Torres
LT/dbb

APPENDIX C
EARTHWORK AND GRADING GUIDELINES

EARTHWORK AND GRADING GUIDELINES

GENERAL

These guidelines present general procedures and recommendations for earthwork and grading as required on the approved grading plans, including preparation of areas to be filled, placement of fill and installation of subdrains and excavations. The recommendations contained in the geotechnical report are applicable to each specific project, are part of the earthwork and grading guidelines and would supersede the provisions contained hereafter in the case of conflict. Observations and/or testing performed by the consultant during the course of grading may result in revised recommendations which could supersede these guidelines or the recommendations contained in the geotechnical report. Figures A through O are provided at the back of this appendix, exhibiting generalized cross sections relating to these guidelines.

The contractor is responsible for the satisfactory completion of all earthworks in accordance with provisions of the project plans and specifications. The project soil engineer and engineering geologist (geotechnical consultant) or their representatives should provide observation and testing services, and geotechnical consultation throughout the duration of the project.

EARTHWORK OBSERVATIONS AND TESTING

Geotechnical Consultant

Prior to the commencement of grading, a qualified geotechnical consultant (a soil engineer and engineering geologist) should be employed for the purpose of observing earthwork procedures and testing the fills for conformance with the recommendations of the geotechnical report, the approved grading plans, and applicable grading codes and ordinances.

The geotechnical consultant should provide testing and observation so that determination may be made that the work is being completed as specified. It is the responsibility of the contractor to assist the consultant and keep them aware of work schedules and predicted changes, so that the consultant may schedule their personnel accordingly.

All removals, prepared ground to receive fill, key excavations, and subdrains should be observed and documented by the project engineering geologist and/or soil engineer prior to placing any fill. It is the contractor's responsibility to notify the engineering geologist and soil engineer when such areas are ready for observation.

Laboratory and Field Tests

Maximum dry density tests to determine the degree of compaction should be performed in accordance with American Standard Testing Materials test method ASTM designation D-1557-78. Random field compaction tests should be performed in accordance with test method ASTM designations D-1556-82, D-2937 or D-2922 & D-3017, at intervals of approximately two (2) feet of fill height per 10,000 sq. ft. or every one thousand cubic yards of fill placed. These criteria would vary depending on the soil conditions and the size of the project. The location and frequency of testing would be at the discretion of the geotechnical consultant

Contractor's Responsibility

All clearing, site preparation, and earthwork performed on the project should be conducted by the contractor, with observation by geotechnical consultants and staged approval by the appropriate governing agencies. It is the contractor's responsibility to prepare the ground surface to receive the fill to the satisfaction of the soil engineer, and to place, spread, moisture condition, mix and compact the fill in accordance with the recommendations of the soil engineer. The contractor should also remove all major deleterious material considered unsatisfactory by the soil engineer.

It is the sole responsibility of the contractor to provide adequate equipment and methods to accomplish the earthwork in accordance with applicable grading guidelines, codes or agency ordinances, and approved grading plans. Sufficient watering apparatus and compaction equipment should be provided by the contractor with due consideration for the fill material, rate of placement, and climatic conditions. If, in the opinion of the geotechnical consultant, unsatisfactory conditions such as questionable weather, excessive oversized rock, deleterious material or insufficient support equipment are resulting in a quality of work that is not acceptable, the consultant will inform the contractor, and the contractor is expected to rectify the conditions, and if necessary, stop work until conditions are satisfactory.

The contractor will properly grade all surfaces to maintain good drainage and prevent ponding of water. The contractor will take action to control surface water and to prevent erosion control measures that have been installed.

SITE PREPARATION

All vegetation including brush, trees, thick grasses, organic debris, and other deleterious material should be removed and disposed of offsite, and must be concluded prior to placing fill. Existing fill, soil, alluvium, colluvium, or rock materials determined by the soil engineer or engineering geologist as unsuitable for structural in-place support should be removed prior to fill placement. Depending upon the soil conditions, these materials may be reused as compacted fills. Any materials incorporated as part of the compacted fills should be approved by the soil engineer.

Any underground structures such as cesspools, cisterns, mining shafts, tunnels, septic tanks, wells, pipelines, or other structures not located prior to grading are to be removed or treated in a manner recommended by the soil engineer. Soft, dry, spongy, highly fractured, or otherwise unsuitable ground extending to such a depth that surface processing cannot adequately improve the condition should be over excavated down to firm ground and approved by the soil engineer before compaction and filling operations continue. Over excavated and processed soils which have been properly mixed and moisture-conditioned should be recompacted to the minimum relative compaction as specified in these guidelines.

Existing ground which is determined to be satisfactory for support of the fills should be scarified to a minimum depth of six (6) inches, or as directed by the soil engineer. After the scarified ground is brought to optimum moisture (or greater) and mixed, the materials should be compacted as specified herein. If the scarified zone is greater than 6 inches in depth, it may be necessary to remove the excess and place the material in lifts restricted to six (6) inches in compacted thickness.

Existing grind which is not satisfactory to support compacted fill should be over excavated as required in the geotechnical report or by the onsite soils engineer and/or engineering geologists. Scarification, discing, or other acceptable form of mixing should continue until the soils are broken down and free of large fragments or clods, until the working surface is reasonably uniform and free from ruts, hollows, hummocks, or other uneven features which would inhibit compaction as described above.

Where fills are to be placed on ground with slopes steeper than 5:1 (horizontal to vertical) gradient, the ground should be benched. The lowest bench, which will act as a key, should be a minimum of 12 feet wide and should be at least two (2) feet deep into competent material, approved by the soil engineer and/or engineering geologist. In fill over cut slope conditions, the recommended minimum width of the lowest bench or key is at least 15 feet with the key excavated on competent material, as designated by the Geotechnical Consultant. As a general rule, unless superseded by the Soil Engineer, the minimum width of fill keys should be approximately equal to one-half (½) the height of the slope.

Standard benching is typically four feet (minimum) vertically, exposing competent material. Benching may be used to remove unsuitable materials, although it is understood that the vertical height of the bench may exceed four feet. Pre stripping may be considered for removal of unsuitable materials in excess of four feet in thickness.

All areas to receive fill, including processed areas, removal areas, and toe of fill benches should be observed and approved by the soil engineer and/or engineering geologist prior to placement of fill. Fills may then be properly placed and compacted until design grades are attained.

COMPACTED FILLS

Earth materials imported or excavated on the property may be utilized as fill provided that each soil type has been accepted by the soil engineer. These materials should be free of roots, tree branches, other organic matter or other deleterious materials. All unsuitable materials should be removed from the fill as directed by the soil engineer. Soils of poor gradation, undesirable expansion potential, or substandard strength characteristics may be designated unsuitable by the consultant and may require mixing with other earth materials to serve as a satisfactory fill material.

Fill materials generated from benching operations should be dispersed throughout the fill area. Benching operations should not result in the benched material being placed only within a single equipment width away from the fill/bedrock contact.

Oversized materials, defined as rock or other irreducible materials with a maximum size exceeding 12 inches in one dimension, should not be buried or placed in fills unless the location of materials and disposal methods are specifically approved by the soil engineer. Oversized material should be taken offsite or placed in accordance with recommendations of the soil engineer in areas designated as suitable for rock disposal. Oversized material should not be placed vertically within 10 feet of finish grade or horizontally within 20 feet of slope faces.

To facilitate trenching, rock should not be placed within the range of foundation excavations or future utilities unless specifically approved by the soil engineer and/or the representative developers.

If import fill material is required for grading, representative samples of the material should be analyzed in the laboratory by the soil engineer to determine its physical properties. If any material other than that previously analyzed is imported to the fill or encountered during grading, analysis of this material should be conducted by the soil engineer as soon as practical.

Fill material should be placed in areas prepared to receive fill in near-horizontal layers that should not exceed six (6) inches compacted in thickness. The soil engineer may approve thicker lifts if testing indicates the grading procedures are such that adequate compaction is being achieved. Each layer should be spread evenly and mixed to attain uniformity of material and moisture suitable for compaction.

Fill materials at moisture content less than optimum should be watered and mixed, and "wet" fill materials should be aerated by scarification, or should be mixed with drier material. Moisture conditioning and mixing of fill materials should continue until the fill materials have uniform moisture content at or above optimum moisture.

After each layer has been evenly spread, moisture-conditioned and mixed, it should be uniformly compacted to a minimum of 90 percent of maximum density as determined by ASTM test designation, D 1557-78, or as otherwise recommended by the soil engineer. Compaction equipment should be adequately sized and should be reliable to efficiently achieve the required degree of compaction.

Where tests indicate that the density of any layer of fill, or portion thereof, is below the required relative compaction or improper moisture content, the particular layer or portion will be reworked until the required density and/or moisture content has been attained. No additional fill will be placed in an area until the last placed lift of fill has been tested and found to meet the density and moisture requirements, and is approved by the soil engineer.

Compaction of slopes should be accomplished by over-building the outside edge a minimum of three (3) feet horizontally, and subsequently trimming back to the finish design slope configuration. Testing will be performed as the fill is horizontally placed to evaluate compaction as the fill core is being developed. Special efforts may be necessary to attain the specified compaction in the fill slope zone. Final slope shaping should be performed by trimming and removing loose materials with appropriate equipment. A final determination of fill slope compaction should be based on observation and/or testing of the finished slope face.

If an alternative to over-building and cutting back the compacted fill slope is selected, then additional efforts should be made to achieve the required compaction in the outer 10 feet of each lift of fill by undertaking the following:

- Equipment consisting of a heavy short-shanked sheepsfoot should be used to roll (horizontal) parallel to the slopes continuously as fill is placed. The sheepsfoot roller should also be used to roll perpendicular to the slopes, and extend out over the slope to provide adequate compaction to the face slope.

- Loose fill should not be spilled out over the face of the slope as each lift is compacted. Any loose fill spilled over a previously completed slope face should be trimmed off or be subject to re-rolling.

- Field compaction tests will be made in the outer two (2) to five (5) feet of the slope at two (2) to three (3) foot vertical intervals, subsequent to compaction operations.

- After completion of the slope, the slope face should be shaped with a small dozer and then re-rolled with a sheepsfoot to achieve compaction to near the slope face. Subsequent to testing to verify compaction, the slopes should be grid-rolled to achieve adequate compaction to the slope face. Final testing should be used to confirm compaction after grid rolling.

- Where testing indicates less than adequate compaction, the contractor will be responsible to process, moisture condition, mix and recompact the slope materials as necessary to achieve compaction. Additional testing should be performed to verify compaction.

- Erosion control and drainage devices should be designed by the project civil engineer in compliance with the ordinances of the controlling governmental agencies, and/or in accordance with the recommendations of the soil engineer or engineering geologist.

EXCAVATIONS

Excavations and cut slopes should be observed and mapped during grading by the engineering geologist. If directed by the engineering geologist, further excavations or over-excavation and refilling of cut areas should be performed. When fills over cut slopes are to be graded, the cut portion of the slope should be observed by the engineering geologist prior to placement of the overlying fill portion of the slope. The engineering geologist should observe all cut slopes and should be notified by the contractor when cut slopes are started.

If, during the course of grading, unanticipated adverse or potentially adverse geologic conditions are encountered, the engineering geologist and soil engineer should investigate, evaluate and make recommendations to mitigate (or limit) these conditions. The need for cut slope buttressing or stabilizing should be based on as-grading evaluations by the engineering geologist, whether anticipated previously or not.

Unless otherwise specified in soil and geological reports, no cut slopes should be excavated higher or steeper than that allowed by the ordinances of controlling governmental agencies. Additionally, short-term stability of temporary cut slopes is the contractor's responsibility.

Erosion control and drainage devices should be designed by the project civil engineer and should be constructed in compliance with the ordinances of the controlling governmental agencies, and/or in accordance with the recommendations of the soil engineer or engineering geologist.

SUBDRAIN INSTALLATION

Subdrains should be installed in accordance with the approved embedment material, alignment and details indicated by the geotechnical consultant. Subdrain locations or construction materials should not be changed or modified without approval of the geotechnical consultant. The soil engineer and/or engineering geologist may recommend and direct changes in subdrain line, grade and drain material in the field, pending exposed conditions. The location of constructed subdrains should be recorded by the project civil engineer.

COMPLETION

Consultation, observation and testing by the geotechnical consultant should be completed during grading operations in order to state an opinion that all cut and filled areas are graded in accordance with the approved project specifications.

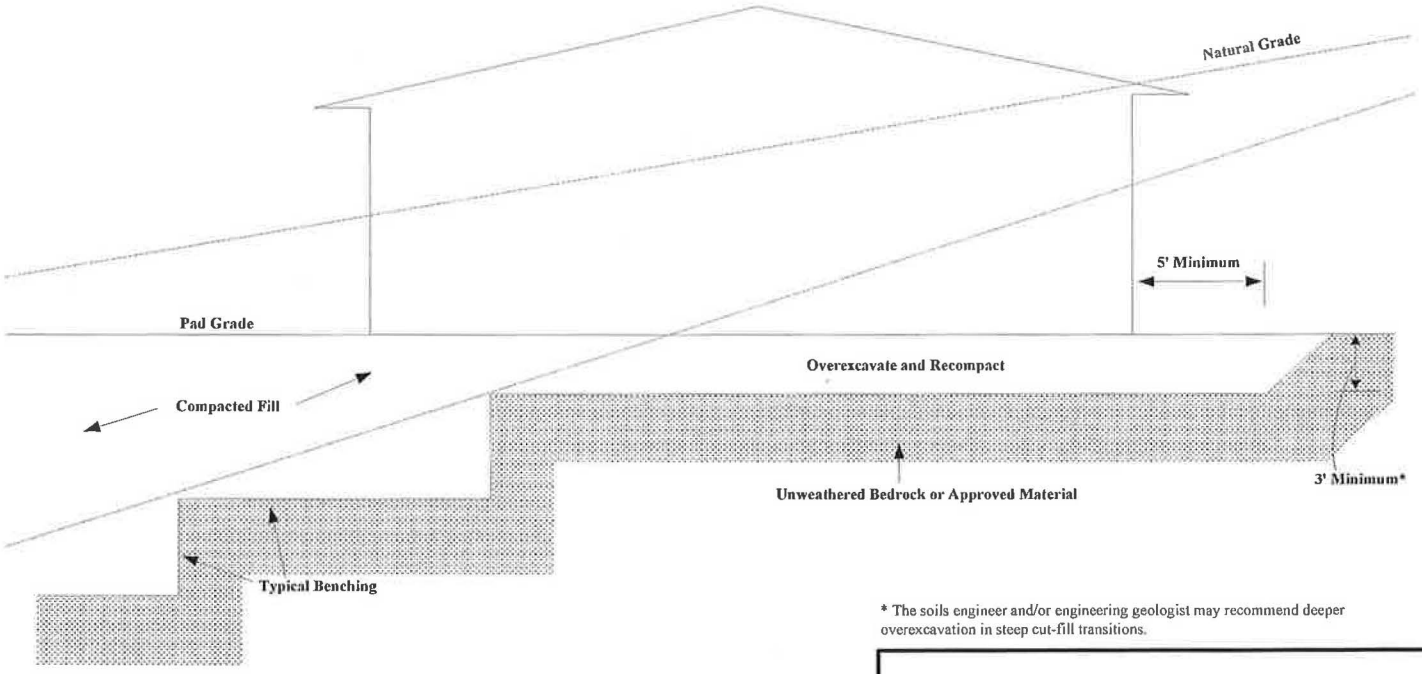
After completion of grading and after the soil engineer and engineering geologist have finished their observations, final reports should be submitted subject to review by the controlling governmental agencies. No additional grading should be undertaken without prior notification of the soil engineer and/or engineering geologist.

All finished cut and fill slopes should be protected from erosion, including but not limited to planting in accordance with the plan design specifications and/or as recommended by a landscape architect. Such protection and/or planning should be undertaken as soon as possible after completion of grading.

ATTACHMENTS

Figure A – Transition Lot Detail Cut Lot
Figure B – Transition Lot Detail Cut - Fill
Figure C – Rock Disposal Pits
Figure D – Detail for Fill Slope Toeing out on a Flat Alluviated Canyon
Figure E – Removal Adjacent to Existing Fill
Figure F – Daylight Cut Lot Detail
Figure G – Skin Fill of Natural Ground
Figure H – Typical Stabilization Buttress Fill Design
Figure I – Stabilization Fill for Unstable Material Exposed in Portion of Cut Slope
Figure J – Fill Over Cut Detail
Figure K – Fill Over Natural Detail
Figure L – Oversize Rock Disposal
Figure M – Canyon Subdrain Detail
Figure N – Canyon Subdrain Alternate Details
Figure O – Typical Stabilization Buttress Subdrain Detail
Figure P – Retaining Wall Backfill

TRANSITION LOT DETAIL
CUT LOT – MATERIAL TYPE TRANSITION



* The soils engineer and/or engineering geologist may recommend deeper overexcavation in steep cut-fill transitions.

