GEOTECHNICAL INVESTIGATION
PROPOSED SKYLINE RETIREMENT CENTER
NORTHWEST OF 11330 CAMPO ROAD
LA MESA, CALIFORNIA 91941

PREPARED FOR:

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CTE JOB NO. 10-13295G

OCTOBER 3, 2016
REVISED OCTOBER 17, 2016

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1.0 INTRODUCTION AND SCOPE OF SERVICES

1.1 Introduction

This report presents the results of CTE's preliminary geotechnical investigation providing preliminary conclusions and recommendations for design and construction of the proposed Skyline Retirement Center. The report is revised from the October 3, 2016 report to provide corrected calculated infiltration rates as attached. This work is authorized on August 24, 2016 by Mr. Daniel Grant through CTE proposal number G-3866 dated August 18, 2106. .

Preliminary geotechnical recommendations for excavations, fill placement, and foundation design for the proposed structures and improvements are presented in this report. The investigation included at review of selected documents, field exploration, laboratory testing, geologic hazard evaluation, geotechnical engineering analysis, and preparation of this report. Reviewed selected references are presented in Appendix A. Exploration logs are provided in Appendix B. Laboratory test methods and results are presented in Appendix C. Standard Specifications for Grading are presented in Appendix D. Percolation testing and calculated infiltration rates are presented in Appendix E. Figure 1 is an index map showing the approximate site location. Figure 2 shows the approximate locations of subsurface explorations and the preliminary site layout. Figures 2A and 2B provide geologic cross sections through selected portions of the project. Figure 3 provides a regional fault and seismicity map. Figure 4 illustrates conceptual retaining wall drainage provisions.

1.2 Scope of Services

The implemented scope of services includes:

- Review of readily available geologic information.
- Field exploration with a truck mounted hollow stem auger drill rig to evaluate site subsurface conditions.
- Laboratory testing of selected soil samples to provide data for evaluation of geotechnical characteristics of site soils.
- Assessment of geologic conditions pertinent to the site.
- Percolation testing, calculation of infiltration rates, and an evaluation of potential impacts associated with stormwater infiltration into the subsurface.
- Preparation of this report providing a summary of the geotechnical investigation performed, and conclusions and preliminary recommendations for the site development.

2.0 SITE BACKGROUND

2.1 Site Location and Description

The site is located in the County of San Diego, east of the City of La Mesa, California (Figure 1). The site is a triangular shaped lot bounded by Campo Road to the south and southwest, a residential community to the north and a private road leading to Skyline Wesleyan Church to the east and southeast. Adjacent to the northern approximate one-half of the site, is a graded 2:1 (horizontal to vertical) surface ratio slope that ascends approximately 30 vertical feet to a residential development. The subject lot is undeveloped land supporting native vegetation. It descends at up to a 3:1 (H:V) surface ratio toward Campo Road on the south to southeast. Surface topography near Campo Road flattens to approximately a one percent surface gradient. Surface topography generally slopes in a regular undulating fashion. However, a steeply incised, narrow, south descending drainage is at the east margin of the site near the private road leading to the Westleyan Church. Site elevations were obtained from the rough grading plan, which is used as the base map for the attached Figure 2, and found to range from approximately 550 feet on the north margin of the site to 480 feet above mean sea level (msl) on the south margin of the site near Campo Road.

2.2 Proposed Development

Based on the Proposed Site and Preliminary Grading Plan (REC Consultants, Inc.), the proposed development is to include construction of a large foot print, three story, multiple wing assisted and independent living residence building with an at or partially below grade basement parking garage to underlie the independent living wing. Two duplex buildings are planned within the northeastern portion of the site, with there additional duplex buildings near the northwest portion of the site. Paved parking and drives are planned on all sides of the residence building to form a drive loop around the structure. Entrance to the site is from a private road servicing the Wesleyan Church to the east. This private road provides transportation access to Campo Road on the south margin of the site.

3.0 FIELD AND LABORATORY INVESTIGATION

3.1 Field Investigation

Field exploration, performed on August 31, 2016, included site reconnaissance and advancement of nine exploratory borings for geotechnical purposes and six percolation test holes. Soils were logged in the field by a CTE geologist and visually classified according to the Unified Soil Classification System. Bulk and relatively undisturbed soil samples were transported to the CTE geotechnical laboratory in Escondido, California for testing.

Exploration logs including descriptions of the soils encountered are provided in Appendix B. The field descriptions shown on the exploration logs have been modified, where appropriate, to reflect laboratory test results. Approximate exploration locations are shown on Figure 2.

3.2 Laboratory Investigation

Laboratory tests were conducted on representative soil samples for classification purposes and to evaluate physical properties and engineering characteristics of site soils. Laboratory tests conducted for this geotechnical investigation include: in place moisture and density, remolded shear strength, grain-size determination, maximum dry density and optimum moisture content (Modified Proctor), chemical analyses, Expansion Index, and R-Value. Test method descriptions and laboratory results are presented in Appendix C.

4.0 GEOLOGY

4.1 General Physiographic Setting

San Diego 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 San Diego Region can be further subdivided into the coastal plain area, central mountain–valley area, and eastern mountain-valley area. The project site lies within the boundary of the coastal plain area and the central mountain-valley area. The coastal plain ranges in elevation from approximately sea level to 1,200 feet above mean sea level and is characterized by Cretaceous and Tertiary sedimentary deposits that overlap an eroded basement surface consisting of Jurassic and Cretaceous geologic age crystalline rocks. The coastal plain has been eroded by ocean processes over geologic time to form terraces at various elevations. The terraces are typically underlain by marine and non-marine sedimentary rocks. The central mountain area ranges in elevation from approximately 500 to 5,000 feet above mean sea level and is characterized by Cretaceous and Jurassic crystalline ridges and mountains with intermountain basins that are generally underlain with moderate thickness of alluvium and residual soils.

4.2 Geologic Conditions

Based on regional geologic mapping by Kennedy and Tan (2005 and 2008) and subsurface explorations, the site is underlain by Quaternary Recent Alluvium, Undivided Alluvium and Colluvium, and Cretaceous Santiago Peak Volcanics. Based on explorations by CTE, the geologic conditions at the site are consistent with the regional mapping with the addition of local shallow Quaternary Undocumented Fill.

4.2.1 Quaternary Undocumented Fill (Qudf)

Local, shallow deposits of Quaternary Undocumented Fill were encountered adjacent to the single-family residences near the northeastern margin of the site, and as end dump mounds within an enclosure on the east margin of the site. The undocumented fill consisted of medium dense to dense, slightly moist, medium brown silty fine to medium sand with scattered angular gravel. Fill materials located along the eastern margin of the site include concrete washout and debris.

4.2.2 Quaternary Recent Alluvium (Qal)

Quaternary Recent Alluvium was observed within a steeply incised north-south trending alluvial channel at the east portion of the site. The recent alluvial deposits are expected to consist of loose clayey to silty sand with gravels and cobbles.

4.2.3 Quaternary Alluvium and Colluvium, undivided (Qu)

Quaternary Undivided Alluvium and Colluvium at the site consist primarily of medium dense to very dense, dry to slightly moist, medium brown to red brown, clayey to silty fine to medium sand with gravels and cobbles. Density of these deposits generally increased with depth.

4.2.4 Cretaceous Santiago Peak Volcanics (Ksp)

Cretaceous Santiago Peak Volcanics is mapped by Todd (2004) and Kennedy, Tan, et. al (2005) to underlie the site. The Santiago Peak Volcanics encountered by the site subsurface explorations was weathered to dense to very dense silty sand that with depth became very dense crystalline bedrock that offered refusal to drill progress. It is noted that Tan (2002) mapped the site to be underlain by Cretaceous Gabbro. The nomenclature of Todd (2004) and Kennedy, Tan, et. al. (2005) is utilized herein as it supersedes the Tan (2002) date. This material is expected to offer heavy resistance to excavation where encountered as unweathered crystalline bedrock.

4.3 Groundwater Conditions

Groundwater was not observed at the time of CTE explorations. However, it is possible that the narrow natural drainage on the east side of the site allows subsurface water flow. Groundwater levels will likely vary with seasonal fluctuations. Saturated soils could impact grading or construction activities, especially during or after periods of sustained precipitation. However, groundwater is not anticipated to adversely impact the proposed development, provide site drainage is properly designed, constructed, and maintained as per the project civil engineer of record. In addition, localized typical subdrains could be required during rough grading.

4.4 Geologic Hazards and Assessment

Following is a consideration of geologic hazards pertinent to the site. An assessment of potential impacts to the site is also provided.

4.4.1 Local and Regional Faulting

The California Geological Survey broadly groups faults as "Class A" or "Class B" (CDMG, 1996). Class A faults are identified based upon relatively well-defined paleoseismic activity, and a fault-slip rate of more than 5 millimeters per year (mm/yr). In contrast, Class B faults have comparatively less defined paleoseismic activity and are considered to have a fault-slip rate less than 5 mm/yr. The nearest Class B fault is the Spanish Bight segment of the Rose Canyon Fault, approximately 17.2 kilometers to the west based on the State of California Earthquake Fault Zones map. The nearest known Class A fault is the Julian segment of the Elsinore Fault which is located approximately 54.6 kilometers northeast of the site (Blake, T.F., 2000). The following Table 1 presents the six faults nearest to the site, including their maximum earthquake magnitude and fault classification.

TABLE 4.4.1 NEAR SITE FAULT PARAMETERS			
FAULT NAME	DISTANCE FROM SITE (KM)	MAXIMUM EARTHQUAKE MAGNITUDE	CLASSIFICATION
Rose Canyon	17.2	7.2	B
Coronado Bank	38.5	7.6	B
Elsinore-Julian	54.6	7.1	A
Earthquake Valley	60.6	6.5	B
Elsinore- Coyote Mountain	61.4	6.8	A
Newport-Inglewood (Offshore)	65.6	7.1	B

The site could be subjected to significant shaking in the event of a major earthquake on any of the faults listed above or other regional faults in the southern California or northern Baja California area.

4.4.2 Liquefaction and Seismic Settlement Evaluation

Liquefaction occurs when saturated fine-grained sands or silts lose their physical strengths during earthquake induced shaking and behave as 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. Lateral spread occurs when there is widespread liquefaction and a modest slope, or a free face toward which lateral spreading may occur.

Because of medium dense to dense alluvial soils and dense to very dense bedrock underlying the site, the lack of shallow a groundwater table and the anticipated grading of the site, the potential for liquefaction, seismic settlement, and lateral spread is considered negligible.

4.4.3 Tsunamis and Seiche Evaluation

According to McCulloch (1985), the tsunami potential in the San Diego County coastal area for one-in-100 and one-in-500 year tsunami waves are approximately four and six feet. This indicates that there is a negligible probability of site damage due to the elevation of the site, more than 540 feet above msl, and distance to the ocean. In addition, damage caused by oscillatory waves (seiche) is considered unlikely due to the site elevation and lack of nearby, upgradient, open bodies of water.

4.4.4 Landsliding

Based on mapping by Tan (1995), the site area is considered “generally susceptible” to landsliding. However, active landslides were not encountered during this investigation, and have not been mapped on the site. Consequently, it is expected the potential for landsliding at the site is low, provided surface drainage is controlled and maintained, and site grading is performed as recommended herein.

4.4.5 Compressible and Expansive Soils

Near surface soils including the recent alluvium in the east drainage are dry, and loosened by vegetation and animal burrows. It is recommended that near surface soils be overexcavated and processed for placement as compacted in fill in areas of structure and fill support. Deeper alluvium, weathered bedrock and bedrock are anticipated to not be significantly compressible with respect to the intended site development, but should be grades as recommended herein.

Based on geologic observation and laboratory test results the near-surface materials at the site are anticipated to have a very low to low expansion potential.

4.4.6 Corrosive Soils

Chemical testing was performed to evaluate the potential effects that site soils may have on concrete foundations and various types of 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 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 (1,000 ppm). These guidelines include low water to 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 to buried metallic utilities and untreated conduits.

Chemical test results for representative site soils indicate a soluble sulfate content of less than 0.1 percent and a pH value of 7.49, that indicate a negligible corrosion potential for Portland cement concrete improvements. The obtained minimum resistivity value of 8,340 ohm-cm and soluble chloride value of 20.3 ppm indicate the tested soils generally have a low to mild corrosion potential for buried uncoated metallic conduits. The results of the chemical tests performed are presented in the attached Appendix C. CTE does not practice corrosion engineering. Therefore, a corrosion engineer should be contacted if site specific issues are of concern.

5.0 CONCLUSIONS AND RECOMMENDATIONS

5.1 General

From a geotechnical perspective, the site is considered suitable for development provided recommendations of this report are followed. Unweathered surficial deposits and underlying Cretaceous Santiago Peak Volcanics at the site are generally considered to be suitable for support of planned structures, improvements, and fill soils. In order to minimize potential differential soil movement, it is generally recommended that structure foundations be founded entirely on bedrock or entirely on compacted fill materials. Based on the site evaluation, soil conditions, and proposed structure layout, it is anticipated that the proposed foundations will be founded entirely upon compacted fill soil. CTE should review design phase grading plans and structural plans as they are available to further evaluate geotechnical recommendations provided herein.

Excavation of the site materials may be locally difficult for standard equipment due to local oversize boulders. Excavation of the underlying site dense to very dense crystalline bedrock will likely require specialized equipment, heavy ripping, rock splitting and/or blasting. Disposal and special handling of oversize material not suitable for placement in foundation or utility zones should be anticipated.

Recommendations for the proposed earthwork and improvements are included in the following sections and Appendix D. Where applicable, recommendations in the text of this report supersede those presented in Appendix D. All recommendations are subject to modification based on CTE's review of applicable design phase documents and geotechnical related conditions encountered during grading. CTE may modify recommendations provided herein should site conditions and development plans change.

5.2 Grading and Earthwork

Upon commencement of work, CTE personnel should continuously observe grading and earthwork operations for this project. CTE personnel should perform observation and testing of soil overexcavation, processing, and placement during grading as they pertain to CTE's professional opinions and recommendations contained herein.

