PRELIMINARY GEOTECHNICAL INVESTIGATION
PROPOSED VORTEX FARMS
SOUTHEAST OF SAGE ROAD AND MINTO WAY
RIVERSIDE, CALIFORNIA

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1.0 INTRODUCTION AND SCOPE OF SERVICES

1.1 Introduction

Construction Testing and Engineering, Inc. (CTE) has completed a geotechnical investigation and report providing conclusions and recommendations for the proposed Vortex Farms improvements in Riverside, California. It is understood that the proposed development is to consist of constructing numerous single-story greenhouse structures with a paved drive, stormwater BMP's, septic system, utilities, and other associated improvements. CTE has performed this work in general accordance with the terms of proposal G-5096B dated September 21, 2020. Preliminary geotechnical recommendations for excavations, fill placement, and foundation design for the proposed improvements are presented herein.

1.2 Scope of Services

The scope of services provided included:

- Review of readily available geologic and geotechnical reports.
- Coordination of utility mark-out and location.
- Percolation testing in accordance with Riverside County Low Impact Development BMP Design Handbook.
- Excavation of exploratory borings and soil sampling utilizing a truck-mounted drill rig and limited-access manual excavation equipment.
- Laboratory testing of selected soil samples.
- Description of site geology and evaluation of potential geologic hazards.
- Preparation of this preliminary geotechnical investigation report.

2.0 SITE DESCRIPTION

The subject site is located southeast of Sage Road and Minto Way in Riverside, California (Figure 1). The site is bounded by Sage Road to the west, Minto Way to the north, and undeveloped land to the south and east. Existing site conditions are illustrated on Figures 1 and 2. The proposed improvement area is currently undeveloped. Based on reconnaissance and review of site topography, the proposed structural improvement area generally descends to the southwest with elevations ranging from approximately 1,950 feet above mean sea level (msl) in the northeast to approximately 1,915 feet msl to the southwest.

3.0 FIELD INVESTIGATION AND LABORATORY TESTING

3.1 Field Investigation

CTE performed the recent subsurface investigation on September 30 and October 1, 2020 to evaluate underlying soil conditions. This fieldwork consisted of site reconnaissance, surface mapping of exposed geologic units on site slopes, and the excavation of five exploratory soil borings, three BMP percolation test holes, and four septic percolation test holes. The borings were advanced to a maximum explored depth of approximately 20 feet below ground surface (bgs). Bulk samples were collected from the cuttings, and relatively undisturbed samples were collected by driving Standard Penetration Test (SPT) and Modified California (CAL) samplers. Borings B-1through B-4 and the BMP percolation test holes were excavated with a CME-75 truck-mounted drill rig equipped with eight-inch-diameter, hollow-stem augers. Due to limited access, borings B-5 and B-6 and the septic test holes were advanced with a manually operated auger that extended to a maximum depth of approximately 7.1 feet bgs. Approximate locations of the soil borings and percolation test holes are shown on the attached Figure 2.

Soils were logged in the field by a CTE Engineering Geologist, and were visually classified in general accordance with the Unified Soil Classification System. The field descriptions have been modified, where appropriate, to reflect laboratory test results. Boring logs, including descriptions of the soils encountered, are included in Appendix B.

3.2 Laboratory Testing

Laboratory tests were conducted on selected soil samples for classification purposes, and to evaluate physical properties and engineering characteristics. Laboratory tests included: In-place Moisture and Density, Modified Proctor, Expansion Index, Resistance "R"-Value, Grain Size Analysis, Consolidation, and Chemical Characteristics. Test descriptions and laboratory test results are included in Appendix C.

4.0 PERCOLATION TESTING

Three percolation tests were performed within the proposed BMP infiltration area. The percolation test holes were excavated to depths ranging from approximately 2.8 to 4.8 feet below the ground surface (bgs). The attached Figure 2 shows the approximate percolation test locations. The testing was performed in general accordance with the Riverside County – Low Impact Development BMP Design Handbook. Percolation testing of the septic holes was performed by others, and the results will be presented in a separate report.

4.1 Percolation Test Methods

The percolation tests were performed in general accordance with methods approved by Riverside County Low Impact Development BMP Design Handbook Appendix A after the required pre-soaking. Percolation test results and calculated infiltration rates are presented below in Table 4.2. Field Data and percolation to infiltration calculations are included in Appendix E.

4.2 Calculated Infiltrated Rate

As per the Riverside County Low Impact Development BMP Design Handbook Appendix-A, infiltration rates are to be evaluated using the Porchet Method. The intent of calculating the infiltration via the Porchet Method is to take into account bias inherent in percolation test borehole sidewall infiltration that would not occur at a basin bottom where such sidewalls are not present.

The infiltration rate (I_t) is derived by the equation:

$$I_t = \frac{\Delta H \, \pi r2 \; 60}{\Delta t(\pi r2 + 2\pi r H_{avg})} = \frac{\Delta H \; 60 \; r}{\Delta t(r + 2H_{avg})}$$

Where:

I_t = tested infiltration rate, inches/hour
ΔH = change in head over the time interval, inches
Δt = time interval, minutes
* r = effective radius of test hole
H_{avg} = average head over the time interval, inches

Given the measured percolation rates, the calculated infiltration rates are presented with and without a Factor of Safety applied in Table 4.2 below. The civil engineer of record should determine an appropriate factor of safety to be applied. CTE does not recommend using a factor of safety of less than 2.0.

TABLE 4.2 SUMMARY OF PERCOLATION AND INFILTRATION TEST RESULTS						
Test Location	Soil Type	Riverside County Percolation Procedure	Depth (inches)	Percolation Rate (inches/hour)	Infiltration Rate (inches/hour)	Recommended Rate for Design* (inches/hour)
P-1	Qyf	Non Sandy	35	6.000	0.600	0.300
P-2	Qyf	Non Sandy	58	7.000	0.644	0.322
P-3	Qyf	Sandy	34	38.250	1.934	0.967

NOTES Water level was measured from a fixed point at the top of the hole.
Weather was sunny during percolation testing.
Qyf = Quaternary Young Alluvial Fan Deposits
The test holes were eight inches in diameter.

5.0 GEOLOGY

5.1 General Setting

The Riverside area is located within the Peninsular Ranges physiographic province that is characterized by northwest-trending mountain ranges, intervening valleys, and predominantly northwest trending regional faults. The region can be further subdivided into the coastal plain area, central mountain–valley area and eastern mountain and valley area. The site is located within the central mountain–valley area that is near the western edge of the Peninsular Range Batholith (PRB) and generally consists of Cretaceous igneous rocks and localized Jurassic igneous rocks. The PRB contains remnant blocks of pre-Cretaceous metamorphic rocks that are locally covered with post-Cretaceous volcanic rocks, and marine and non-marine deposits. Throughout the batholith, colluvium and alluvium are present on mountain slopes and intervening valleys.

5.2 Geologic Conditions

Regional geologic mapping by Morton and Matti (2005) indicates the near surface geologic unit underlying the site consists of Quaternary Young Alluvial Fan Deposits and Cretaceous Tonalite of Coahuila Valley. Based on the recent subsurface evaluation, Residual Soil was observed over the Tonalite. Descriptions of the geologic units encountered are presented below.

5.2.1 Quaternary Young Alluvial Fan Deposits

Quaternary Young Alluvial Fan Deposits were observed in borings B-1, B-2, and B-4. This material was generally found to consist of loose to medium dense, grayish brown, silty fine to medium grained sand. This unit was observed to a maximum depth of approximately 11.5 feet bgs. Isolated areas with deeper Young Alluvial Fan Deposits may be encountered during grading.

5.2.2 Residual Soil

Residual Soil was observed in borings B-3, B-5 and B-6. This material was generally found to consist of loose to medium dense, grayish brown clayey fine to medium grained sand. This unit is relatively thin and blankets the underlying tonalite bedrock.

5.2.2 Cretaceous Tonalite of the Coahuila Valley

Cretaceous Tonalite of the Coahuila Valley (Granitic Rock) was observed at depths ranging from approximately 0.9 to 11.5 feet bgs. This bedrock unit was generally found to consist of very dense, reddish gray tonalite that excavates to silty fine to medium grained sand. This unit is anticipated at depth throughout the site.

5.3 Groundwater Conditions

Groundwater was not encountered in the recent borings that were advanced to a maximum explored depth of approximately 20 feet bgs. While groundwater conditions may vary, especially following periods of sustained precipitation or irrigation, it is generally not anticipated to adversely affect shallow construction activities or the completed improvements, if irrigation is limited and proper site drainage is designed, installed, and maintained per the recommendations of the project civil engineer. However, groundwater could have the potential to perch on the underlying granitic bedrock, especially during or following the rainy season. Such occurrences could impact foundation excavations and grading.

5.4 Geologic Hazards

Geologic hazards that were considered to have potential impacts to site development were evaluated based on field observations, literature review, and laboratory test results. It appears that geologic hazards at the site are primarily limited to those caused by shaking from earthquake-generated ground motions. The following paragraphs discuss the geologic hazards considered and their potential risk to the site.

5.4.1 Surface Fault Rupture

In accordance with the Alquist-Priolo Earthquake Fault Zoning Act, (ACT), the State of California established Earthquake Fault Zones around known active faults. The purpose of the ACT is to regulate the development of structures intended for human occupancy near active fault traces in order to mitigate hazards associated with surface fault rupture.

According to the California Geological Survey (Special Publication 42, Revised 2018), a fault that has had surface displacement within the last 11,700 years is defined as a Holocene-active fault and is either already zoned or pending zonation in accordance with the ACT. There are several other definitions of fault activity that are used to regulate dams, power plants, and other critical facilities, and some agencies designate faults that are documented as older than Holocene (last 11,700 years) and younger than late Quaternary (1.6 million years) as potentially active faults that are subject to local jurisdictional regulations.

Based on the site reconnaissance and review of referenced literature, the site is not located within a local or State-designated Earthquake Fault Zone, no known active fault traces underlie or project toward the site, and no known potentially active fault traces project toward the site. Therefore fault surface rupture potential is considered to be low at the subject site.

5.4.2 Local and Regional Faulting

The United States Geological Survey (USGS), with support of State Geological Surveys, and reviewed published work by various researchers, have developed a Quaternary Fault and Fold Database of faults and associated folds that are believed to be sources of earthquakes with magnitudes greater than 6.0 that have occurred during the Quaternary (the past 1.6 million years). The faults and folds within the database have been categorized into four Classes (Class A-D) based on the level of evidence confirming that a Quaternary fault is of tectonic origin and whether the structure is exposed for mapping or inferred from fault related deformational features. Class A faults have been mapped and categorized based on age of documented activity ranging from Historical faults (activity within last 150 years), Latest Quaternary faults (activity within last 15,000 years), Late Quaternary (activity within last 130,000 years), to Middle to late Quaternary (activity within last 1.6 million years). The Class A faults are considered to have the highest potential to generate earthquakes and/or surface rupture, and the earthquake and surface rupture potential generally increases from oldest to youngest. The evidence for Quaternary deformation and/or tectonic activity progressively decreases for Class B and Class C faults. When geologic evidence indicates that a fault is not of tectonic origin it is considered to be a Class D structure. Such evidence includes joints, fractures, landslides, or erosional and fluvial scarps that resemble fault features, but demonstrate a non-tectonic origin.

The nearest known Class A fault is the San Felipe Fault Zone (<1.6 million years), which is approximately 13.6 kilometers southwest of the site. The attached Figure 3 shows regional faults and seismicity with respect to the subject site.

5.4.3 Liquefaction and Seismic Settlement Evaluation

Liquefaction occurs when saturated fine-grained sands or silts lose their physical strengths during earthquake-induced shaking and behave like a liquid. This is due to loss of point-to-point grain contact and transfer of normal stress to the pore water. Liquefaction potential varies with water level, soil type, material gradation, relative density, and probable intensity and duration of ground shaking. Seismic settlement can occur with or without liquefaction; it results from densification of loose soils.

Groundwater was not encountered in any of the borings that extended to a depth of 20 feet bgs, and the improvement area is generally underlain at shallow depths by medium dense alluvial fan deposits and very dense granitic rock. Based on these conditions, the potential for liquefaction or significant seismic settlement at the site is generally considered to be low.

5.4.4 Landsliding

According to mapping Morton and Matti (2005), no landslides are mapped in the site area and were not encountered during the recent field exploration. Based on the preliminary investigation findings, landsliding is not considered to be a significant geologic hazard at the site.

5.4.5 Flooding

Based on Federal Emergency Management Agency mapping (FEMA 2012), site improvement areas are located within Zone X, which is defined as: "Areas determined to be outside the 0.2% annual chance floodplain". Therefore, subject to the review of the project civil engineer, the potential for flooding at the site is generally considered to be low.

5.4.6 Compressible and Expansive Soils

The near surface soils are considered to be potentially compressible in their current condition. Therefore, it is recommended that these soils be overexcavated, where necessary, and properly compacted beneath proposed improvement areas as recommended herein and as determined to be necessary during construction.

Based on observed site conditions and investigation findings, the shallow alluvial fan deposits may be marginally susceptible to hydro-collapse where exposed to increased moisture content. Recommendations provided herein are intended to minimize effects associated with potential consolidation of near surface soils.

Based on laboratory analysis, geologic observation, and the generally granular nature of site soils, the near-surface materials are generally anticipated to exhibit a very low expansion potential (Expansion Index of 20 or less). Verification of expansion potential should be performed during site excavations and grading.

5.4.7 Corrosive Soils

Testing of representative site area soils was performed to evaluate the potential corrosive effects on concrete foundations and buried metallic utilities. Soil environments detrimental to concrete generally have elevated levels of soluble sulfates and/or pH levels less than 5.5. According to the American Concrete Institute (ACI) Table 318 4.3.1, specific guidelines have been provided for concrete where concentrations of soluble sulfate (SO_4) in soil exceed 0.10 percent by weight. These guidelines include low water/cement ratios, increased compressive strength, and specific cement-type requirements. A minimum resistivity value less than approximately 5,000 ohm-cm and/or soluble chloride levels in excess of 200 ppm generally indicate a corrosive environment for buried metallic utilities and untreated conduits.

Chemical test results indicate that near-surface soils at the site area generally present a negligible corrosion potential for Portland cement concrete. Based on resistivity testing, the soils have been interpreted to have a low corrosivity potential to buried metallic improvements. As such, it would likely be prudent for buried utilities to utilize plastic piping and/or conduits, where feasible. However, CTE does not practice corrosion engineering. Therefore, if corrosion of improvements is of more significant concern, a qualified corrosion engineer could be consulted.

6.0 CONCLUSIONS AND RECOMMENDATIONS

6.1 General

CTE concludes that the proposed improvements on the site are feasible from a geotechnical standpoint, provided the preliminary recommendations in this report are incorporated into the design and construction of the project. Recommendations for the proposed earthwork and improvements are included in the following sections and Appendix D. However, recommendations in the text of this report supersede those presented in Appendix D should conflicts exist. These preliminary recommendations should either be confirmed as appropriate or updated following required excavations and observations during site preparation.

6.2 Site Preparation

Prior to grading, areas to receive distress sensitive improvements should be cleared of existing debris and deleterious materials. Objectionable materials, such as vegetation not suitable for structural backfill should be properly disposed of off-site.

In the areas of proposed structures, overexcavation should extend to a minimum depth of three feet below the bottom of proposed foundations or to the depth of competent native materials, whichever is greatest. If loose or otherwise unsuitable materials are encountered at the base of overexcavations, additional excavation to the depth of suitable material may be necessary. Remedial excavations should extend laterally at least five feet beyond the limits of the proposed improvements or the distance resulting from a 1:1 (horizontal: vertical) extended down to suitable material, where feasible. If overexcavations encroach upon property lines the temporary excavation should generally be sloped at a 1:1 (horizontal to vertical) or flatter, to the prescribed overexcavation depth. Depending upon proximity and condition of exposed soils, overexcavation in slot cuts may be recommended by the geotechnical engineer.

Overexcavations for proposed surface improvement areas, such as pavement or flatwork should be conducted to a minimum depth of two feet below existing or proposed subgrade or to the depth of suitable material, whichever is shallower.

A geotechnical representative from CTE should observe the exposed ground surface prior to placement of compacted fill or improvements, to verify the competency of exposed subgrade materials. After approval by this office, the exposed subgrades to receive fill should be scarified a minimum of eight inches, moisture conditioned, and properly compacted prior to fill placement.

6.3 Site Excavation

Generally, excavation of site materials may be accomplished with heavy-duty construction equipment under normal conditions; however, the underlying weathered bedrock will become increasingly difficult to excavate with depth and deeper excavations may not be feasible with standard heavy-duty equipment. In addition, large hard and dense “core stones” could be encountered in weathered bedrock masses resulting in localized, very difficult to impenetrable excavation conditions that may require specialized equipment.

In addition, excavations within the Young Alluvial Fan Deposits could encounter zones that are sensitive to caving and/or erosion, and may not effectively remain standing vertical or near-vertical, even at shallow or minor heights and for short periods of time.

6.4 Fill Placement and Compaction

Following the recommended overexcavation and removal of loose or disturbed soils, areas to receive fills should be scarified approximately eight inches, moisture conditioned, and properly compacted. Fill and backfill should be compacted to a minimum relative compaction of 90 percent at above optimum moisture content, as evaluated by ASTM D 1557. The optimum lift thickness for fill soil depends on the type of compaction equipment used. Generally, backfill should be placed in uniform, horizontal lifts not exceeding eight inches in loose thickness. Fill placement and compaction should be conducted in conformance with local ordinances, and should be observed and tested by a CTE geotechnical representative.

6.5 Fill Materials

Properly moisture conditioned, very low to low expansion potential soils derived from the on-site materials are considered suitable for reuse on the site as compacted fill. If used, these materials should be screened of organics and materials generally greater than three inches in maximum dimension. Irreducible materials greater than three inches in maximum dimension should not be used in shallow fills (within three feet of proposed grades). In utility trenches, adequate bedding should surround pipes.

Imported fill beneath structures and flatwork should have an Expansion Index of 20 or less (ASTM D 4829). Imported fill soils for use in structural or slope areas should be evaluated by the soils engineer before being imported to the site.

For retaining walls, backfill located within a 45-degree wedge extending up from the bottom of the heel foundation of the wall should consist of soil having an Expansion Index of 20 or less (ASTM D 4829) with less than 30 percent passing the No. 200 sieve. The upper 12 to 18 inches of wall backfill should consist of lower permeability soils, in order to reduce surface water infiltration behind walls.

The project structural engineer and/or architect should detail proper wall backdrains, including gravel drain zones, fills, filter fabric and perforated drain pipes. A conceptual wall drainage detail is provided in Figure 4.

6.6 Temporary Construction Slopes

The following recommended slopes should be relatively stable against deep-seated failure, but may experience localized sloughing. On-site soils are considered Type B and Type C soils with recommended slope ratios as set forth in Table 6.6.

TABLE 6.6
RECOMMENDED TEMPORARY SLOPE RATIOS

SOIL TYPE	SLOPE RATIO (Horizontal: vertical)	MAXIMUM HEIGHT
B (Granitic Rock)	1:1 (OR FLATTER)	10 Feet
C (Young Alluvial Fan Deposits and Residual Soil)	1.5:1 (OR FLATTER)	10 Feet

Actual field conditions and soil type designations must be verified by a "competent person" while excavations exist, according to Cal-OSHA regulations. In addition, the above sloping recommendations do not allow for surcharge loading at the top of slopes by vehicular traffic, equipment or materials. Appropriate surcharge setbacks must be maintained from the top of all unshored slopes.

6.7 Foundation and Slab Recommendations

The following recommendations are for preliminary design purposes only. These foundation recommendations should be re-evaluated after review of the project grading and foundation plans, and after completion of rough grading of the building pad areas. Upon completion of rough pad grading, Expansion Index of near surface soils should be verified, and these recommendations should be updated, if necessary.

6.7.1 Foundations

Foundation recommendations presented herein are based on the anticipated low expansion potential of near surface soils after remedial site grading is performed (Expansion Index of 50 or less).

Following the recommended preparatory grading, continuous and isolated spread footings are anticipated to be suitable for use at this site. Foundation dimensions and reinforcement should be based on allowable bearing values of 2,000 pounds per square foot (psf) for minimum 15-inch wide footings embedded a minimum of 24inches below lowest adjacent subgrade elevation. Isolated footings should be at least 24 inches in minimum dimension. The provided bearing value may be increased by 250 psf for each additional six inches of embedment up to a maximum static value of 2,500 psf. The allowable bearing value may be increased by one-third for short-duration loading, which includes the effects of wind or seismic forces. Based on the recommended preparatory grading, it is anticipated that all footings will be founded entirely in properly compacted fill materials. Footings should not span cut to fill interfaces.

Minimum reinforcement for continuous footings should consist of four No. 5 reinforcing bars; two placed near the top and two placed near the bottom, or as per the project structural engineer. The structural engineer should design isolated footing reinforcement. An uncorrected subgrade modulus of 130 pounds per cubic inch is considered suitable for elastic foundation design.

The structural engineer should provide recommendations for reinforcement of any spread footings and footings with pipe penetrations. Footing excavations should generally be maintained at above optimum moisture content until concrete placement.