Note: Figure not to scale

EARTHWORK AND GRADING GUIDELINES
TRANSITION LOT DETAIL
CUT LOT – MATERIAL TYPE TRANSITION

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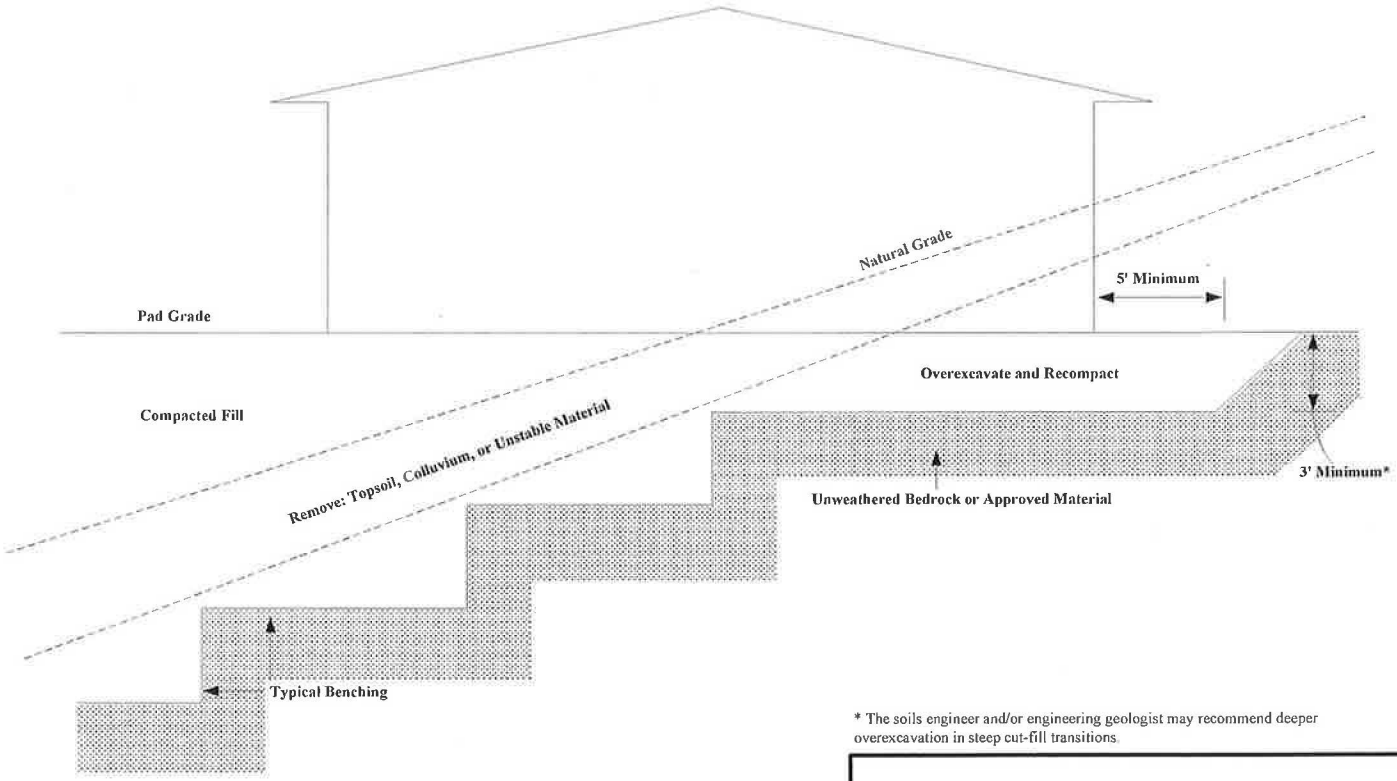
FIGURE A

TRANSITION LOT DETAIL
CUT – FILL – DAYLIGHT TRANSITION



* The soils engineer and/or engineering geologist may recommend deeper overexcavation in steep cut-fill transitions.

Note: Figure not to scale

EARTHWORK AND GRADING GUIDELINES
TRANSITION LOT DETAIL
CUT – FILL – DAYLIGHT TRANSITION

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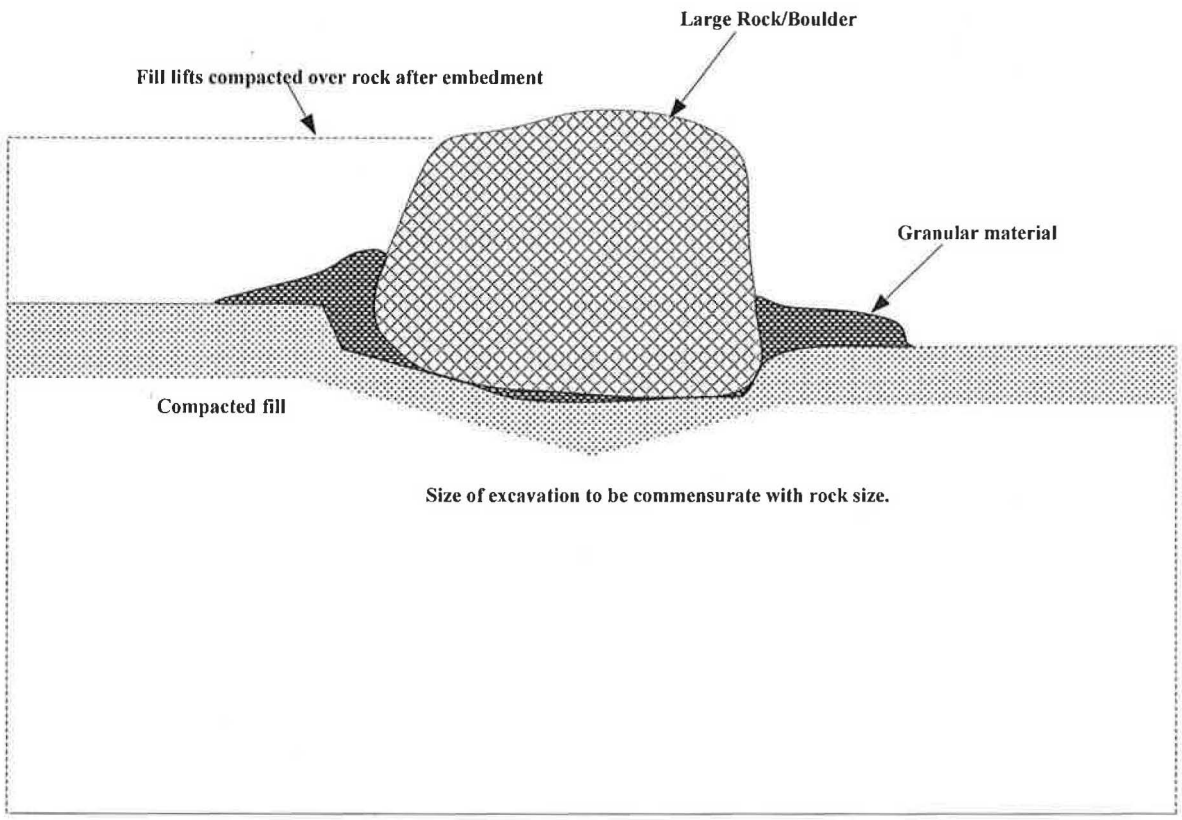
FIGURE B

ROCK DISPOSAL PITS



Note:
(1) Large rock is defined as having a diameter larger than 3 feet in maximum size.
(2) Pit shall be excavated into compacted fill to a depth equal to half of the rock size.
(3) Granular soil shall be pushed into the pit and then flooded around the rock using a sheepsfoot to help with compaction.
(4) A minimum of 3 feet of compacted fill should be laid over each pit.
(5) Pits shall have at least 15 feet of separation between one another, horizontally.
(6) Pits shall be placed at least 20 feet from any fill slope.
(7) Pits shall be used only in deep fill areas.

Note: Figure not to scale

EARTHWORK AND GRADING GUIDELINES
ROCK DISPOSAL PITS

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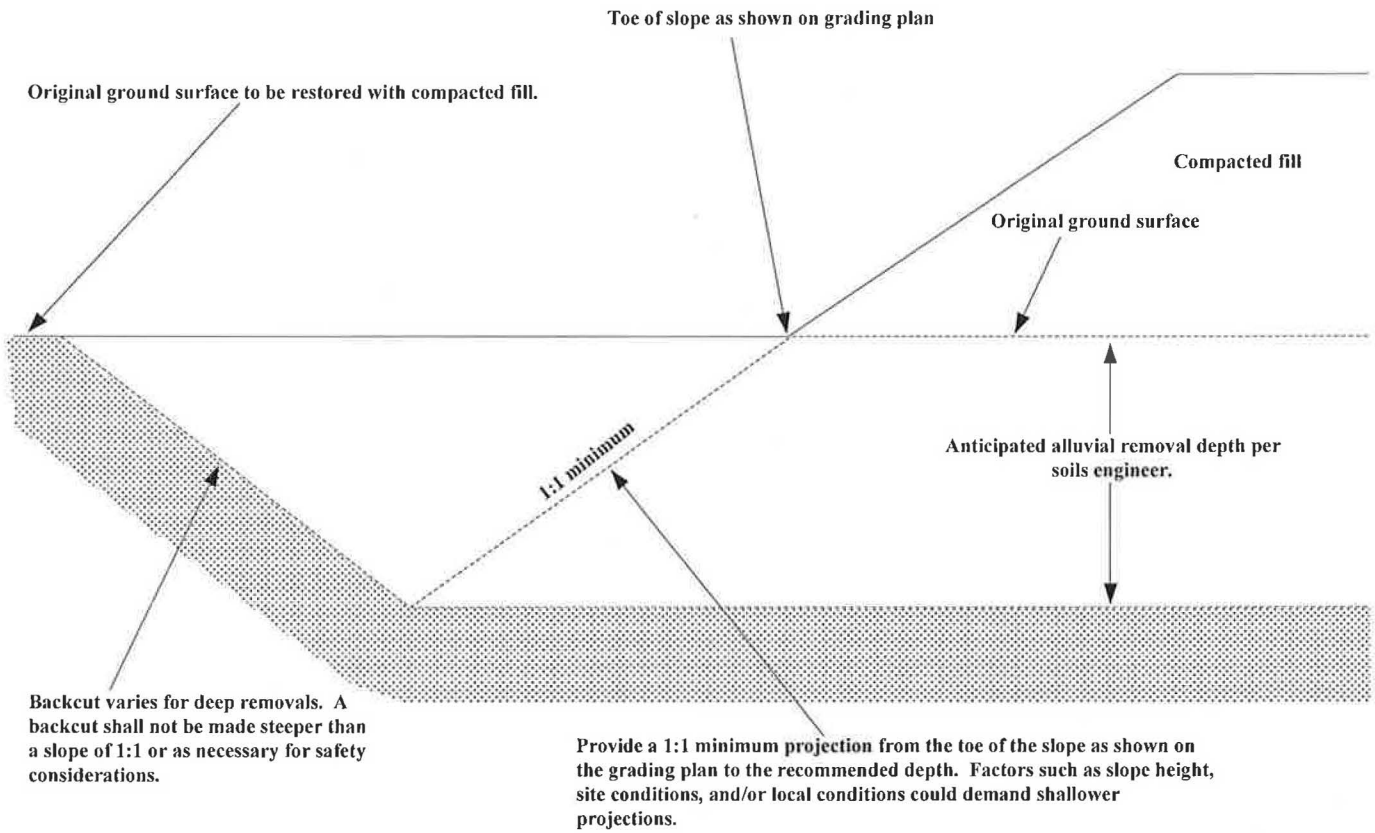
FIGURE C

DETAIL FOR FILL SLOPE TOEING OUT ON FLAT ALLUVIATED CANYON



Note: Figure not to scale

EARTHWORK AND GRADING GUIDELINES
DETAIL FOR FILL SLOPE TOEING OUT ON A FLAT ALLUVIATED CANYON

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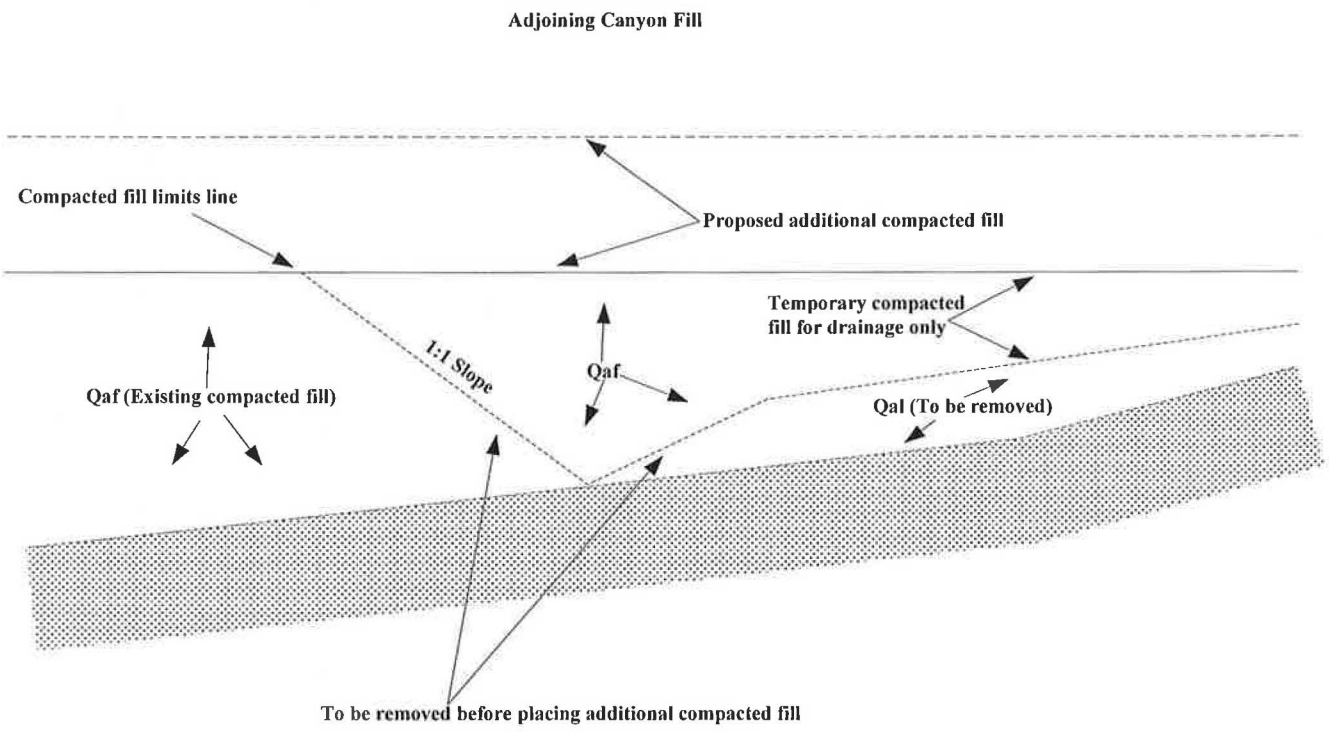
FIGURE D

REMOVAL ADJACENT TO EXISTING FILL



Legend		
Qaf -	Artificial Fill	
Qal -	Alluvium	

Note: Figure not to scale

EARTHWORK AND GRADING GUIDELINES
REMOVAL ADJACENT TO EXISTING FILL

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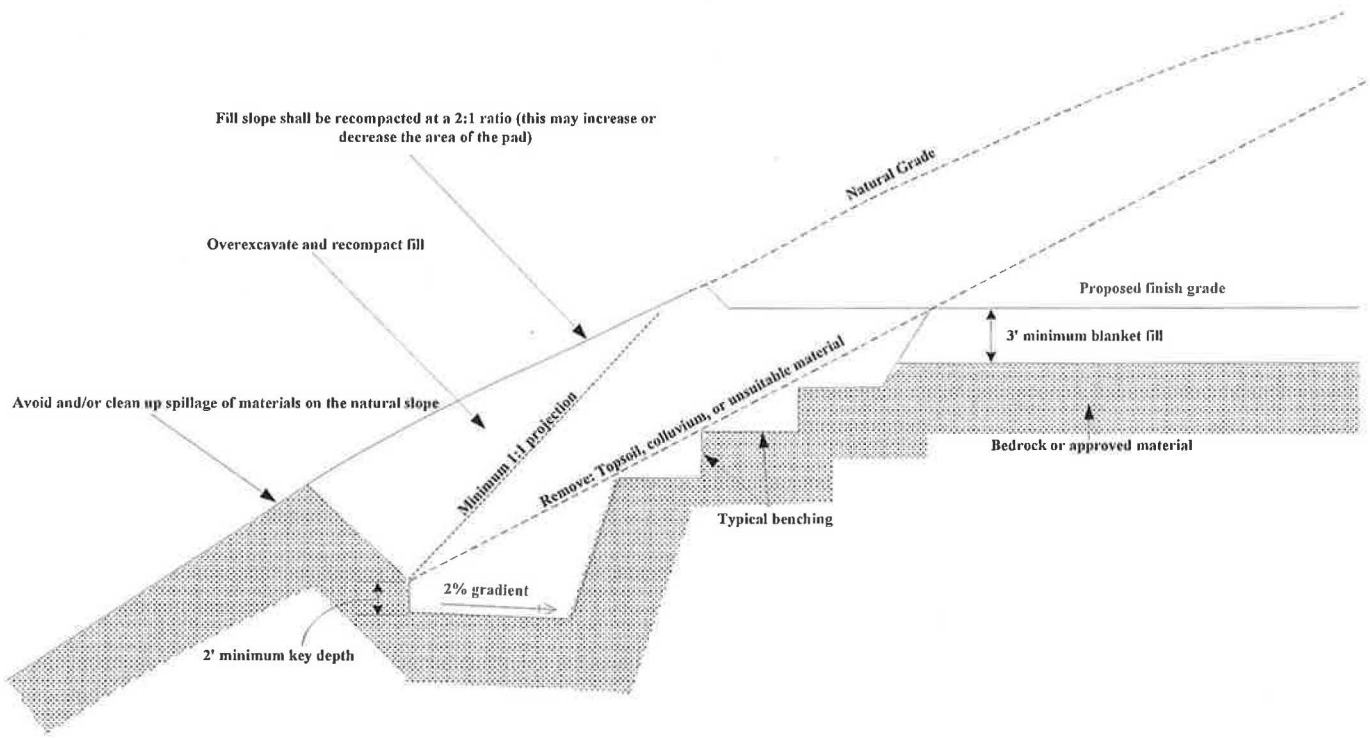
FIGURE E

DAYLIGHT CUT LOT DETAIL



Note: (1) Subdrain and key width requirements shall be determined based on exposed subsurface conditions and the thickness of overburden.