5.3 Site Preparation

5.3.1 General

Before grading, the site should be cleared of any existing debris and other deleterious materials. Near surface vegetation should by "grubbed" from the site and disposed of properly; these materials are not suitable for incorporation in compacted fill soils. In areas to receive structures, distress-sensitive improvements and fills, expansive, surficial eroded, desiccated, burrowed, or otherwise loose or disturbed soils should be overexcavated to the depth of moist, competent dense formational materials at a minimum overexcavation depth of three feet below existing grades, and a minimum two feet below bottom of foundations. Deeper overexcavation may be necessary based on encountered site conditions.

Organic and other deleterious materials not suitable for structural fill should be properly disposed of offsite. Existing loose and disturbed near surface fill soil should be overexcavated and compacted (as necessary) under the observation and testing of a CTE geotechnical representative. Overexcavation should extend at least five feet beyond the building or improvement limits, where feasible.

5.3.2 Excavations

Due to the possible presence of oversize irreducible (rock) materials in the undivided alluvium and colluvium, and incorporated in fill soils, excavations may be difficult and result in over digging and/or irregularly-shaped excavations. If deep excavations are proposed, underlying bedrock materials are expected to provide very difficult excavation conditions. It is anticipated that these dense to very dense materials will require very large heavy-duty excavation equipment, rock breaking equipment, hydraulic or chemical splitting, and/or blasting to excavate and process. Large irreducible materials would likely result from excavation of the underlying unweathered very dense bedrock or large boulders incorporated into site native soils. Special handling or disposal of large rock could be necessary.

Irreducible materials generally greater than three to six inches in diameter should not be used in shallow fills on the site. However, such materials (and larger materials) could be placed at depth as per the recommendations in Appendix D, and as recommended by CTE during construction. CTE should observe excavation as it progresses, as well as the exposed surfaces prior to fill placement.

5.3.3 Fill Placement and Compaction

CTE should observe that proper site preparation is prepared prior to fill placement. Areas to receive fills should be scarified and moisture conditioned as recommended herein. Fill and backfill should be compacted to a minimum relative compaction of 90 percent as evaluated by ASTM D1557 at a moisture content at least two percent above optimum. In proposed pavement areas, the upper foot of fill soil (and all aggregate base) should be compacted to a minimum relative compaction of 95 percent of maximum dry density at a moisture content of at least two percent above optimum.

The optimum lift thickness for backfill soil will depend on the type of compaction equipment used. Generally, backfill should be placed in uniform lifts not exceeding eight inches in loose thickness. Backfill placement and compaction should be performed in conformance with geotechnical recommendations and local ordinances.

5.3.4 Fill Materials

Very low to low expansion potential soils derived from the onsite materials are generally considered suitable for reuse on the site as shallow engineered compacted fill. If used, these materials should be screened of significant construction debris, vegetation matter, and oversize materials generally greater than six inches in maximum dimension within five feet of finish grade. Although not anticipated, adverse effects of moderately to highly expansive clay soils should be mitigated by blending these soils with less expansive materials and compacting at moisture contents well above optimum.

Imported fill beneath structures, pavements and walks should have an Expansion Index of 20 or less with less than 35 percent passing the no. 200 sieve. Imported fill soils for use in structural or slope areas should be evaluated by CTE before placement on the site.

5.4 Temporary Construction Slopes

Recommendations for unshored temporary excavations without seepage are provided herein. The recommended slopes should be relatively stable against deep-seated failure, but may experience localized sloughing. It is assumed the proposed slopes are homogenous and free of joints, fractures and bedding surfaces. Care should be taken to prevent destabilization of boulders if present within and above temporary excavations. Recommended slope ratios are set forth in Table 6.4 below.

TABLE 5.4
RECOMMENDED TEMPORARY SLOPE RATIOS

SOILS TYPE	SLOPE RATIO (Horizontal: Vertical)	MAXIMUM HEIGHT (without seepage)
B (Cretaceous Santiago Peak Volcanics)	1:1 (MAXIMUM)	10 FEET
C (Alluvium and Colluvium, undivided)	1 ½:1 (MAXIMUM)	5 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 recommendations do not allow for potential water seepage, surcharge loading at the top of slopes by vehicular traffic, equipment or materials and or defects and weaknesses in the excavated mass. Appropriate surcharge setbacks must be maintained from the top of all unshored slopes.

5.5 Temporary Shoring

Temporary shoring is not anticipated to be necessary based on the currently proposed development. However, if such improvements become necessary, CTE will provide proper design and construction recommendations, upon request.

5.6 Foundations and Slab Recommendations

Geotechnical recommendations for structure slabs and spread foundations are provided herein. These recommendations should be considered preliminary and subject to revision as project design plans are developed.

5.6.1 General

Standard spread foundations are considered suitable for support of the proposed structures provided all footings are embedded entirely on either competent dense native bedrock or embedded into and entirely underlain by at least two feet of compacted fill materials, as recommended herein. Based on site conditions, it is anticipated that the structures will be founded entirely on compacted fill materials. However, these recommendations should be reviewed as project layout and foundation depths are developed.

5.6.2 Shallow Spread Foundations

Standard spread foundations founded entirely in compacted fill should be embedded at least 18 inches below the lowest adjacent subgrade, and underlain by at least 24 inches of compacted fill measured from the foundation bottom. Spread foundations should be at least 18 inches in width. Foundations as recommended herein may be designed to impose an allowable bearing pressure of 2,500 pounds per square foot (psf). Proposed footings should be designed such that the horizontal distance from the face of adjacent descending fill slopes to the outer edge of the footing is at least 10 feet.

The bearing values above may be increased by 250 psf for each additional six inches of depth or width beyond the minimum, up to a maximum bearing pressure of 3,500 psf. However, if footings are planned to be deepened, CTE should review the proposed conditions to further evaluate the shallow to deep fill transitional bearing conditions. For bearing values herein, a one-third increase may also be used when evaluating short duration wind or seismic loads. The weight of any soil backfill may be neglected when determining the downward load on the footings.

If elastic design methods are used, an uncorrected modulus of subgrade reaction on the order of 150 pci is considered appropriate for the anticipated site conditions and foundations bearing on competent compacted fills.

Minimum reinforcement for continuous footings should consist of four No. 4 reinforcing bars; two placed near the top and two placed near the bottom or as per more stringent requirements provided by the project structural engineer. The structural engineer should provide recommendations for reinforcement of isolated footings and footings with pipe penetrations. Pipe penetrations should be adequately sealed to prevent moisture intrusion into slab subsoils. Footing excavations should be maintained at above optimum moisture content until concrete placement. Foundations and/or any other structural elements should not be below a 3:1 plane extending upward from the excavation bottom (bottom below filter media) of an unlined infiltration basin to the bottom of the closest structural element edge.

Based on observations of the underlying site materials and the bearing pressures recommended above, it is anticipated total settlement of structural footings designed as recommended herein to be on the order of one inch. Differential settlements for spread and strip footings are expected to be on the order of 0.5 inch. The allowable bearing and anticipated settlement values should be re-evaluated after foundation plan and design loads have been finalized.

5.6.3 Lateral Resistance

Lateral loads for structures supported on spread or mat foundations may be resisted by soil friction and by the passive resistance of the adjacent soils. A coefficient of friction of 0.30 may be used between foundations and the supporting soils. The passive resistance of the fill l soils may be assumed to be equal to the pressure developed by a fluid with a density of 250 pcf, up to a maximum pressure of 2,000 psf. A one-third increase in the passive value may be used for wind or seismic loads. The frictional resistance and the passive resistance may be combined without reduction in determining the total lateral resistance provided the passive resistance does not exceed two-thirds of the total allowable resistance.

5.6.4 Concrete Slabs-On-Grade

Concrete slabs-on-grade should be designed for the anticipated loading; however, based on the underlying fill thickness conditions, should measure a minimum of 4.5 inches in thickness for the proposed structures. Slab areas subject to heavy loading or vehicle traffic should be designed by the structural engineer based on specific requirements, but measure a minimum five inches in thickness.

If elastic design is used, a modulus of subgrade reaction of 125 pci is considered appropriate for slabs on dense fill materials compacted to a minimum 90% relative dry density as specified herein. The anticipated light to moderately loaded concrete slabs-on-grade should be reinforced with minimum #3 reinforcing bars placed on maximum 18-inch centers, each way, at or above mid-slab height and with proper concrete cover. Reinforcement of concrete slabs subjected to heavier loads should be detailed by the project structural engineer.

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 crushed aggregate or gravel, as per the 2013 CBC/Green Building Code, is recommended. 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, if necessary. 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.

It is recommended that all concrete slabs be moist-cured for at least five days in accordance with methods recommended by the American Concrete Institute. Onsite concrete quality control should be utilized during the concrete cure period.

5.7 Seismic Design Criteria

The seismic ground motion values listed in the following Table 5.7 were derived in accordance with the ASCE 7-10 Standard. This was accomplished by establishing the Site Class based on the soil properties at the site, and then calculating the site coefficients and parameters using the United States Geological Survey Seismic Design Maps application for the 2013 and 2016 CBC values. These values are intended for the design of structures to resist the effects of earthquake ground motions for the site coordinates 32.74477° latitude and –116.95538° longitude, as underlain by soils corresponding to site Class D. The following values assume the site is not a critical life support facility.

TABLE 5.7 SEISMIC GROUND MOTION VALUES		
PARAMETER	VALUE	CBC REFERENCE (2013)
Site Class	D	ASCE 7, Chapter 20
Mapped Spectral Response Acceleration Parameter, S_S	0.848 g	Figure 1613.3.1 (1)
Mapped Spectral Response Acceleration Parameter, S_1	0.329 g	Figure 1613.3.1 (2)
Seismic Coefficient, F_a	1.161 g	Table 1613.3.3 (1)
Seismic Coefficient, F_v	1.742 g	Table 1613.3.3 (2)
MCE Spectral Response Acceleration Parameter, S_{MS}	0.984 g	Section 1613.3.3
MCE Spectral Response Acceleration Parameter, S_{M1}	0.573 g	Section 1613.3.3
Design Spectral Response Acceleration, Parameter S_{DS}	0.656 g	Section 1613.3.4
Design Spectral Response Acceleration, Parameter S_{D1}	0.382 g	Section 1613.3.4
Peak Ground Acceleration PGA_M	0.379 g	ASCE 7, Section 11.8.3

5.8 Retaining Walls

Retaining walls up to approximately 15 feet high and backfilled using granular soils may be designed using the equivalent fluid weights given on Table 5.8 below.

TABLE 5.8 EQUIVALENT FLUID UNIT WEIGHTS (pounds per cubic foot)		
WALL TYPE	LEVEL BACKFILL	SLOPE BACKFILL 2:1 (HORIZONTAL: VERTICAL)
CANTILEVER WALL (YIELDING)	35	60
RESTRAINED WALL	60	85

Lateral pressures on cantilever retaining walls (yielding walls) due to earthquake motions may be calculated based on work by Seed and Whitman (1970). The total lateral thrust 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 thrust may be similarly calculated based on work by Wood (1973):

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

Where P_A = Static Active Thrust (given previously Table 6.8)
P_K = Static Restrained Wall Thrust (given previously Table 6.8)
ΔP_{AE} = Dynamic Active Thrust Increment = $(3/8)\ k_h\ \gamma H^2$
ΔP_{KE} = Dynamic Restrained Thrust Increment = $k_h\ \gamma H^2$
k_h = 2/3 Peak Ground Acceleration = 2/3 (PGA_M)
H = Total Height of the Wall
γ = Total Unit Weight of Soil ≈ 135 pounds per cubic foot

The increment of dynamic thrust in both cases should be distributed triangularly with a line of action located at H/3 above the bottom of the wall (SEAOC, 2013).

In addition to the recommended earth pressure, the upper 10 feet of subterranean walls adjacent to streets or other traffic loads should be designed to resist a uniform lateral pressure of 100 psf. This is the result of an assumed 300-psf surcharge behind the walls due to typical traffic loading. If the traffic loads are set back at least 10 feet from the subject walls, the traffic surcharge may be neglected.

CTE recommends that all walls be backfilled with soil having an Expansion Index of 20 or less. The backfill area should include the zone defined by a 1:1 sloping plane, extended back from the base of the wall. Retaining wall backfill should be compacted to between 90 and 95 percent relative compaction, based on ASTM D1557-91. Backfill should not be placed until walls have achieved adequate structural strength. Heavy equipment, which could cause distress to walls, should not be used for compaction of soils behind retaining walls. 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 or the waterproofing specialty consultant.

5.9 Vehicular Pavements

The proposed development will include automobile drive and parking areas, as well as truck drive and loading areas. Preliminary pavement sections presented below are based on a soil sample from Boring 1 at a depth between the surface and five feet below existing grade. Existing compacted fill materials should be prepared as indicated in the previous sections of this report. Subgrade and all aggregate base materials in pavement areas should be compacted to a minimum of 95% relative compaction at a moisture content slightly above optimum. After grading, testing of subgrade soils for Resistance "R"-Value should be performed to assist in pavement recommendations based upon as-built conditions.

TABLE 5.9
PRELIMINARY RECOMMENDED PAVEMENT THICKNESS

Traffic Area	Assumed Traffic Index	Assumed Subgrade "R"-Value	AC Thickness (inches)	Class II Aggregate Base Thickness (inches)	Full Depth Concrete (inches)
Truck Drive/ Loading Areas	6.0	<5	5.0	10.0	7.0
Auto Parking & Drive Areas	5.0	<5	4.0	8.0	6.0

* Auto parking and drive areas sections are anticipated to be adequate for infrequently used fire lanes or similar for the pavement as currently prepared.

Concrete pavements should have a modulus of rupture of at least 600 psi. PCC pavement can be constructed with No. 4 reinforcing bars placed at no more than 24 inches on center, each way, at or above mid-pavement height. As an alternative, pavements may be constructed without reinforcement if construction or expansion/contraction joints are spaced no greater than a distance equal to 24 times the pavement thickness, in both directions. Concrete pavement details should be in accordance with, for example, the recommendations of the American Concrete Institute or other widely recognized authority, particularly with regard to thickened edges, joint spacing, doweling, and drainage. The closest bottom edge of structural foundations and curb stops should be below a

3:1 (horizontal to vertical) ratio plane extending upward from the bottom of unlined infiltration basins.