6.7.2 Foundation Settlement

The maximum total static settlement is expected to be on the order of 1.0 inch and the maximum differential settlement is expected to be on the order of 0.5 inch.

6.7.3 Foundation Setback

Footings for structures should be designed such that the horizontal distance from the face of adjacent slopes to the outer edge of the footing is at least 12 feet. In addition, footings should bear beneath a 1:1 plane extended up from the nearest bottom edge of adjacent trenches and/or excavations. Deepening of affected footings may be a suitable means of attaining the prescribed setbacks.

6.7.4 Interior Concrete Slabs

Lightly loaded interior concrete slabs for non-traffic areas should be a minimum of 5.0 inches thick. Minimum slab reinforcement should consist of #4 reinforcing bars placed on maximum 15-inch centers, each way, at or above mid-slab height, but with proper cover. More stringent recommendations per the project structural engineer supersede these recommendations, as applicable.

In moisture-sensitive floor areas, a suitable vapor retarder of at least 15-mil thickness (with all laps or penetrations sealed or taped) overlying a four-inch layer of consolidated aggregate base or gravel (with SE of 30 or more) should be installed. An optional maximum two-inch layer of similar material may be placed above the vapor retarder to help protect the membrane during steel and concrete placement. This recommended protection is generally considered typical in the industry. If proposed floor areas or coverings are considered especially sensitive to moisture emissions, additional recommendations from a specialty consultant could be obtained. CTE is not an expert at preventing moisture penetration through slabs. A qualified architect or other experienced professional should be contacted if moisture penetration is a more significant concern.

Slabs subjected to heavier loads, racking, or vehicular traffic will require thicker structural slab sections and/or increased reinforcement. A 110-pci subgrade modulus is considered suitable for elastic design of minimally embedded improvements such as slabs-on-grade.

Subgrade materials should be maintained or brought to a minimum of two percent or greater above optimum moisture content until slab underlayment and concrete are placed.

6.8 Seismic Design Criteria

The seismic ground motion values listed in the table below were derived in accordance with the ASCE 7-16 Standard that is incorporated into the 2019 California Building Code. This was accomplished by establishing the Site Class based on the soil properties at the site, and calculating site coefficients and parameters using the using the SEAOC-OSHPD U.S. Seismic Design Maps application. Seismic ground motion values are based on the approximate site coordinates of 33.6489° latitude and –116.9407° longitude. These values are intended for the design of structures to resist the effects of earthquake ground motions.

TABLE 6.8 SEISMIC GROUND MOTION VALUES (CODE-BASED) 2019 CBC AND ASCE 7-16		
PARAMETER	VALUE	2019 CBC/ASCE 7-16 REFERENCE
Site Class	C	ASCE 16, Chapter 20
Mapped Spectral Response Acceleration Parameter, S_S	1.500	Figure 1613.2.1 (1)
Mapped Spectral Response Acceleration Parameter, S_1	0.600	Figure 1613.2.1 (2)
Seismic Coefficient, F_a	1.200	Table 1613.2.3 (1)
Seismic Coefficient, F_v	1.400	Table 1613.2.3 (2)
MCE Spectral Response Acceleration Parameter, S_{MS}	1.800	Section 1613.2.3
MCE Spectral Response Acceleration Parameter, S_{M1}	0.840	Section 1613.2.3
Design Spectral Response Acceleration, Parameter S_{DS}	1.200	Section 1613.2.5(1)
Design Spectral Response Acceleration, Parameter S_{D1}	0.560	Section 1613.2.5 (2)
Peak Ground Acceleration PGA_M	0.740	ASCE 16, Section 11.8.3

6.9 Lateral Resistance and Earth Pressures

Lateral loads acting against structures may be resisted by friction between the footings and the supporting soil or passive pressure acting against structures. If frictional resistance is used, allowable coefficients of friction of 0.30 (total frictional resistance equals the coefficient of friction multiplied by the dead load) for concrete cast directly against compacted fill or native material is recommended. A design passive resistance value of 250 pounds per square foot per foot of depth (with a maximum value of 2,000 pounds per square foot) may be used. The allowable lateral resistance can be taken as the sum of the frictional resistance and the passive resistance, provided the passive resistance does not exceed two-thirds of the total allowable resistance.

If proposed, retaining walls backfilled using granular soils may be designed using the equivalent fluid unit weights given in Table 6.9 below.

TABLE 6.9 EQUIVALENT FLUID UNIT WEIGHTS (G_h) (pounds per cubic foot)		
WALL TYPE	LEVEL BACKFILL	SLOPE BACKFILL 2:1 (HORIZONTAL: VERTICAL)
CANTILEVER WALL (YIELDING)	45	55
RESTRAINED WALL	55	65

Lateral pressures on cantilever retaining walls (yielding walls) over six feet high due to earthquake motions may be calculated based on work by Seed and Whitman (1970). The total lateral earth pressure against a properly drained and backfilled cantilever retaining wall above the groundwater level can be expressed as:

$$P_{AE} = P_A + \Delta P_{AE}$$

For non-yielding (or "restrained") walls, the total lateral earth pressure may be similarly calculated based on work by Wood (1973):

$$P_{KE} = P_K + \Delta P_{KE}$$

Where P_A/b = Static Active Earth Pressure = $G_hH^2/2$

P_K/b = Static Restrained Wall Earth Pressure = $G_hH^2/2$

$\Delta P_{AE}/b$ = Dynamic Active Earth Pressure Increment = $(3/8)\ k_h\ \gamma H^2$

$\Delta P_{KE}/b$ = Dynamic Restrained Earth Pressure Increment = $k_h\ \gamma H^2$

b = unit length of wall (usually 1 foot)

$k_h = 1/2^*\ PGA_m$ (PGA_m given previously Table 6.8)

G_h = Equivalent Fluid Unit Weight (given previously Table 6.9)

H = Total Height of the retained soil

γ = Total Unit Weight of Soil ≈ 135 pounds per cubic foot

*It is anticipated that the 1/2 reduction factor will be appropriate for proposed walls that are not substantially sensitive to movement during the design seismic event. Proposed walls that are more sensitive to such movement could utilize a 2/3 reduction factor. If any proposed walls require minimal to no movement during the design seismic event, no reduction factor to the peak ground acceleration should be used. The project structural engineer of record should determine the appropriate reduction factor to use (if any) based on the specific proposed wall characteristics.

The static and increment of dynamic earth pressure in both cases may be applied with a line of action located at H/3 above the bottom of the wall (SEAOC, 2013).

These values assume non-expansive backfill and free-draining conditions. Measures should be taken to prevent moisture buildup behind all retaining walls. Drainage measures should include free-draining backfill materials and sloped, perforated drains. These drains should discharge to an appropriate off-site location. Waterproofing should be as specified by the project architect.

6.10 Exterior Flatwork

Flatwork should be installed with crack-control joints at appropriate spacing as designed by the project architect to reduce the potential for cracking in exterior flatwork caused by minor movement of subgrade soils and concrete shrinkage. Additionally, it is recommended that flatwork be installed with at least number 4 reinforcing bars at 18-inch centers, each way, at or above mid-height of slab, but with proper concrete cover, or with other reinforcement per the applicable project designer. Flatwork that should be installed with crack control joints, includes driveways, sidewalks, and architectural features. All subgrades should be prepared according to the earthwork recommendations previously given before placing concrete. Positive drainage should be established and maintained next to all flatwork. Subgrade materials should be maintained at a minimum of two percent above optimum moisture content until the time of concrete placement.

6.11 Vehicular Pavement

The proposed improvements include paved vehicle drive and parking areas. Presented in Table 6.11 are preliminary pavement sections utilizing laboratory determined Resistance "R" Value. Actual traffic area slab sections to be provided by the structural designer based on anticipated loading. Beneath proposed pavement areas, the upper 12 inches of subgrade and all base materials should be compacted to 95% relative compaction in accordance with ASTM D1557, and at a minimum of two percent above optimum moisture content.

TABLE 6.11
RECOMMENDED PAVEMENT THICKNESS

Traffic Area	Assumed Traffic Index	Preliminary Subgrade "R"-Value	Asphalt Pavements		Portland Cement Concrete Pavements, on Subgrade Soils (inches)
			AC Thickness (inches)	Class II Aggregate Base Thickness (inches)	
Drive Areas	6.0	40+	4.0	5.0	7.0
Parking Areas	5.0	40+	3.0	4.0	6.5

* Caltrans Class 2 aggregate base
** Concrete should have a modulus of rupture of at least 600 psi

Following rough site grading, CTE laboratory testing of representative subgrade soils for as-graded "R"-Value should be performed to verify adequacy of pavement sections.

Asphalt paved areas should be designed, constructed, and maintained in accordance with the recommendations of the Asphalt Institute, or other widely recognized authority. Concrete paved areas should be designed and constructed in accordance with the recommendations of the American Concrete Institute or other widely recognized authority, particularly with regard to thickened edges, joints, and drainage. The Standard Specifications for Public Works construction ("Greenbook") or Caltrans Standard Specifications may be referenced for pavement materials specifications.

6.12 Drainage

Surface runoff should be collected and directed away from improvements by means of appropriate erosion-reducing devices and positive drainage should be established around the proposed improvements. Positive drainage should be directed away from improvements at a gradient of at least two percent for a distance of at least five feet. However, the project civil engineers should evaluate the on-site drainage and make necessary provisions to keep surface water from affecting the site.

Generally, CTE recommends against allowing water to infiltrate building pads or adjacent to slopes. CTE understands that some agencies are encouraging the use of storm-water cleansing devices. Use of such devices tends to increase the possibility of adverse effects associated with high groundwater including slope instability and liquefaction. See Appendix E for further discussion of site infiltration.

6.13 Slopes

Based on anticipated soil strength characteristics slopes, if proposed, should be constructed at ratios of 2:1 (horizontal: vertical) or flatter. These slope inclinations should exhibit factors of safety greater than 1.5.

Although properly constructed slopes on this site should be grossly stable, the soils will be somewhat erodible. Therefore, runoff water should not be permitted to drain over the edges of slopes unless that water is confined to properly designed and constructed drainage facilities. Erosion-resistant vegetation should be maintained on the face of all slopes.

Typically, soils along the top portion of a fill slope face will creep laterally. CTE recommends against building distress-sensitive hardscape improvements within five feet of slope crests, and against using thickened edges in this area.

6.14 Controlled Low Strength Materials (CLSM)

Controlled Low Strength Materials (CLSM) may be used in deepened footing excavation areas, building pads, and/or adjacent to retaining walls or other structures, provided the appropriate following recommendations are also incorporated. Minimum overexcavation depths recommended herein beneath slabs, flatwork, and other areas may be applicable beneath CLSM if/where CLSM is to be used, and excavation bottoms should be observed by CTE prior to placement of CLSM. Prior to CLSM placement, the excavation should be free of debris, loose soil materials, and water. Once specific areas to utilize CLSM have been determined, CTE should review the locations to determine if additional recommendations are appropriate.

CLSM should consist of a minimum three-sack cement/sand slurry with a minimum 28-day compressive strength of 100 psi (or equal to or greater than the maximum allowable short term soil bearing pressure provided herein, whichever is higher) as determined by ASTM D4832. If re-excavation is anticipated, the compressive strength of CLSM should generally be limited to a maximum of 150 psi per ACI 229R-99. Where re-excavation is required, two-sack cement/sand slurry may be used to help limit the compressive strength. The allowable soils bearing pressure and coefficient of friction provided herein should still govern foundation design. CLSM may not be used in lieu of structural concrete where required by the structural engineer.

6.15 Plan Review

CTE should be authorized to review the project grading and foundation plans prior to commencement of earthwork in order to provide additional recommendations, if necessary.

6.16 Construction Observation

The recommendations provided in this report are based on preliminary design information for the proposed construction and the subsurface conditions observed in the soil borings. The interpolated subsurface conditions should be confirmed by CTE during construction with respect to anticipated conditions. Upon completion of precise grading, if necessary, soil samples will be collected to evaluate as-built Expansion Index. Foundation recommendations may be revised upon completion of grading, and as-built laboratory tests results. Additionally, soil samples should be taken in pavement subgrade areas upon rough grading to refine pavement recommendations as necessary.

Recommendations provided in this report are based on the understanding and assumption that CTE will provide the observation and testing services for the project. All earthwork should be observed and tested in accordance with recommendations contained within this report. CTE should evaluate footing excavations before reinforcing steel placement.

7.0 LIMITATIONS OF INVESTIGATION

The field evaluation, laboratory testing and geotechnical analysis presented in this report have been conducted according to current engineering practice and the standard of care exercised by reputable geotechnical consultants performing similar tasks in this area. No other warranty, expressed or implied, is made regarding the conclusions, recommendations and opinions expressed in this report. Variations may exist and conditions not observed or described in this report may be encountered during construction. This report is prepared for the project as described. It is not prepared for any other property or party.

The recommendations provided herein have been developed in order to reduce the post-construction movement of site improvements related to soil settlement. However, even with the design and construction recommendations presented herein, some post-construction movement and associated distress may occur.

The findings of this report are valid as of the present date. However, changes in the conditions of a property can occur with the passage of time, whether they are due to natural processes or the works of man on this or adjacent properties. In addition, changes in applicable or appropriate standards may occur, whether they result from legislation or the broadening of knowledge. Accordingly, the findings of this report may be invalidated wholly or partially by changes outside CTE's involvement. Therefore, this report is subject to review and should not be relied upon after a period of three years.

CTE's conclusions and recommendations are based on an analysis of the observed conditions. If conditions different from those described in this report are encountered, CTE should be notified and additional recommendations, if required, will be provided subject to CTE remaining as authorized geotechnical consultant of record. This report is for use of the project as described. It should not be utilized for any other project.

The percolation test results were obtained in accordance with regional standards and were performed with the standard of care practiced by other professionals practicing in the area. However, percolation test results can significantly vary laterally and vertically due to slight changes in soil type, degree of weathering, secondary mineralization, and other physical and chemical variabilities. As such, the test results are only considered as an estimate of percolation and converted infiltration rates for design purposes. No guarantee is made based on the percolation testing to the actual functionality or longevity of associated infiltration basins or other BMP devices designed from the presented infiltration rates.

CTE appreciates this opportunity to be of service on this project. If you have any questions regarding this report, please do not hesitate to contact the undersigned.

Respectfully submitted,

CONSTRUCTION TESTING & ENGINEERING, INC.

Dan T. Math, GE #2665
Principal Engineer

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Principal Engineering Geologist

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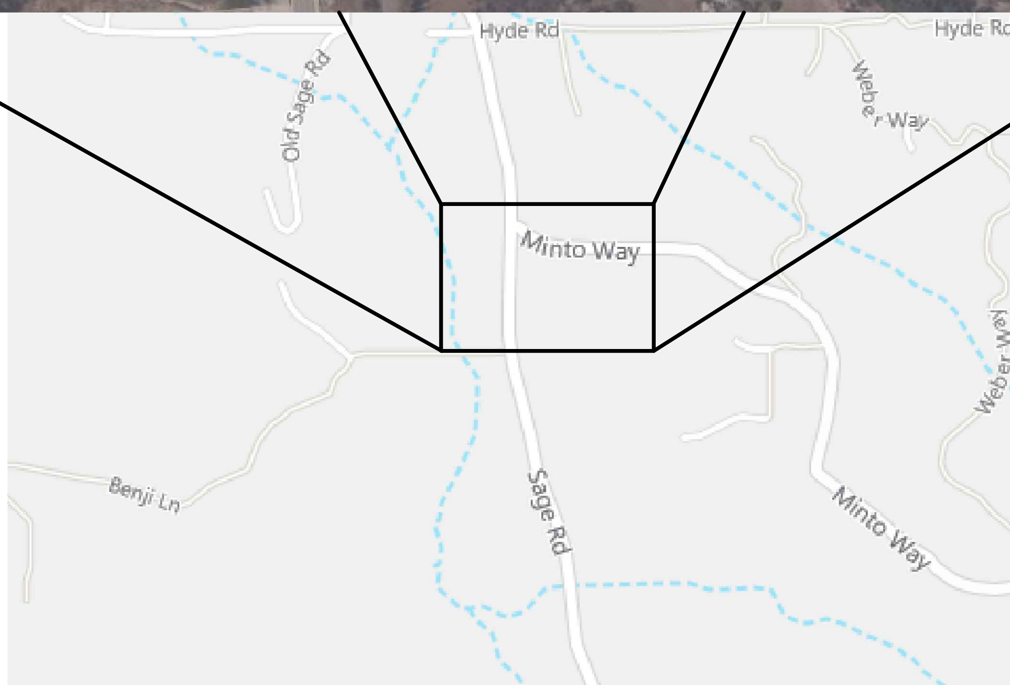
Certified Engineering Geologist

AJB/JFL/DTM:ach



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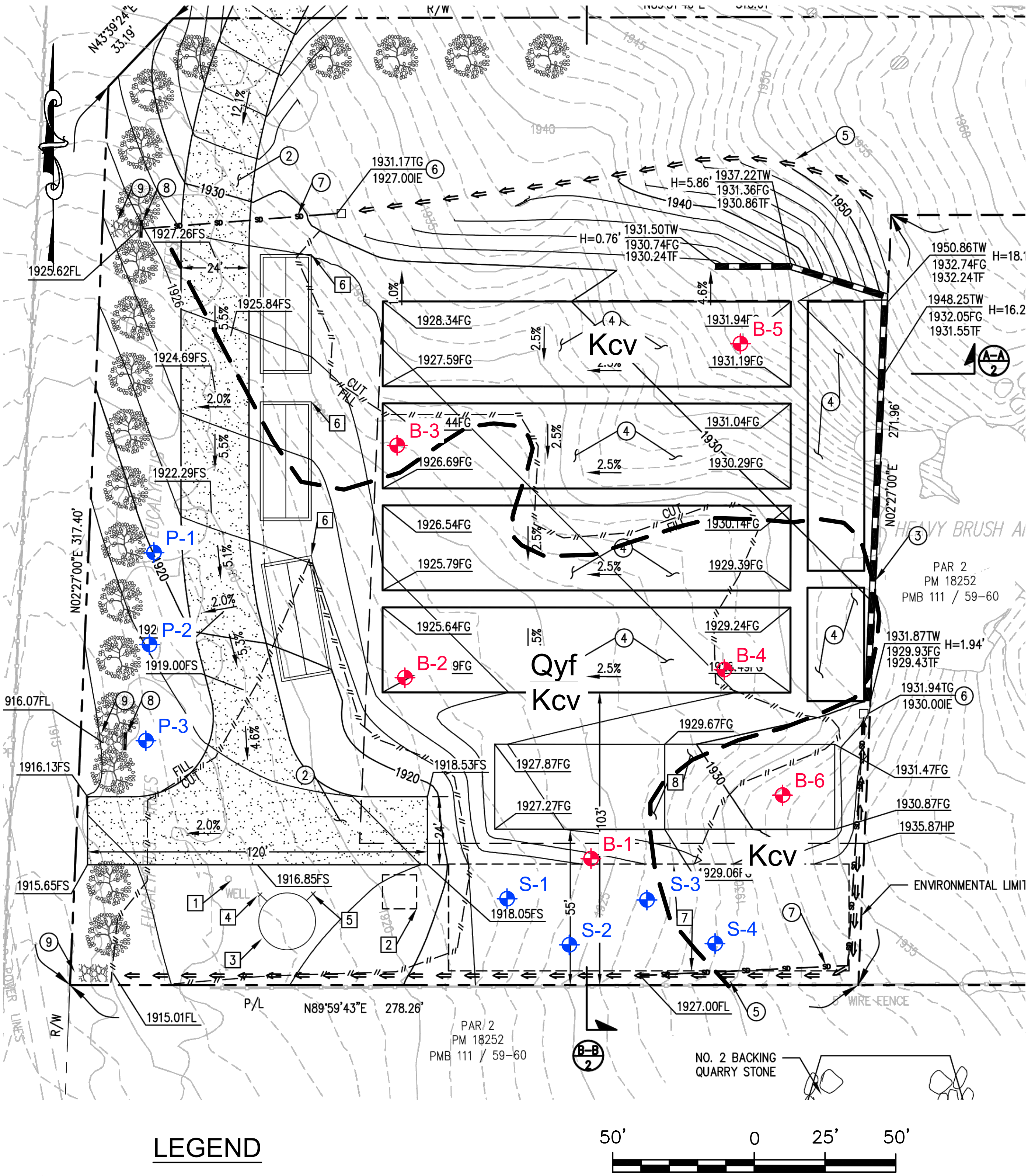
SITE INDEX MAP PROPOSED VORTEX FARMS SOUTHEAST OF SAGE ROAD AND MINTO WAY RIVERSIDE, CALIFORNIA	SCALE: AS SHOWN	DATE: 10/20
	CTE JOB NO.: 10-12390T	FIGURE: 1