(2) Pad overexcavation and recompaction shall be completed if determined as necessary by the soils engineer and/or engineering geologist.

Note: Figure not to scale

EARTHWORK AND GRADING GUIDELINES
DAYLIGHT CUT LOT DETAIL

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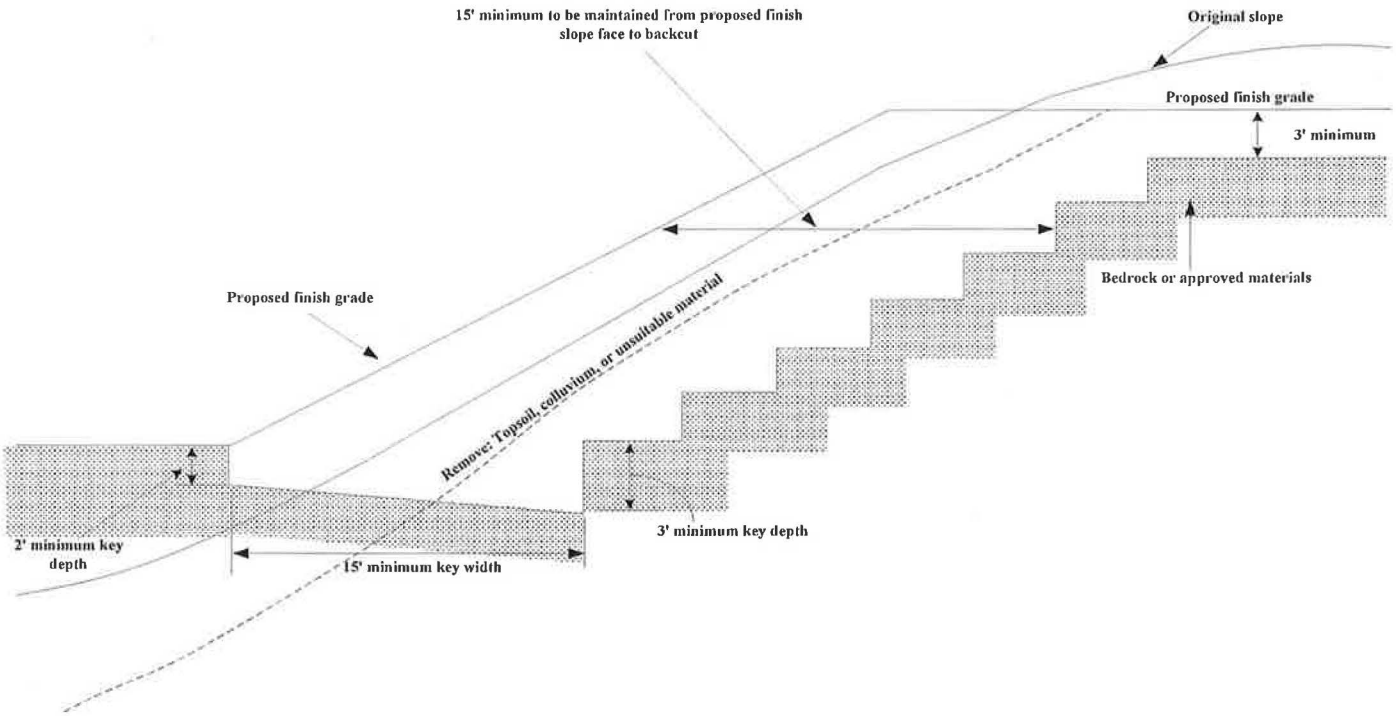
FIGURE F

SKIN FILL OF NATURAL GROUND



Note: (1) The need and disposition of drains will be determined by the soils engineer and/or engineering geologist based on site conditions.

(2) Pad overexcavation and recompaction shall be completed if determined as necessary by the soils engineer and/or engineering geologist.

Note: Figure not to scale

EARTHWORK AND GRADING GUIDELINES
SKIN FILL OF NATURAL GROUND

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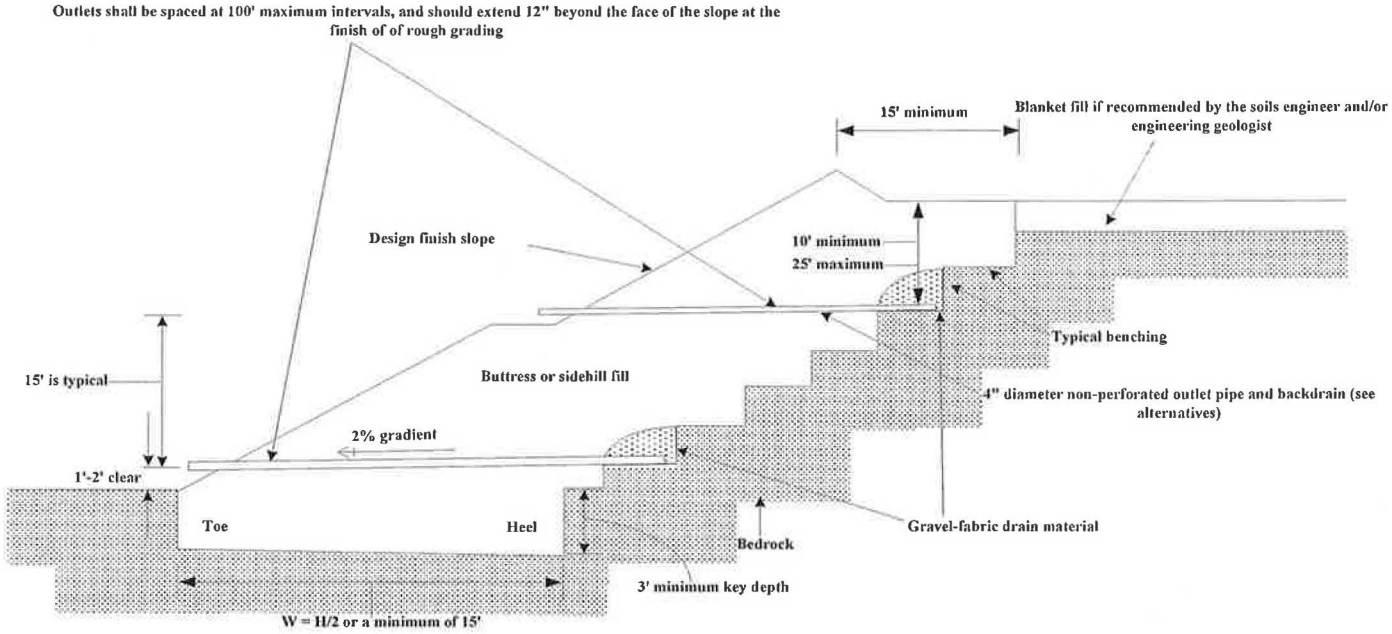
FIGURE G

TYPICAL STABILIZATION BUTTRESS FILL DESIGN



Note: Figure not to scale

EARTHWORK AND GRADING GUIDELINES
TYPICAL STABILIZATION BUTTRESS FILL DESIGN

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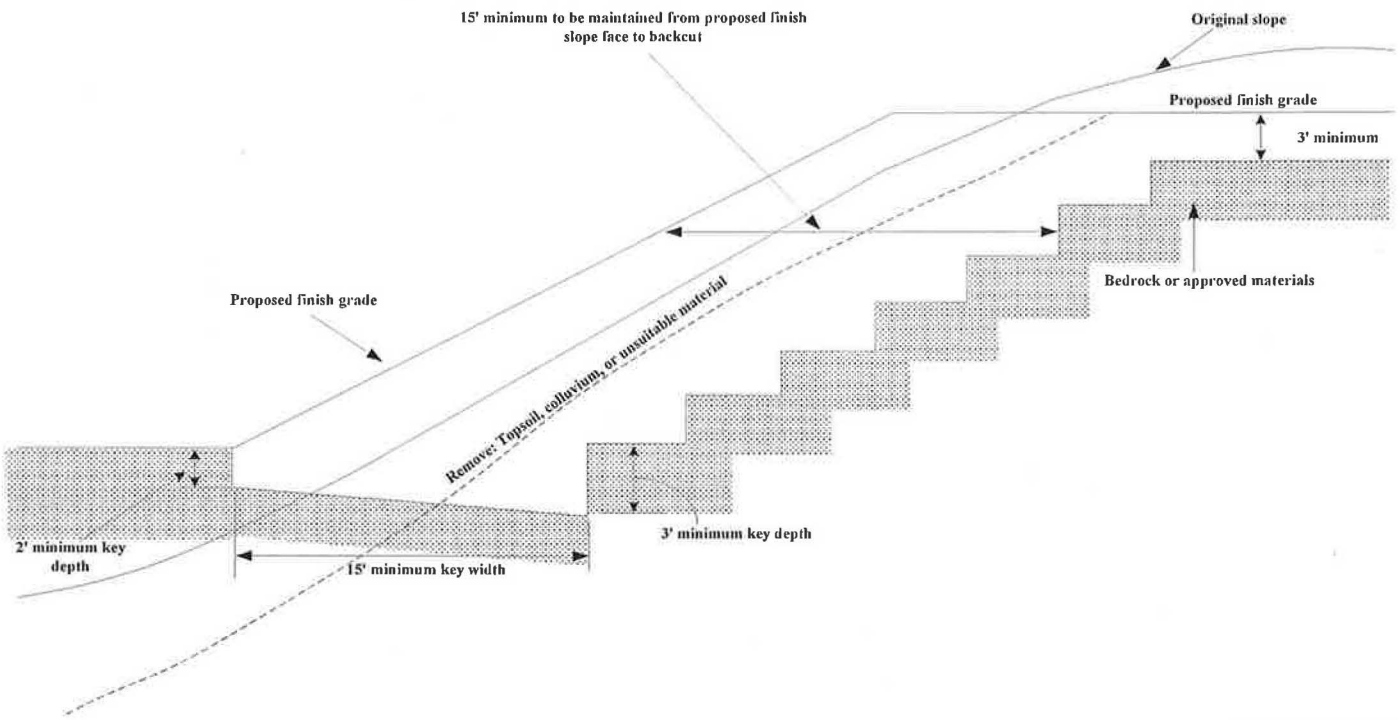
FIGURE H

SKIN FILL OF NATURAL GROUND



Note: (1) The need and disposition of drains will be determined by the soils engineer and/or engineering geologist based on site conditions.

(2) Pad overexcavation and recompaction shall be completed if determined as necessary by the soils engineer and/or engineering geologist.

Note: Figure not to scale

EARTHWORK AND GRADING GUIDELINES
SKIN FILL OF NATURAL GROUND

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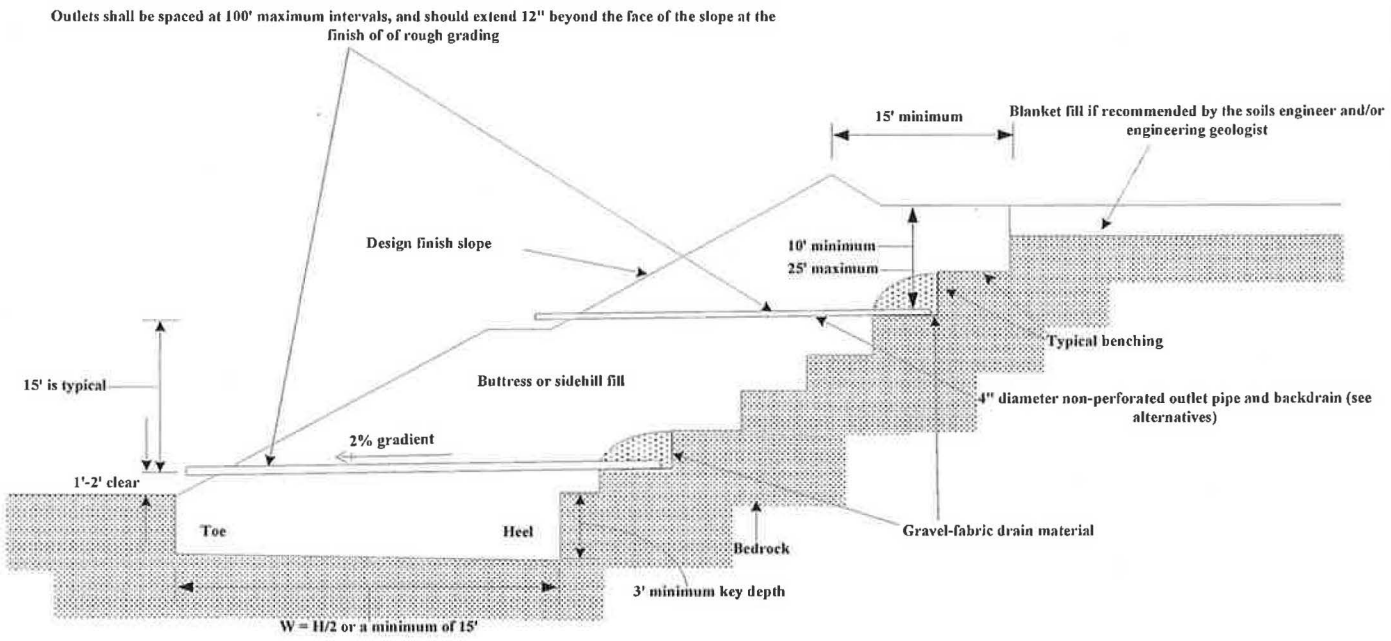
FIGURE G