5.10 Exterior Concrete Flatwork

Exterior concrete slabs for pedestrian loads should measure a minimum four inches thick and have minimal reinforcement of #3 rebar on 24-inch centers (both ways), or equivalent pre-fabricated reinforcement. Soils to a depth of at least one foot below the concrete should be processed to 90 percent of maximum dry density at least two over optimum moisture content. As applicable, native soils below concrete slab subgrade should be over excavated, and a properly compacted moisture conditioned subgrade placed in the resulting volume. Exterior flatwork subgrade soils to a depth of one foot should be at or above a two percent of optimum moisture just prior to concrete pour. Reinforcement should be placed at or above mid-height in the slab, but with proper cover, or as recommended by the project engineer or architect. Flatwork should be installed with reinforcement and crack control joints spaced as recommended by the project engineer or architect. Positive drainage to convey water away from all flatwork should be established and maintained. However, site drainage should be designed and detailed by the project civil engineer.

5.11 Drainage

Surface runoff should be collected and directed away from improvements by means of erosion control 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 engineer should evaluate the on-site drainage and make necessary provisions to keep surface water from affecting the site.

CTE generally recommends against allowing water to infiltrate building pads or adjacent to slopes and improvements. It is understood that some agencies are encouraging the use of storm-water cleansing devices. Therefore, if storm water cleansing devices must be used, it is generally recommended that they be underlain by an impervious barrier and that the infiltrate be collected via subsurface piping and discharged off site. If infiltration must occur, water should infiltrate as far away from structural improvements as feasible and distress in the infiltrate area should be anticipated. Attached Appendix E provides calculated infiltration rates for bio-retention basins planned at the site.

5.12 Slopes

Existing slopes at the site are considered to be generally stable. Slopes at this site should be constructed at 2:1 (horizontal: vertical) or flatter ratio. Surface 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 graded slopes.

5.13 Plan Review

CTE should review all project grading and foundation plans during the project design phase, before the start of earthwork. Depending upon the results of the review, an addendum report may be necessary based upon development-specific conditions.

5.14 Construction Observation

The recommendations provided in this report are based on preliminary design information for the proposed earthwork and the subsurface conditions found in the exploration locations. The interpolated subsurface conditions should be further evaluated in the field during construction.

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 geotechnical related work should be observed and tested by CTE. All soil preparation and foundation excavations should be evaluated by a designated CTE representative.

6.0 LIMITATIONS OF INVESTIGATION

The field evaluation, laboratory testing, and geotechnical analysis presented in this report have been conducted according to current geotechnical 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.

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 be 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 of CTE's control. Therefore, this report is subject to review and should not be relied upon after a period of three years or project development plans change. This report is prepared for the project described. It is not suitable for use on any other projects.

The recommendations presented herein have been developed in order to minimize the potential adverse affects of the anticipated shallow to deep fill transitional bearing conditions. However, even with the design and construction recommendations herein, some post-construction differential movement could occur.

The conclusions and recommendations of this report are based on an analysis of the observed conditions. If conditions different from those described in this report are encountered, this office should be notified and additional recommendations, if required, will be provided upon request.

This report has been prepared as per County of San Diego storm water control requirements as presented in Appendix E of this report. CTE does not accept any liabilities toward preparation of the County of San Diego required design requirements. It is noted that implementation of the information provided herein is subject to interpretation and approval of the County of San Diego who has adopted the subject storm water requirements. CTE does not accept the rationale of the subject County of San Diego storm water design requirements other that preparation of this report as required by the County of San Diego storm water design documents.

The opportunity to be of service on this project is appreciated. 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

Gregory F. Rzonca, CEG #1191
Senior Certified Engineering Geologist

MM/GFR/DTM:nri

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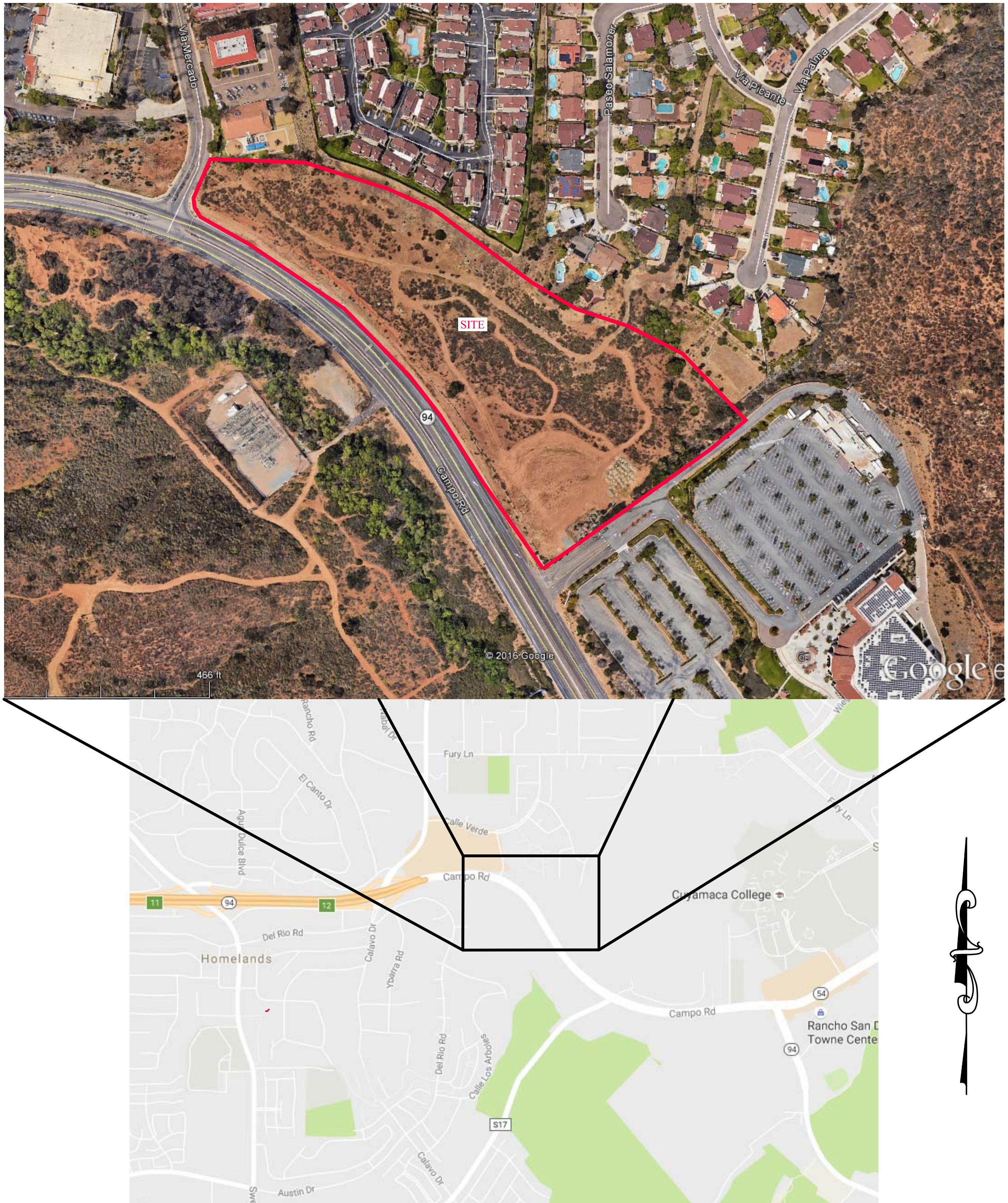

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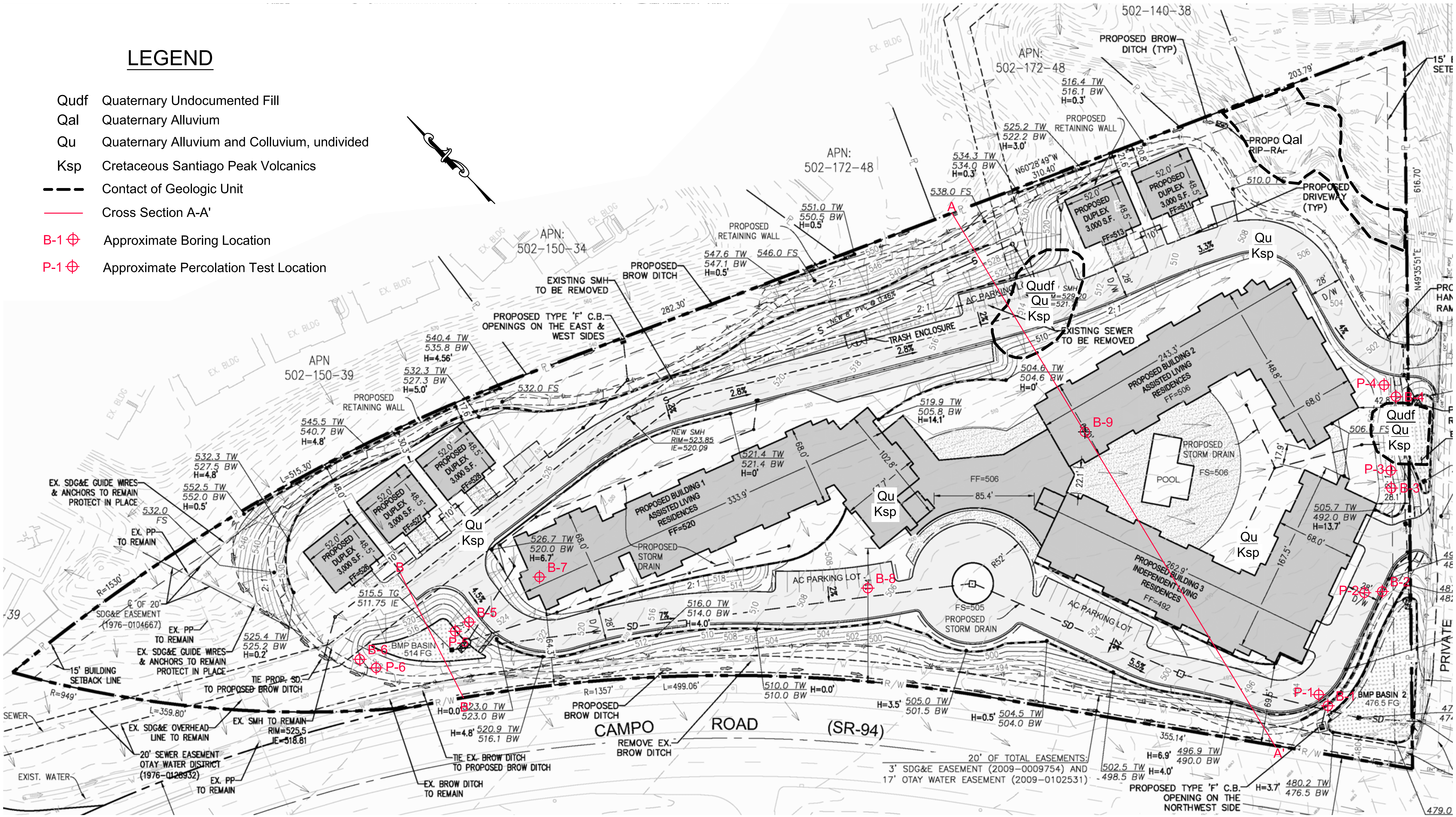
SITE INDEX MAP SKYLINE RETIREMENT CENTER 11330 CAMPO ROAD LA MESA, CALIFORNIA	SCALE: AS SHOWN	DATE: 10/16
	CTE JOB NO.: 10-13295G	FIGURE: 1

LEGEND

Qudf	Quaternary Undocumented Fill
Qal	Quaternary Alluvium
Qu	Quaternary Alluvium and Colluvium, undivided
Ksp	Cretaceous Santiago Peak Volcanics
– – –	Contact of Geologic Unit
——	Cross Section A-A'
B-1 ⊕	Approximate Boring Location
P-1 ⊕	Approximate Percolation Test Location



\\Esc_server\projects\10-13295G\Figures\Figure 2 Geotechnical Map.dwg

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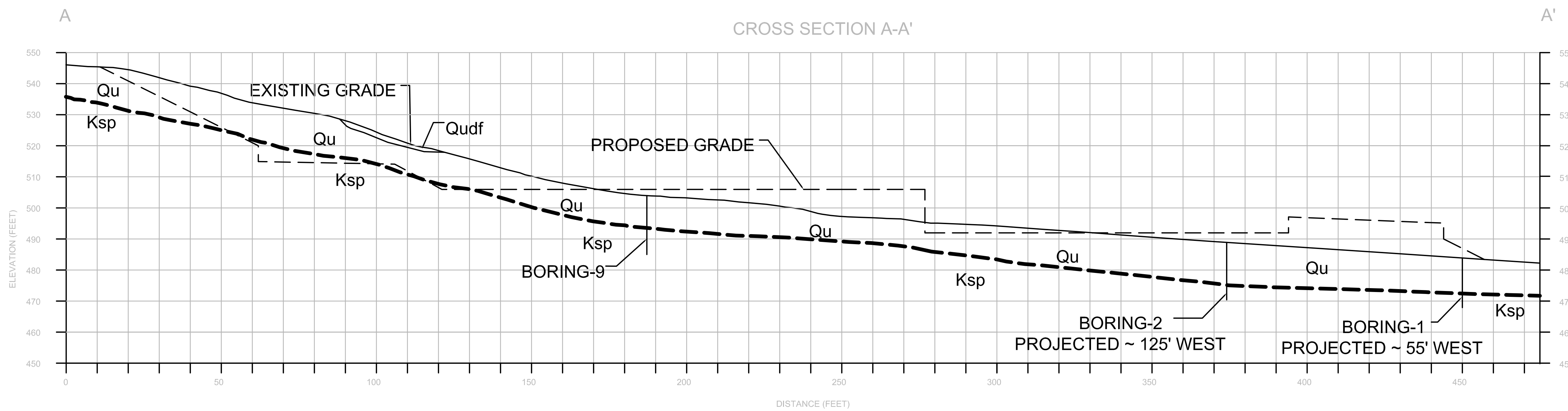
GEOTECHNICAL MAP
SKYLINE RETIREMENT CENTER
11330 CAMPO ROAD
LA MESA, CALIFORNIA