LEGEND

B-1 Approximate Boring Location

P-3 Approximate BMP Perc Test Location

S-4 Approximate Septic Perc Test Location

Qyf / Kcv Young Alluvial Fan Deposits over Tonalite of Coahuila Valley

Approximate Geologic Contact

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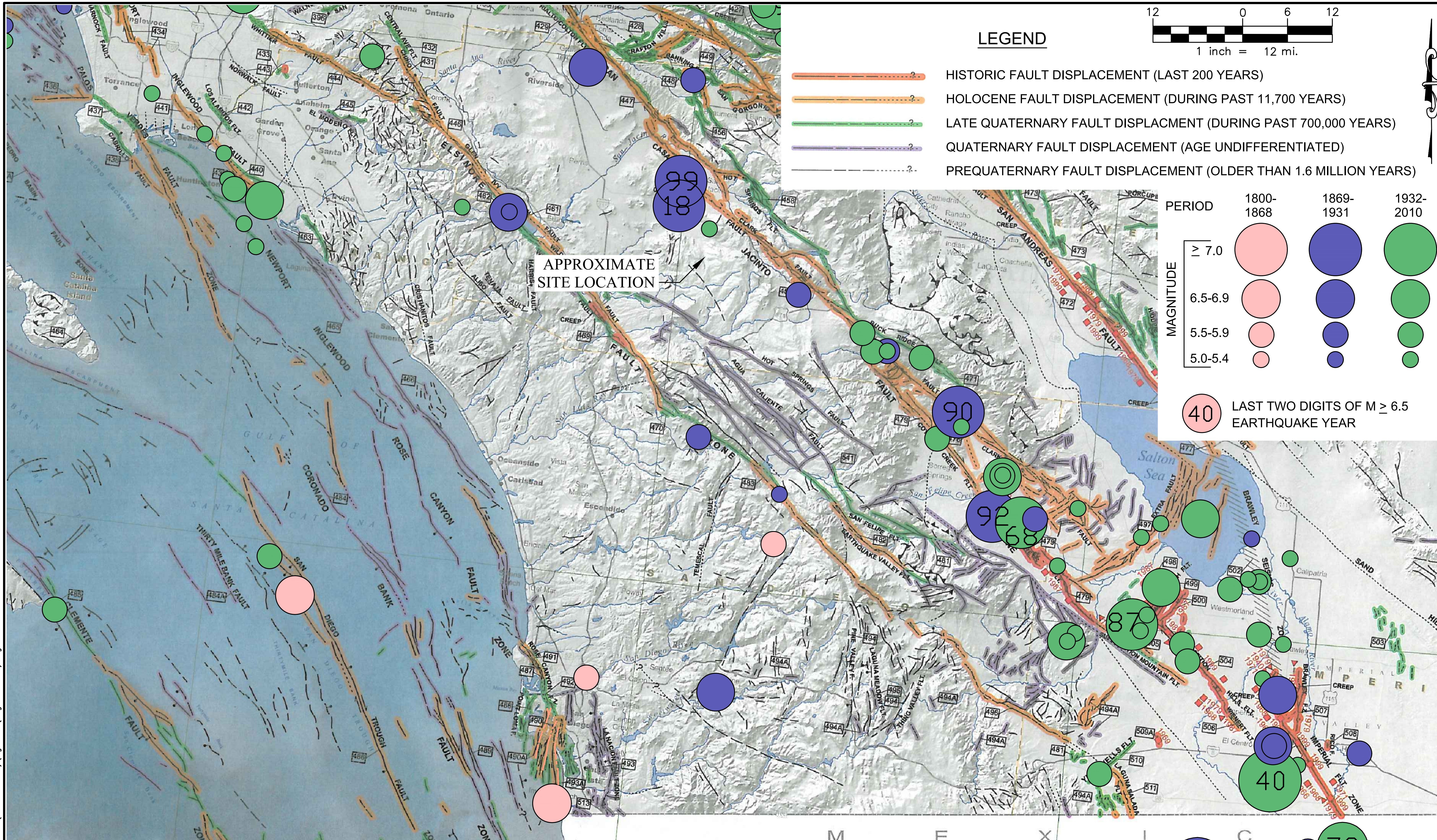
GEOLOGIC/EXPLORATION LOCATION MAP	SCALE: 1"=50'	DATE: 10/20
PROPOSED VORTEX FARMS SOUTHEAST CORNER OF SAGE ROAD AND MINTO WAY RIVERSIDE, CALIFORNIA	CTE JOB NO.: 10-15741G	FIGURE: 2

S:\Projects\10-15741G (Vortex Farms)\Figure 2.dwg



NOTES: FAULT ACTIVITY MAP OF CALIFORNIA, 2010, CALIFORNIA GEOLOGIC DATA MAP SERIES MAP NO. 6;
EPICENTERS OF AND AREAS DAMAGED BY M≥5 CALIFORNIA EARTHQUAKES, 1800-1999 ADAPTED AFTER TOPPOZADA, BRANUM, PETERSEN, HALLSTORM, CRAMER, AND REICHLE, 2000, CDMG MAP SHEET 49
REFERENCE FOR ADDITIONAL EXPLANATION; MODIFIED WITH CISN AND USGS SEISMIC MAPS

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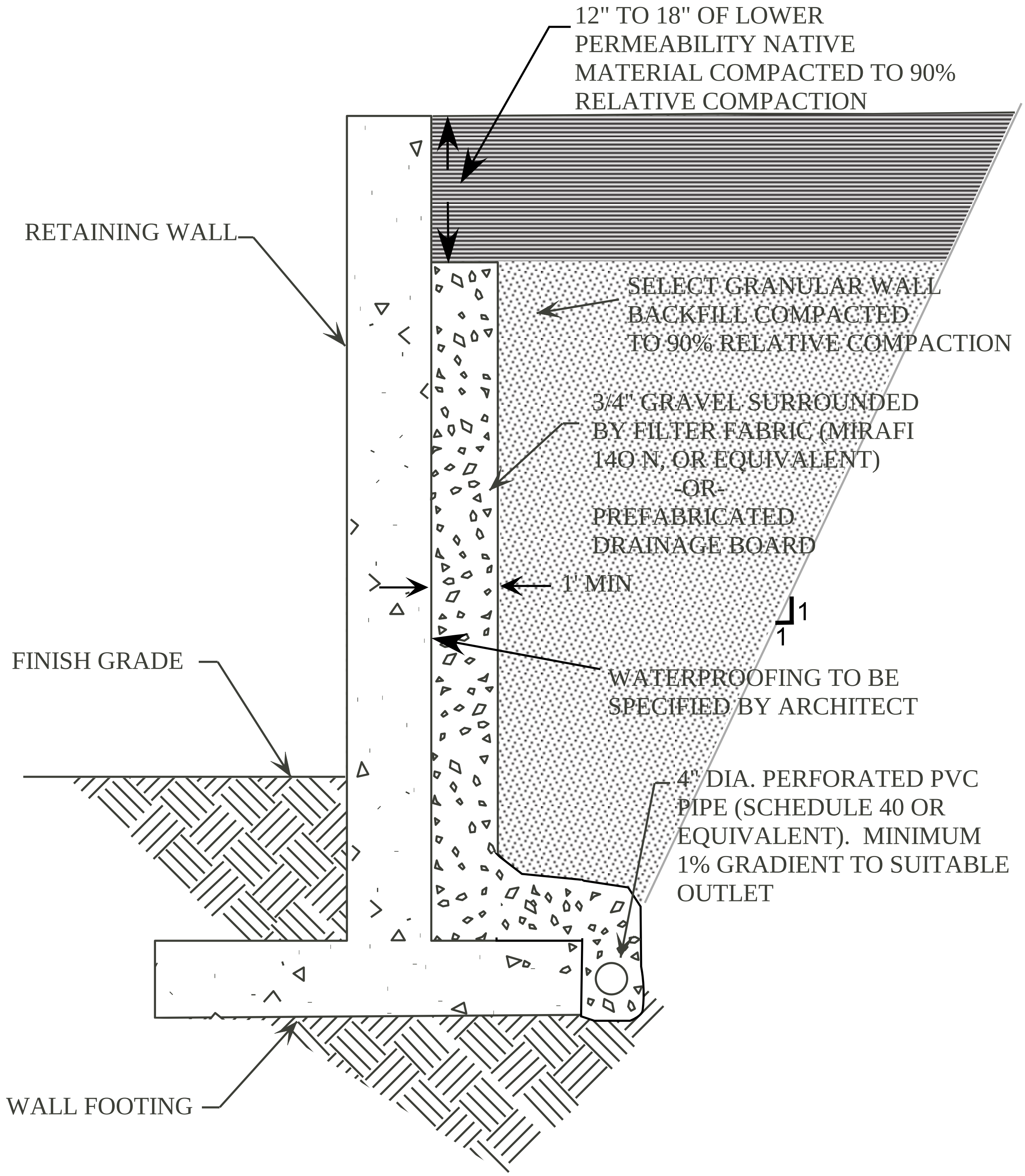
REGIONAL FAULT AND SEISMICITY MAP
PROPOSED VORTEX FARMS
SOUTHEAST OF SAGE RFOAD AND MINTO WAY
RIVERSIDE, CALIFORNIA

CTE JOB NO:	10-15741G
SCALE:	1 inch = 12 miles
DATE: 10/20	FIGURE: 3

S:\Projects\10-15741G (Vortex Farms)\Figure 3 (Regional Fault).dwg



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RETAINING WALL DRAINAGE DETAIL

CTE JOB NO: 10-15741G		
SCALE: NO SCALE		
DATE: 10/20	FIGURE: 4	

APPENDIX A

REFERENCES

REFERENCES

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APPENDIX B

BORING LOGS

CTE INC.
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DEFINITION OF TERMS

PRIMARY DIVISIONS			SYMBOLS	SECONDARY DIVISIONS
COARSE GRAINED SOILS MORE THAN HALF OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	**GRAVELS** MORE THAN HALF OF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE	CLEAN GRAVELS < 5% FINES	GW	WELL GRADED GRAVELS, GRAVEL-SAND MIXTURES LITTLE OR NO FINES
			GP	POORLY GRADED GRAVELS OR GRAVEL SAND MIXTURES, LITTLE OF NO FINES
		GRAVELS WITH FINES	GM	SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES, NON-PLASTIC FINES
			GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES, PLASTIC FINES
	SANDS MORE THAN HALF OF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE	CLEAN SANDS < 5% FINES	SW	WELL GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
			SP	POORLY GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
		SANDS WITH FINES	SM	SILTY SANDS, SAND-SILT MIXTURES, NON-PLASTIC FINES
			SC	CLAYEY SANDS, SAND-CLAY MIXTURES, PLASTIC FINES
FINE GRAINED SOILS MORE THAN HALF OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	**SILTS AND CLAYS** LIQUID LIMIT IS LESS THAN 50		ML	INORGANIC SILTS, VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS, SLIGHTLY PLASTIC CLAYEY SILTS
			CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY, SANDY, SILTS OR LEAN CLAYS
			OL	ORGANIC SILTS AND ORGANIC CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS LIQUID LIMIT IS GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SANDY OR SILTY SOILS, ELASTIC SILTS
			CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
			OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTY CLAYS
HIGHLY ORGANIC SOILS			PT	PEAT AND OTHER HIGHLY ORGANIC SOILS

GRAIN SIZES

BOULDERS	COBBLES	GRAVEL		SAND			SILTS AND CLAYS
		COARSE	FINE	COARSE	MEDIUM	FINE	

12" 3" 3/4" 4 10 40 200

CLEAR SQUARE SIEVE OPENING U.S. STANDARD SIEVE SIZE

ADDITIONAL TESTS

(OTHER THAN TEST PIT AND BORING LOG COLUMN HEADINGS)

MAX- Maximum Dry Density
GS- Grain Size Distribution
SE- Sand Equivalent
EI- Expansion Index
CHM- Sulfate and Chloride
Content , pH, Resistivity
COR - Corrosivity
SD- Sample Disturbed

PM- Permeability
SG- Specific Gravity
HA- Hydrometer Analysis
AL- Atterberg Limits
RV- R-Value
CN- Consolidation
CP- Collapse Potential
HC- Hydrocollapse
REM- Remolded

PP- Pocket Penetrometer
WA- Wash Analysis
DS- Direct Shear
UC- Unconfined Compression
MD- Moisture/Density
M- Moisture
SC- Swell Compression
OI- Organic Impurities

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PROJECT:	DRILLER:	SHEET: of
CTE JOB NO:	DRILL METHOD:	DRILLING DATE:
LOGGED BY:	SAMPLE METHOD:	ELEVATION:

BORING LEGEND

Depth (Feet)	Bulk Sample	Driven Type	Blows/Foot	Dry Density (pcf)	Moisture (%)	U.S.C.S. Symbol	Graphic Log	DESCRIPTION	Laboratory Tests
0								Block or Chunk Sample	
								Bulk Sample	
5									
								Standard Penetration Test	
10								Modified Split-Barrel Drive Sampler (Cal Sampler)	
								Thin Walled Army Corp. of Engineers Sample	
15								Groundwater Table	
								Soil Type or Classification Change	
20								? ? ? ? ? ? ?	
								Formation Change [(Approximate boundaries queried (?)]	
25						"SM"		Quotes are placed around classifications where the soils exist in situ as bedrock	

FIGURE: BL2

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PROJECT:	VORTEX FARMS	DRILLER:	BAJA EXPLORATION	SHEET:	1 of 1
CTE JOB NO:	10-15741G	DRILL METHOD:	HOLLOW-STEM AUGER	DRILLING DATE:	9/30/2020
LOGGED BY:	AJB	SAMPLE METHOD:	RING, SPT and BULK	ELEVATION:	~1924 FEET

BORING: B-1

Depth (Feet)	Bulk Sample	Driven Type	Blows/6"	Dry Density (pcf)	Moisture (%)	U.S.C.S. Symbol	Graphic Log	DESCRIPTION	Laboratory Tests
0						SM		**QUATERNARY YOUNG ALLUVIAL FAN DEPOSITS:** Loose to medium dense, dry, grayish brown, silty fine to coarse grained SAND, friable, massive.	
						"SM"		**CRETACEOUS TONALITE OF THE COAHUILA VALLEY:** Very dense, slightly moist, reddish gray tonalite that excavates to silty fine to medium grained SAND, oxidized, severely weathered.	
5			28 29 50/6"						
10			50/6"						
15			50/2"						
								Total Depth: 15.2' No Groundwater Encountered	
20									
25									

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PROJECT:	VORTEX FARMS	DRILLER:	BAJA EXPLORATION	SHEET:	1 of 1
CTE JOB NO:	10-15741G	DRILL METHOD:	HOLLOW-STEM AUGER	DRILLING DATE:	9/30/2020
LOGGED BY:	AJB	SAMPLE METHOD:	RING, SPT and BULK	ELEVATION:	~1922 FEET

BORING: B-2

Depth (Feet)	Bulk Sample	Driven Type	Blows/6"	Dry Density (pcf)	Moisture (%)	U.S.C.S. Symbol	Graphic Log	DESCRIPTION	Laboratory Tests
0	X (0–5)					SM		**QUATERNARY YOUNG ALLUVIAL FAN DEPOSITS:** Loose to medium dense, dry, grayish brown, silty fine to coarse grained SAND, friable, massive.	
~3									EI, RV
5		Ring	8 11 13						GS
10		SPT	18 17 32						MD, CN
~11.5						"SM"		**CRETACEOUS TONALITE OF THE COAHUILA VALLEY:** Very dense, slightly moist, reddish gray tonalite that excavates to silty fine to medium grained SAND, oxidized, severely weathered.	
15		Ring	50/3"						GS
20								Total Depth: 20' No Groundwater Encountered	
25									

PROJECT:	VORTEX FARMS	DRILLER:	BAJA EXPLORATION	SHEET:	1 of 1
CTE JOB NO:	10-15741G	DRILL METHOD:	HOLLOW-STEM AUGER	DRILLING DATE:	9/30/2020
LOGGED BY:	AJB	SAMPLE METHOD:	RING, SPT and BULK	ELEVATION:	~1926 FEET

BORING: B-3

Depth (Feet)	Bulk Sample	Driven Type	Blows/6"	Dry Density (pcf)	Moisture (%)	U.S.C.S. Symbol	Graphic Log	DESCRIPTION	Laboratory Tests
0						SM		**RESIDUAL SOIL:** Loose to medium dense, dry, grayish brown, silty fine to coarse grained SAND, friable, massive.	
						"SM"		**CRETACEOUS TONALITE OF THE COAHUILA VALLEY:** Very dense, slightly moist, reddish gray tonalite that excavates to silty fine to medium grained SAND, oxidized, severely weathered.	
5			14 31 50/4"						
								Total Depth: 6.4' No Groundwater Encountered	
10									
15									
20									
25									

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PROJECT:	VORTEX FARMS	DRILLER:	BAJA EXPLORATION	SHEET:	1 of 1
CTE JOB NO:	10-15741G	DRILL METHOD:	HOLLOW-STEM AUGER	DRILLING DATE:	9/30/2020
LOGGED BY:	AJB	SAMPLE METHOD:	RING, SPT and BULK	ELEVATION:	~1928 FEET

BORING: B-4

Depth (Feet)	Bulk Sample	Driven Type	Blows/6"	Dry Density (pcf)	Moisture (%)	U.S.C.S. Symbol	Graphic Log	DESCRIPTION	Laboratory Tests
0	X					SM		**QUATERNARY YOUNG ALLUVIAL FAN DEPOSITS:** Loose to medium dense, dry, grayish brown, silty fine to coarse grained SAND, friable, massive.	MAX, CHM
5		X	13 10 13						
						"SM"		**CRETACEOUS TONALITE OF THE COAHUILA VALLEY:** Very dense, slightly moist, reddish gray tonalite that excavates to silty fine to medium grained SAND, oxidized, severely weathered.	
10		X	50/6"						
15		X	50/3"						
								Total Depth: 15.3' No Groundwater Encountered	
20									
25									

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PROJECT:	VORTEX FARMS	DRILLER:	BAJA EXPLORATION	SHEET:	1 of 1
CTE JOB NO:	10-15741G	DRILL METHOD:	HOLLOW-STEM AUGER	DRILLING DATE:	9/30/2020
LOGGED BY:	AJB	SAMPLE METHOD:	RING, SPT and BULK	ELEVATION:	~1940 FEET

BORING: B-5

Depth (Feet)	Bulk Sample	Driven Type	Blows/6"	Dry Density (pcf)	Moisture (%)	U.S.C.S. Symbol	Graphic Log	DESCRIPTION	Laboratory Tests
0	X					SM		**RESIDUAL SOIL:** Loose to medium dense, dry, grayish brown, silty fine to coarse grained SAND, friable, massive.	
						"SM"		**CRETACEOUS TONALITE OF THE COAHUILA VALLEY:** Very dense, slightly moist, reddish gray tonalite that excavates to silty fine to medium grained SAND, oxidized, severely weathered.	
								Total Depth: 1.4' 9Refusal in bedrock) No Groundwater Encountered	
5									
10									
15									
20									
25									

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PROJECT:	VORTEX FARMS	DRILLER:	BAJA EXPLORATION	SHEET:	1 of 1
CTE JOB NO:	10-15741G	DRILL METHOD:	HOLLOW-STEM AUGER	DRILLING DATE:	9/30/2020
LOGGED BY:	AJB	SAMPLE METHOD:	RING, SPT and BULK	ELEVATION:	~1932 FEET

BORING: B-6

Depth (Feet)	Bulk Sample	Driven Type	Blows/6"	Dry Density (pcf)	Moisture (%)	U.S.C.S. Symbol	Graphic Log	DESCRIPTION	Laboratory Tests
0	X					SM		**<u>QUATERNARY YOUNG ALLUVIAL FAN DEPOSITS</u>:** Loose to medium dense, dry, grayish brown, silty fine to coarse grained SAND, friable, massive.	
5									
						"SM"		**<u>CRETACEOUS TONALITE OF THE COAHUILA VALLEY</u>:** Very dense, slightly moist, reddish gray tonalite that excavates to silty fine to medium grained SAND, oxidized, severely weathered.	
								Total Depth: 7.1' (Refusal in bedrock) No Groundwater Encountered	
10									
15									
20									
25									

APPENDIX C

LABORATORY METHODS AND RESULTS

APPENDIX C
LABORATORY METHODS AND RESULTS

Laboratory Testing Program

Laboratory tests were performed on representative soil samples to detect their relative engineering properties. Tests were performed following test methods of the American Society for Testing Materials or other accepted standards. The following presents a brief description of the various test methods used.

Classification

Soils were classified visually according to the Unified Soil Classification System. Visual classifications were supplemented by laboratory testing of selected samples according to ASTM D2487. The soil classifications are shown on the Exploration Logs in Appendix B.

In-Place Moisture/Density

The in-place moisture content and dry unit weight of selected samples were determined using relatively undisturbed chunk soil samples.

Modified Proctor

Laboratory maximum dry density and optimum moisture content were evaluated according to ASTM D 1557, Method A. A mechanically operated rammer was used during the compaction process.

Expansion Index

Expansion testing was performed on selected samples of the matrix of the on-site soils according to ASTM D 4829.

Resistance "R" Value

The resistance "R"-value was measured by the California Test. 301. The graphically determined "R" value at an exudation pressure of 300 pounds per square inch is the value used for pavement section calculation.

Particle-Size Analysis

Particle-size analyses were performed on selected representative samples according to ASTM D 422.

Consolidation

To assess their compressibility and volume change behavior when loaded and wetted, relatively undisturbed samples of representative samples from the investigation were subject to consolidation tests in accordance with ASTM D 2435.