TYPICAL STABILIZATION BUTTRESS FILL DESIGN



Note: Figure not to scale

EARTHWORK AND GRADING GUIDELINES
TYPICAL STABILIZATION BUTTRESS FILL DESIGN

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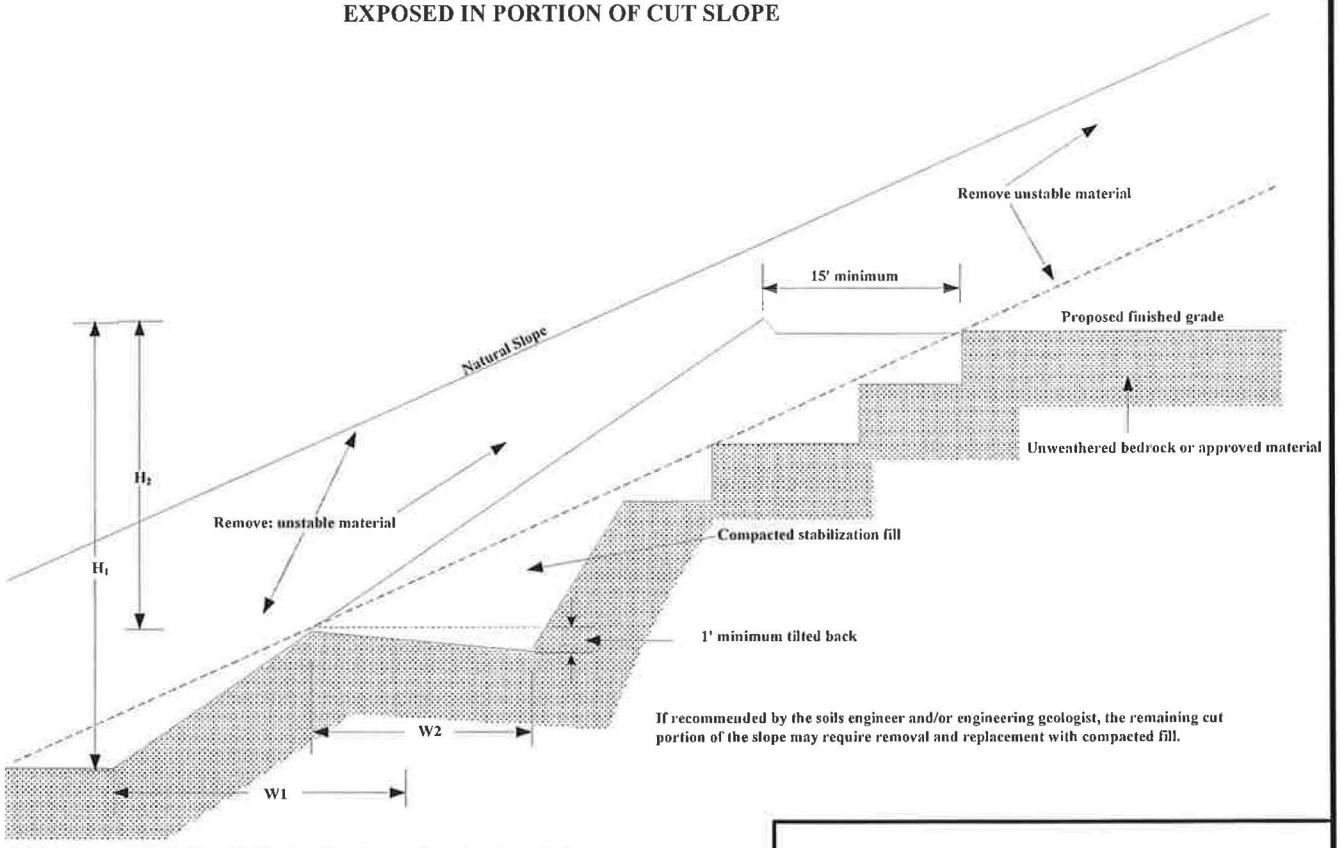
FIGURE H

STABILIZATION FILL FOR UNSTABLE MATERIAL EXPOSED IN PORTION OF CUT SLOPE



Note: (1) Subdrains are required only if specified by the soils engineer and/or engineering geologist.
(2) "W" shall be the equipment width (15') for slope heights less than 25 feet. For slopes greater than 25 feet "W" shall be determined by the project soils engineer and/or the engineering geologist. "W" shall never be less than H/2.

Note: Figure not to scale

EARTHWORK AND GRADING GUIDELINES
STABILIZATION FILL FOR UNSTABLE MATERIAL EXPOSED IN PORTION OF CUT SLOPE

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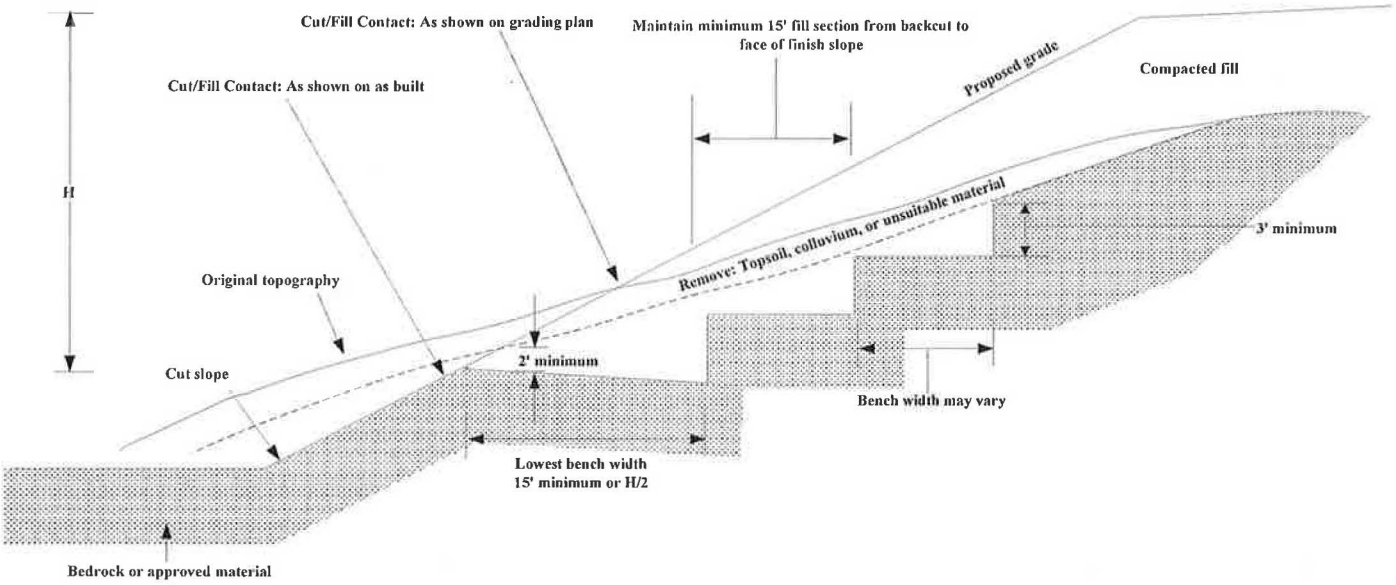
FIGURE I

FILL OVER CUT DETAIL



Note: The cut sectioin shall be excavated and evaluated by the soils engineer/engineering geologist prior to constructing the fill portion.

Note: Figure not to scale

EARTHWORK AND GRADING GUIDELINES
FILL OVER CUT DETAIL

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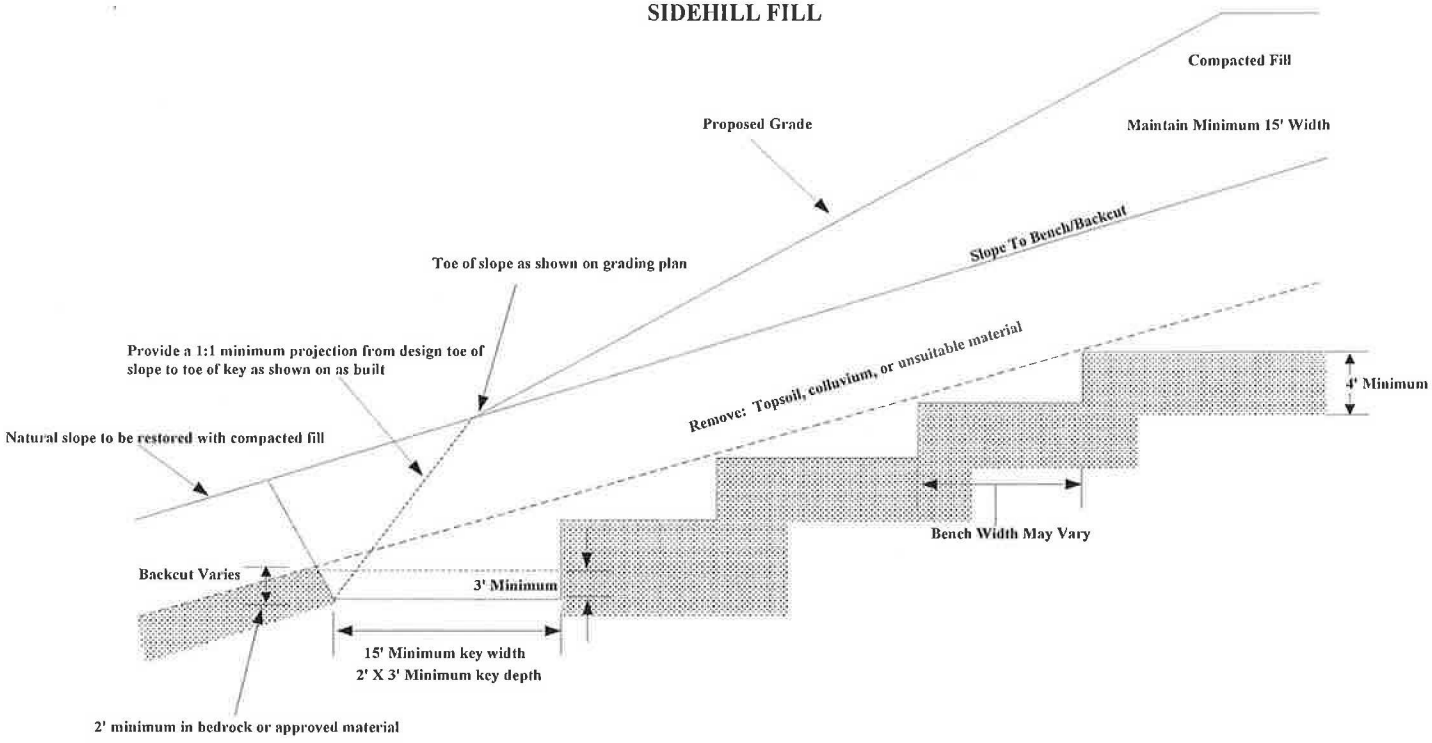
FIGURE J

FILL OVER NATURAL DETAIL
SIDEHILL FILL



Note: (1) Special recommendations shall be provided by the soils engineer/engineering geologist where the natural slope approaches or exceeds the design slope ratio.
(2) The need for and disposition of drains would be determined by the soils engineer/engineering geologist based upon exposed conditions.

Note: Figures not to scale

EARTHWORK AND GRADING GUIDELINES
FILL OVER NATURAL DETAIL
SIDEHILL FILL

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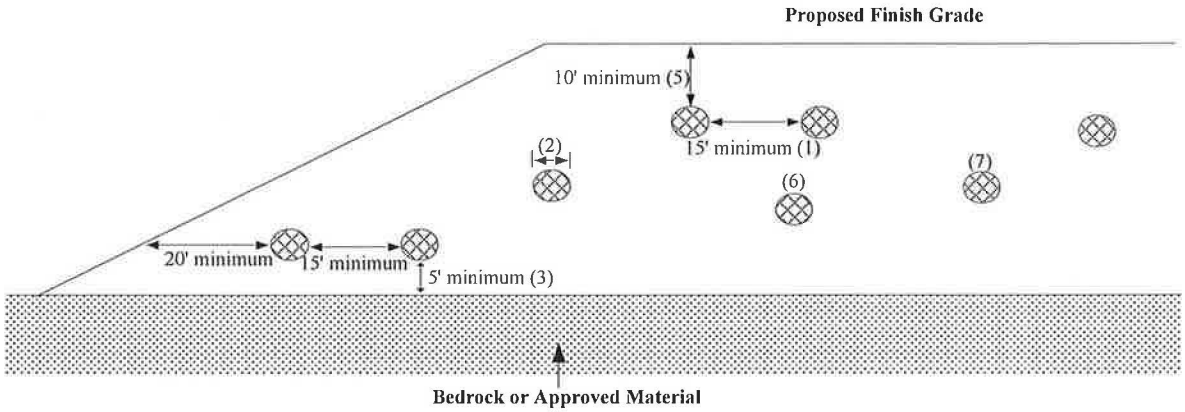
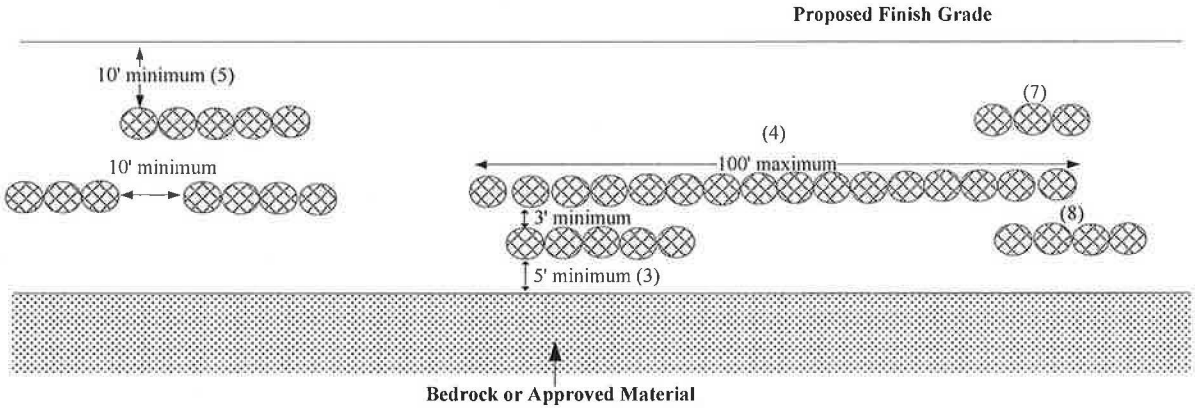
FIGURE K

OVERSIZE ROCK DISPOSAL

View Normal to Slope Face



View Parallel to Slope Face



Note:
(1) One Equipment width or a minimum of 15 feet.
(2) Height and width may vary depending on rock size and type of equipment used. Length of windrow shall be no greater than 100 feet maximum.
(3) If approved by the soils engineer and/or engineering geologist.
(4) Orientation of windrows may vary but shall be as recommended by the soils engineer and/or engineering geologist. Unless recommended staggering of windrows is not necessary.
(5) Areas shall be cleared for utility trenches, foundations, and swimming pools.
(6) Voids in windrows shall be filled by flooding granular soil into place. Granular soil shall be any soil which has a unified soil classification system (Universal Building Code (UBC) 29-1). Designation of SM, SP, SW, GP, or GW.
(7) After fill between windrows is placed and compacted with the lift of fill covering windrow, windrow shall be proof rolled with a D-9 dozer or equivalent.
(8) Oversized rock is defined as larger than 12", and less than 4 feet in size.

Approximate Scale: 1" = 30'

Note: All distances are approximate

EARTHWORK AND GRADING GUIDELINES
OVERSIZE ROCK DISPOSAL

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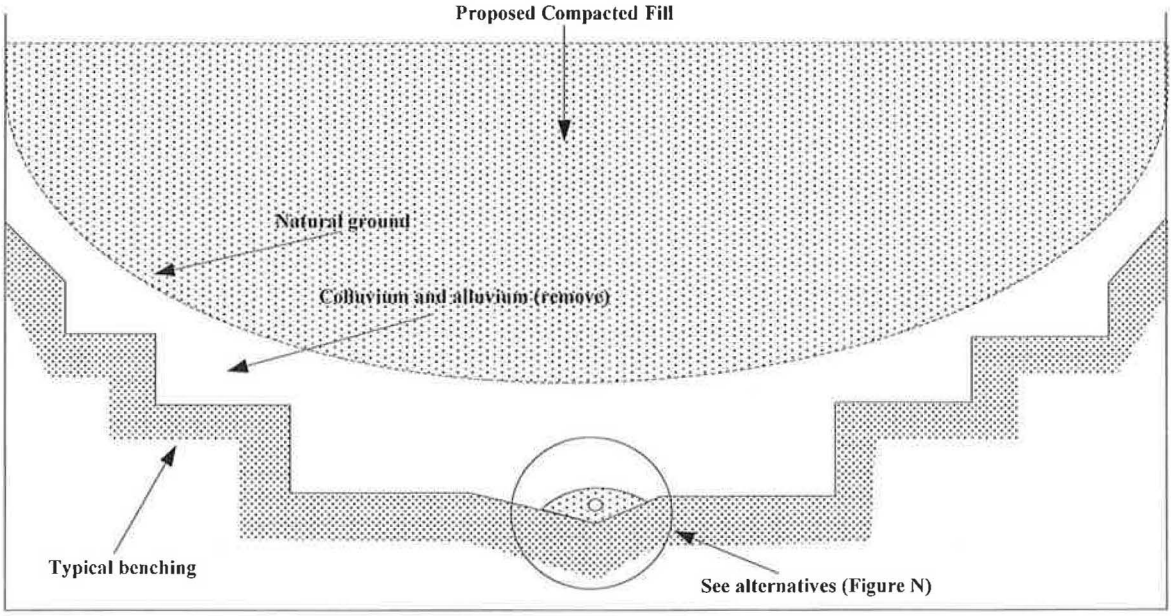
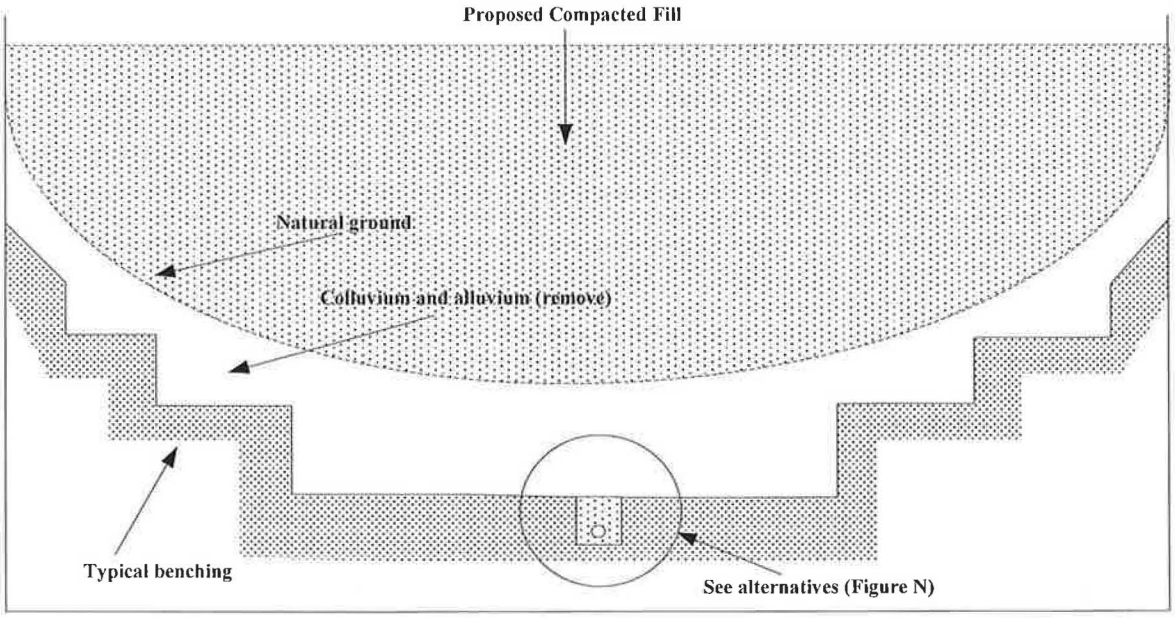
FIGURE L

CANYON SUBDRAIN DETAIL

Type A



Type B



Note: Alternatives, locations, and extent of subdrains should be determined by the soils engineer and/or engineering geologist during actual grading.