CTE JOB NO:	10-13295G
SCALE:	1" = 80'
DATE: 10/16	FIGURE: 2



LEGEND

Qudf	Quaternary Undocumented Fill
Qu	Quaternary Alluvium and Colluvium, undivided
Ksp	Cretaceous Santiago Peak Volcanics
— — —	Contact of Geologic Unit

\\Esc_server\projects\10-13295G\Figures\Figure 2A Cross Section A-A'.dwg

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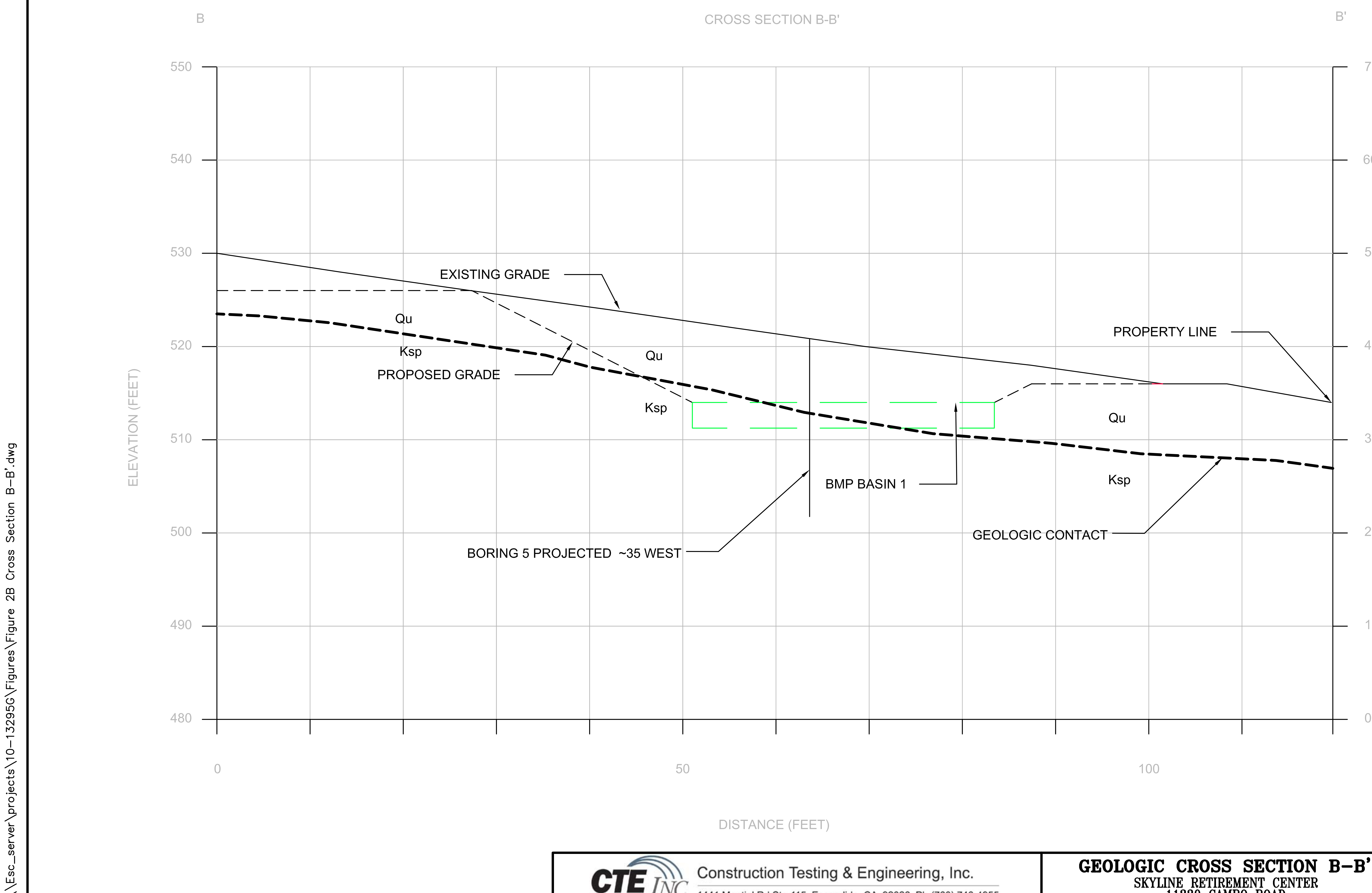
GEOLOGIC CROSS SECTION A-A'
SKYLINE RETIREMENT CENTER
11330 CAMPO ROAD
LA MESA, CALIFORNIA

CTE JOB NO: 10-13295G	
SCALE: 1" = 40'	
DATE: 10/16	FIGURE: 2A



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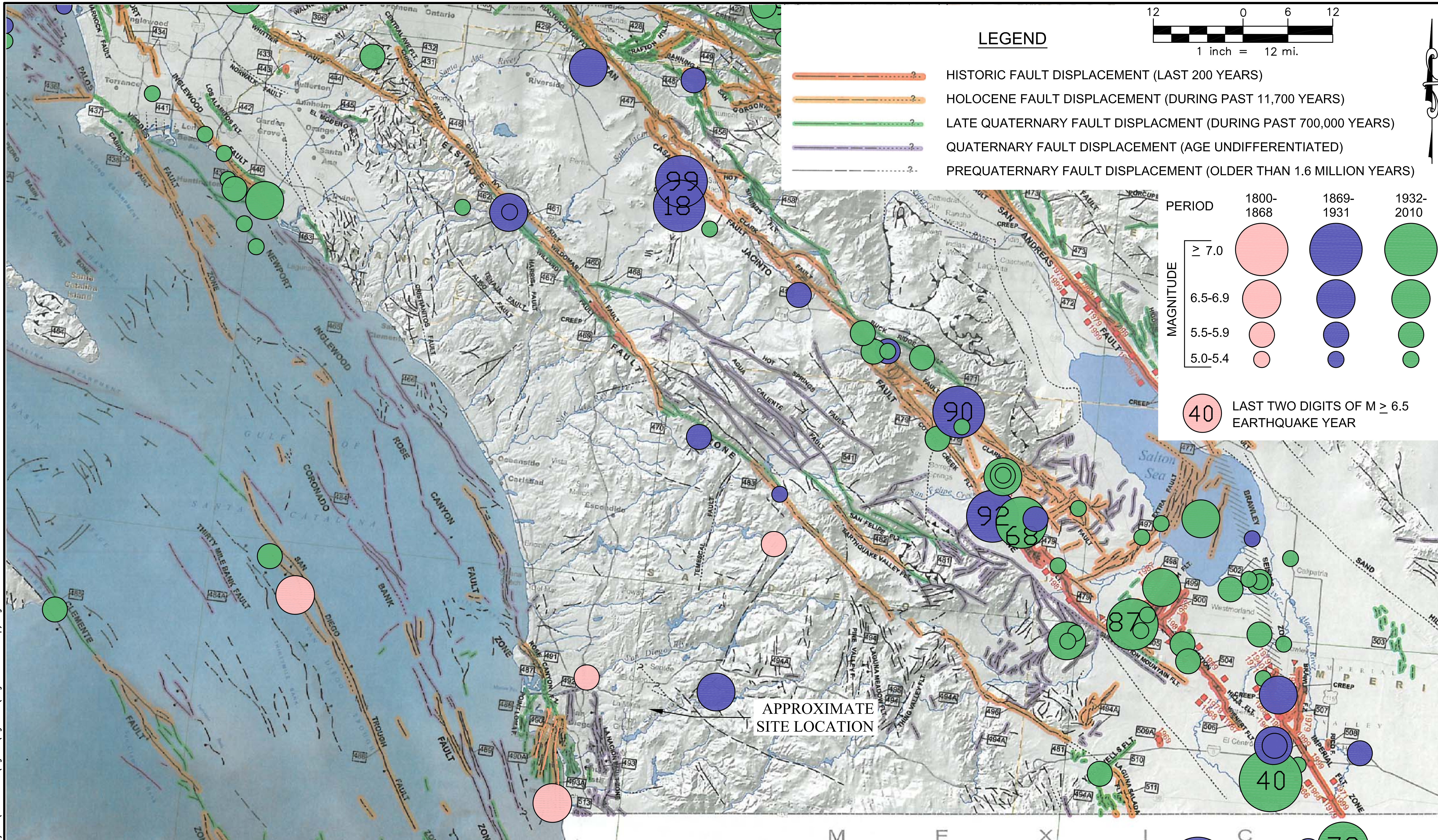
GEOLOGIC CROSS SECTION B-B'
SKYLINE RETIREMENT CENTER
11330 CAMPO ROAD
LA MESA, CALIFORNIA

CTE JOB NO: 10-13295G	
SCALE: 1" = 10'	
DATE: 10/16	FIGURE: 2B



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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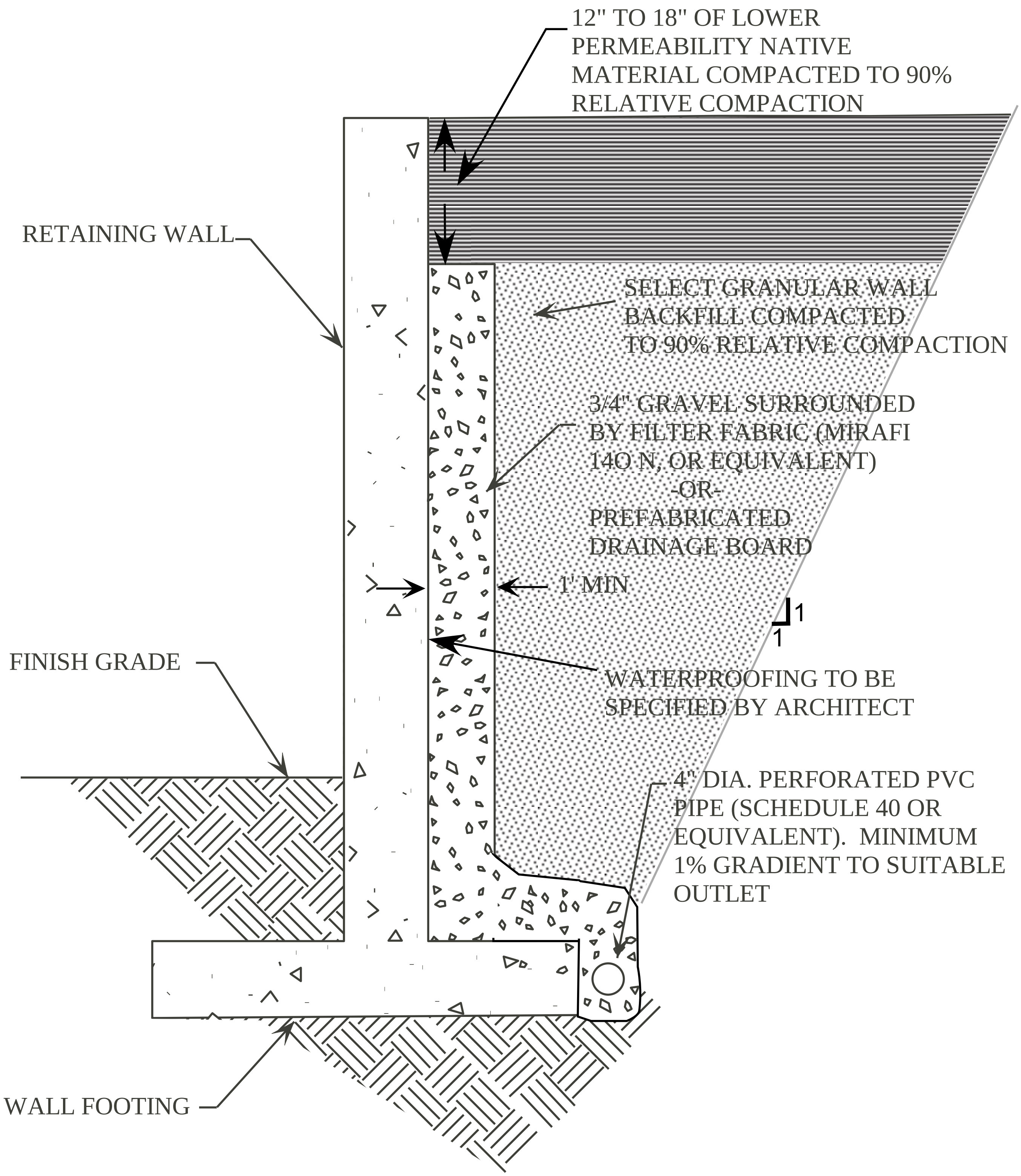
REGIONAL FAULT AND SEISMICITY MAP

SKYLINE RETIREMENT CENTER
11330 CAMPO ROAD
LA MESA, CA

CTE JOB NO:	10-13295G
SCALE:	1 inch = 12 miles
DATE: 10/16	FIGURE: 3

\\ESC_SERVER\Projects\10-13295G\Figures\Figure 3 (Regional Fault Map).dwg



Construction Testing & Engineering, Inc.
Inspection | Testing | Geotechnical | Environmental & Construction Engineering | Civil Engineering | Surveying

RETAINING WALL DRAINAGE DETAIL

CTE JOB NO: 10-13295G	
SCALE: NO SCALE	
DATE: 10/16	FIGURE: 4

APPENDIX A

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REFERENCES CITED

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APPENDIX B

FIELD EXPLORATION LOGS

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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

FIGURE: BL1

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

Depth (Feet)	Bulk Sample	Driven Type	Blows/Foot	Dry Density (pcf)	Moisture (%)	U.S.C.S. Symbol	Graphic Log	BORING LEGEND 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:	SKYLINE RETIREMENT CENTER	DRILLER:	BAJA EXPLORATION	SHEET:	1 of 1
CTE JOB NO:	10-13295G	DRILL METHOD:	HOLLOW-STEM AUGER	DRILLING DATE:	8/31/2016
LOGGED BY:	MM	SAMPLE METHOD:	RING, SPT and BULK	ELEVATION:	~ 481'