Chemical Analysis

Soil materials were collected with sterile sampling equipment and tested for Sulfate and Chloride content, pH, Corrosivity, and Resistivity.

EXPANSION INDEX TEST

ASTM D 4829

LOCATION	DEPTH (feet)	EXPANSION INDEX	EXPANSION POTENTIAL
B-2	0-5	4	VERY LOW

IN-PLACE MOISTURE AND DENSITY

LOCATION	DEPTH (feet)	% MOISTURE	DRY DENSITY
B-2	10	2.8	115.8

RESISTANCE "R"-VALUE

CALTEST 301

LOCATION	DEPTH (feet)	R-VALUE
B-2	0-5	48

SULFATE

LOCATION	DEPTH (feet)	RESULTS ppm
B-4	0-5	200.31

CHLORIDE

LOCATION	DEPTH (feet)	RESULTS ppm
B-4	0-5	28.4

p.H.

LOCATION	DEPTH (feet)	RESULTS
B-4	0-5	6.4

RESISTIVITY

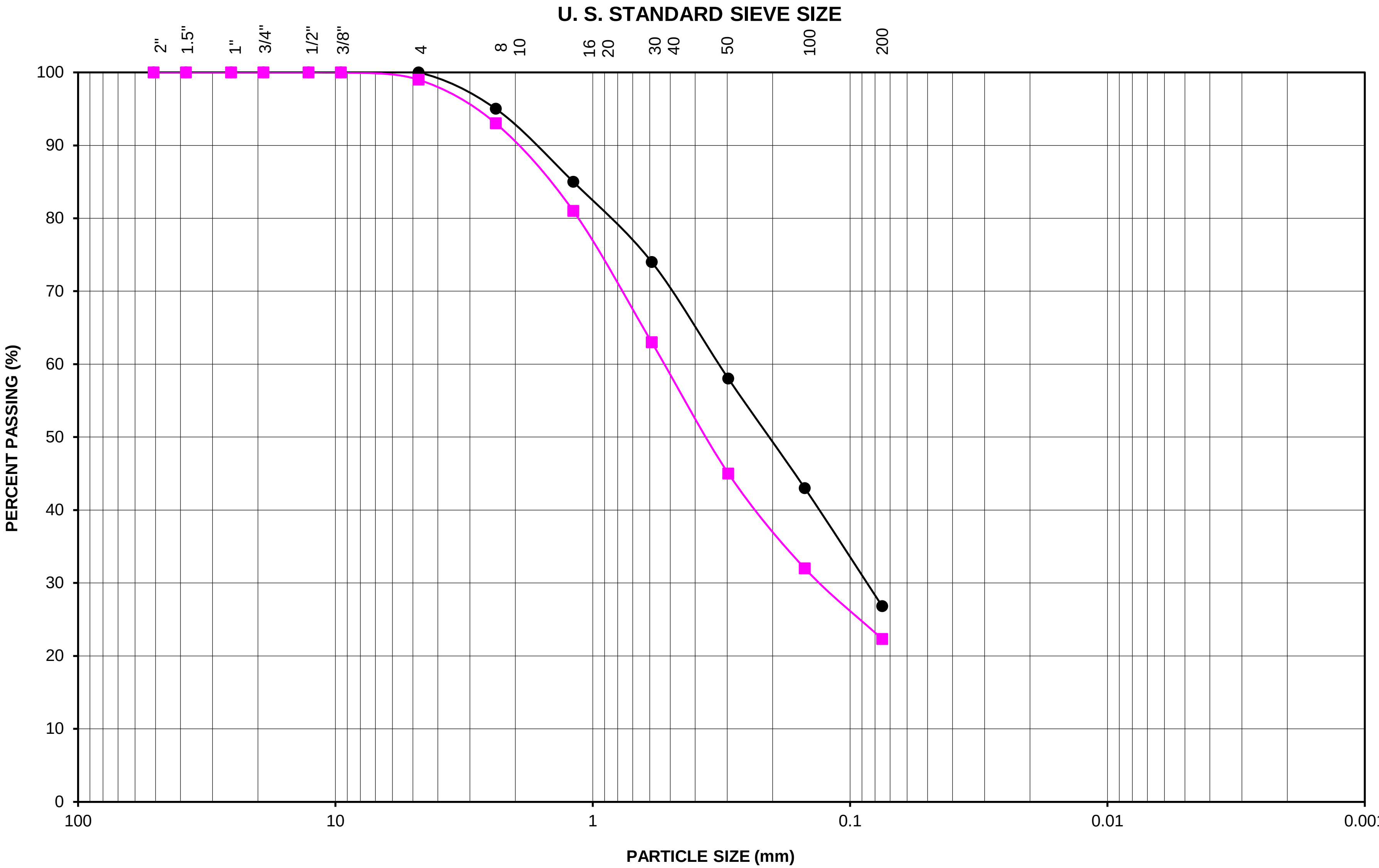
CALIFORNIA TEST 424

LOCATION	DEPTH (feet)	RESULTS ohms-cm
B-4	0-5	39500

MODIFIED PROCTOR

ASTM D 1557

LOCATION	DEPTH (feet)	MAXIUM DRY DENSITY (PCF)	OPTIMUM MOISTURE (%)
B-4	0-6.5	119.5	8.4



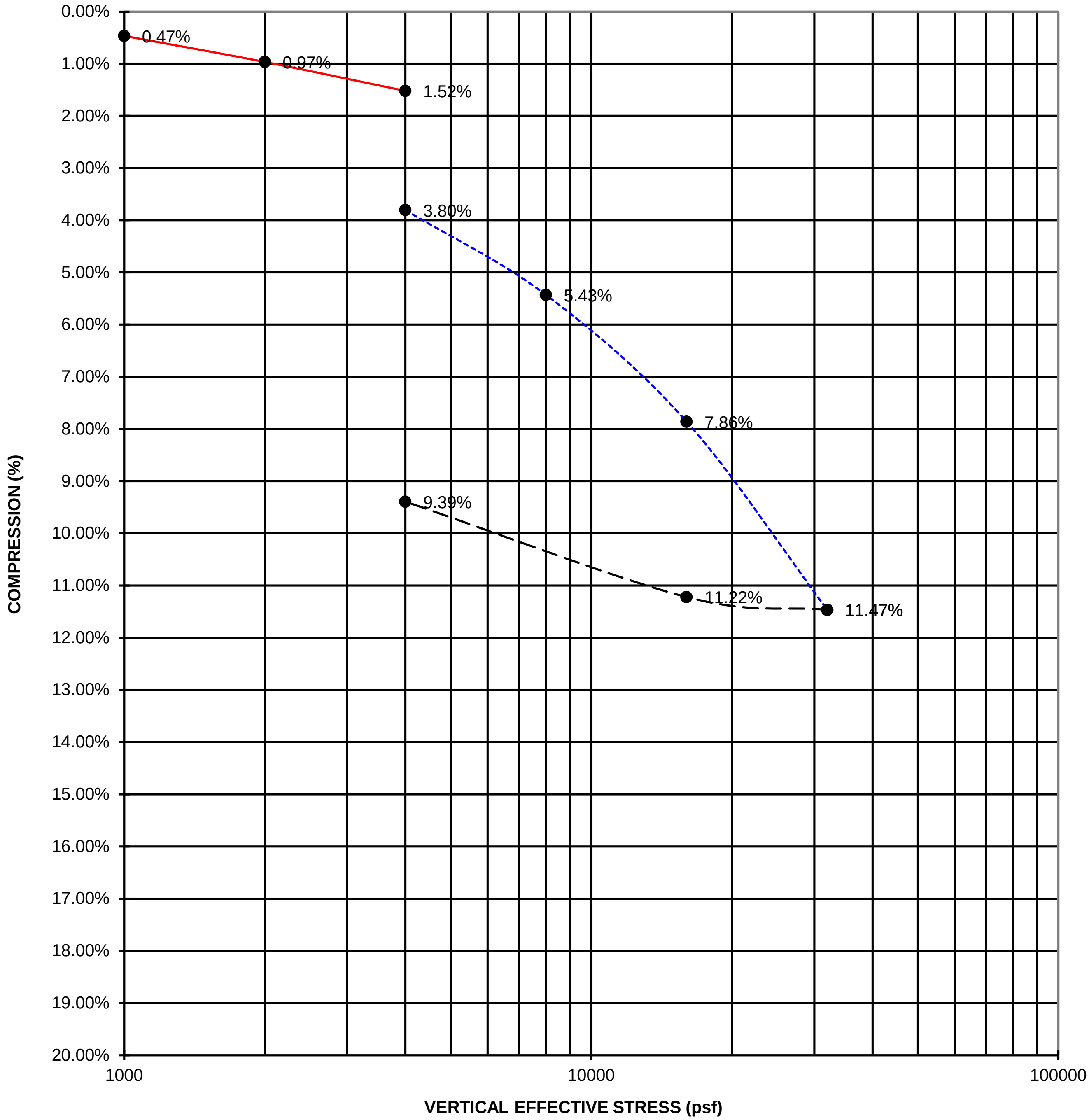
PARTICLE SIZE ANALYSIS

CTE INC.
Construction Testing & Engineering, Inc.
1441 Montiel Rd Ste 115, Escondido, CA 92026 Ph (760) 746-4955

Sample Designation	Sample Depth (feet)	Symbol	Liquid Limit (%)	Plasticity Index	Classification
B-2	5	●			SM
B-2	15	■			SM
CTE JOB NUMBER:		10-15741G		FIGURE:	C-1

Construction Testing & Engineering, Inc.

Inspection | Testing | Geotechnical | Environmental & Construction Engineering | Civil Engineering | Surveying



Consolidation Test ASTM D2435

Project Name:	Vortex Farms				
Project Number:	10-15741G	Sample Date:	9/30/2020	Initial Moisture (%):	2.8
Lab Number:	31320	Test Date:	10/5/2020	Final Moisture (%):	9.0
Sample Location:	B-2 @ 10'	Tested By:	JH	Initial Dry Density (PCF):	115.8
Sample Description:	Moderate Brown (SM)			Final Dry Density (PCF):	127.8

APPENDIX D

STANDARD SPECIFICATIONS FOR GRADING

Section 1 - General

Construction Testing & Engineering, Inc. presents the following standard recommendations for grading and other associated operations on construction projects. These guidelines should be considered a portion of the project specifications. Recommendations contained in the body of the previously presented soils report shall supersede the recommendations and or requirements as specified herein. The project geotechnical consultant shall interpret disputes arising out of interpretation of the recommendations contained in the soils report or specifications contained herein.

Section 2 - Responsibilities of Project Personnel

The geotechnical consultant should provide observation and testing services sufficient to general conformance with project specifications and standard grading practices. The geotechnical consultant should report any deviations to the client or his authorized representative.

The Client should be chiefly responsible for all aspects of the project. He or his authorized representative has the responsibility of reviewing the findings and recommendations of the geotechnical consultant. He shall authorize or cause to have authorized the Contractor and/or other consultants to perform work and/or provide services. During grading the Client or his authorized representative should remain on-site or should remain reasonably accessible to all concerned parties in order to make decisions necessary to maintain the flow of the project.

The Contractor is responsible for the safety of the project and satisfactory completion of all grading and other associated operations on construction projects, including, but not limited to, earth work in accordance with the project plans, specifications and controlling agency requirements.

Section 3 - Preconstruction Meeting

A preconstruction site meeting should be arranged by the owner and/or client and should include the grading contractor, design engineer, geotechnical consultant, owner’s representative and representatives of the appropriate governing authorities.

Section 4 - Site Preparation

The client or contractor should obtain the required approvals from the controlling authorities for the project prior, during and/or after demolition, site preparation and removals, etc. The appropriate approvals should be obtained prior to proceeding with grading operations.

Clearing and grubbing should consist of the removal of vegetation such as brush, grass, woods, stumps, trees, root of trees and otherwise deleterious natural materials from the areas to be graded. Clearing and grubbing should extend to the outside of all proposed excavation and fill areas.

Demolition should include removal of buildings, structures, foundations, reservoirs, utilities (including underground pipelines, septic tanks, leach fields, seepage pits, cisterns, mining shafts, tunnels, etc.) and other man-made surface and subsurface improvements from the areas to be graded. Demolition of utilities should include proper capping and/or rerouting pipelines at the project perimeter and cutoff and capping of wells in accordance with the requirements of the governing authorities and the recommendations of the geotechnical consultant at the time of demolition.

Trees, plants or man-made improvements not planned to be removed or demolished should be protected by the contractor from damage or injury.

Debris generated during clearing, grubbing and/or demolition operations should be wasted from areas to be graded and disposed off-site. Clearing, grubbing and demolition operations should be performed under the observation of the geotechnical consultant.

Section 5 - Site Protection

Protection of the site during the period of grading should be the responsibility of the contractor. Unless other provisions are made in writing and agreed upon among the concerned parties, completion of a portion of the project should not be considered to preclude that portion or adjacent areas from the requirements for site protection until such time as the entire project is complete as identified by the geotechnical consultant, the client and the regulating agencies.

Precautions should be taken during the performance of site clearing, excavations and grading to protect the work site from flooding, ponding or inundation by poor or improper surface drainage. Temporary provisions should be made during the rainy season to adequately direct surface drainage away from and off the work site. Where low areas cannot be avoided, pumps should be kept on hand to continually remove water during periods of rainfall.

Rain related damage should be considered to include, but may not be limited to, erosion, silting, saturation, swelling, structural distress and other adverse conditions as determined by the geotechnical consultant. Soil adversely affected should be classified as unsuitable materials and should be subject to overexcavation and replacement with compacted fill or other remedial grading as recommended by the geotechnical consultant.

The contractor should be responsible for the stability of all temporary excavations. Recommendations by the geotechnical consultant pertaining to temporary excavations (e.g., backcuts) are made in consideration of stability of the completed project and, therefore, should not be considered to preclude the responsibilities of the contractor. Recommendations by the geotechnical consultant should not be considered to preclude requirements that are more restrictive by the regulating agencies. The contractor should provide during periods of extensive rainfall plastic sheeting to prevent unprotected slopes from becoming saturated and unstable. When deemed appropriate by the geotechnical consultant or governing agencies the contractor shall install checkdams, desilting basins, sand bags or other drainage control measures.

In relatively level areas and/or slope areas, where saturated soil and/or erosion gullies exist to depths of greater than 1.0 foot; they should be overexcavated and replaced as compacted fill in accordance with the applicable specifications. Where affected materials exist to depths of 1.0 foot or less below proposed finished grade, remedial grading by moisture conditioning in-place, followed by thorough recompaction in accordance with the applicable grading guidelines herein may be attempted. If the desired results are not achieved, all affected materials should be overexcavated and replaced as compacted fill in accordance with the slope repair recommendations herein. If field conditions dictate, the geotechnical consultant may recommend other slope repair procedures.

Section 6 - Excavations

6.1 Unsuitable Materials

Materials that are unsuitable should be excavated under observation and recommendations of the geotechnical consultant. Unsuitable materials include, but may not be limited to, dry, loose, soft, wet, organic compressible natural soils and fractured, weathered, soft bedrock and nonengineered or otherwise deleterious fill materials.

Material identified by the geotechnical consultant as unsatisfactory due to its moisture conditions should be overexcavated; moisture conditioned as needed, to a uniform at or above optimum moisture condition before placement as compacted fill.

If during the course of grading adverse geotechnical conditions are exposed which were not anticipated in the preliminary soil report as determined by the geotechnical consultant additional exploration, analysis, and treatment of these problems may be recommended.

6.2 Cut Slopes

Unless otherwise recommended by the geotechnical consultant and approved by the regulating agencies, permanent cut slopes should not be steeper than 2:1 (horizontal: vertical).

The geotechnical consultant should observe cut slope excavation and if these excavations expose loose cohesionless, significantly fractured or otherwise unsuitable material, the materials should be overexcavated and replaced with a compacted stabilization fill. If encountered specific cross section details should be obtained from the Geotechnical Consultant.

When extensive cut slopes are excavated or these cut slopes are made in the direction of the prevailing drainage, a non-erodible diversion swale (brow ditch) should be provided at the top of the slope.

6.3 Pad Areas

All lot pad areas, including side yard terrace containing both cut and fill materials, transitions, located less than 3 feet deep should be overexcavated to a depth of 3 feet and replaced with a uniform compacted fill blanket of 3 feet. Actual depth of overexcavation may vary and should be delineated by the geotechnical consultant during grading, especially where deep or drastic transitions are present.

For pad areas created above cut or natural slopes, positive drainage should be established away from the top-of-slope. This may be accomplished utilizing a berm drainage swale and/or an appropriate pad gradient. A gradient in soil areas away from the top-of-slopes of 2 percent or greater is recommended.

Section 7 - Compacted Fill

All fill materials should have fill quality, placement, conditioning and compaction as specified below or as approved by the geotechnical consultant.

7.1 Fill Material Quality

Excavated on-site or import materials which are acceptable to the geotechnical consultant may be utilized as compacted fill, provided trash, vegetation and other deleterious materials are removed prior to placement. All import materials anticipated for use on-site should be sampled tested and approved prior to and placement is in conformance with the requirements outlined.

Rocks 12 inches in maximum and smaller may be utilized within compacted fill provided sufficient fill material is placed and thoroughly compacted over and around all rock to effectively fill rock voids. The amount of rock should not exceed 40 percent by dry weight passing the 3/4-inch sieve. The geotechnical consultant may vary those requirements as field conditions dictate.

Where rocks greater than 12 inches but less than four feet of maximum dimension are generated during grading, or otherwise desired to be placed within an engineered fill, special handling in accordance with the recommendations below. Rocks greater than four feet should be broken down or disposed off-site.

7.2 Placement of Fill

Prior to placement of fill material, the geotechnical consultant should observe and approve the area to receive fill. After observation and approval, the exposed ground surface should be scarified to a depth of 6 to 8 inches. The scarified material should be conditioned (i.e. moisture added or air dried by continued discing) to achieve a moisture content at or slightly above optimum moisture conditions and compacted to a minimum of 90 percent of the maximum density or as otherwise recommended in the soils report or by appropriate government agencies.

Compacted fill should then be placed in thin horizontal lifts not exceeding eight inches in loose thickness prior to compaction. Each lift should be moisture conditioned as needed, thoroughly blended to achieve a consistent moisture content at or slightly above optimum and thoroughly compacted by mechanical methods to a minimum of 90 percent of laboratory maximum dry density. Each lift should be treated in a like manner until the desired finished grades are achieved.

The contractor should have suitable and sufficient mechanical compaction equipment and watering apparatus on the job site to handle the amount of fill being placed in consideration of moisture retention properties of the materials and weather conditions.

When placing fill in horizontal lifts adjacent to areas sloping steeper than 5:1 (horizontal: vertical), horizontal keys and vertical benches should be excavated into the adjacent slope area. Keying and benching should be sufficient to provide at least six-foot wide benches and a minimum of four feet of vertical bench height within the firm natural ground, firm bedrock or engineered compacted fill. No compacted fill should be placed in an area after keying and benching until the geotechnical consultant has reviewed the area. Material generated by the benching operation should be moved sufficiently away from the bench area to allow for the recommended review of the horizontal bench prior to placement of fill.

Within a single fill area where grading procedures dictate two or more separate fills, temporary slopes (false slopes) may be created. When placing fill adjacent to a false slope, benching should be conducted in the same manner as above described. At least a 3-foot vertical bench should be established within the firm core of adjacent approved compacted fill prior to placement of additional fill. Benching should proceed in at least 3-foot vertical increments until the desired finished grades are achieved.

Prior to placement of additional compacted fill following an overnight or other grading delay, the exposed surface or previously compacted fill should be processed by scarification, moisture conditioning as needed to at or slightly above optimum moisture content, thoroughly blended and recompacted to a minimum of 90 percent of laboratory maximum dry density. Where unsuitable materials exist to depths of greater than one foot, the unsuitable materials should be over-excavated.

Following a period of flooding, rainfall or overwatering by other means, no additional fill should be placed until damage assessments have been made and remedial grading performed as described herein.

Rocks 12 inch in maximum dimension and smaller may be utilized in the compacted fill provided the fill is placed and thoroughly compacted over and around all rock. No oversize material should be used within 3 feet of finished pad grade and within 1 foot of other compacted fill areas. Rocks 12 inches up to four feet maximum dimension should be placed below the upper 10 feet of any fill and should not be closer than 15 feet to any slope face. These recommendations could vary as locations of improvements dictate. Where practical, oversized material should not be placed below areas where structures or deep utilities are proposed. Oversized material should be placed in windrows on a clean, overexcavated or unyielding compacted fill or firm natural ground surface. Select native or imported granular soil (S.E. 30 or higher) should be placed and thoroughly flooded over and around all windrowed rock, such that voids are filled. Windrows of oversized material should be staggered so those successive strata of oversized material are not in the same vertical plane.

It may be possible to dispose of individual larger rock as field conditions dictate and as recommended by the geotechnical consultant at the time of placement.

The contractor should assist the geotechnical consultant and/or his representative by digging test pits for removal determinations and/or for testing compacted fill. The contractor should provide this work at no additional cost to the owner or contractor's client.