EARTHWORK AND GRADING GUIDELINES
CANYON SUBDRAIN DETAIL

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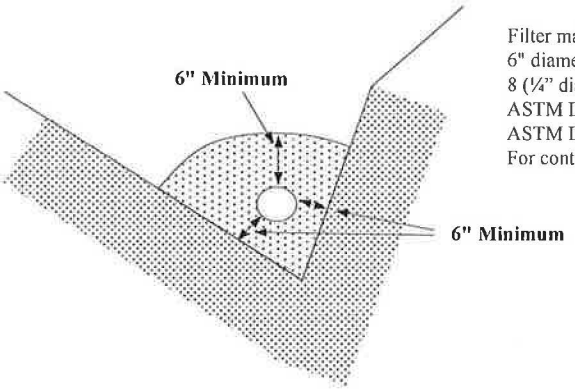
FIGURE M

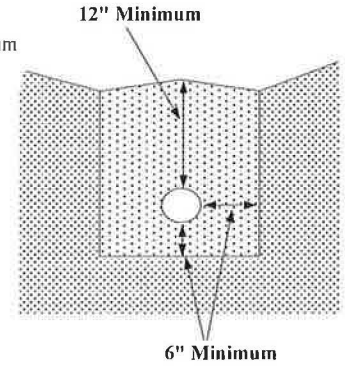
Note: Figures not to scale

CANYON SUBDRAIN ALTERNATE DETAILS

Alternate 1: Perforated Pipe and Filter Material



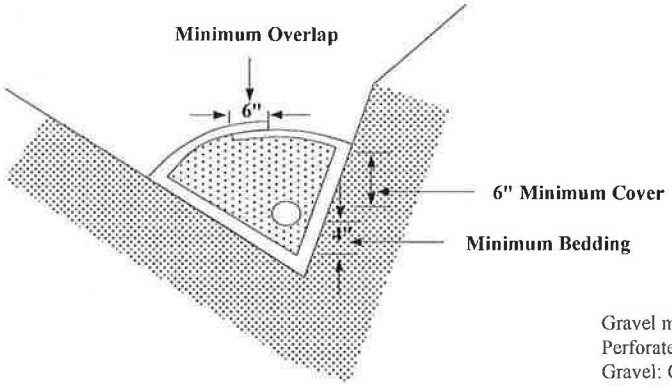
Filter material: Minimum volume of 9 feet^3/linear foot.
6" diameter ABS or PVC pipe or approved substitute with minimum 8 (¼" diameter) perforations per linear foot in bottom half of pipe.
ASTM D 2751, SDR 35 or ASTM D 1527, Schedule 40.
ASTM D 3034, SDR 35 or ASTM D 1785, Schedule 40.
For continuous run in excess of 500 feet use 8" diameter pipe.

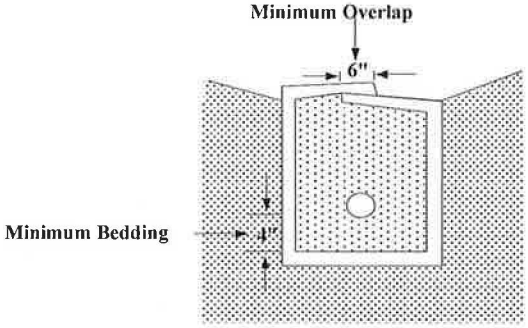


Filter Material

Sieve Size	Percent Passing
1"	100
¾"	90-100
3/8"	40-100
No. 4	25-40
No. 8	18-33
No. 30	5-15
No. 50	0-7
No. 200	0-3

Alternate 2: Perforated Pipe, Gravel and Filter Fabric





Gravel material 9 feet^3/linear foot.
Perforated pipe: see alternate 1.
Gravel: Clean ¾" rock or approved substitute.
Filter Fabric: Mirafi 140 or approved substitute.

Note: Figures not to scale

EARTHWORK AND GRADING GUIDELINES
CANYON SUBDRAIN ALTERNATE DETAILS

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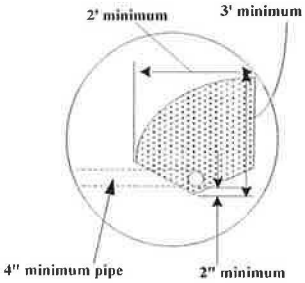
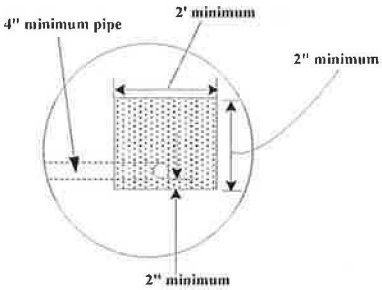
FIGURE N

TYPICAL STABILIZATION BUTTRESS SUBDRAIN DETAIL





Filter Material: Minimum of 5 ft^3/linear foot of pipe or 4 ft^3/linear foot of pipe when placed in square cut trench.

Alternative In Lieu Of Filter Material: Gravel may be encased in approved filter fabric. Filter fabric shall be mirafi 140 or equivalent. Filter fabric shall be lapped a minimum of 12" on all joints.

Minimum 4" Diameter Pipe: ABS-ASTM D-2751, SDR 35 or ASTM D-1527 schedule 40 PVC-ASTM D-3034, SDR 35 or ASTM D-1785 schedule 40 with a crushing strength of 1,000 pounds minimum, and a minimum of 8 uniformly spaced perforations per foot of pipe installed with perforations at bottom of pipe. Provide cap at upstream end of pipe. Slope at 2% to outlet pipe. Outlet pipe shall be connected to the subdrain pipe with tee or elbow.

Note:
(1) Trench for outlet pipes shall be backfilled with onsite soil.
(2) Backdrains and lateral drains shall be located at the elevation of every bench drain. First drain shall be located at the elevation just above the lower lot grade. Additional drains may be required at the discretion of the soils engineer and/or engineering geologist.

Filter Material – Shall be of the following specification or an approved equivalent:

Filter Material

Sieve Size	Percent Passing
1"	100
¾"	90-100
3/8"	40-100
No. 4	25-40
No. 8	18-33
No. 30	5-15
No. 50	0-7
No. 200	0-3

Gravel - Shall be of the following specification or an approved equivalent:

Filter Material

Sieve Size	Percent Passing
1½"	100
No. 4	50
No. 200	8

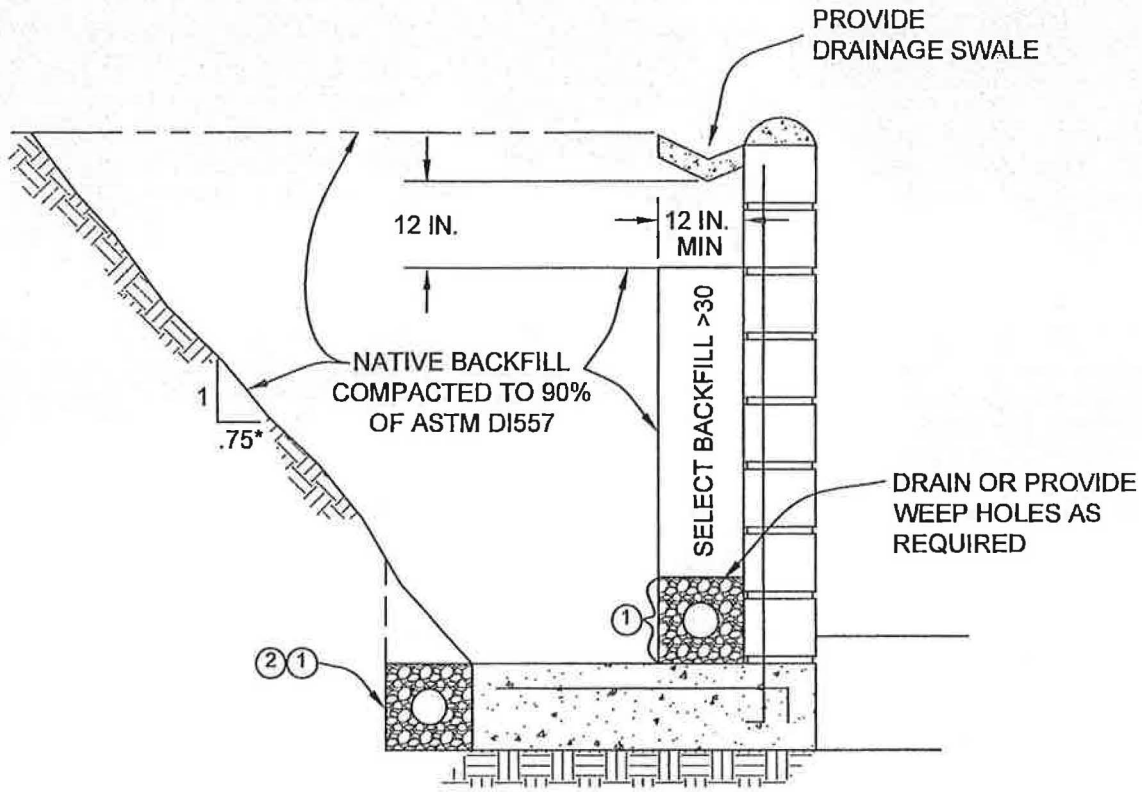
Sand equivalent: Minimum of 50

Note: Figures not to scale

EARTHWORK AND GRADING GUIDELINES
TYPICAL STABILIZATION BUTTRESS SUBDRAIN DETAIL

EEI
Expertise . . Service . . *Solutions*

FIGURE O



* OR AS REQUIRED FOR SAFETY

NOTES

1. 4-INCH PERFORATED PVC SCHEDULE 40 OR APPROVED ALTERNATE. PLACE PERFORATION DOWN AND SURROUND WITH A MINIMUM OF 1 CUBIC FOOT PER LINEAL FOOT (1 FT. /FT.) OF 3/4 INCH ROCK OR APPROVED ALTERNATE AND WRAPPED IN FILTER FABRIC.
2. PLACE DRAIN AS SHOWN WHERE MOISTURE MIGRATION THROUGH THE WALL IS UNDESIRABLE.

NOTE: FIGURE NOT TO SCALE

EARTHWORK & GRADING GUIDELINES

TYPICAL RETAINING WALL BACKFILL

EEI

Expertise...Service...*Solutions*

FIGURE P

Moreno Valley Ironwood Rockfall Investigation

City of Moreno Valley
Riverside County, California

March 15, 2016

Project No. KGT 16-05

KANE GeoTech, Inc.
Geoengineering Consultants

Moreno Valley Ironwood Rockfall Investigation

Riverside County, California

March 15, 2016

Project No. KGT 16-05

Prepared for:

Global Investment and Development, LLC
Administrative and Project Assistant
3470 Wilshire Blvd. Ste. 1020
Los Angeles, CA 90010

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Suite 1115
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www.kanegeotech.com

Moreno Valley Ironwood Rockfall Investigation

Riverside County, California

Project No. KGT 16-05

1. INTRODUCTION

1.1 General

KANE GeoTech, Inc. (KANE GeoTech) was retained by Global Investment and Development, LLC to investigate any potential rockfall hazard affecting the location of a planned residences in the City of Moreno Valley on Ironwood Avenue, located in Riverside County, California. This report was prepared by KANE GeoTech to provide detailed information on the assessment of potential rockfall hazards at the Project site. The Project location, Tentative Tract No. 37001, is shown in Figure 1 with an aerial overview of the site in Figure 2. A lot map of the planned development is included as Appendix A.

1.2 Purpose

The purpose of this report is to present the results of the field investigation and rockfall analyses performed to assess the potential rockfall hazards at the Project site.

2. SCOPE OF WORK

2.1 Scope

The scope of services provided by KANE GeoTech included the following:

1. *Literature Review*. KANE GeoTech reviewed existing geotechnical information, reports, and maps pertinent to the project area.

2. *Site Investigation.* KANE GeoTech visited the site to evaluate the site conditions and gather data necessary to perform a rockfall analyses.

3. *Engineering Analysis.* KANE GeoTech performed a rockfall analyses using the Colorado Rockfall Simulation Program to assess the potential rockfall hazard present at the site.

Figure 1. Project location.

Figure 2. Aerial overview of project site.

4. *Report of Findings.* KANE GeoTech provides this Report of Findings stamped by a Licensed California Civil Engineer experienced in rockfall. This Report contains a summary of the site investigation and engineering analysis.

3. SITE DESCRIPTION

3.1 Overview

The Project site, Tentative Tract No. 37001, is a planned residential area located at approximately latitude 33°56'56'' N and longitude 117°11'16'' W, in Moreno Valley, Riverside County, California. Several of the planned residences are on a flat area at the base of a rocky outcrop. This slope is the source of any potential rockfall. At the time of our visit, most of the area was covered by short grass.

3.2 Regional Geology

The project site is generally located south of the San Gregorio Mountains in Southern California, northeast of the Santa Ana Mountains, and southeast of Box Springs Mountain. This area is a part of the Perris Block which is bounded by the Elsinore Fault, located to the southwest and the Jacinto Fault to the northeast (City, 2006). The area is a part of the Southern California batholith that is composed of felsic rich, intrusive igneous bedrock.

3.3 Site Geology

The site is located at the southeastern base of the Kalmia Hills in the northern section of the Perris Block. The bedrock present at the site is mainly biotite-hornblende tonalite not associated with an specific pluton (USGS, 1967). The tonalite is grey, medium-grained and in some areas contains mafic inclusions.

The slope adjacent to the planned resident locations contains spheroidally weathered, large, rounded boulders. These boulders are composed of the tonalite described above. The boulders are heavily weathered and when broken down, form the sandy soil present at the site. The majority of these boulders are embedded in the sediment or are actually exposed bedrock. There are some areas of exposed bedrock indicating the depth to bedrock, although varies, is shallow.

Figure 3. Large embedded boulders and exposed bedrock outcrops.

4. SITE EVALUATION

4.1 Background

KANE GeoTech visited the project site on February 2, 2016. The purpose of the visit and field investigation was to collect data required for analyses to determine the nature and extent of any rockfall hazards. Details and data were recorded with photos and in a field notebook. Rock type, boulder size, and probable paths of rockfall, and soil cover were noted.

4.2 Observations

The areas of concern consist of embedded rounded boulders. Approximately 95% of them are embedded in a soft sediment, Figure 3. These boulders weather into smaller spheroidal boulders with a maximum diameter of 1-ft, Figure 4. These 1-ft boulders were observed sporadically throughout the project site. There is minimal vegetation present at the site and consists of shrubs and grasses. Six different slopes have been observed and are presented in Appendix B.