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	X					SC		**QUATERNARY ALLUVIUM AND COLLUVIUM, undivided**: Medium dense to dense, slightly moist, medium brown, clayey fine to medium SAND, trace gravel.	RV, MAX
5		Ring	15 16 25					Dense, slightly moist, reddish brown, clayey fine SAND with trace medium sand, indurated.	
10		SPT	18 36 50/5"			SM/SC		Very dense, slightly moist, reddish brown to olive gray, silty fine to medium SAND, trace clay.	
								CRETACEOUS SANTIAGO PEAK VOLCANICS: Becomes olive gray, medium to coarse grained sand.	
15		Ring	50/3"			SM		Very dense, dry to slightly moist, olive gray, silty fine SAND, minimal return, dark gray, crystalline bedrock within sampler.	
								Total Depth: 16' No groundwater encountered Boring Backfilled 8/31/16	
20									
25									

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PROJECT:	SKYLINE RETIREMENT CENTER	DRILLER:	BAJA EXPLORATION	SHEET:	1 of 1
CTE JOB NO:	10-13295G	DRILL METHOD:	HOLLOW-STEM AUGER	DRILLING DATE:	8/31/2016
LOGGED BY:	MM	SAMPLE METHOD:	RING, SPT and BULK	ELEVATION:	~ 487'

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						SC		**QUATERNARY ALLUVIUM AND COLLUVIUM, undivided**: Medium dense, slightly moist, medium brown, clayey fine to medium SAND.	
		Ring	19 21 21					Dense, slightly moist, medium brown, clayey fine to medium SAND, indurated.	
5								Becomes dark brown.	
		SPT	21 50/6"					Very dense, slightly moist, reddish brown to dark brown, clayey fine to medium SAND, trace coarse sand, well indurated.	
10								Becomes clayey.	
		Ring	9 18 50/5"					Very dense, slightly moist to moist, dark reddish brown, clayey fine to medium sand.	
15						SC		**CRETACEOUS SANTIAGO PEAK VOLCANICS**: Within sampler at 14': Becomes olive brown, clayey fine to medium SAND with trace coarse SAND, micaceous. Becomes olive gray, sandy. Very dense, slightly moist, olive gray to drak gray, fine to medium SAND, minimal fines, trace coarse SAND.	
		Ring	50/4"			SM			
20								Total Depth: 18.5' No groundwater encountered Boring Backfilled 8/31/16	
25									

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PROJECT:	SKYLINE RETIREMENT CENTER	DRILLER:	BAJA EXPLORATION	SHEET:	1 of 1
CTE JOB NO:	10-13295G	DRILL METHOD:	HOLLOW-STEM AUGER	DRILLING DATE:	8/31/2016
LOGGED BY:	MM	SAMPLE METHOD:	RING, SPT and BULK	ELEVATION:	~ 494

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						SC		**QUATERNARY ALLUVIUM AND COLLUVIUM, undivided**: Medium dense to dense, slightly moist, medium brown, clayey fine to medium SAND, abundant gravel.	
		Ring	9 12 12	111.8	3.7			Medium dense, sllightly moist, orange brown to reddish brown, clayey fine to medium SAND, trace coarse SAND and gravel.	MD, GS
5									
								Becomes darker reddish brown.	
10		Ring	16 24 39					Very dense, slightly moist, dark reddish brown, clayey fine to medium SAND, indurated.	
								Becomes dark brown, clayey.	
15		SPT	12 14 12					Medium dense, moist, dark reddish brown, clayey fine to medium SAND.	GS
								Total Depth: 16.5' No groundwater encountered Boring Backfilled 8/31/16	
20									
25									

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PROJECT:	SKYLINE RETIREMENT CENTER	DRILLER:	BAJA EXPLORATION	SHEET:	1 of 1
CTE JOB NO:	10-13295G	DRILL METHOD:	HOLLOW-STEM AUGER	DRILLING DATE:	8/31/2016
LOGGED BY:	MM	SAMPLE METHOD:	RING, SPT and BULK	ELEVATION:	~ 497'

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					SC		**QUATERNARY ALLUVIUM AND COLLUVIUM, undivided**: Medium dense, slightly moist, medium brown, clayey fine SAND, some gravel.	
	X	X	13 14 14					Becomes orangeish brown. Medium dense, slightly moist, orangeish brown, clayey fine to medium SAND.	
5									
								Becomes reddish brown.	
		X	11 11 13					Medium dense, slightly moist, dark brown, clayey fine to medium SAND, indurated.	
10									
		X	9 10 15					Medium dense, slightly moist, dark brown, clayey fine to medium SAND, trace coarse sand.	GS
15								Total Depth: 14.5' No groundwater encountered Boring Backfilled 8/31/16	
20									
25									

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PROJECT:	SKYLINE RETIREMENT CENTER	DRILLER:	BAJA EXPLORATION	SHEET:	1 of 1
CTE JOB NO:	10-13295G	DRILL METHOD:	HOLLOW-STEM AUGER	DRILLING DATE:	8/31/2016
LOGGED BY:	MM	SAMPLE METHOD:	RING, SPT and BULK	ELEVATION:	~ 525'

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						SC		**QUATERNARY ALLUVIUM AND COLLUVIUM, undivided**: Medium dense, slightly moist, medium brown, clayey fine to medium SAND, trace gravel.	
								Becomes orangeish brown.	
5			20 19 11					Dense, slightly moist, dark reddish brown, clayey fine to medium SAND, trace coarse sand, well indurated.	
								Becomes olive gray, silty.	
								CRETACEOUS SANTIAGO PEAK VOLCANICS:	
10			43 50			SM		Very dense, slightly moist, olive gray to gray, silty fine to coarse SAND, micaceous.	
								Becomes lighter gray, slightly coarser grained.	
15			50/5"			SM		Very dense, slightly moist, olive gray to gray, silty fine to coarse SAND, trace angular gravel.	
20								Total Depth: 19.5' No groundwater encountered Boring Backfilled 8/31/16	
25									

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PROJECT:	SKYLINE RETIREMENT CENTER	DRILLER:	BAJA EXPLORATION	SHEET:	1 of 1
CTE JOB NO:	10-13295G	DRILL METHOD:	HOLLOW-STEM AUGER	DRILLING DATE:	8/31/2016
LOGGED BY:	MM	SAMPLE METHOD:	RING, SPT and BULK	ELEVATION:	~ 532'

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						SC		**QUATERNARY ALLUVIUM AND COLLUVIUM, undivided**: Medium dense, slightly moist, medium brown, clayey fine to medium SAND, trace gravel.	
								Becomes orangeish brown.	
5			42 50/3"					Very dense, slightly moist, orangeish brown, clayey fine to medium SAND, minimal return, crystalline rock within sampler.	
								Becomes reddish brown.	
								CRETACEOUS SANTIAGO PEAK VOLCANICS:	
								Becomes olive, silty.	
			50/3"			SM		Very dense, slightly moist, olive brown to olive gray, silty fine to medium SAND, trace coarse SAND.	
10									
								Becomes olive, silty.	
15			50/2"			SM		Very dense, slightly moist, olive, silty fine to medium SAND, some medium sand, trace coarse sand.	
			50/3"			SM		Very dense, slightly moist, olive, silty fine to medium SAND, trace coarse sand.	
20								Total Depth: 18.5' No groundwater encountered Boring Backfilled 8/31/16	
25									

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PROJECT:	SKYLINE RETIREMENT CENTER	DRILLER:	BAJA EXPLORATION	SHEET:	1 of 1
CTE JOB NO:	10-13295G	DRILL METHOD:	HOLLOW-STEM AUGER	DRILLING DATE:	8/31/2016
LOGGED BY:	MM	SAMPLE METHOD:	RING, SPT and BULK	ELEVATION:	~ 532'

BORING: B-7

Depth (Feet)	Bulk Sample	Driven Type	Blows/6"	Dry Density (pcf)	Moisture (%)	U.S.C.S. Symbol	Graphic Log	DESCRIPTION	Laboratory Tests
0	X					SC		**QUATERNARY ALLUVIUM AND COLLUVIUM, undivided**: Very dense to dense, slightly moist, medium brown, clayey fine to medium SAND, trace coarse SAND.	MAX, EI, CHM, DS
								Becomes orangeish brown.	
5			50/4"					Very dense, slightly moist, reddish brown, clayey fine to medium SAND, indurated.	
								CRETACEOUS SANTIAGO PEAK VOLCANICS:	
								Becomes olive gray.	
								Difficult drilling.	
10			50/2"			SM		Minimal return, fine grained crystalline rock within sampler.	
								Refusal at 10.5' Total Depth: 10.5' No groundwater encountered Boring Backfilled 8/31/16	
15									
20									
25									

PROJECT:	SKYLINE RETIREMENT CENTER	DRILLER:	BAJA EXPLORATION	SHEET:	1 of 1
CTE JOB NO:	10-13295G	DRILL METHOD:	HOLLOW-STEM AUGER	DRILLING DATE:	8/31/2016
LOGGED BY:	MM	SAMPLE METHOD:	RING, SPT and BULK	ELEVATION:	~ 485'

BORING: B-8

Depth (Feet)	Bulk Sample	Driven Type	Blows/6"	Dry Density (pcf)	Moisture (%)	U.S.C.S. Symbol	Graphic Log	DESCRIPTION	Laboratory Tests
0						SC		**QUATERNARY ALLUVIUM AND COLLUVIUM, undivided**: Medium dense, slightly moist, medium brown, clayey fine SAND.	
								Becomes dense, orangeish brown.	
5			22 46 50/3"	118.4	5.6			Very dense, slightly moist, reddish brown to dark brown, clayey fine to medium SAND, trace coarse SAND, well indurated, micaceous.	MD
10			50/3"					Very dense, slightly moist, reddish brown to olive gray, clayey fine to medium SAND, indurated.	
								CRETACEOUS SANTIAGO PEAK VOLCANICS:	
								Becomes olive gray, silty to sandy.	
15			50/4"			SM		Very dense, slightly moist, olivre gray, silty fine to medium SAND, abundant coarse sand.	
			50/3"			SM			
20								Total Depth: 18.5' No groundwater encountered Boring Backfilled 8/31/16	
25									

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PROJECT:	SKYLINE RETIREMENT CENTER	DRILLER:	BAJA EXPLORATION	SHEET:	1 of 1
CTE JOB NO:	10-13295G	DRILL METHOD:	HOLLOW-STEM AUGER	DRILLING DATE:	8/31/2016
LOGGED BY:	MM	SAMPLE METHOD:	RING, SPT and BULK	ELEVATION:	~

BORING: B-9

Depth (Feet)	Bulk Sample	Driven Type	Blows/6"	Dry Density (pcf)	Moisture (%)	U.S.C.S. Symbol	Graphic Log	DESCRIPTION	Laboratory Tests
0						SC		**QUATERNARY ALLUVIUM AND COLLUVIUM, undivided**: Medium dense to dense, slightly moist, medium brown, clayey fine to medium SAND, trace gravel.	
								Becomes reddish brown.	
5			15 15 19					Dense, slightly moist, reddish brown, clayey fine to medium SAND, indurated.	
								Becomes olive gray, silty to sandy.	
						SC/SM		Very dense, slightly moist, reddish brown to olive gray, clayey to silty fine to coarse SAND.	
10			50/2"						
								CRETACEOUS SANTIAGO PEAK VOLCANICS:	
								Becomes lighter gray, silty.	
15			50/4"			SM		Very dense, slightly moist, olive gray to gray, silty fine to coarse SAND, micaceous.	
			50/3"			SM		Very dense, slightly moist, olive gray, silty fine to medium SAND, trace coarse sand.	
20								Total Depth: 19.5' No groundwater encountered. Boring Backfilled 8/31/16	
25									

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PROJECT:	SKYLINE RETIREMENT CENTER	DRILLER:	BAJA EXPLORATION	SHEET:	1 of 1
CTE JOB NO:	10-13295G	DRILL METHOD:	HOLLOW-STEM AUGER	DRILLING DATE:	8/31/2016
LOGGED BY:	MM	SAMPLE METHOD:	RING, SPT and BULK	ELEVATION:	~ 481'

BORING: P-1

Depth (Feet)	Bulk Sample	Driven Type	Blows/6"	Dry Density (pcf)	Moisture (%)	U.S.C.S. Symbol	Graphic Log	DESCRIPTION	Laboratory Tests
0						SC		**<u>QUATERNARY ALLUVIUM AND COLLUVIUM, undivided</u>**: Medium dense to dense, slightly moist, medium brown, clayey fine to medium SAND, trace gravel.	
								Becomes reddish brown.	
5									
								Total Depth: 8' No groundwater encountered. Boring Backfilled 8/31/16	
10									
15									
20									
25									

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PROJECT:	SKYLINE RETIREMENT CENTER	DRILLER:	BAJA EXPLORATION	SHEET:	1 of 1
CTE JOB NO:	10-13295G	DRILL METHOD:	HOLLOW-STEM AUGER	DRILLING DATE:	8/31/2016
LOGGED BY:	MM	SAMPLE METHOD:	RING, SPT and BULK	ELEVATION:	~ 487'

BORING: P-2

Depth (Feet)	Bulk Sample	Driven Type	Blows/6"	Dry Density (pcf)	Moisture (%)	U.S.C.S. Symbol	Graphic Log	DESCRIPTION	Laboratory Tests
0						SC		**QUATERNARY ALLUVIUM AND COLLUVIUM, undivided**: Medium dense to dense, slightly moist, medium brown, clayey fine to medium SAND, trace gravel.	
5								Becomes reddish brown.	
								Becomes dark brown.	
								Total Depth: 8' No groundwater encountered Boring Backfilled 8/31/16	
10									
15									
20									
25									