Fill should be tested by the geotechnical consultant for compliance with the recommended relative compaction and moisture conditions. Field density testing should conform to ASTM Method of Test D 1556-00, D 2922-04. Tests should be conducted at a minimum of approximately two vertical feet or approximately 1,000 to 2,000 cubic yards of fill placed. Actual test intervals may vary as field conditions dictate. Fill found not to be in conformance with the grading recommendations should be removed or otherwise handled as recommended by the geotechnical consultant.

7.3 Fill Slopes

Unless otherwise recommended by the geotechnical consultant and approved by the regulating agencies, permanent fill slopes should not be steeper than 2:1 (horizontal: vertical).

Except as specifically recommended in these grading guidelines compacted fill slopes should be over-built two to five feet and cut back to grade, exposing the firm, compacted fill inner core. The actual amount of overbuilding may vary as field conditions dictate. If the desired results are not achieved, the existing slopes should be overexcavated and reconstructed under the guidelines of the geotechnical consultant. The degree of overbuilding shall be increased until the desired compacted slope surface condition is achieved. Care should be taken by the contractor to provide thorough mechanical compaction to the outer edge of the overbuilt slope surface.

At the discretion of the geotechnical consultant, slope face compaction may be attempted by conventional construction procedures including backrolling. The procedure must create a firmly compacted material throughout the entire depth of the slope face to the surface of the previously compacted firm fill intercore.

During grading operations, care should be taken to extend compactive effort to the outer edge of the slope. Each lift should extend horizontally to the desired finished slope surface or more as needed to ultimately established desired grades. Grade during construction should not be allowed to roll off at the edge of the slope. It may be helpful to elevate slightly the outer edge of the slope. Slough resulting from the placement of individual lifts should not be allowed to drift down over previous lifts. At intervals not exceeding four feet in vertical slope height or the capability of available equipment, whichever is less, fill slopes should be thoroughly dozer trackrolled.

For pad areas above fill slopes, positive drainage should be established away from the top-of-slope. This may be accomplished using a berm and pad gradient of at least two percent.

Section 8 - Trench Backfill

Utility and/or other excavation of trench backfill should, unless otherwise recommended, be compacted by mechanical means. Unless otherwise recommended, the degree of compaction should be a minimum of 90 percent of the laboratory maximum density.

Within slab areas, but outside the influence of foundations, trenches up to one foot wide and two feet deep may be backfilled with sand and consolidated by jetting, flooding or by mechanical means. If on-site materials are utilized, they should be wheel-rolled, tamped or otherwise compacted to a firm condition. For minor interior trenches, density testing may be deleted or spot testing may be elected if deemed necessary, based on review of backfill operations during construction.

If utility contractors indicate that it is undesirable to use compaction equipment in close proximity to a buried conduit, the contractor may elect the utilization of light weight mechanical compaction equipment and/or shading of the conduit with clean, granular material, which should be thoroughly jetted in-place above the conduit, prior to initiating mechanical compaction procedures. Other methods of utility trench compaction may also be appropriate, upon review of the geotechnical consultant at the time of construction.

In cases where clean granular materials are proposed for use in lieu of native materials or where flooding or jetting is proposed, the procedures should be considered subject to review by the geotechnical consultant. Clean granular backfill and/or bedding are not recommended in slope areas.

Section 9 - Drainage

Where deemed appropriate by the geotechnical consultant, canyon subdrain systems should be installed in accordance with CTE's recommendations during grading.

Typical subdrains for compacted fill buttresses, slope stabilization or sidehill masses, should be installed in accordance with the specifications.

Roof, pad and slope drainage should be directed away from slopes and areas of structures to suitable disposal areas via non-erodible devices (i.e., gutters, downspouts, and concrete swales).

For drainage in extensively landscaped areas near structures, (i.e., within four feet) a minimum of 5 percent gradient away from the structure should be maintained. Pad drainage of at least 2 percent should be maintained over the remainder of the site.

Drainage patterns established at the time of fine grading should be maintained throughout the life of the project. Property owners should be made aware that altering drainage patterns could be detrimental to slope stability and foundation performance.

Section 10 - Slope Maintenance

10.1 - Landscape Plants

To enhance surficial slope stability, slope planting should be accomplished at the completion of grading. Slope planting should consist of deep-rooting vegetation requiring little watering. Plants native to the southern California area and plants relative to native plants are generally desirable. Plants native to other semi-arid and arid areas may also be appropriate. A Landscape Architect should be the best party to consult regarding actual types of plants and planting configuration.

10.2 - Irrigation

Irrigation pipes should be anchored to slope faces, not placed in trenches excavated into slope faces.

Slope irrigation should be minimized. If automatic timing devices are utilized on irrigation systems, provisions should be made for interrupting normal irrigation during periods of rainfall.

10.3 - Repair

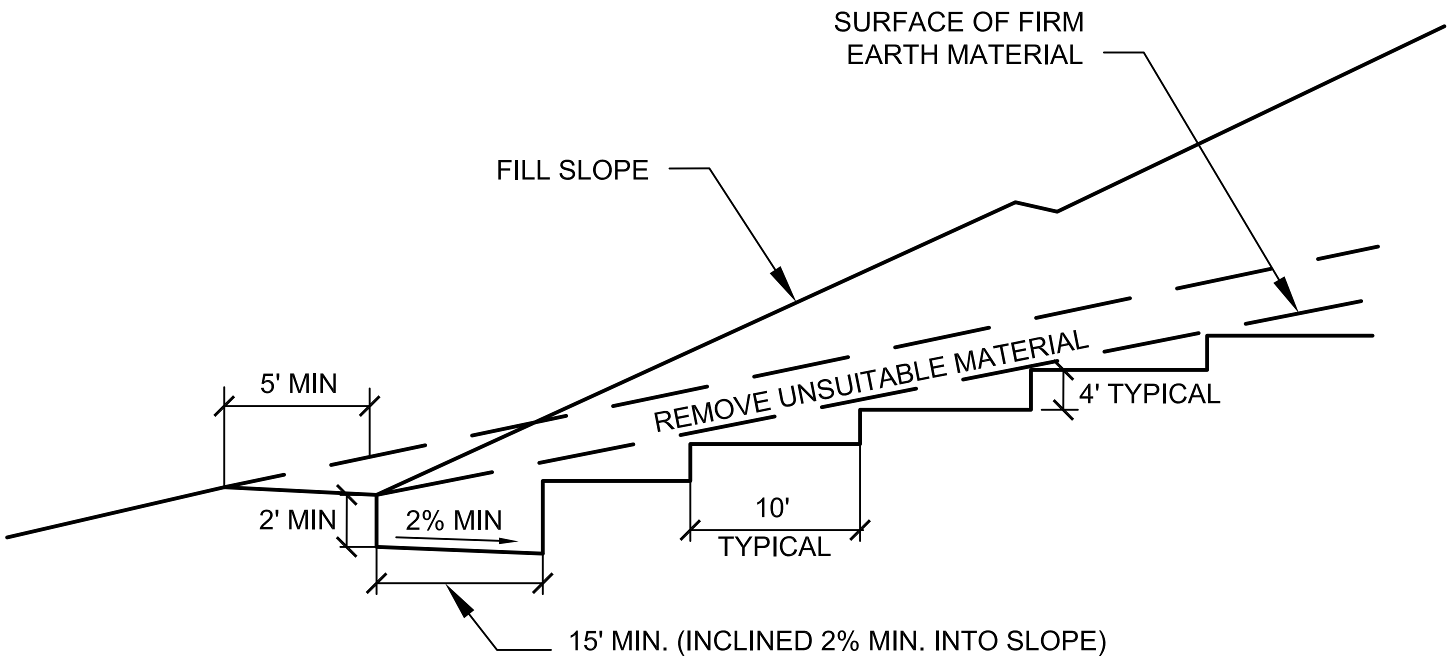
As a precautionary measure, plastic sheeting should be readily available, or kept on hand, to protect all slope areas from saturation by periods of heavy or prolonged rainfall. This measure is strongly recommended, beginning with the period prior to landscape planting.

If slope failures occur, the geotechnical consultant should be contacted for a field review of site conditions and development of recommendations for evaluation and repair.

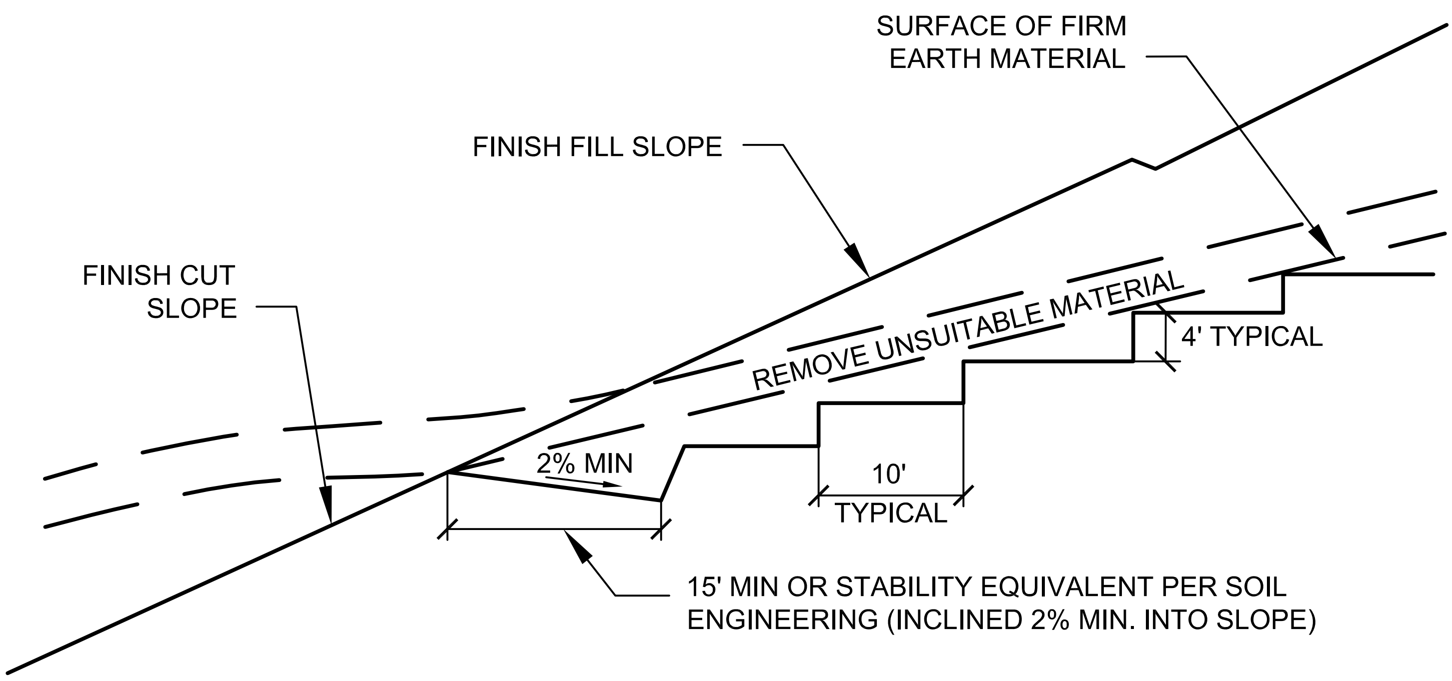
If slope failures occur as a result of exposure to period of heavy rainfall, the failure areas and currently unaffected areas should be covered with plastic sheeting to protect against additional saturation.

In the accompanying Standard Details, appropriate repair procedures are illustrated for superficial slope failures (i.e., occurring typically within the outer one foot to three feet of a slope face).

BENCHING FILL OVER NATURAL

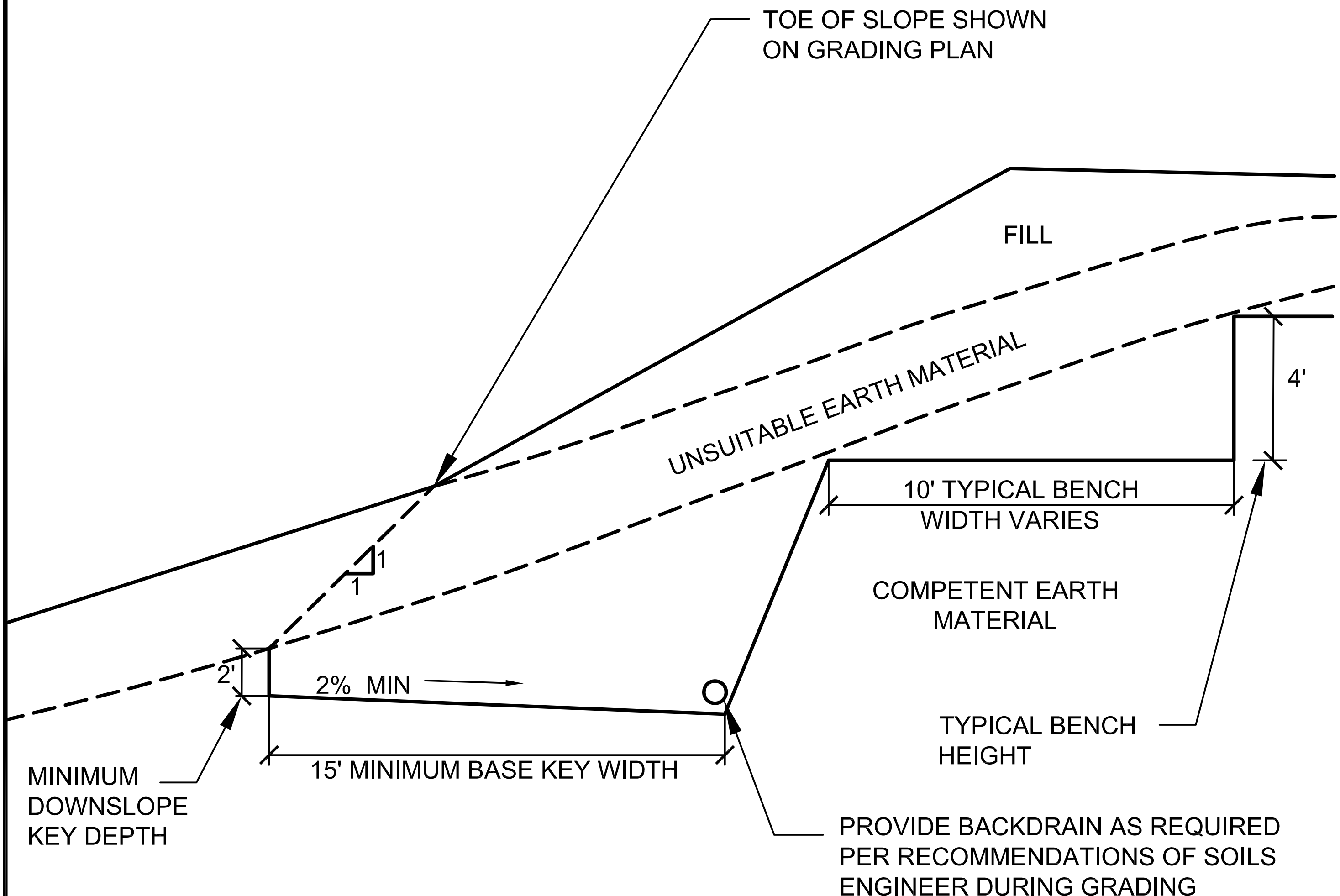


BENCHING FILL OVER CUT



NOT TO SCALE

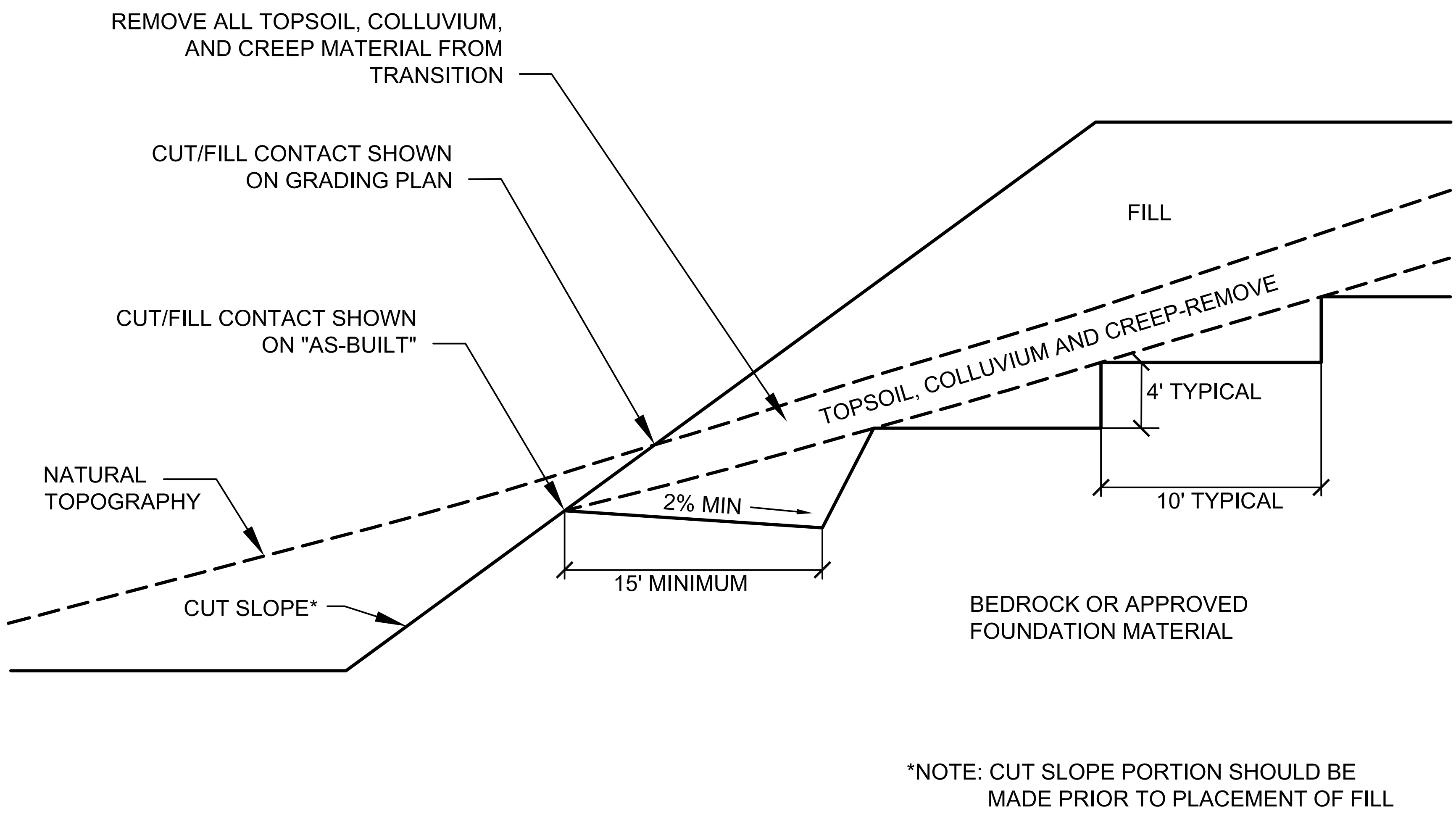
BENCHING FOR COMPACTED FILL DETAIL



WHERE NATURAL SLOPE GRADIENT IS 5:1 OR LESS, BENCHING IS NOT NECESSARY. FILL IS NOT TO BE PLACED ON COMPRESSIBLE OR UNSUITABLE MATERIAL.