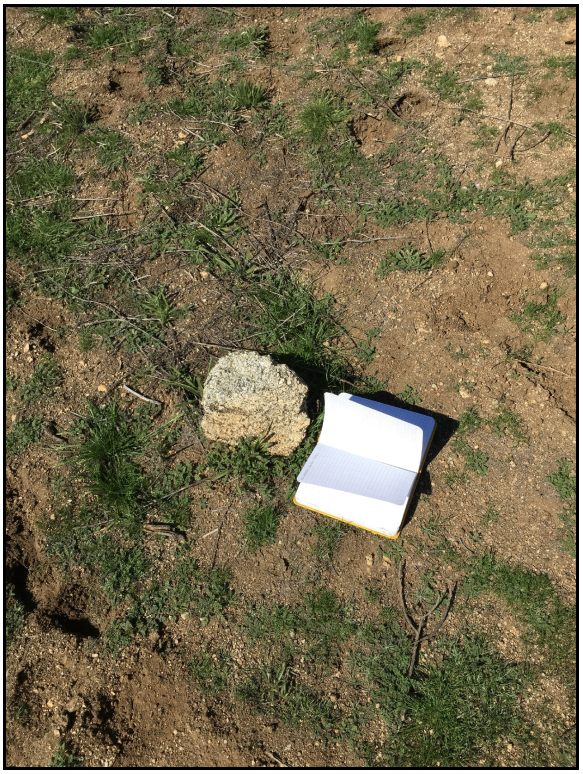

Figure 4. Typical rockfall observed at the site.

Lots 36 through 42 mostly consisted of large embedded boulders. Also present at this location were blocky, rounded boulders that could potentially mobilize during a seismic event. These boulders may detach during such events, but are very unlikely to impact the planned residence locations. There were no indications of rockfall exceeding the 1-ft diameter boulders at any of the planned lot locations.

A large, feldspar vein was observed at the site near Lots 41 and 42. This vein maybe be continuous throughout the site, but was only exposed in this

area. The vein is hard, and resistant to erosion. The thickness is unknown, Figure 5.

Lot 171, located on the east end of the project site was an additional area of concern. Lot 171 is composed of the same rounded, embedded boulders as the west end of the site. No boulders exceeding 1-ft in diameter was found at the location of planned residence construction.

Figure 5. Exposed feldspar vein .

5. ROCKFALL ANALYSES

5.1 Method of Analyses

Rockfall analyses of the slope utilized computer modeling. The Colorado Rockfall Simulation Program (CRSP) was used to simulate and analyze rockfall events.

5.2 Colorado Rockfall Simulation Program (CRSP)

Common practice in the analyses of slope rockfall is to use CRSP (Jones, et al., 2000). It is also possible to estimate how far a rock will travel along a slope by conducting actual rock rolling tests. While these tests may be useful in verifying criteria, they are expensive and are limited by the small number of rocks that can be rolled. In contrast, thousands of simulated rolls can be made using CRSP.

CRSP uses a computer algorithm based on actual rockfall tests to predict the distance a rock will stop from the toe of a slope, the velocity of the rock, how high the rock is likely to bounce, and the kinetic energy of the rock at any point. CRSP requires a slope profile, and an estimate of parameters such as rock unit weight, size, and slope roughness in addition to normal and tangential coefficients of restitution along the slope. CRSP provides an image of the slope profile with simulated rockfall, Figure 6, and can then compute the dynamic parameters of rockfall events, that is, the velocity, kinetic energy, and bounce height.

By modeling a slope using CRSP, it is possible to make some reasoned judgements on the need for, or the design of, a rockfall fence or impact wall. The CRSP algorithm has been validated by field data. It is routinely used as a design tool by the California Department of Transportation (Caltrans) and many other state highway departments.

After evaluating the site, it was determined that certain areas were most susceptible to rockfall and were chosen for the analyses. Rockfall parameters used in the analyses were based on field data and topographic maps. The collected field data included determination of surface conditions, boulder sizes, and soil properties.

Six slope profiles were analyzed with surface characteristics based on observed site conditions. Figure 6 shows a typical profile. Each profile was analyzed with surface characteristics based on observed slope conditions and boulder size. CRSP rockfall analyses were performed for the slope using 1,000 simulated rock rolls with rock shape and size applicable to the most hazardous boulders located in that zone. For these analyses, vegetation was not considered a factor in energy dissipation.

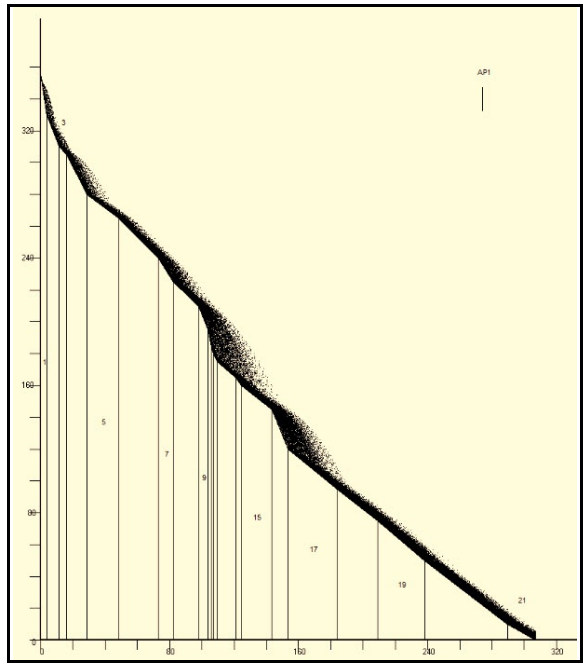

Figure 6. Typical CRSP profile..

The model boulders were assumed to have a unit weight of 165-pcf, typical of a intrusive, felsic, igneous rock (Hunt,1984), and maximum dimensions of 2-ft by 3-ft. Velocity, bounce heights, and kinetic energies were determined along each profile.

The profiles were assigned Analyses Points (AP) that were placed at the planned residence locations (AP1) to determine the probable rockfall hazard. Velocity, bounce heights, and kinetic energies were determined at the AP for each profile.

6. RESULTS AND DISCUSSION

6.1 Overview

Field investigations and rockfall analyses were performed and conclusions were made using generally accepted geotechnical engineering principles and incorporating currently available information, equipment and methods.

6.2 CRSP Results and Discussion

CRSP was utilized to model six profiles, chosen due to slope geometry and boulder locations. After the completion of the analyses, it was determined that the planned residences after not expected to be impacted by rockfall. The full results from CRSP, including all inputs and slope geometry, can be found in the Appendix A.

7. CONCLUSIONS AND RECOMMENDATIONS

7.1 Conclusions

Based on the results of the field studies and rockfall analyses, the following conclusions and recommendations are reported below. It is our opinion that:

1. **Some minor rockfall onto the slope may occur.** The rockfall source will continue to weather and erode and is likely to produce rockfall onto the slope. Based on the our observations and CRSP modeling, the proposed locations of the residences should not be affected.

2. **Rockfall mitigation is not necessary.** After rockfall simulation analyses, it is our opinion that rockfall mitigation is not necessary for the proposed location of the residences. It will, however, be beneficial to construct reinforced concrete or block privacy walls at Lots 36, 37, 38, 39, and 40 to provide supplementary protection and prevent any small, nuisance rockfall from accumulating in residential areas.

8. REFERENCES

City of Moreno Valley. (2006). "Moreno Valley General Plan. Final Program EIR. 2.6 Geology and Soils". July 2006.

Hunt, R. E. (1984). *Geotechnical Engineering Investigation Manual*. McGraw-Hill Book Company, New York, 195 p.

Jennings, C.W., Strand, R.G., and Rogers, T.H., 1977, Geologic map of California: California Division of Mines and Geology, scale 1:750,000.

Jennings, C.W., 1985, An explanatory text to accompany the 1:750,000 scale fault and geologic maps of California: California Division of Mines and Geology, Bulletin 201, 197 p.

Jones, C. L., Higgins, J. D., Andrew, R. D. (2000). "Colorado Rockfall Simulation Program Version 4.0." Colorado Department of Transportation, Denver, CO, 127 p.

USGS., (1967). PR1980 Sunnymead Quadrangle, 7.5 Minute Series, Scale 1"= 2000'.

9. LIMITATIONS

The analyses, conclusions and recommendations contained in this report are based on the site conditions observed by us and derived from the information provided to us. If there is a substantial lapse of time between the submission of our report and the start of any work at the site, or if conditions have changed due to natural causes, mining or construction operations at or adjacent to the site, we urge that our report be reviewed to determine the applicability of the conclusions and recommendations considering the changed conditions and time lapse. This report is applicable only for the project and sites studied. This report should not be used after three years.

Our professional services were performed, our findings obtained, and our recommendations proposed in accordance with generally accepted engineering principles and practices. This warranty is in lieu of all other warranties either expressed or implied. Findings and statements of professional opinion do not constitute a guarantee or warranty, expressed or implied.

***KANE* GeoTech, Inc.**

William F. Kane

William F. Kane, PhD, PG, PE
California Licensed Civil Engineer No. 55714

REGISTERED PROFESSIONAL ENGINEER
WILLIAM F. KANE
3-15-2016
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Exp. 12-31-16
CIVIL
STATE OF CALIFORNIA

Appendix A
Project Lot Map

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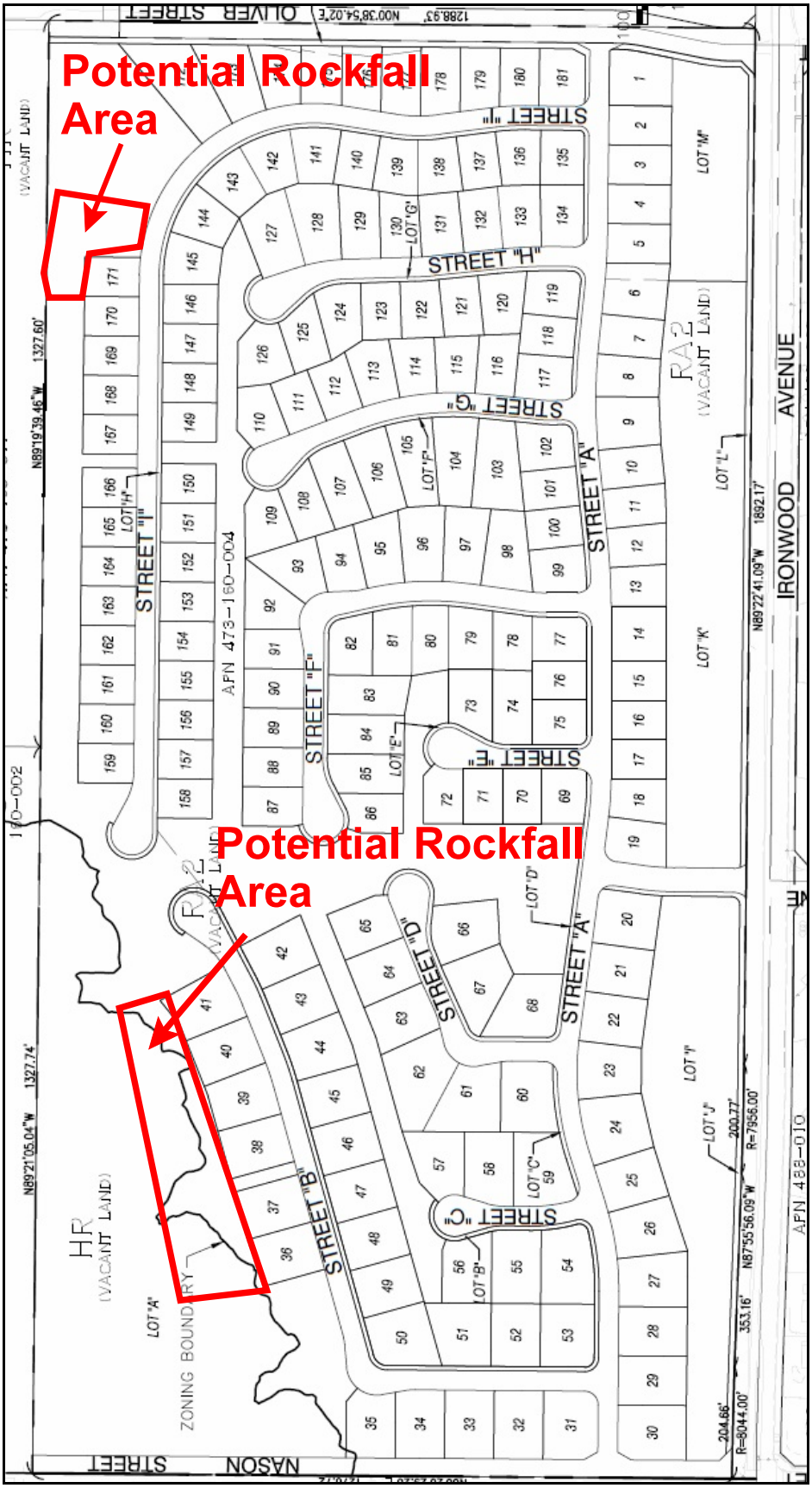

PLATE A-1. Project Lot Map

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Appendix B
CRSP Analyses

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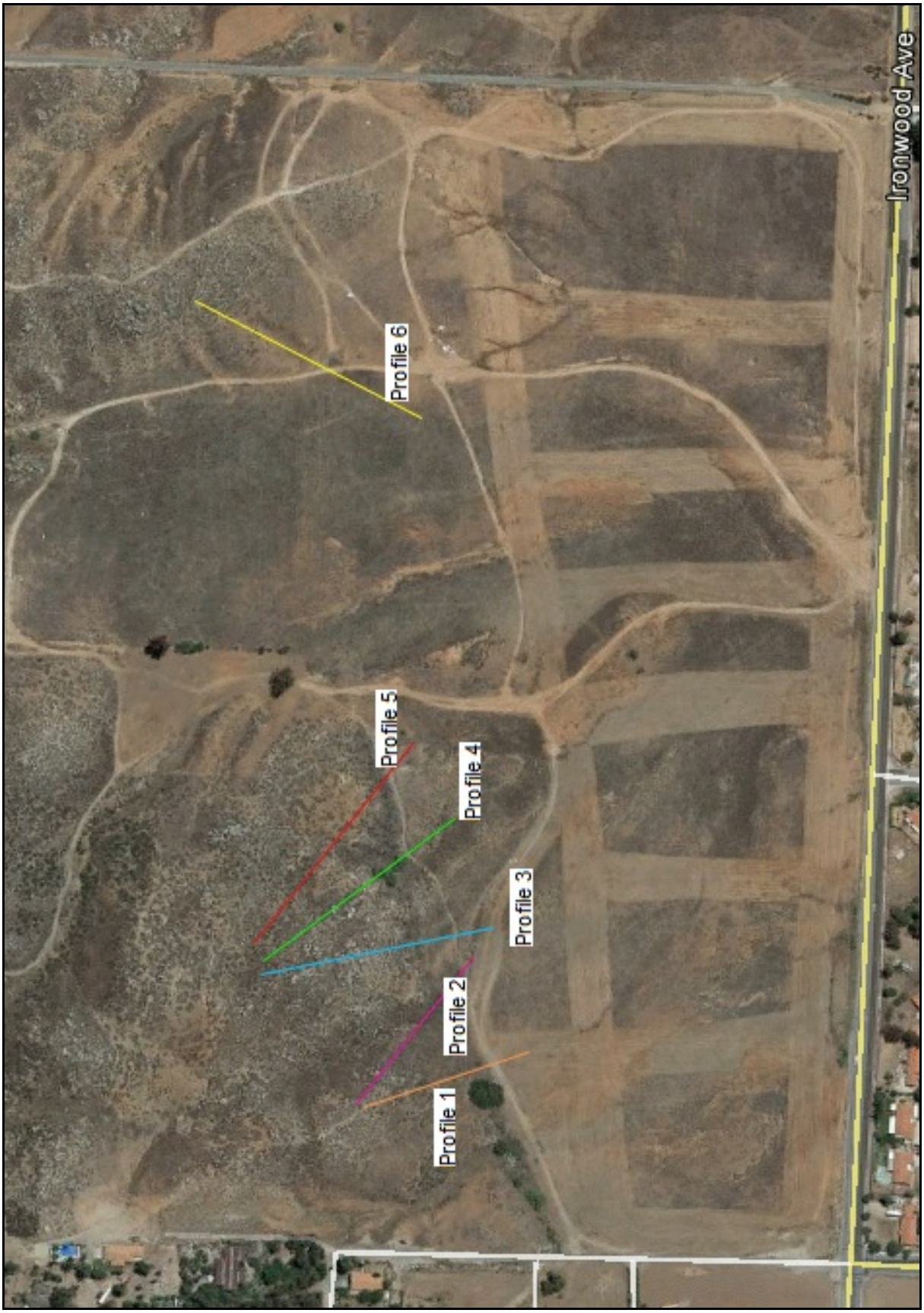