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PROJECT:	SKYLINE RETIREMENT CENTER	DRILLER:	BAJA EXPLORATION	SHEET:	1 of 1
CTE JOB NO:	10-13295G	DRILL METHOD:	HOLLOW-STEM AUGER	DRILLING DATE:	8/31/2016
LOGGED BY:	MM	SAMPLE METHOD:	RING, SPT and BULK	ELEVATION:	~ 494

BORING: P-3

Depth (Feet)	Bulk Sample	Driven Type	Blows/6"	Dry Density (pcf)	Moisture (%)	U.S.C.S. Symbol	Graphic Log	DESCRIPTION	Laboratory Tests
0						SC		**QUATERNARY ALLUVIUM AND COLLUVIUM, undivided**: Medium dense to dense, slightly moist, medium brown, clayey fine to medium SAND, abundant gravel. Becomes reddish brown.	
								Total Depth: 3.0' No groundwater encountered Boring Backfilled 8/31/16	
5									
10									
15									
20									
25									

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PROJECT:	SKYLINE RETIREMENT CENTER	DRILLER:	BAJA EXPLORATION	SHEET:	1 of 1
CTE JOB NO:	10-13295G	DRILL METHOD:	HOLLOW-STEM AUGER	DRILLING DATE:	8/31/2016
LOGGED BY:	MM	SAMPLE METHOD:	RING, SPT and BULK	ELEVATION:	~ 497

BORING: P-4

Depth (Feet)	Bulk Sample	Driven Type	Blows/6"	Dry Density (pcf)	Moisture (%)	U.S.C.S. Symbol	Graphic Log	DESCRIPTION	Laboratory Tests
0						SC		**QUATERNARY ALLUVIUM AND COLLUVIUM, undivided**: Medium dense to dense, slightly moist, medium brown, clayey fine to medium SAND, abundant gravel. Becomes reddish brown.	
								Total Depth: 3.0' No groundwater encountered Boring Backfilled 8/31/16	
5									
10									
15									
20									
25									

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PROJECT:	SKYLINE RETIREMENT CENTER	DRILLER:	BAJA EXPLORATION	SHEET:	1 of 1
CTE JOB NO:	10-13295G	DRILL METHOD:	HOLLOW-STEM AUGER	DRILLING DATE:	8/31/2016
LOGGED BY:	MM	SAMPLE METHOD:	RING, SPT and BULK	ELEVATION:	~ 525'

BORING: P-5

Depth (Feet)	Bulk Sample	Driven Type	Blows/6"	Dry Density (pcf)	Moisture (%)	U.S.C.S. Symbol	Graphic Log	DESCRIPTION	Laboratory Tests
0						SC		**QUATERNARY ALLUVIUM AND COLLUVIUM, undivided**: Medium dense, slightly moist, medium brown, clayey fine to medium SAND, trace gravel.	
								Becomes orangeish brown.	
5								Becomes reddish brown.	
								Becomes olive gray, silty.	
						SM		**CRETACEOUS SANTIAGO PEAK VOLCANICS**: Very dense, slightly moist, olive gray to gray, silty fine to coarse SAND.	
10								Total Depth: 9' No groundwater encountered Boring Backfilled 8/31/16	
15									
20									
25									

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PROJECT:	SKYLINE RETIREMENT CENTER	DRILLER:	BAJA EXPLORATION	SHEET:	1 of 1
CTE JOB NO:	10-13295G	DRILL METHOD:	HOLLOW-STEM AUGER	DRILLING DATE:	8/31/2016
LOGGED BY:	MM	SAMPLE METHOD:	RING, SPT and BULK	ELEVATION:	~ 525'

BORING: P-6

Depth (Feet)	Bulk Sample	Driven Type	Blows/6"	Dry Density (pcf)	Moisture (%)	U.S.C.S. Symbol	Graphic Log	DESCRIPTION	Laboratory Tests
0						SC		**<u>QUATERNARY ALLUVIUM AND COLLUVIUM, undivided</u>**: Medium dense, slightly moist, medium brown, clayey fine to medium SAND, trace angular gravel.	
								Becomes orangeish brown.	
5								Becomes reddish brown.	
						SM		**<u>CRETACEOUS SANTIAGO PEAK VOLCANICS</u>**: Very dense, slightly moist, olive gray to gray, silty fine to coarse SAND.	
10								Total Depth: 9' No groundwater encountered Boring Backfilled 8/31/16	
15									
20									
25									

APPENDIX C

LABORATORY METHODS AND RESULTS

APPENDIX C
LABORATORY METHODS AND RESULTS

Laboratory tests were performed on selected soil samples to evaluate their engineering properties. Tests were performed following test methods of the American Society for Testing and Materials, or other accepted standards. The following presents a brief description of the various test methods used. Laboratory results are presented in the following section of this Appendix.

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 D 2487.

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

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.

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.

Chemical Analysis
Soil materials were collected and tested for Sulfate and Chloride content, pH, Corrosivity, and Resistivity.

Direct Shear
Direct shear tests were performed on either samples direct from the field or on samples recompacted to a specific density. Direct shear testing was performed in accordance with ASTM D 3080. The samples were inundated during shearing to represent adverse field conditions.

In-Place Moisture/Density

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

EXPANSION INDEX TEST

ASTM D 4829

LOCATION	DEPTH (feet)	EXPANSION INDEX	EXPANSION POTENTIAL
B-7	0-5	13	VERY LOW

IN-PLACE MOISTURE AND DENSITY

LOCATION	DEPTH (feet)	% MOISTURE	DRY DENSITY
B-3	3	3.7	111.8
B-8	5	5.5	118.4

RESISTANCE "R"-VALUE

CALTEST 301

LOCATION	DEPTH (feet)	R-VALUE
B-1	0-5	5

SULFATE

CALIFORNIA TEST 417

LOCATION	DEPTH (feet)	RESULTS ppm
B-7	0-5	54.1

CHLORIDE

CALIFORNIA TEST 422

LOCATION	DEPTH (feet)	RESULTS ppm
B-7	0-5	20.3

p.H.

CALIFORNIA TEST 643

LOCATION	DEPTH (feet)	RESULTS
B-7	0-5	7.49

RESISTIVITY

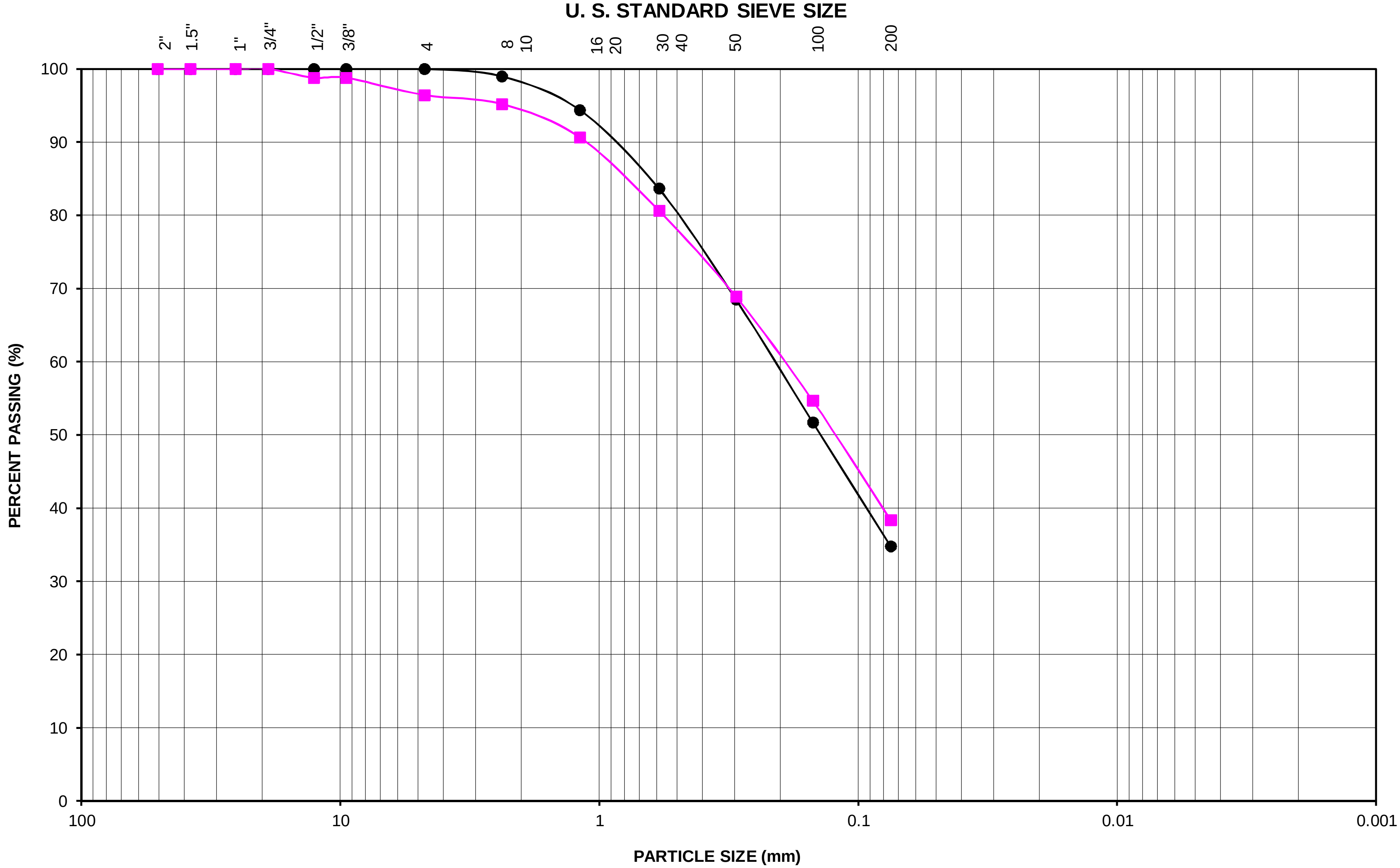
CALIFORNIA TEST 643

LOCATION	DEPTH (feet)	RESULTS ohms-cm
B-7	0-5	8340

MODIFIED PROCTOR

ASTM D 1557

LOCATION	DEPTH (feet)	MAXIUM DRY DENSITY (PCF)	OPTIMUM MOISTURE (%)
B-1	0-1	126.2	11.0
B-7	0-5	133.1	9.1



PARTICLE SIZE ANALYSIS

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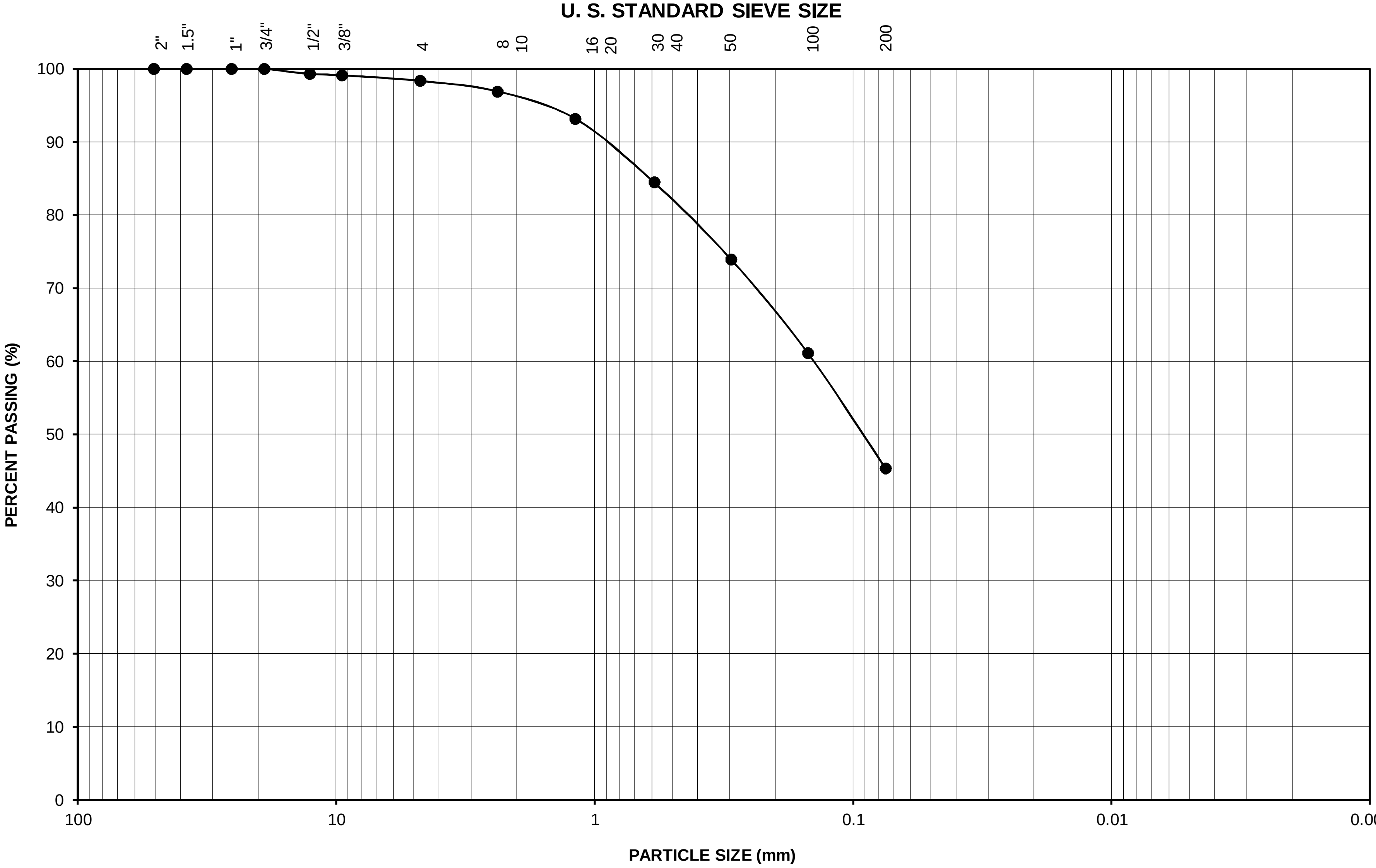
Sample Designation	Sample Depth (feet)	Symbol	Liquid Limit (%)	Plasticity Index	Classification
B-3	3	●	-	-	SC
B-3	15	■	-	-	SC
CTE JOB NUMBER:		10-13295G		FIGURE:	C-1