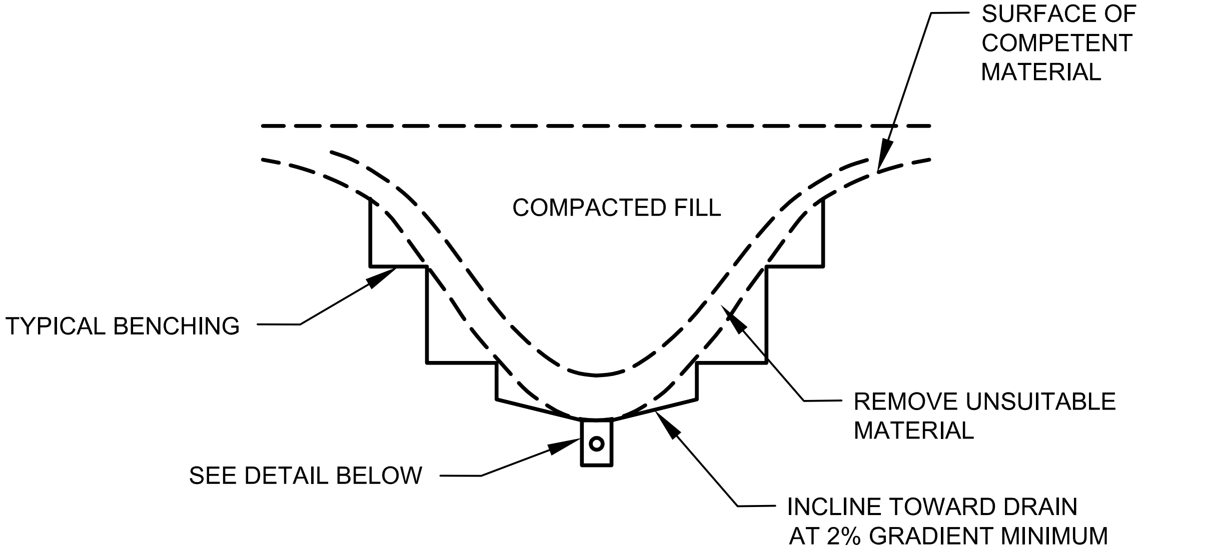
NOT TO SCALE

FILL SLOPE ABOVE NATURAL GROUND DETAIL

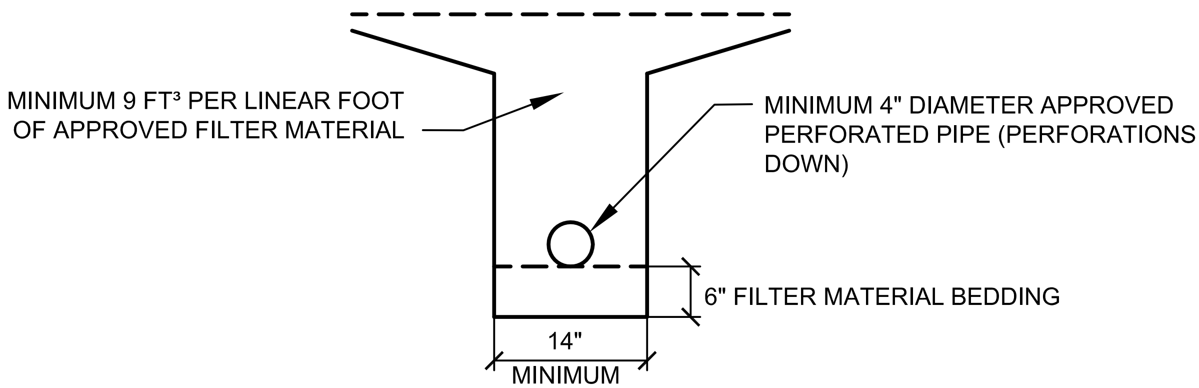


NOT TO SCALE

FILL SLOPE ABOVE CUT SLOPE DETAIL



DETAIL



CALTRANS CLASS 2 PERMEABLE MATERIAL FILTER MATERIAL TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUAL:

SIEVE SIZE	PERCENTAGE PASSING
1"	100
¾"	90-100
⅜"	40-100
NO. 4	25-40
NO. 8	18-33
NO. 30	5-15
NO. 50	0-7
NO. 200	0-3

NOT TO SCALE

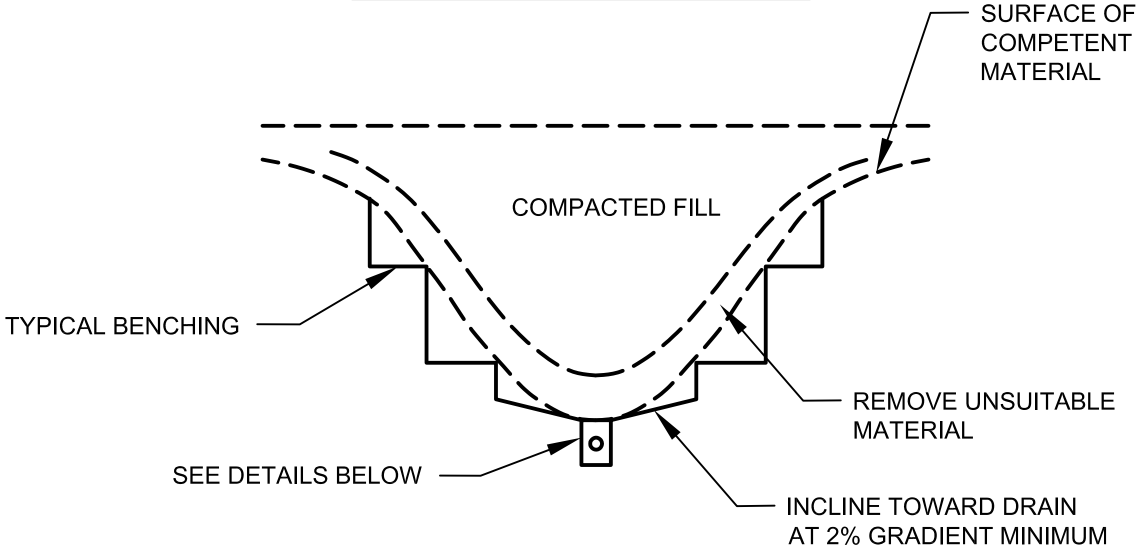
APPROVED PIPE TO BE SCHEDULE 40 POLY-VINYL-CHLORIDE (P.V.C.) OR APPROVED EQUAL. MINIMUM CRUSH STRENGTH 1000 psi

PIPE DIAMETER TO MEET THE FOLLOWING CRITERIA, SUBJECT TO FIELD REVIEW BASED ON ACTUAL GEOTECHNICAL CONDITIONS ENCOUNTERED DURING GRADING

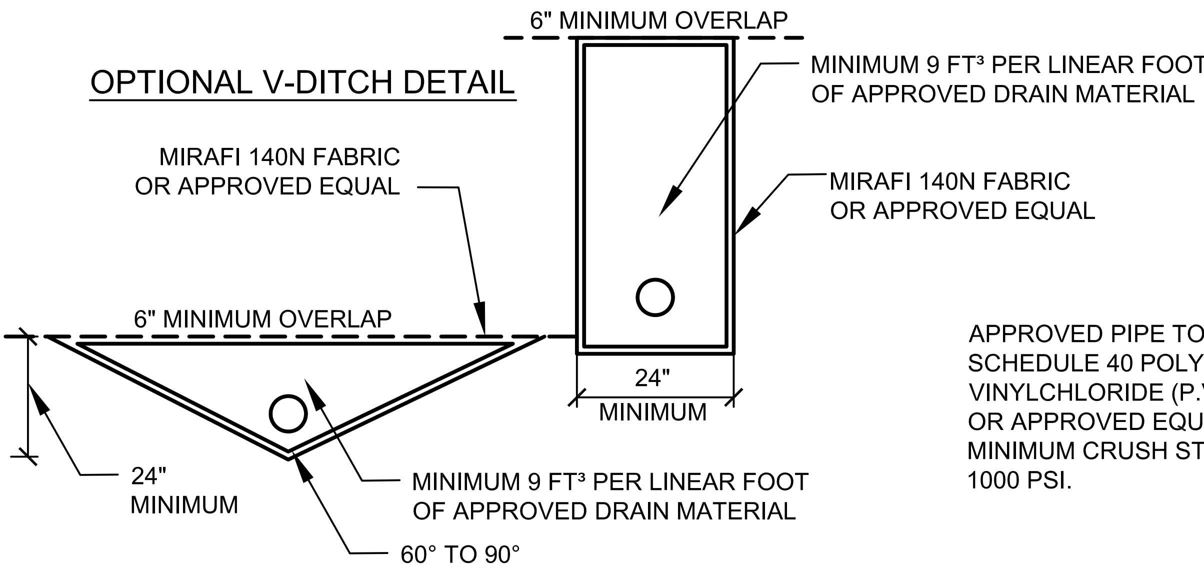
LENGTH OF RUN	PIPE DIAMETER
INITIAL 500'	4"
500' TO 1500'	6"
> 1500'	8"

TYPICAL CANYON SUBDRAIN DETAIL

CANYON SUBDRAIN DETAILS



TRENCH DETAILS



APPROVED PIPE TO BE SCHEDULE 40 POLY-VINYLCHLORIDE (P.V.C.) OR APPROVED EQUAL. MINIMUM CRUSH STRENGTH 1000 PSI.

DRAIN MATERIAL TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUAL:

SIEVE SIZE	PERCENTAGE PASSING
1 ½"	88-100
1"	5-40
¾"	0-17
⅜"	0-7
NO. 200	0-3

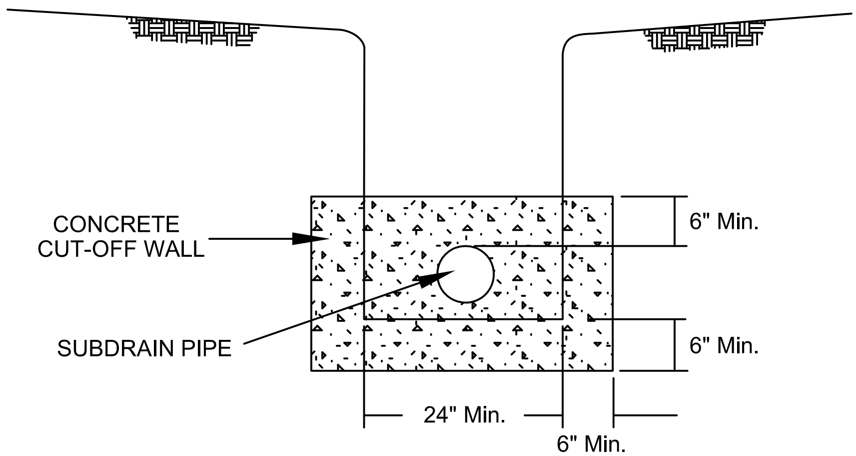
PIPE DIAMETER TO MEET THE FOLLOWING CRITERIA, SUBJECT TO FIELD REVIEW BASED ON ACTUAL GEOTECHNICAL CONDITIONS ENCOUNTERED DURING GRADING

LENGTH OF RUN	PIPE DIAMETER
INITIAL 500'	4"
500' TO 1500'	6"
> 1500'	8"

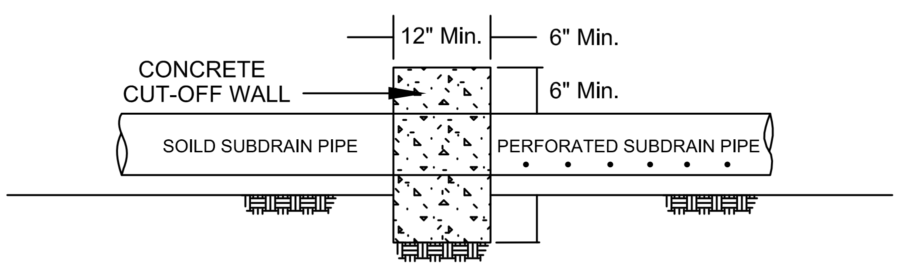
NOT TO SCALE

GEOFABRIC SUBDRAIN

FRONT VIEW



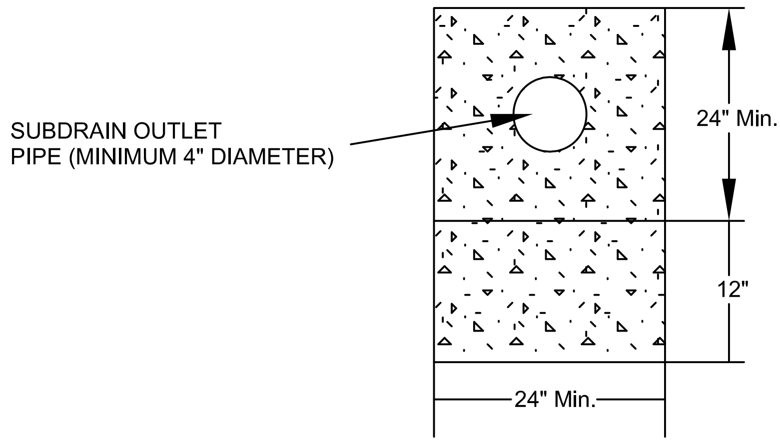
SIDE VIEW



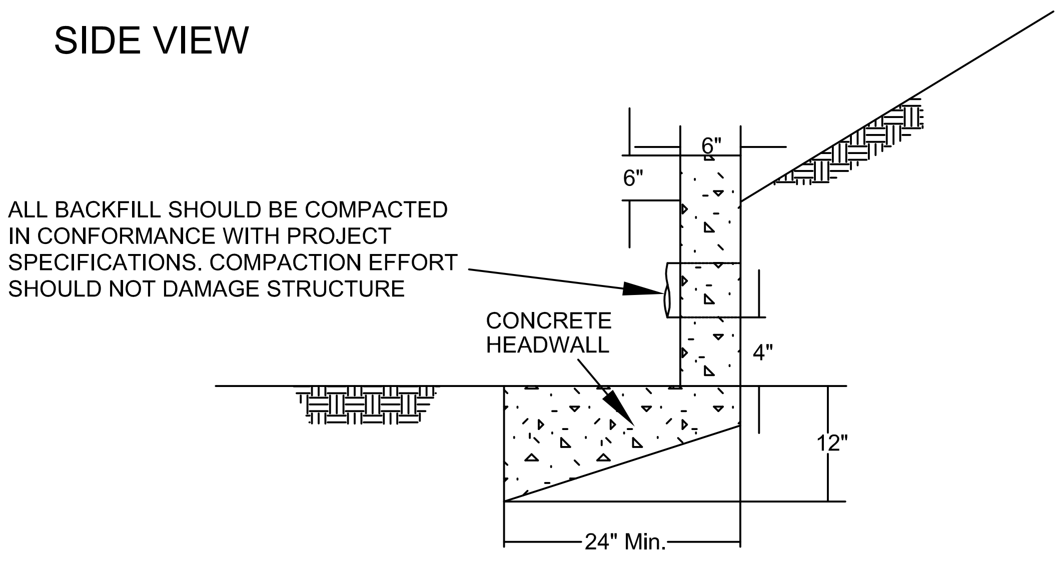
NOT TO SCALE

RECOMMENDED SUBDRAIN CUT-OFF WALL

FRONT VIEW



SIDE VIEW



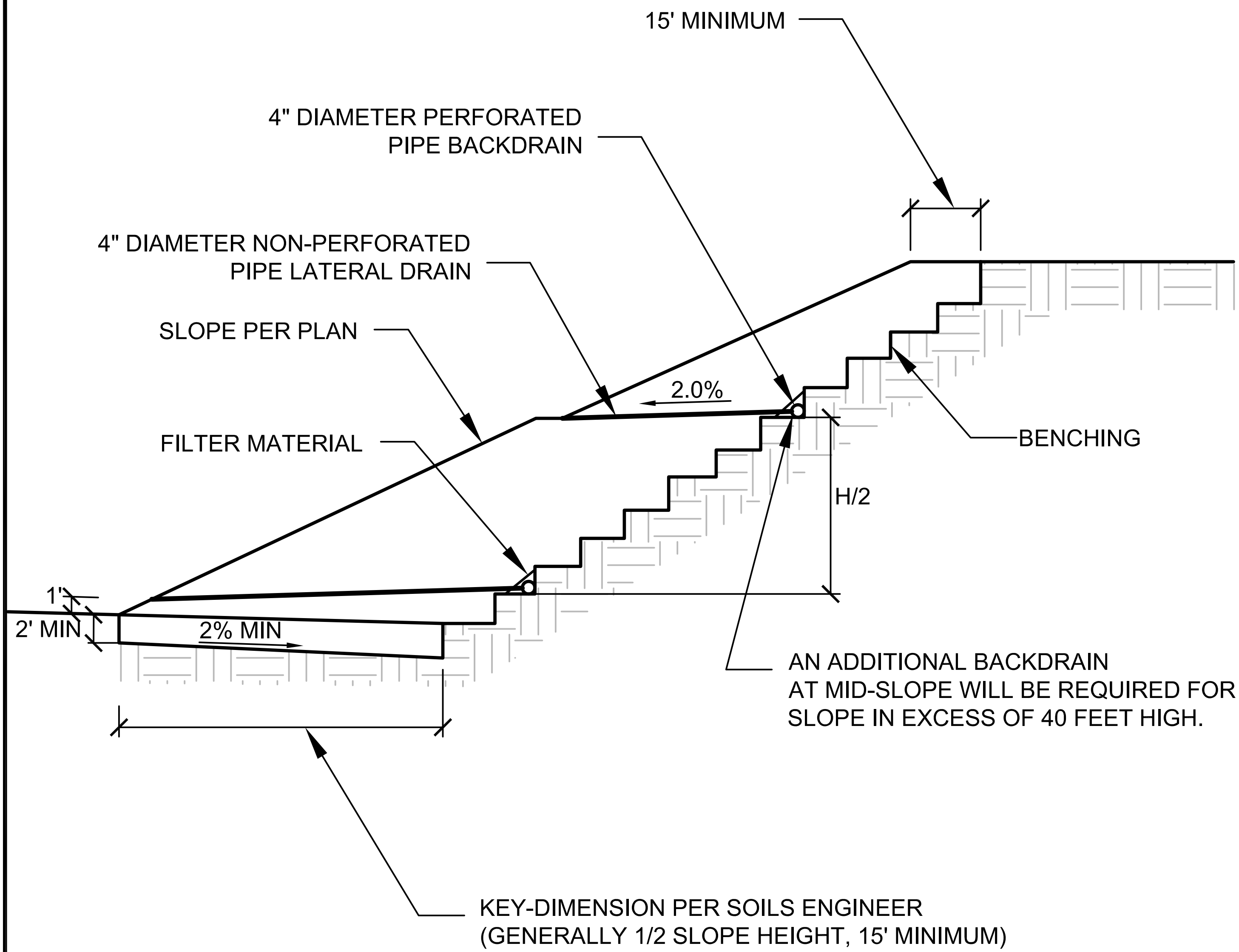
NOTE: HEADWALL SHOULD OUTLET AT TOE OF SLOPE OR INTO CONTROLLED SURFACE DRAINAGE DEVICE