PLATE B-1. CRSP Profile Locations

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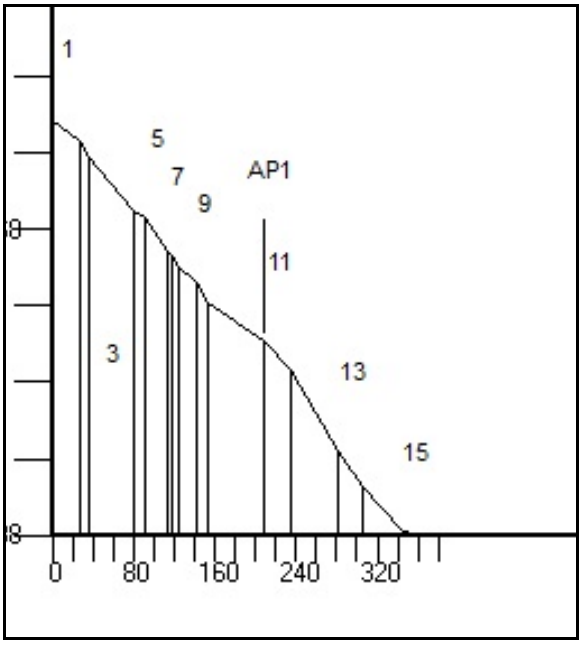
CRSP Analysis Profile 1



CRSP Input File -\\KANESERVER\Kane GeoTech Folder\KANE GeoTech Projects\2016\KGT16-05 Moreno Valley Ironwood Rockfall\CRSP\CRSP Profiles\Profile 1\Profile 1.dat

Input File Specifications

Units of Measure: U.S.
Total Number of Cells: 15
Analysis Point 1 X-Coordinate: 208.75
Analysis Point 2 X-Coordinate: 0
Analysis Point 3 X-Coordinate: 0
Initial Y-Top Starting Zone Coordinate: 1996
Initial Y-Base Starting Zone Coordinate: 1991

Remarks:

Cell Data

Cell No.	S.R.	Tang. C.	Norm. C.	Begin X	Begin Y	End X	End Y
1	3	.75	.15	0	1996	26.083	1991
2	3	.75	.15	26.083	1991	35.917	1987
3	3	.75	.15	35.917	1987	80.667	1973
4	3	.75	.15	80.667	1973	91.75	1971
5	3	.75	.15	91.75	1971	112.75	1962
6	3	.75	.15	112.75	1962	117.167	1961
7	3	.75	.15	117.167	1961	125.25	1958
8	3	.75	.15	125.25	1958	141.667	1954
9	3	.75	.15	141.667	1954	153.5	1949
10	3	.75	.15	153.5	1949	208.75	1939
11	3	.75	.15	208.75	1939	234.416	1931
12	3	.75	.15	234.416	1931	281.083	1910
13	3	.75	.15	281.083	1910	306.333	1901
14	3	.75	.15	306.333	1901	345.917	1889
15	3	.75	.15	345.917	1889	365.917	1888

CRSP Simulation Specifications: Used with \\KANESERVER\Kane GeoTech Folder\KANE GeoTech Projects\2016\KGT16-05 Moreno Valley Ironwood Rockfall\CRSP\CRSP Profiles\Profile 1\Profile 1.dat

Total Number of Rocks Simulated: 1000

Starting Velocity in X-Direction: 1 ft/sec
Starting Velocity in Y-Direction: -1 ft/sec
Starting Cell Number: 1
Ending Cell Number: 15
Rock Density: 165 lb/ft^3
Rock Shape: Spherical
Diameter: 1 ft
CRSP Analysis Point 1 Data - \\KANESERVER\Kane GeoTech Folder\KANE GeoTech Projects\2016\KGT16-05 Moreno Valley Ironwood Rockfall\CRSP\CRSP Profiles\Profile 1\Profile 1.dat

Analysis Point 1: X = 208.75, Y = 1939

NO ROCKS PAST ANALYSIS POINT 1

CRSP Data Collected at End of Each Cell - \\KANESERVER\Kane GeoTech Folder\KANE GeoTech Projects\2016\KGT16-05 Moreno Valley Ironwood Rockfall\CRSP\CRSP Profiles\Profile 1\Profile 1.dat

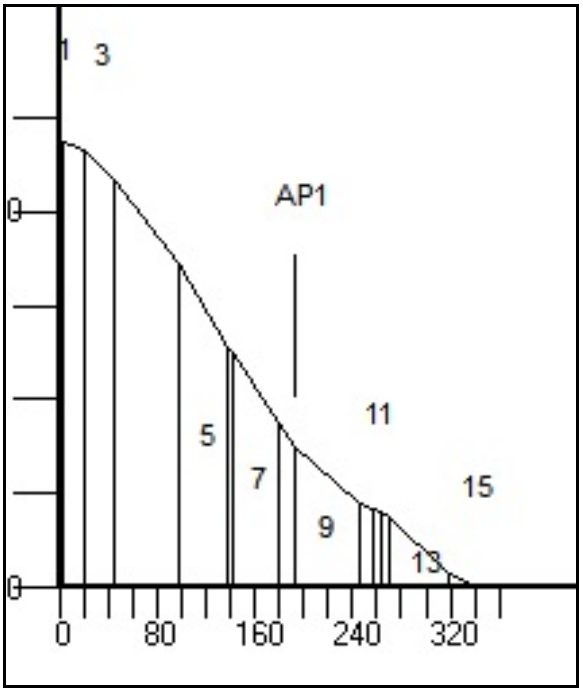
Velocity Units: ft/sec Bounce Height Units: ft

Cell #	Max. Vel.	Avg. Vel.	S.D. Vel.	Max. Bounce Ht.	Avg. Bounce Ht.
1	No rocks	past end of cell			
2	No rocks	past end of cell			
3	No rocks	past end of cell			
4	No rocks	past end of cell			
5	No rocks	past end of cell			
6	No rocks	past end of cell			
7	No rocks	past end of cell			
8	No rocks	past end of cell			
9	No rocks	past end of cell			
10	No rocks	past end of cell			
11	No rocks	past end of cell			
12	No rocks	past end of cell			
13	No rocks	past end of cell			
14	No rocks	past end of cell			
15	No rocks	past end of cell			

CRSP Rocks Stopped Data - \\KANESERVER\Kane GeoTech Folder\KANE GeoTech Projects\2016\KGT16-05 Moreno Valley Ironwood Rockfall\CRSP\CRSP Profiles\Profile 1\Profile 1.dat

X Interval	Rocks Stopped
0 To 10 ft	1000
10 To 20 ft	0
20 To 30 ft	0
30 To 40 ft	0
40 To 50 ft	0
50 To 60 ft	0
60 To 70 ft	0
70 To 80 ft	0
80 To 90 ft	0
90 To 100 ft	0
100 To 110 ft	0
110 To 120 ft	0
120 To 130 ft	0
130 To 140 ft	0
140 To 150 ft	0
150 To 160 ft	0
160 To 170 ft	0
170 To 180 ft	0
180 To 190 ft	0
190 To 200 ft	0
200 To 210 ft	0
210 To 220 ft	0
220 To 230 ft	0
230 To 240 ft	0
240 To 250 ft	0
250 To 260 ft	0
260 To 270 ft	0
270 To 280 ft	0
280 To 290 ft	0
290 To 300 ft	0
300 To 310 ft	0
310 To 320 ft	0
320 To 330 ft	0
330 To 340 ft	0
340 To 350 ft	0
350 To 360 ft	0
360 To 365.917 ft	0

CRSP Analysis Profile 2



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Input File Specifications

Units of Measure: U.S.
Total Number of Cells: 15
Analysis Point 1 X-Coordinate: 192
Analysis Point 2 X-Coordinate: 0
Analysis Point 3 X-Coordinate: 0
Initial Y-Top Starting Zone Coordinate: 1996
Initial Y-Base Starting Zone Coordinate: 1995

Remarks:

Cell Data

Cell No.	S.R.	Tang. C.	Norm. C.	Begin X	Begin Y	End X	End Y
1	3	.75	.15	0	1996	3.583	1995
2	3	.75	.15	3.583	1995	19.667	1993
3	3	.75	.15	19.667	1993	44	1987
4	3	.75	.15	44	1987	97.75	1969
5	3	.75	.15	97.75	1969	138.5	1951
6	3	.75	.15	138.5	1951	141.416	1950
7	3	.75	.15	141.416	1950	180.416	1935
8	3	.75	.15	180.416	1935	192	1930
9	3	.75	.15	192	1930	246.416	1918
10	3	.75	.15	246.416	1918	256	1917
11	3	.75	.15	256	1917	264.167	1916
12	3	.75	.15	264.167	1916	269.583	1915
13	3	.75	.15	269.583	1915	319.667	1903
14	3	.75	.15	319.667	1903	331.667	1901
15	3	.75	.15	331.667	1901	346.5	1900

CRSP Simulation Specifications: Used with X:\KANE GeoTech Projects\2016\KGT16-05 Moreno Valley Ironwood Rockfall\CRSP\CRSP Profiles\Profile 2\Profile 2.dat

Total Number of Rocks Simulated: 100
Starting Velocity in X-Direction: 1 ft/sec
Starting Velocity in Y-Direction: -1 ft/sec
Starting Cell Number: 1
Ending Cell Number: 15
Rock Density: 165 lb/ft^3
Rock Shape: Spherical
Diameter: 1 ft

CRSP Analysis Point 1 Data - X:\KANE GeoTech Projects\2016\KGT16-05 Moreno Valley Ironwood Rockfall\CRSP\CRSP Profiles\Profile 2\Profile 2.dat

Analysis Point 1: X = 192, Y = 1930

NO ROCKS PAST ANALYSIS POINT 1

CRSP Data Collected at End of Each Cell - X:\KANE GeoTech Projects\2016\KGT16-05 Moreno Valley Ironwood Rockfall\CRSP\CRSP Profiles\Profile 2\Profile 2.dat

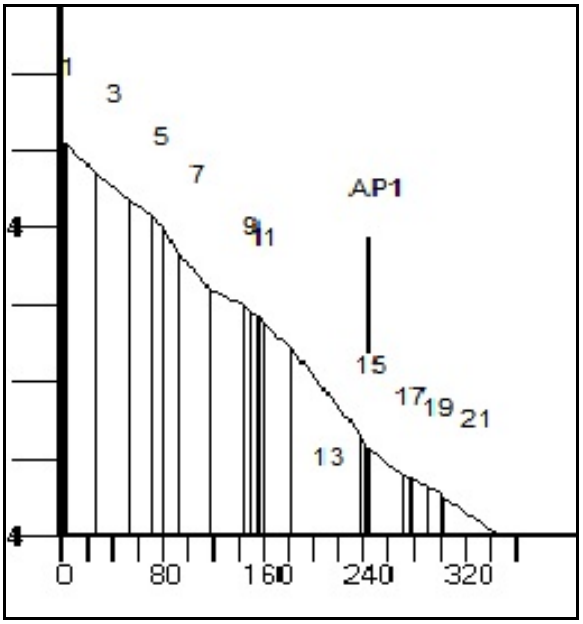
Velocity Units: ft/sec Bounce Height Units: ft

Cell #	Max. Vel.	Avg. Vel.	S.D. Vel.	Max. Bounce Ht.	Avg. Bounce Ht.
1	No rocks	past end of cell			
2	No rocks	past end of cell			
3	No rocks	past end of cell			
4	No rocks	past end of cell			
5	No rocks	past end of cell			
6	No rocks	past end of cell			
7	No rocks	past end of cell			
8	No rocks	past end of cell			
9	No rocks	past end of cell			
10	No rocks	past end of cell			
11	No rocks	past end of cell			
12	No rocks	past end of cell			
13	No rocks	past end of cell			
14	No rocks	past end of cell			
15	No rocks	past end of cell			

CRSP Rocks Stopped Data - X:\KANE GeoTech Projects\2016\KGT16-05 Moreno Valley Ironwood Rockfall\CRSP\CRSP Profiles\Profile 2\Profile 2.dat

X Interval	Rocks Stopped
0 To 10 ft	100
10 To 20 ft	0
20 To 30 ft	0
30 To 40 ft	0
40 To 50 ft	0
50 To 60 ft	0
60 To 70 ft	0
70 To 80 ft	0
80 To 90 ft	0
90 To 100 ft	0
100 To 110 ft	0
110 To 120 ft	0
120 To 130 ft	0
130 To 140 ft	0
140 To 150 ft	0
150 To 160 ft	0
160 To 170 ft	0
170 To 180 ft	0
180 To 190 ft	0
190 To 200 ft	0
200 To 210 ft	0
210 To 220 ft	0
220 To 230 ft	0
230 To 240 ft	0
240 To 250 ft	0
250 To 260 ft	0
260 To 270 ft	0
270 To 280 ft	0
280 To 290 ft	0
290 To 300 ft	0
300 To 310 ft	0
310 To 320 ft	0
320 To 330 ft	0
330 To 340 ft	0
340 To 346.5 ft	0

CRSP Analysis Profile 3



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Input File Specifications

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Analysis Point 3 X-Coordinate: 0
Initial Y-Top Starting Zone Coordinate: 1997
Initial Y-Base Starting Zone Coordinate: 1996

Remarks:

Cell Data

Cell No.	S.R.	Tang. C.	Norm. C.	Begin X	Begin Y	End X	End Y
1	3	.75	.15	0	1997	3.667	1996
2	3	.75	.15	3.667	1996	26.25	1988
3	3	.75	.15	26.25	1988	54.083	1981
4	3	.75	.15	54.083	1981	72.416	1977
5	3	.75	.15	72.416	1977	80.667	1974
6	3	.75	.15	80.667	1974	93.75	1967
7	3	.75	.15	93.75	1967	118.833	1958
8	3	.75	.15	118.833	1958	145.083	1954
9	3	.75	.15	145.083	1954	150.416	1952
10	3	.75	.15	150.416	1952	155	1951
11	3	.75	.15	155	1951	159.583	1949
12	3	.75	.15	159.583	1949	181.583	1943
13	3	.75	.15	181.583	1943	237.167	1920
14	3	.75	.15	237.167	1920	241.333	1918
15	3	.75	.15	241.333	1918	243.883	1917
16	3	.75	.15	243.883	1917	269.25	1910
17	3	.75	.15	269.25	1910	277.583	1909
18	3	.75	.15	277.583	1909	290.75	1907
19	3	.75	.15	290.75	1907	300.667	1905
20	3	.75	.15	300.667	1905	303.5	1904
21	3	.75	.15	303.5	1904	347.416	1894

CRSP Simulation Specifications: Used with X:\KANE GeoTech Projects\2016\KGT16-05 Moreno Valley Ironwood Rockfall\CRSP\CRSP Profiles\Profile 3\Profile 3.dat

Total Number of Rocks Simulated: 100
Starting Velocity in X-Direction: 1 ft/sec
Starting Velocity in Y-Direction: -1 ft/sec
Starting Cell Number: 1
Ending Cell Number: 21
Rock Density: 165 lb/ft^3
Rock Shape: Spherical
Diameter: 1 ft

CRSP Analysis Point 1 Data - X:\KANE GeoTech Projects\2016\KGT16-05 Moreno Valley Ironwood Rockfall\CRSP\CRSP Profiles\Profile 3\Profile 3.dat

Analysis Point 1: X = 243.833, Y = 1917

NO ROCKS PAST ANALYSIS POINT 1

CRSP Data Collected at End of Each Cell - X:\KANE GeoTech Projects\2016\KGT16-05 Moreno Valley Ironwood Rockfall\CRSP\CRSP Profiles\Profile 3\Profile 3.dat

Velocity Units: ft/sec Bounce Height Units: ft

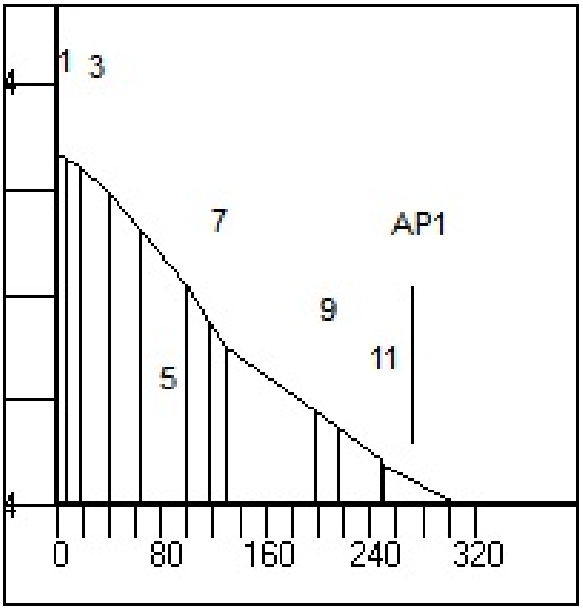
Cell #	Max. Vel.	Avg. Vel.	S.D. Vel.	Max. Bounce Ht.	Avg. Bounce Ht.
1	No rocks	past end of cell			
2	No rocks	past end of cell			
3	No rocks	past end of cell			
4	No rocks	past end of cell			
5	No rocks	past end of cell			
6	No rocks	past end of cell			
7	No rocks	past end of cell			
8	No rocks	past end of cell			
9	No rocks	past end of cell			
10	No rocks	past end of cell			
11	No rocks	past end of cell			
12	No rocks	past end of cell			
13	No rocks	past end of cell			
14	No rocks	past end of cell			
15	No rocks	past end of cell			
16	No rocks	past end of cell			