PARTICLE SIZE ANALYSIS

Construction Testing & Engineering, Inc.
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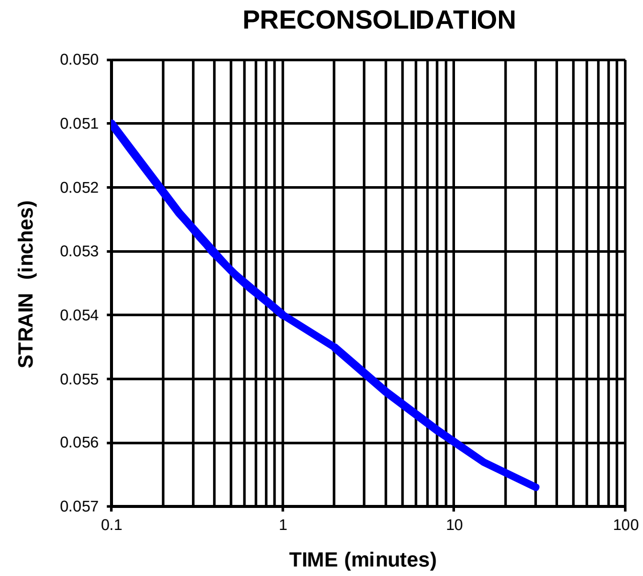
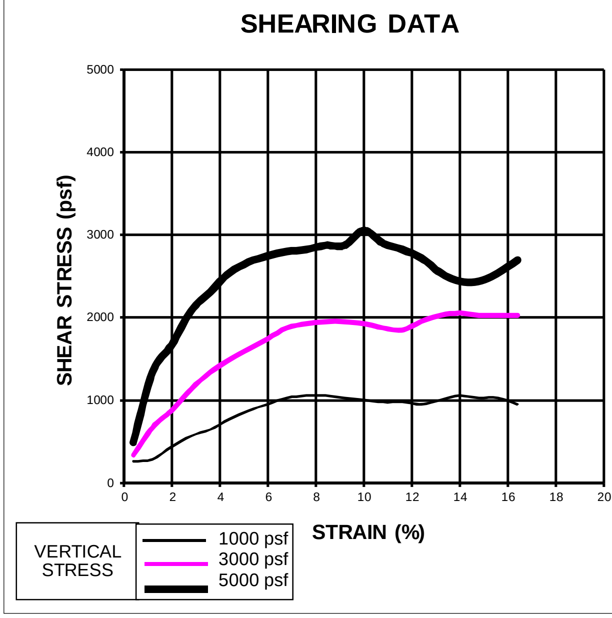
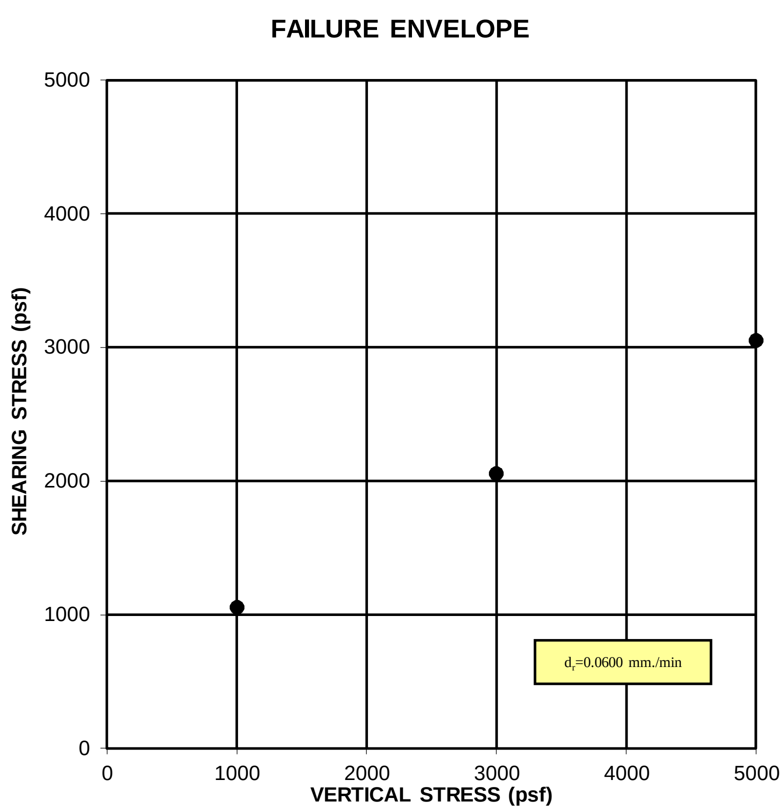
Sample Designation	Sample Depth (feet)	Symbol	Liquid Limit (%)	Plasticity Index	Classification
B-4	13	●	-	-	SC
			-	-	

CTE JOB NUMBER:	10-13295G	FIGURE:	C-2







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SHEAR STRENGTH TEST - ASTM D3080

Job Name: Skyline Retirement Center
Initial Dry Density (pcf): 133.1

Project Number: 10-13295G
Sample Date: 8/31/2016
Initial Moisture (%): 9.1

Lab Number: 26621
Test Date: 9/27/2016
Final Moisture (%): 17.4

Sample Location: B-7 @ 10'
Tested by: Julian Carmona
Cohesion: 550 psf

Sample Description: Reddish Brown SC (Remolded @ 90%)
Angle Of Friction: 26.6

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

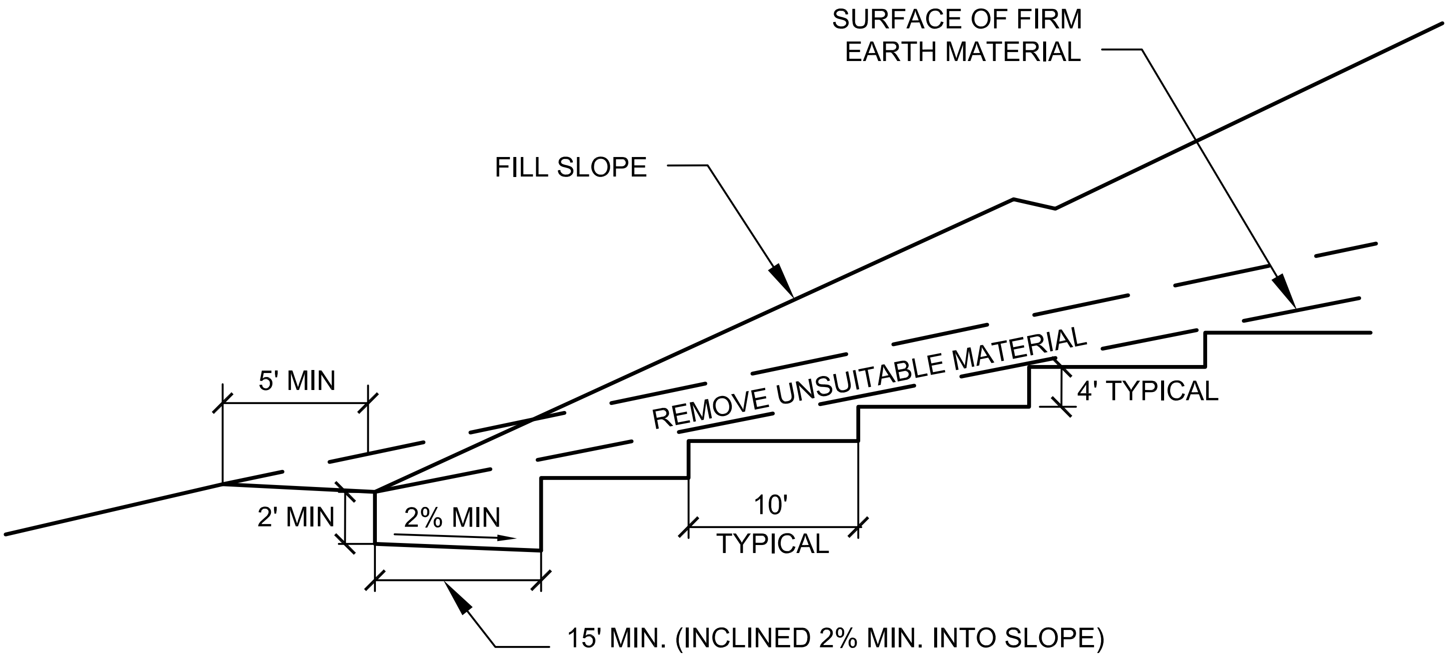
As a precautionary measure, plastic sheeting should be readily available, or kept on hand, to protect all slope areas from saturation by periods of heavy or prolonged rainfall. This measure is strongly recommended, beginning with the period prior to landscape planting.

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.

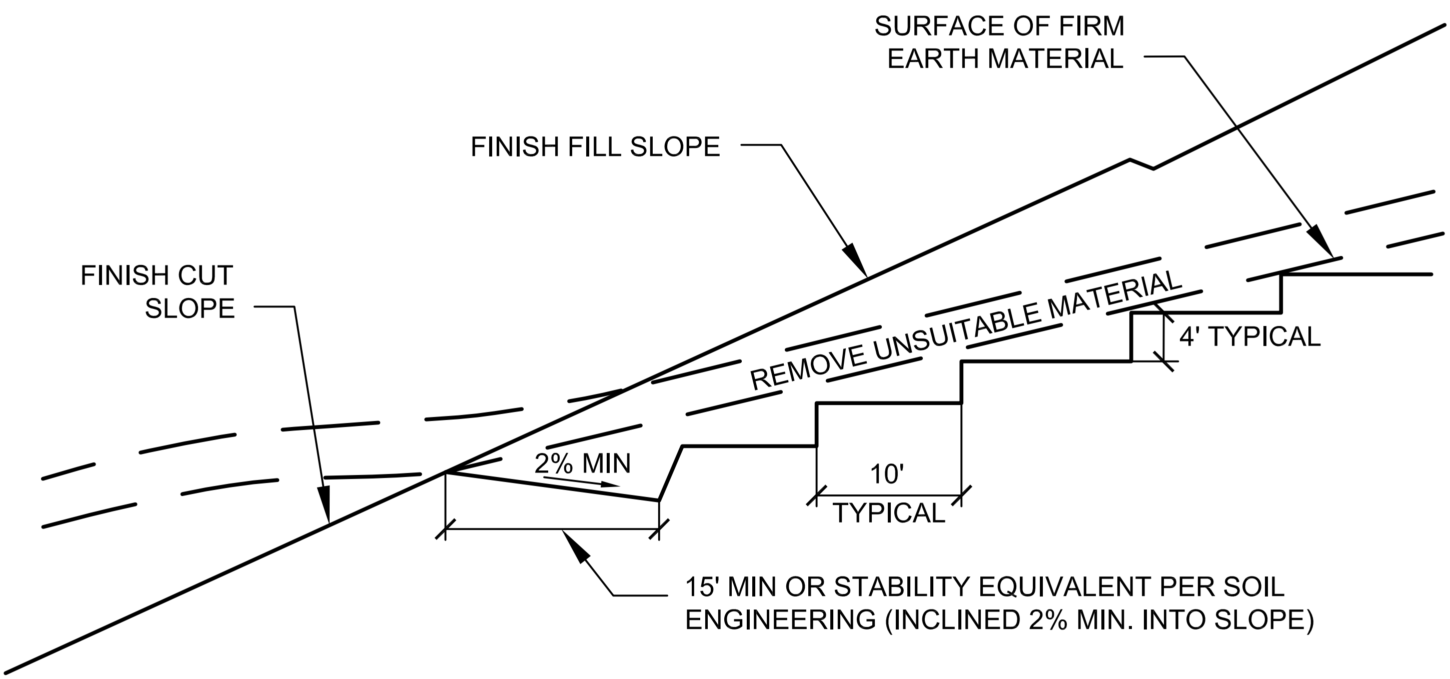
If slope failures occur as a result of exposure to period of heavy rainfall, the failure areas and currently unaffected areas should be covered with plastic sheeting to protect against additional saturation.