ALL DISCHARGE SHOULD BE CONTROLLED

THIS DETAIL IS A MINIMUM DESIGN AND MAY BE MODIFIED DEPENDING UPON ENCOUNTERED CONDITIONS AND LOCAL REQUIREMENTS

NOT TO SCALE

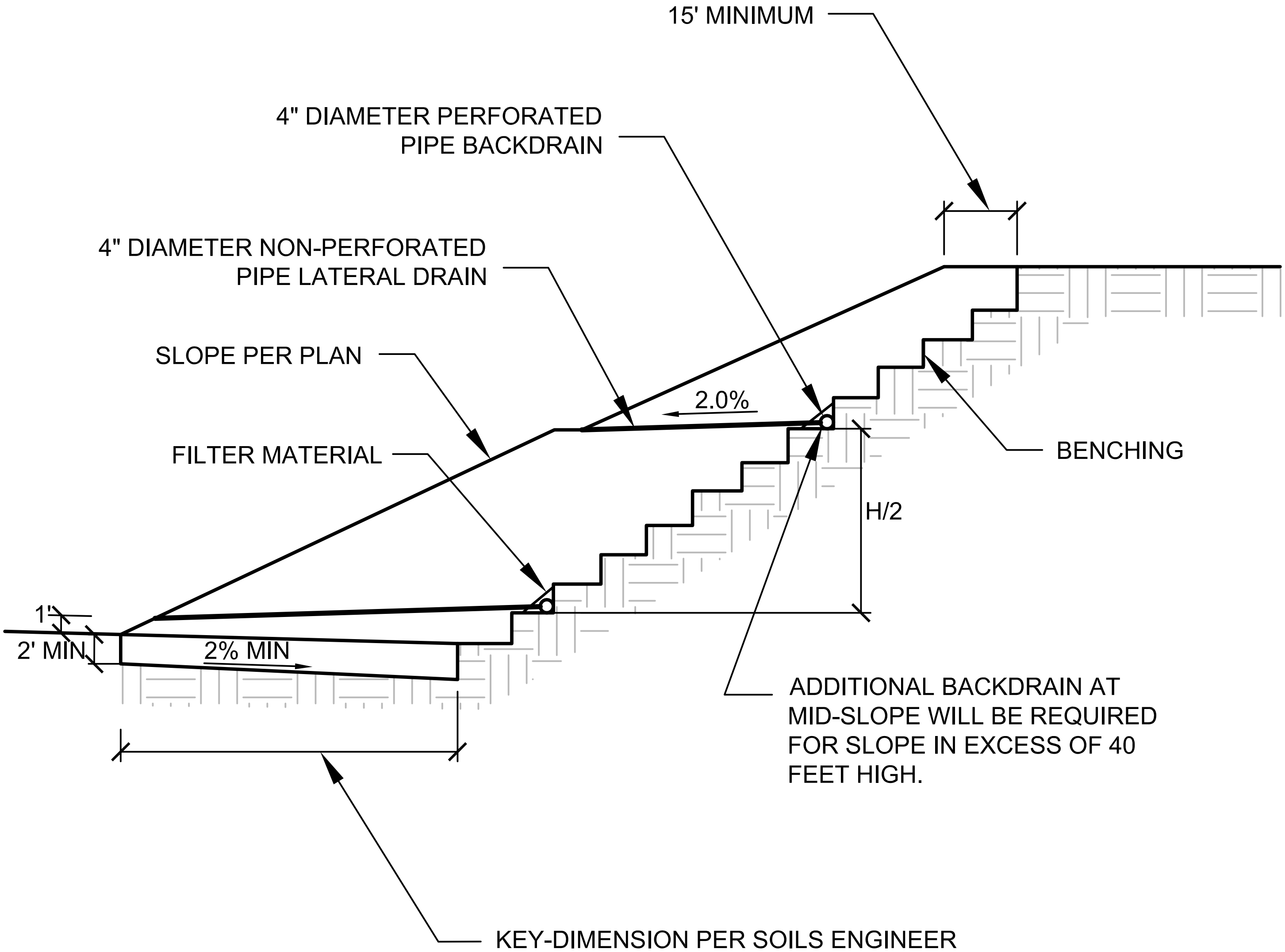
TYPICAL SUBDRAIN OUTLET HEADWALL DETAIL



DIMENSIONS ARE MINIMUM RECOMMENDED

NOT TO SCALE

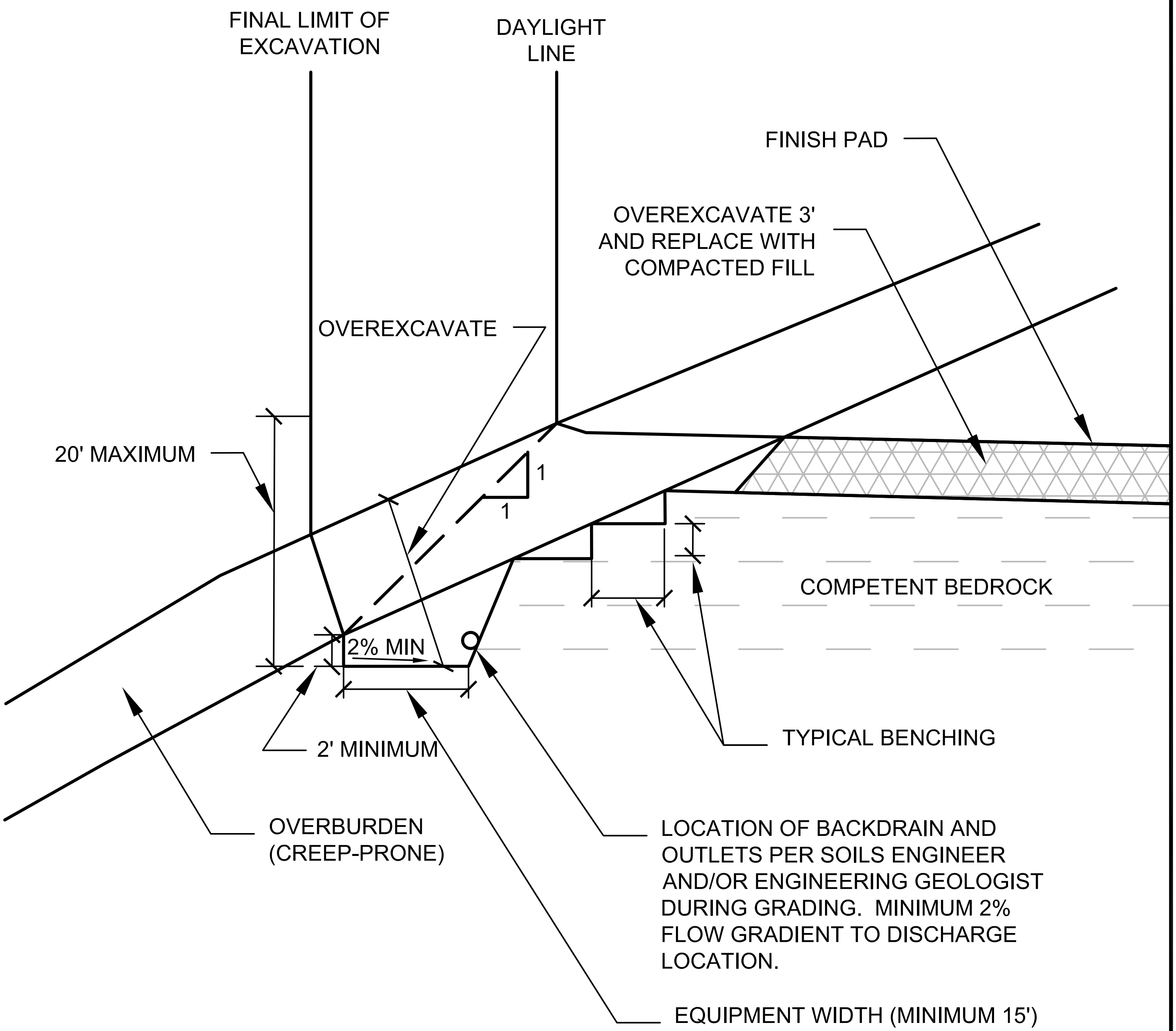
TYPICAL SLOPE STABILIZATION FILL DETAIL



DIMENSIONS ARE MINIMUM RECOMMENDED

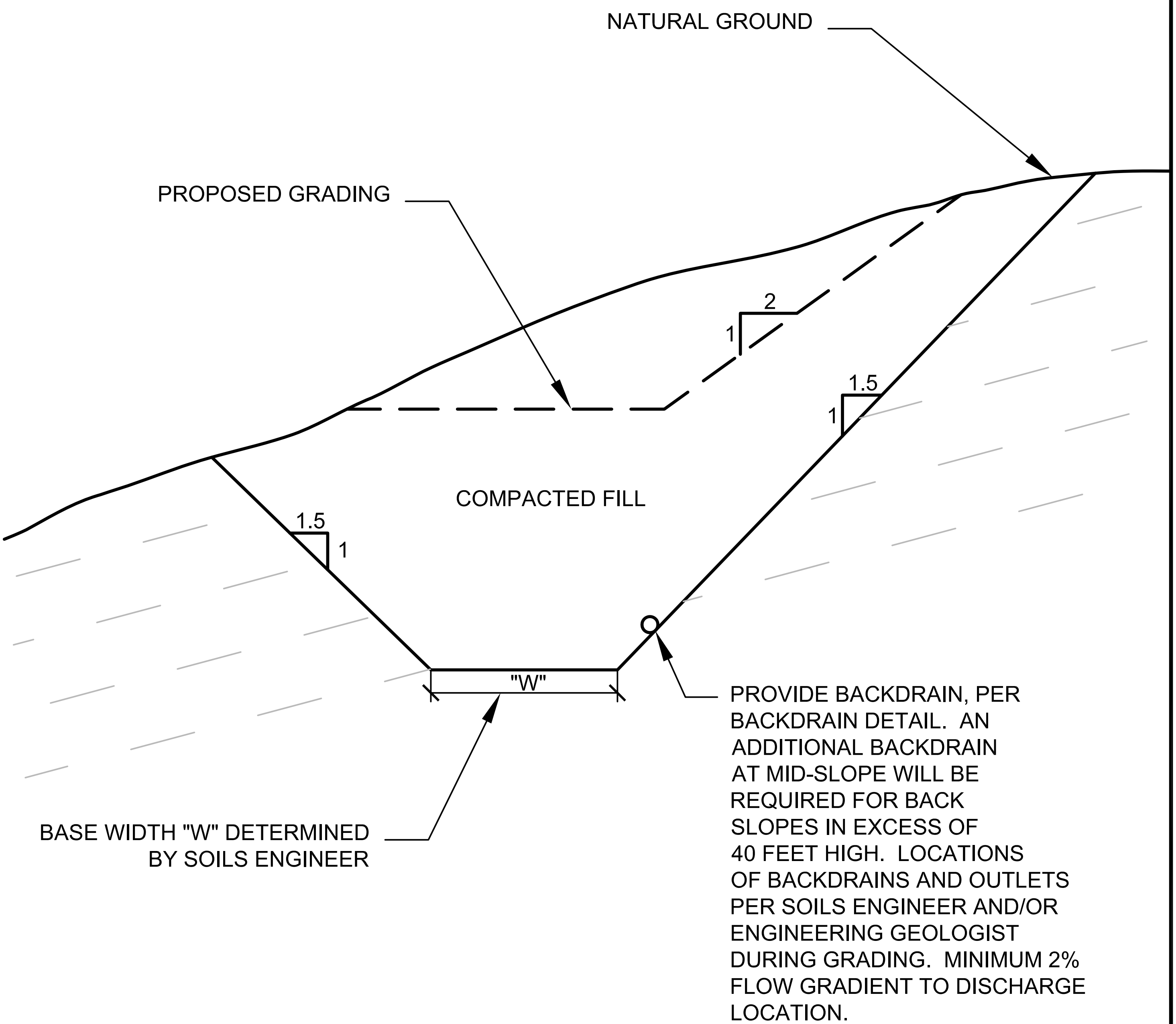
NOT TO SCALE

TYPICAL BUTTRESS FILL DETAIL



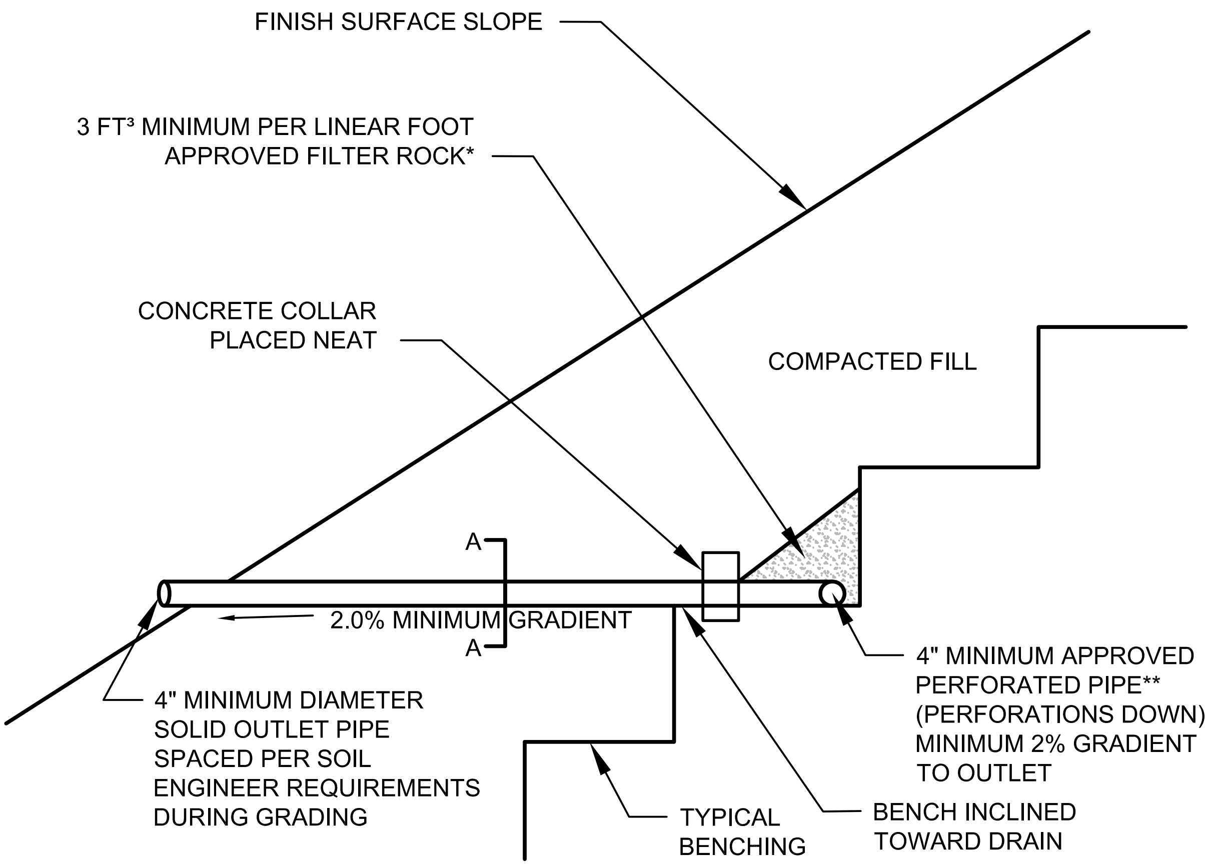
NOT TO SCALE

DAYLIGHT SHEAR KEY DETAIL

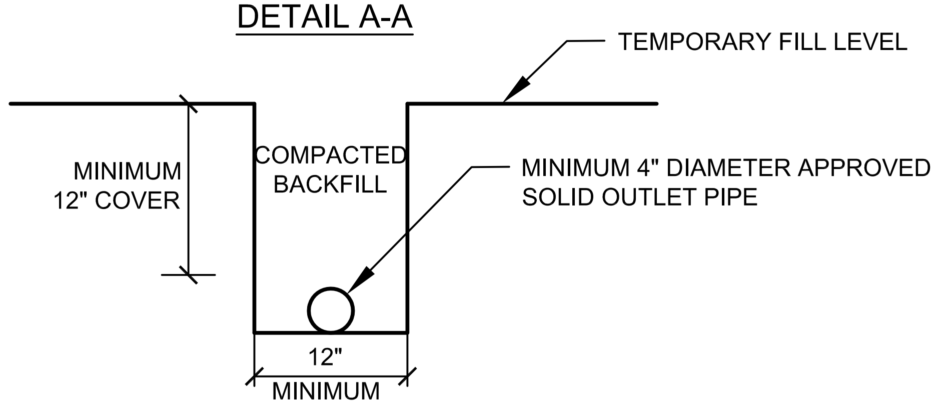


NOT TO SCALE

TYPICAL SHEAR KEY DETAIL



DETAIL A-A



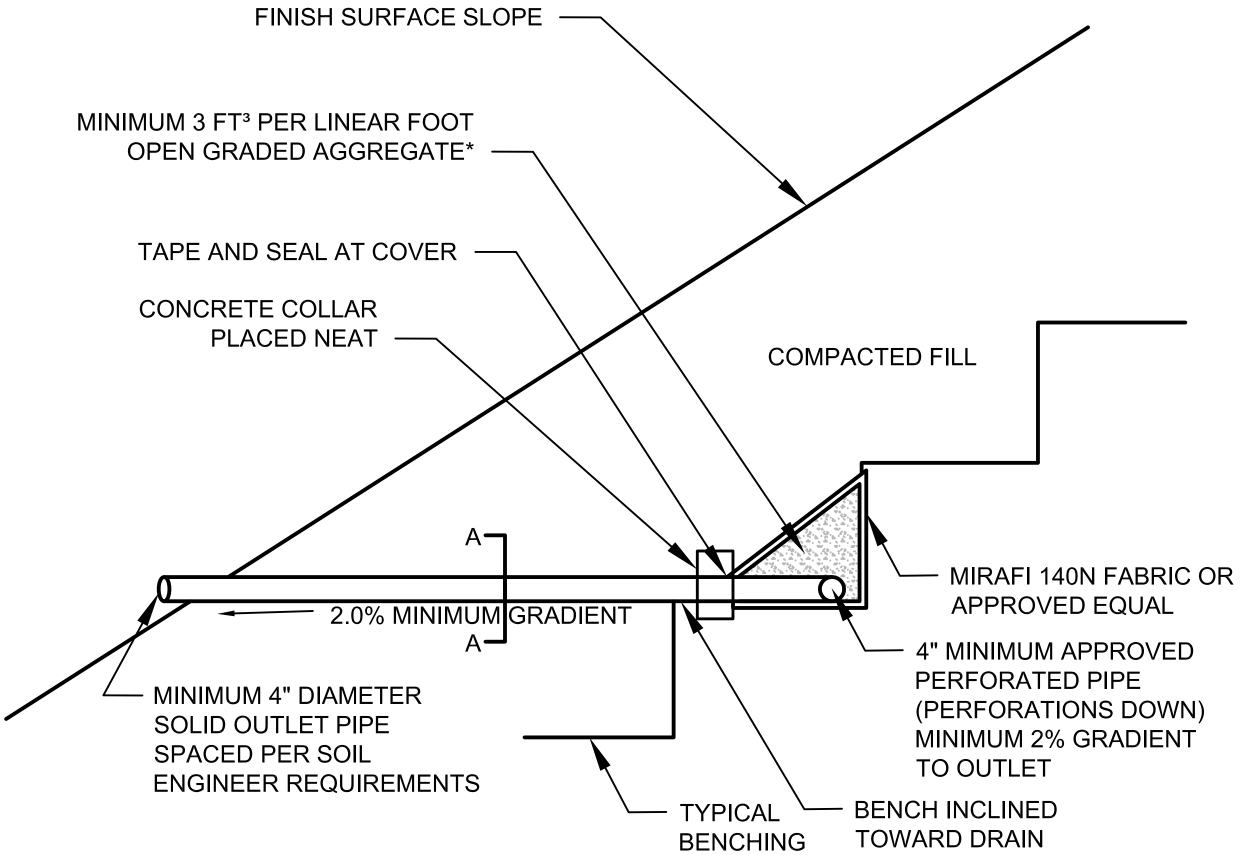
**APPROVED PIPE TYPE:
SCHEDULE 40 POLYVINYL CHLORIDE (P.V.C.) OR APPROVED EQUAL. MINIMUM CRUSH STRENGTH 1000 PSI

*FILTER ROCK TO MEET FOLLOWING SPECIFICATIONS OR APPROVED EQUAL:

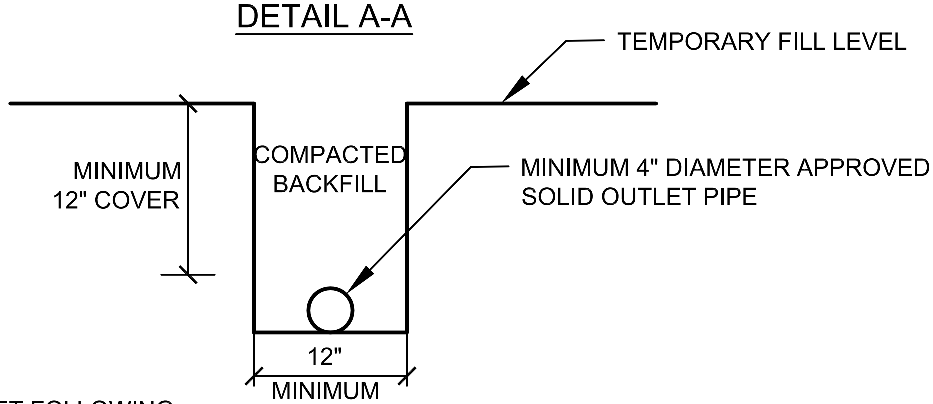
SIEVE SIZE	PERCENTAGE PASSING
1"	100
¾"	90-100
⅜"	40-100
NO. 4	25-40
NO. 30	5-15
NO. 50	0-7
NO. 200	0-3

NOT TO SCALE

TYPICAL BACKDRAIN DETAIL



DETAIL A-A

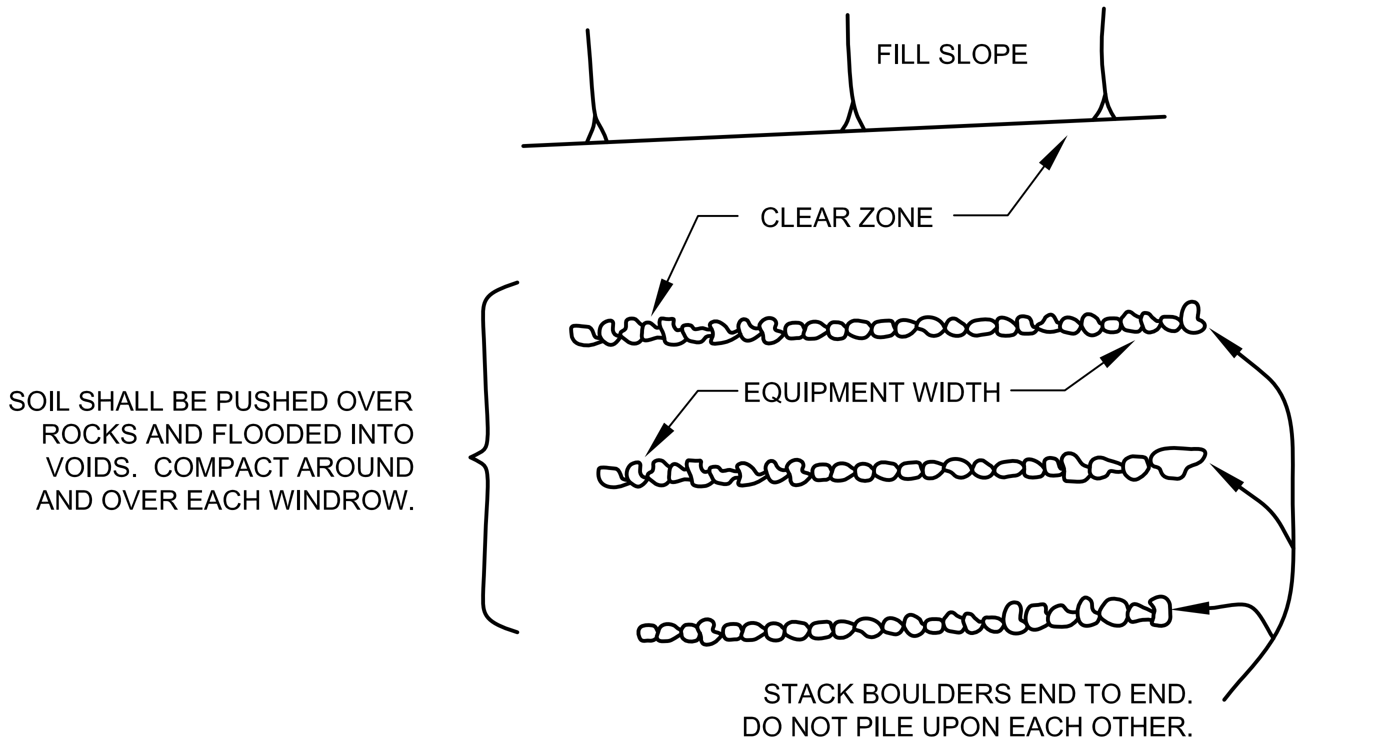


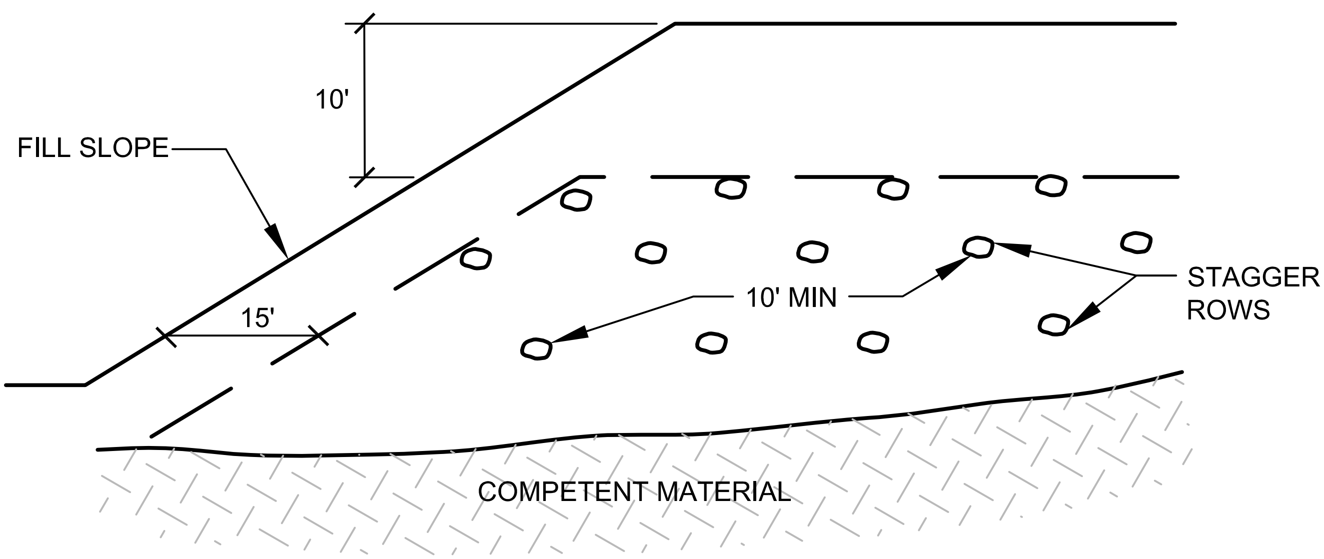
*NOTE: AGGREGATE TO MEET FOLLOWING SPECIFICATIONS OR APPROVED EQUAL:

SIEVE SIZE	PERCENTAGE PASSING
1 ½"	100
1"	5-40
¾"	0-17
⅜"	0-7
NO. 200	0-3

NOT TO SCALE

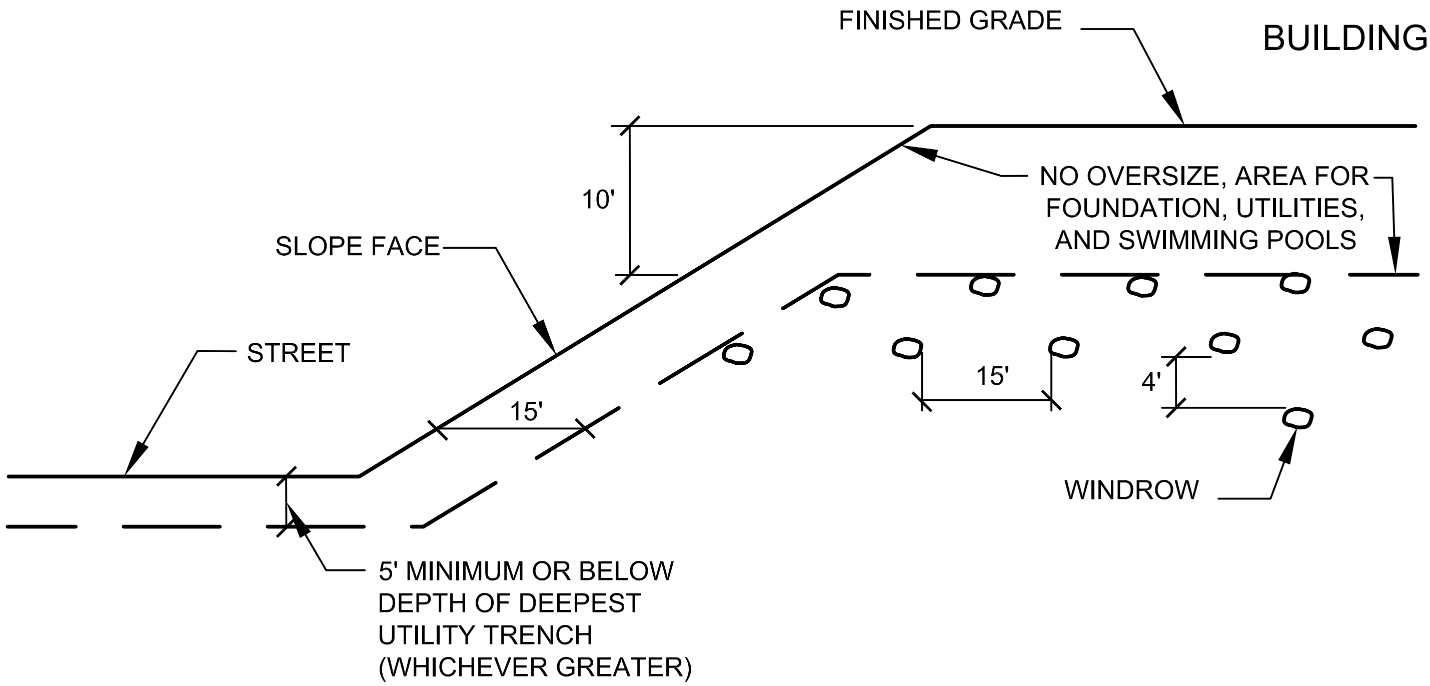
BACKDRAIN DETAIL (GEOFRABIC)



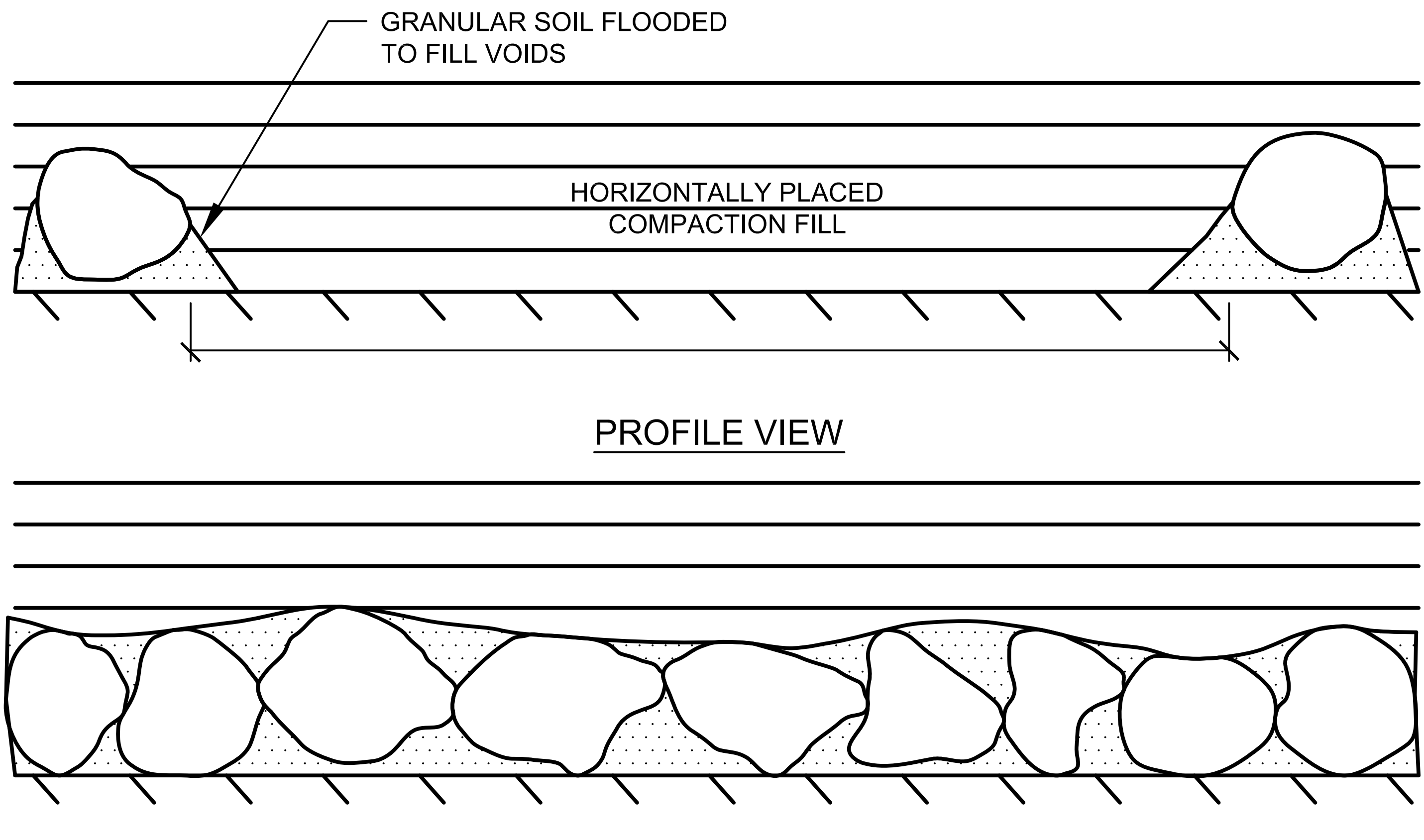


NOT TO SCALE

ROCK DISPOSAL DETAIL



TYPICAL WINDROW DETAIL (EDGE VIEW)



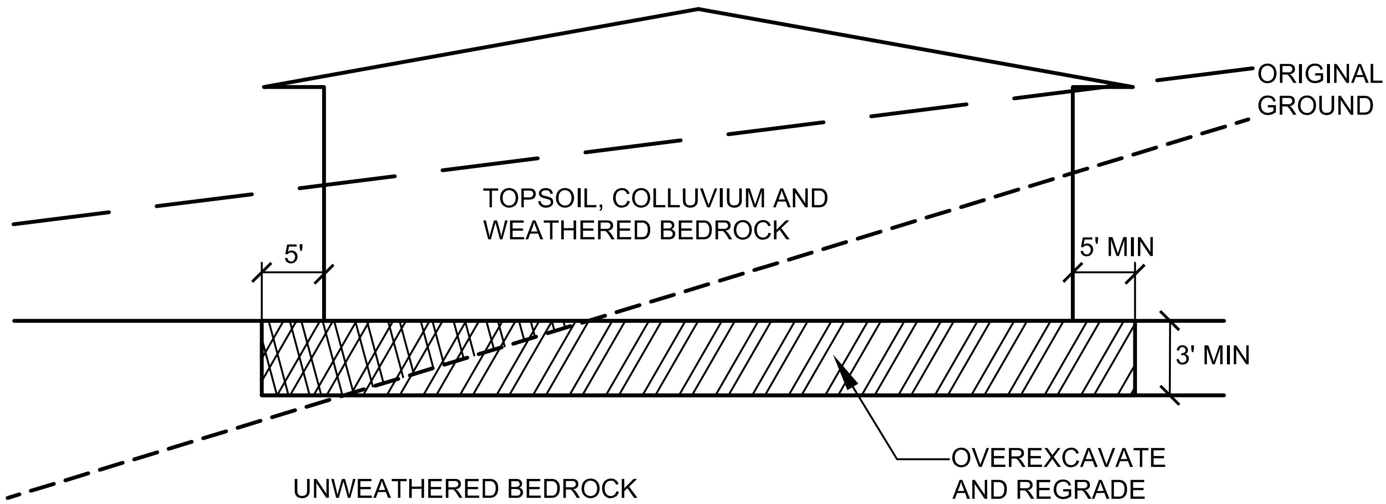
PROFILE VIEW

NOT TO SCALE

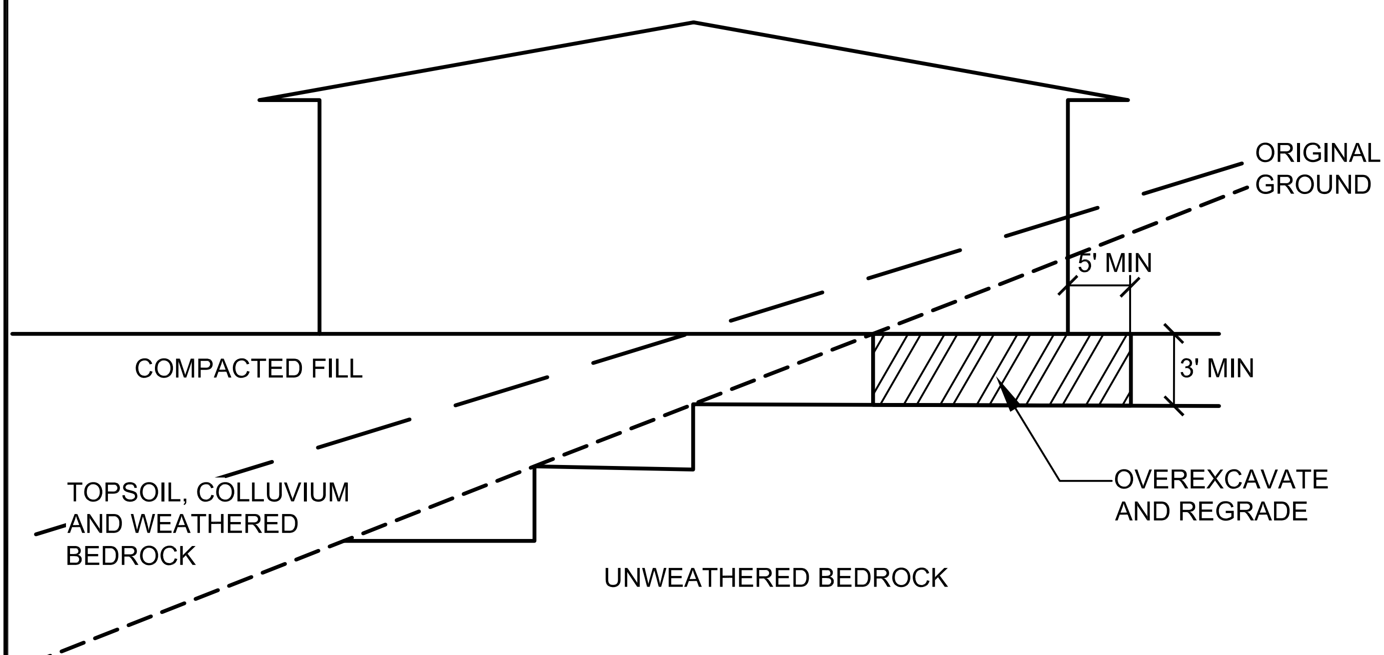
ROCK DISPOSAL DETAIL

GENERAL GRADING RECOMMENDATIONS

CUT LOT



CUT/FILL LOT (TRANSITION)



NOT TO SCALE

TRANSITION LOT DETAIL

APPENDIX E

PERCOLATION TO INFILTRATION CALCULATIONS AND FIELD DATA

Project:	**Vortex Farms**						
Project No.:	**10-15741G**						Tables P-1 - P-3

Percolation Field Data and Calculated Rates

P-1 — **Total Depth: 35 inches**

Time	Test Interval Time (minutes)	Test Refill Depth /Inches	Water Level Initial/Start Depth /Inches	Water Level End/Final Depth /Inches	Incremental Water Level Change (inches)	Percolation Rate inches/minute	Percolation Rate inches/hour
7:45:00	Initial	None	14.00	initial	-		
8:15:00	30	15.5	14.00	18.00	4.00	0.133	8.000
8:45:00	30	15.5	15.50	19.00	3.50	0.117	7.000
9:15:00	30	15.5	15.50	18.50	3.00	0.100	6.000
9:45:00	30	15.5	15.50	18.50	3.00	0.100	6.000
10:15:00	30	15.5	15.50	18.50	3.00	0.100	6.000
10:45:00	30	15.5	15.50	18.50	3.00	0.100	6.000
11:15:00	30	15.5	15.50	18.50	3.00	0.100	6.000
11:45:00	30	15.5	15.50	18.50	3.00	0.100	6.000
12:15:00	30	15.5	15.50	18.50	3.00	0.100	6.000
12:45:00	30	15.5	15.50	18.50	3.00	0.100	6.000
13:15:00	30	15.5	15.50	18.50	3.00	0.100	6.000
13:45:00	30	NO	15.50	18.50	3.00	0.100	6.000

P-2 — **Total Depth: 58 inches**

Time	Test Interval Time (minutes)	Test Refill Depth /Inches	Water Level Initial/Start Depth /Inches	Water Level End/Final Depth /Inches	Incremental Water Level Change (inches)	Percolation Rate inches/minute	Percolation Rate inches/hour
7:50:00	Initial	None	38.00	initial	-		
8:20:00	30	35.5	38.00	47.75	9.750	0.325	19.500
8:50:00	30	36.75	35.50	39.25	3.750	0.125	7.500
9:20:00	30	36.5	36.75	40.50	3.750	0.125	7.500
9:50:00	30	36.25	36.50	40.00	3.500	0.117	7.000
10:20:00	30	36.5	36.25	38.75	2.500	0.083	5.000
10:50:00	30	36.25	36.50	39.75	3.250	0.108	6.500
11:20:00	30	36.5	36.25	39.75	3.500	0.117	7.000
11:50:00	30	36.25	36.50	39.75	3.250	0.108	6.500
12:20:00	30	36.5	36.25	40.25	4.000	0.133	8.000
12:50:00	30	36.5	36.50	40.00	3.500	0.117	7.000
13:20:00	30	36.5	36.50	39.75	3.250	0.108	6.500
13:50:00	30	NO	36.50	40.00	3.500	0.117	7.000

P-3 — **Total Depth: 59.25 inches**

Time	Test Interval Time (minutes)	Test Refill Depth /Inches	Water Level Initial/Start Depth /Inches	Water Level End/Final Depth /Inches	Incremental Water Level Change (inches)	Percolation Rate inches/minute	Percolation Rate inches/hour
7:55:00	Initial	None	7.50	initial	-		
8:20:00	25	6.875	7.50	52.88	45.38	1.815	108.900
8:45:00	25	18.625	6.88	51.00	44.13	1.765	105.900
8:55:00	10	17.25	18.63	26.88	8.25	0.825	49.500
9:05:00	10	17.375	17.25	25.25	8.00	0.800	48.000
9:15:00	10	19.125	17.38	24.88	7.50	0.750	45.000
9:25:00	10	18.625	19.13	25.88	6.75	0.675	40.500
9:35:00	10	18.5	18.63	25.25	6.63	0.663	39.750
9:45:00	10	NO	18.50	24.88	6.38	0.638	38.250

Percolation Rate Conversion P-1

		Inches
Time Interval,	Δt =	30
Final Depth of Water,	D_f =	18.50
Test Hole Radius,	r =	4
Initial Depth to Water,	D_0 =	15.50
Total Depth of Test Hole,	D_T =	35

H_o =	19.5	in
H_f =	16.5	in
$\Delta H = \Delta D$ =	3	in
H_{avg} =	18	in
I_t =	0.600	in/hr

Percolation Rate Conversion P-2

		Inches
Time Interval,	Δt =	30
Final Depth of Water,	D_f =	40.00
Test Hole Radius,	r =	4
Initial Depth to Water,	D_0 =	36.50
Total Depth of Test Hole,	D_T =	58

H_o =	21.5	in
H_f =	18	in
$\Delta H = \Delta D$ =	3.5	in
H_{avg} =	19.75	in
I_t =	0.644	in/hr

Percolation Rate Conversion P-3

		Inches
Time Interval,	Δt =	10
Final Depth of Water,	D_f =	24.88
Test Hole Radius,	r =	4
Initial Depth to Water,	D_0 =	18.50
Total Depth of Test Hole,	D_T =	59.25

H_o =	40.75	in
H_f =	34.375	in
$\Delta H = \Delta D$ =	6.375	in
H_{avg} =	37.5625	in
I_t =	1.934	in/hr

TABLE 2.0 RESULTS OF PERCOLATION TESTING WITH MINIMUM FACTOR OF SAFETY APPLIED					
Test Location	Test Depth (inches)	Soil Type* (USCS Classification)	Percolation Rate (inches per hour)	Infiltration Rate (inches per hour)	Infiltration Rate with FOS of 3 Applied (inches per hour)
P-1	35	Qya	6.00	0.600	0.300
P-2	58	Qya	7.00	0.644	0.322
P-3	59.25	Qya	38.25	1.934	0.967