17	No rocks	past end of cell
18	No rocks	past end of cell
19	No rocks	past end of cell
20	No rocks	past end of cell
21	No rocks	past end of cell

CRSP Rocks Stopped Data - X:\KANE GeoTech Projects\2016\KGT16-05 Moreno Valley Ironwood Rockfall\CRSP\CRSP Profiles\Profile 3\Profile 3.dat

X Interval	Rocks Stopped
0 To 10 ft	100
10 To 20 ft	0
20 To 30 ft	0
30 To 40 ft	0
40 To 50 ft	0
50 To 60 ft	0
60 To 70 ft	0
70 To 80 ft	0
80 To 90 ft	0
90 To 100 ft	0
100 To 110 ft	0
110 To 120 ft	0
120 To 130 ft	0
130 To 140 ft	0
140 To 150 ft	0
150 To 160 ft	0
160 To 170 ft	0
170 To 180 ft	0
180 To 190 ft	0
190 To 200 ft	0
200 To 210 ft	0
210 To 220 ft	0
220 To 230 ft	0
230 To 240 ft	0
240 To 250 ft	0
250 To 260 ft	0
260 To 270 ft	0
270 To 280 ft	0
280 To 290 ft	0
290 To 300 ft	0
300 To 310 ft	0
310 To 320 ft	0
320 To 330 ft	0
330 To 340 ft	0
340 To 347.416 ft	0

CRSP Analysis Profile 4



CRSP Input File -X:\KANE GeoTech Projects\2016\KGT16-05 Moreno Valley Ironwood Rockfall\CRSP\CRSP Profiles\Profile 4\Profile 4.dat

Input File Specifications

Units of Measure: U.S.
Total Number of Cells: 12
Analysis Point 1 X-Coordinate: 272.416
Analysis Point 2 X-Coordinate:
Analysis Point 3 X-Coordinate:
Initial Y-Top Starting Zone Coordinate: 1981
Initial Y-Base Starting Zone Coordinate: 1980

Remarks:

Cell Data

Cell No.	S.R.	Tang. C.	Norm. C.	Begin X	Begin Y	End X	End Y
1	3	.75	.15	0	1981	7.583	1980
2	3	.75	.15	7.583	1980	18.25	1978
3	3	.75	.15	18.25	1978	39.583	1973
4	3	.75	.15	39.583	1973	65.25	1966
5	3	.75	.15	65.25	1966	100	1956
6	3	.75	.15	100	1956	116	1949
7	3	.75	.15	116	1949	130.333	1944
8	3	.75	.15	130.333	1944	197.167	1932
9	3	.75	.15	197.167	1932	215.583	1929
10	3	.75	.15	215.583	1929	247.167	1923
11	3	.75	.15	247.167	1923	250.333	1922
12	3	.75	.15	250.333	1922	306.667	1914

CRSP Simulation Specifications: Used with X:\KANE GeoTech Projects\2016\KGT16-05 Moreno Valley Ironwood Rockfall\CRSP\CRSP Profiles\Profile 4\Profile 4.dat

Total Number of Rocks Simulated: 100
Starting Velocity in X-Direction: 1 ft/sec
Starting Velocity in Y-Direction: -1 ft/sec
Starting Cell Number: 1
Ending Cell Number: 12
Rock Density: 165 lb/ft^3
Rock Shape: Spherical
Diameter: 1 ft

CRSP Analysis Point 1 Data - X:\KANE GeoTech Projects\2016\KGT16-05 Moreno Valley Ironwood Rockfall\CRSP\CRSP Profiles\Profile 4\Profile 4.dat

Analysis Point 1: X = 272.416, Y = 1919

NO ROCKS PAST ANALYSIS POINT 1

CRSP Data Collected at End of Each Cell - X:\KANE GeoTech Projects\2016\KGT16-05 Moreno Valley Ironwood Rockfall\CRSP\CRSP Profiles\Profile 4\Profile 4.dat

Velocity Units: ft/sec Bounce Height Units: ft

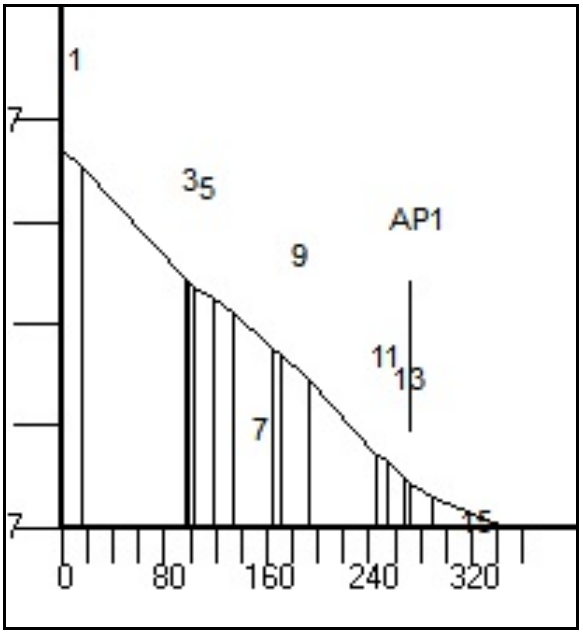
Cell #	Max. Vel.	Avg. Vel.	S.D. Vel.	Max. Bounce Ht.	Avg. Bounce Ht.
1	No rocks	past end of cell			
2	No rocks	past end of cell			
3	No rocks	past end of cell			
4	No rocks	past end of cell			
5	No rocks	past end of cell			
6	No rocks	past end of cell			
7	No rocks	past end of cell			
8	No rocks	past end of cell			
9	No rocks	past end of cell			
10	No rocks	past end of cell			
11	No rocks	past end of cell			
12	No rocks	past end of cell			

CRSP Rocks Stopped Data - X:\KANE GeoTech Projects\2016\KGT16-05 Moreno Valley Ironwood Rockfall\CRSP\CRSP Profiles\Profile 4\Profile 4.dat

X Interval	Rocks Stopped
0 To 10 ft	100
10 To 20 ft	0
20 To 30 ft	0
30 To 40 ft	0
40 To 50 ft	0
50 To 60 ft	0
60 To 70 ft	0
70 To 80 ft	0
80 To 90 ft	0
90 To 100 ft	0
100 To 110 ft	0
110 To 120 ft	0
120 To 130 ft	0

```
130 To  140 ft                  0
140 To  150 ft                  0
150 To  160 ft                  0
160 To  170 ft                  0
170 To  180 ft                  0
180 To  190 ft                  0
190 To  200 ft                  0
200 To  210 ft                  0
210 To  220 ft                  0
220 To  230 ft                  0
230 To  240 ft                  0
240 To  250 ft                  0
250 To  260 ft                  0
260 To  270 ft                  0
270 To  280 ft                  0
280 To  290 ft                  0
290 To  300 ft                  0
300 To  306.667 ft              0
```

CRSP Analysis Profile 5



CRSP Input File -X:\KANE GeoTech Projects\2016\KGT16-05 Moreno Valley Ironwood Rockfall\CRSP\CRSP Profiles\Profile 5\Profile 5.dat

Input File Specifications

Units of Measure: U.S.
Total Number of Cells: 15
Analysis Point 1 X-Coordinate: 271
Analysis Point 2 X-Coordinate:
Analysis Point 3 X-Coordinate:
Initial Y-Top Starting Zone Coordinate: 1981
Initial Y-Base Starting Zone Coordinate: 1978

Remarks:

Cell Data

Cell No.	S.R.	Tang. C.	Norm. C.	Begin X	Begin Y	End X	End Y
1	3	.75	.15	0	1981	16	1978
2	3	.75	.15	16	1978	96.25	1956
3	3	.75	.15	96.25	1956	99.5	1955
4	3	.75	.15	99.5	1955	103.583	1954
5	3	.75	.15	103.583	1954	118.416	1952
6	3	.75	.15	118.416	1952	135	1949
7	3	.75	.15	135	1949	164.667	1942
8	3	.75	.15	164.667	1942	171.25	1941
9	3	.75	.15	171.25	1941	193	1936
10	3	.75	.15	193	1936	246.083	1921
11	3	.75	.15	246.083	1921	253.5	1920
12	3	.75	.15	253.5	1920	268.083	1917
13	3	.75	.15	268.083	1917	271	1916
14	3	.75	.15	271	1916	288.833	1913
15	3	.75	.15	288.833	1913	348.583	1907

CRSP Simulation Specifications: Used with X:\KANE GeoTech Projects\2016\KGT16-05 Moreno Valley Ironwood Rockfall\CRSP\CRSP Profiles\Profile 5\Profile 5.dat
Total Number of Rocks Simulated: 100
Starting Velocity in X-Direction: 1 ft/sec
Starting Velocity in Y-Direction: -1 ft/sec
Starting Cell Number: 1
Ending Cell Number: 15
Rock Density: 165 lb/ft^3
Rock Shape: Spherical
Diameter: 1 ft

CRSP Analysis Point 1 Data - X:\KANE GeoTech Projects\2016\KGT16-05 Moreno Valley Ironwood Rockfall\CRSP\CRSP Profiles\Profile 5\Profile 5.dat

Analysis Point 1: X = 271, Y = 1916

NO ROCKS PAST ANALYSIS POINT 1

CRSP Data Collected at End of Each Cell - X:\KANE GeoTech Projects\2016\KGT16-05 Moreno Valley Ironwood Rockfall\CRSP\CRSP Profiles\Profile 5\Profile 5.dat

Velocity Units: ft/sec Bounce Height Units: ft

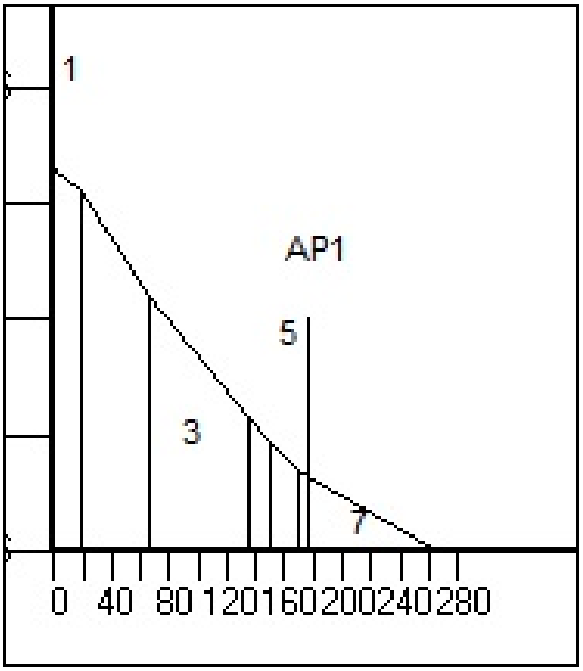
Cell #	Max. Vel.	Avg. Vel.	S.D. Vel.	Max. Bounce Ht.	Avg. Bounce Ht.
1	No rocks	past end of cell			
2	No rocks	past end of cell			
3	No rocks	past end of cell			
4	No rocks	past end of cell			
5	No rocks	past end of cell			
6	No rocks	past end of cell			
7	No rocks	past end of cell			
8	No rocks	past end of cell			
9	No rocks	past end of cell			
10	No rocks	past end of cell			
11	No rocks	past end of cell			
12	No rocks	past end of cell			
13	No rocks	past end of cell			
14	No rocks	past end of cell			
15	No rocks	past end of cell			

CRSP Rocks Stopped Data - X:\KANE GeoTech Projects\2016\KGT16-05 Moreno Valley Ironwood Rockfall\CRSP\CRSP Profiles\Profile 5\Profile 5.dat

X Interval	Rocks Stopped
0 To 10 ft	100
10 To 20 ft	0
20 To 30 ft	0
30 To 40 ft	0
40 To 50 ft	0
50 To 60 ft	0
60 To 70 ft	0
70 To 80 ft	0
80 To 90 ft	0

90 To 100 ft	0
100 To 110 ft	0
110 To 120 ft	0
120 To 130 ft	0
130 To 140 ft	0
140 To 150 ft	0
150 To 160 ft	0
160 To 170 ft	0
170 To 180 ft	0
180 To 190 ft	0
190 To 200 ft	0
200 To 210 ft	0
210 To 220 ft	0
220 To 230 ft	0
230 To 240 ft	0
240 To 250 ft	0
250 To 260 ft	0
260 To 270 ft	0
270 To 280 ft	0
280 To 290 ft	0
290 To 300 ft	0
300 To 310 ft	0
310 To 320 ft	0
320 To 330 ft	0
330 To 340 ft	0
340 To 348.583 ft	0

CRSP Analysis Profile 6



CRSP Input File -X:\KANE GeoTech Projects\2016\KGT16-05 Moreno Valley Ironwood Rockfall\CRSP\CRSP Profiles\Profile 6\Profile 6.dat

Input File Specifications

Units of Measure: U.S.
Total Number of Cells: 7
Analysis Point 1 X-Coordinate: 176.75
Analysis Point 2 X-Coordinate:
Analysis Point 3 X-Coordinate:
Initial Y-Top Starting Zone Coordinate: 1962
Initial Y-Base Starting Zone Coordinate: 1958

Remarks:

Cell Data

Cell No.	S.R.	Tang. C.	Norm. C.	Begin X	Begin Y	End X	End Y
1	3	.75	.15	0	1962	18.25	1958
2	3	.75	.15	18.25	1958	65.75	1940
3	3	.75	.15	65.75	1940	135.416	1919
4	3	.75	.15	135.416	1919	151.25	1915
5	3	.75	.15	151.25	1915	169.667	1910
6	3	.75	.15	169.667	1910	176.75	1909
7	3	.75	.15	176.75	1909	266.833	1896

CRSP Simulation Specifications: Used with X:\KANE GeoTech Projects\2016\KGT16-05 Moreno Valley Ironwood Rockfall\CRSP\CRSP Profiles\Profile 6\Profile 6.dat

Total Number of Rocks Simulated: 100
Starting Velocity in X-Direction: 1 ft/sec
Starting Velocity in Y-Direction: -1 ft/sec
Starting Cell Number: 1
Ending Cell Number: 7
Rock Density: 165 lb/ft^3
Rock Shape: Spherical
Diameter: 1 ft

CRSP Analysis Point 1 Data - X:\KANE GeoTech Projects\2016\KGT16-05 Moreno Valley Ironwood Rockfall\CRSP\CRSP Profiles\Profile 6\Profile 6.dat

Analysis Point 1: X = 176.75, Y = 1909

NO ROCKS PAST ANALYSIS POINT 1

CRSP Data Collected at End of Each Cell - X:\KANE GeoTech Projects\2016\KGT16-05 Moreno Valley Ironwood Rockfall\CRSP\CRSP Profiles\Profile 6\Profile 6.dat

Velocity Units: ft/sec Bounce Height Units: ft

Cell #	Max. Vel.	Avg. Vel.	S.D. Vel.	Max. Bounce Ht.	Avg. Bounce Ht.
1	No rocks	past end of cell			
2	No rocks	past end of cell			
3	No rocks	past end of cell			
4	No rocks	past end of cell			
5	No rocks	past end of cell			
6	No rocks	past end of cell			
7	No rocks	past end of cell			

CRSP Rocks Stopped Data - X:\KANE GeoTech Projects\2016\KGT16-05 Moreno Valley Ironwood Rockfall\CRSP\CRSP Profiles\Profile 6\Profile 6.dat

X Interval	Rocks Stopped
0 To 10 ft	100
10 To 20 ft	0
20 To 30 ft	0
30 To 40 ft	0
40 To 50 ft	0
50 To 60 ft	0
60 To 70 ft	0
70 To 80 ft	0
80 To 90 ft	0
90 To 100 ft	0
100 To 110 ft	0
110 To 120 ft	0
120 To 130 ft	0
130 To 140 ft	0
140 To 150 ft	0
150 To 160 ft	0
160 To 170 ft	0
170 To 180 ft	0
180 To 190 ft	0
190 To 200 ft	0
200 To 210 ft	0
210 To 220 ft	0
220 To 230 ft	0
230 To 240 ft	0
240 To 250 ft	0
250 To 260 ft	0
260 To 266.833 ft	0

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