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

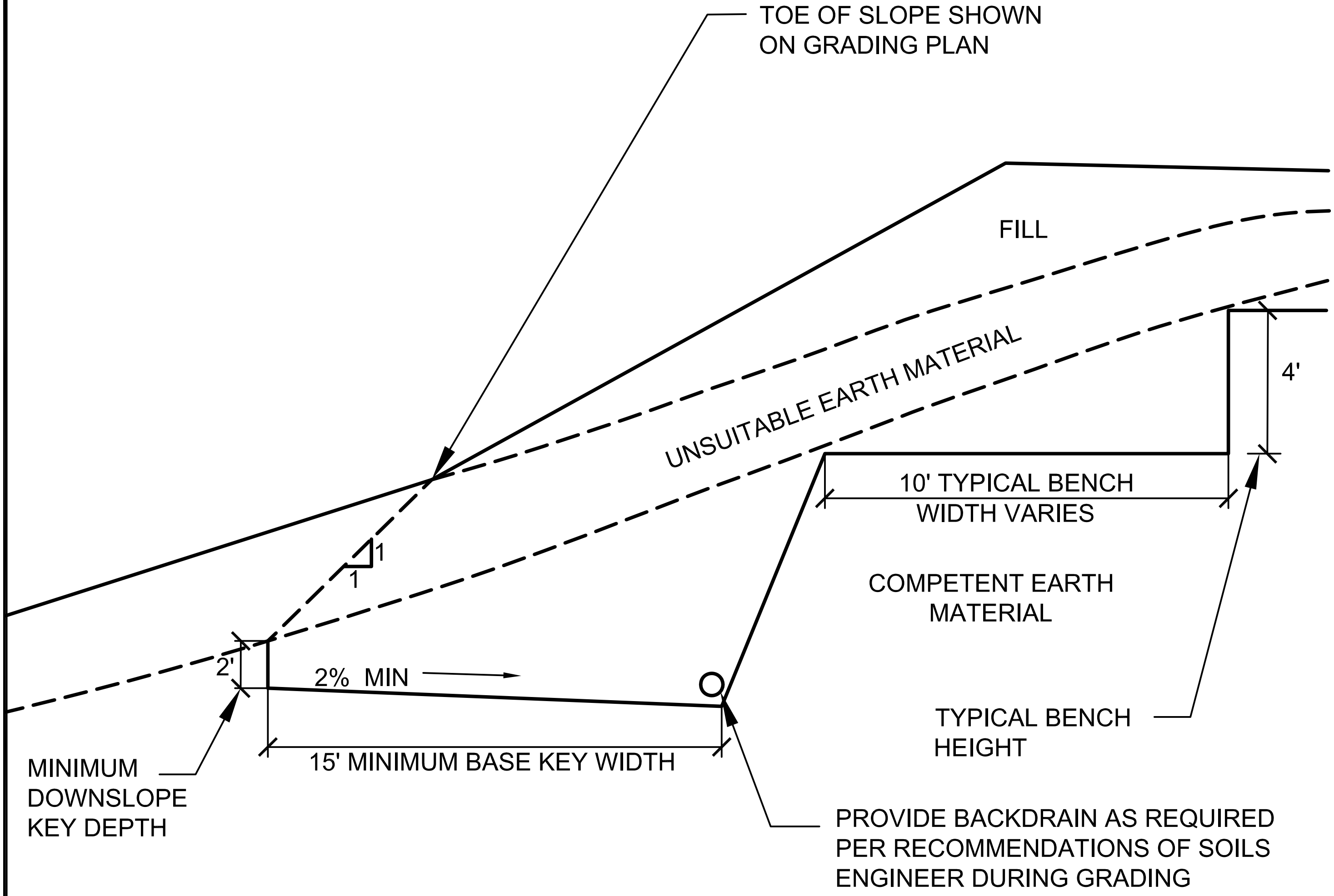


BENCHING FILL OVER CUT



NOT TO SCALE

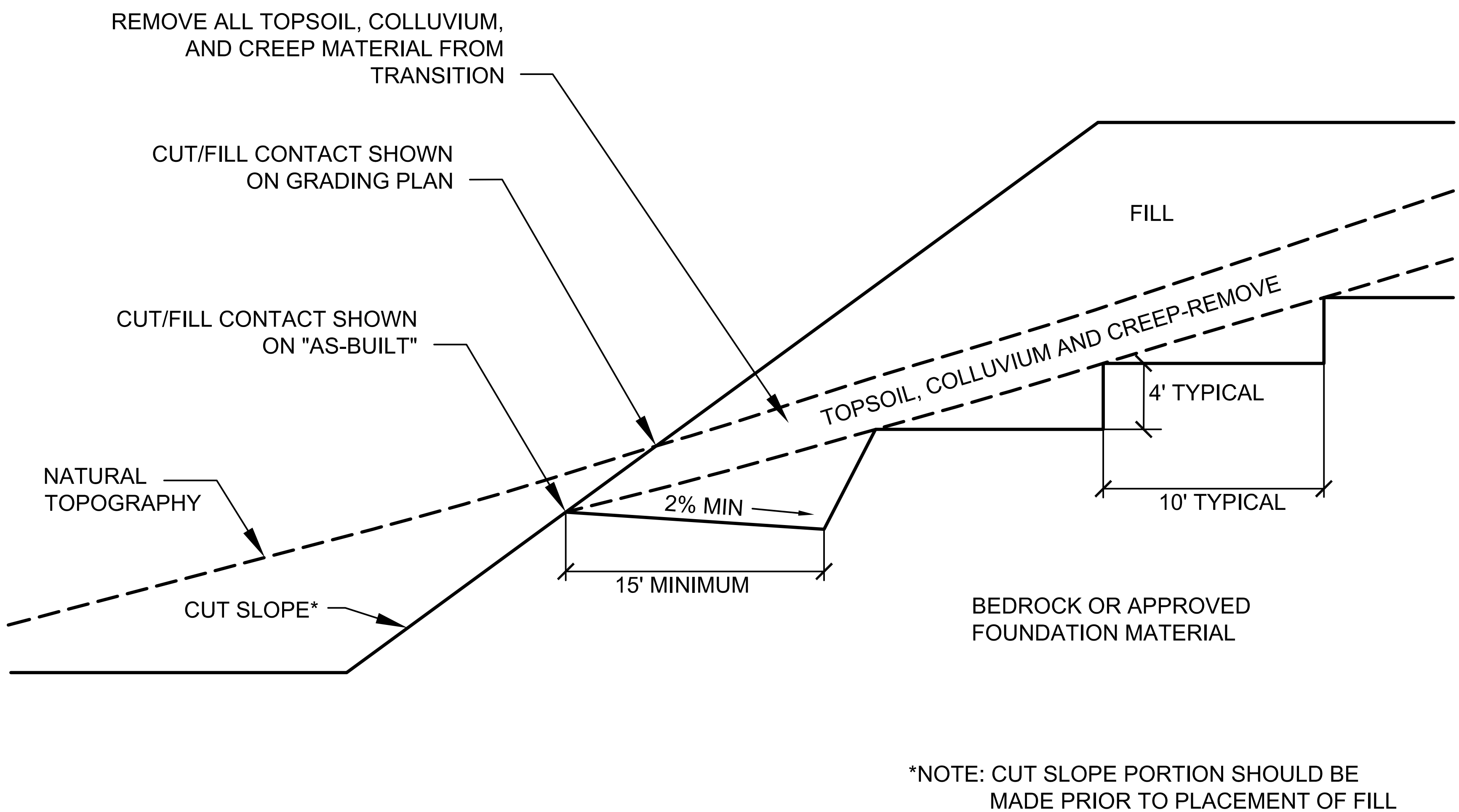
BENCHING FOR COMPACTED FILL DETAIL



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

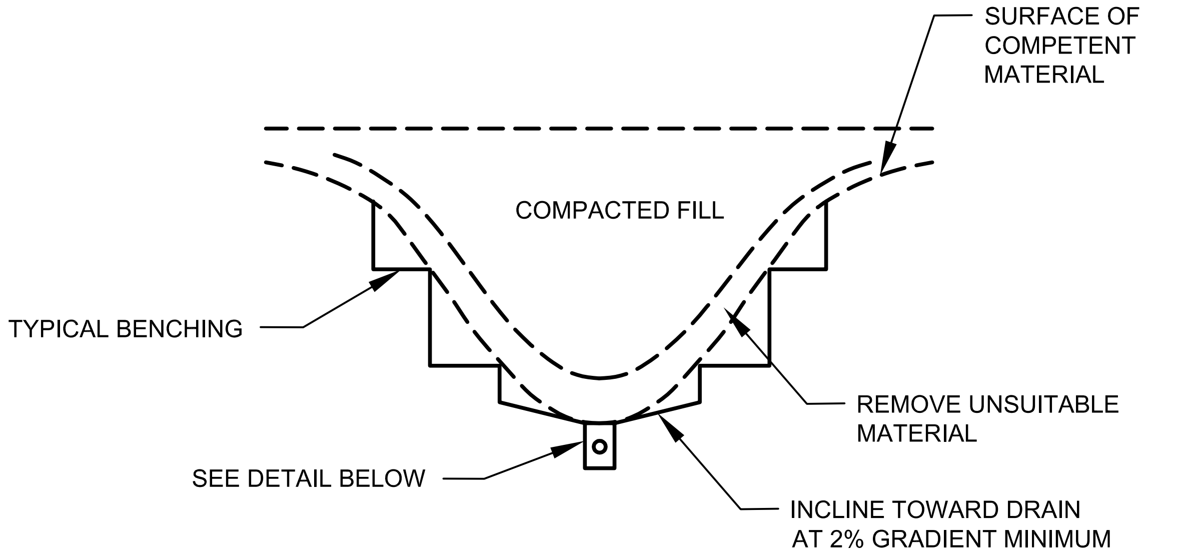
NOT TO SCALE

FILL SLOPE ABOVE NATURAL GROUND DETAIL

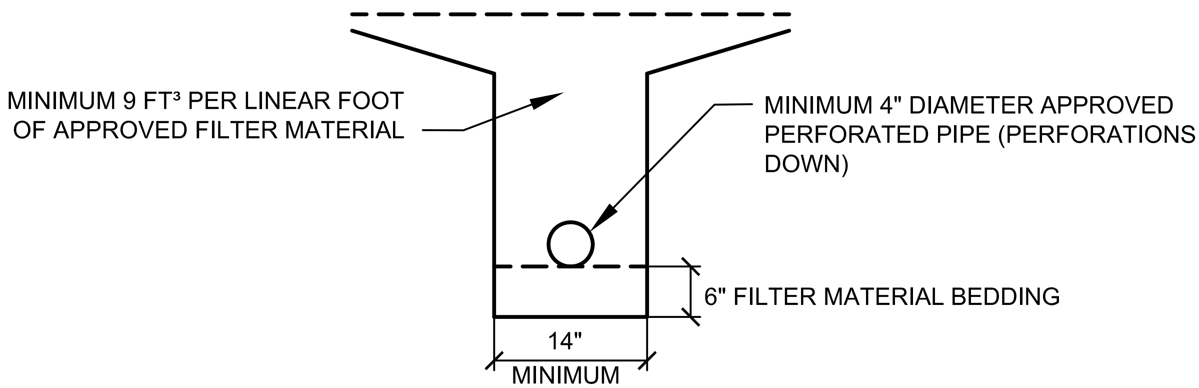


NOT TO SCALE

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

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

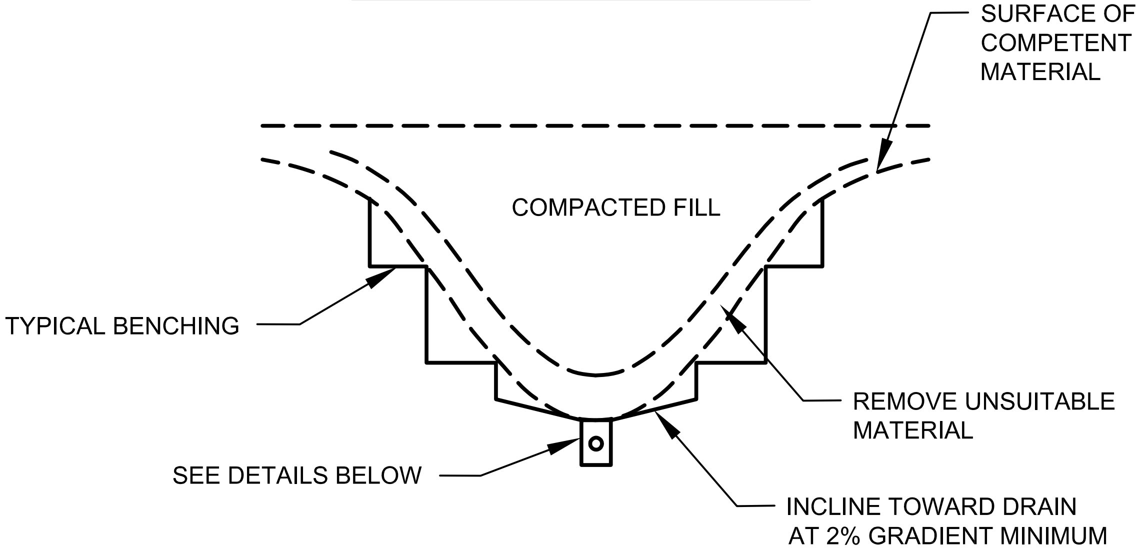
PIPE DIAMETER TO MEET THE FOLLOWING CRITERIA, SUBJECT TO FIELD REVIEW BASED ON ACTUAL GEOTECHNICAL CONDITIONS ENCOUNTERED DURING GRADING

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

NOT TO SCALE

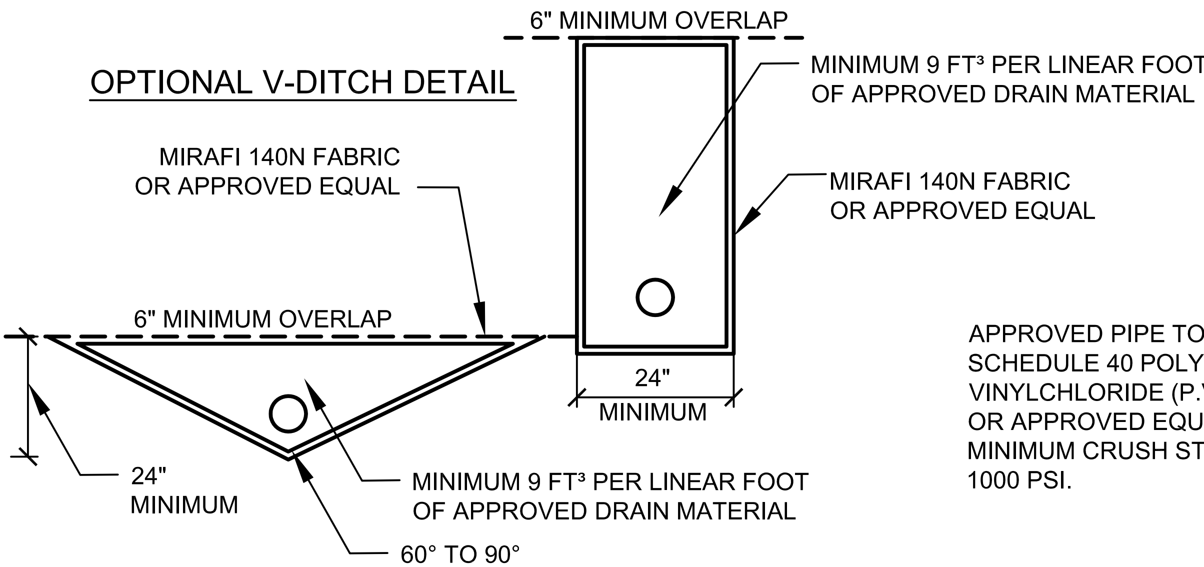
TYPICAL CANYON SUBDRAIN DETAIL

CANYON SUBDRAIN DETAILS



TRENCH DETAILS

OPTIONAL V-DITCH DETAIL



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

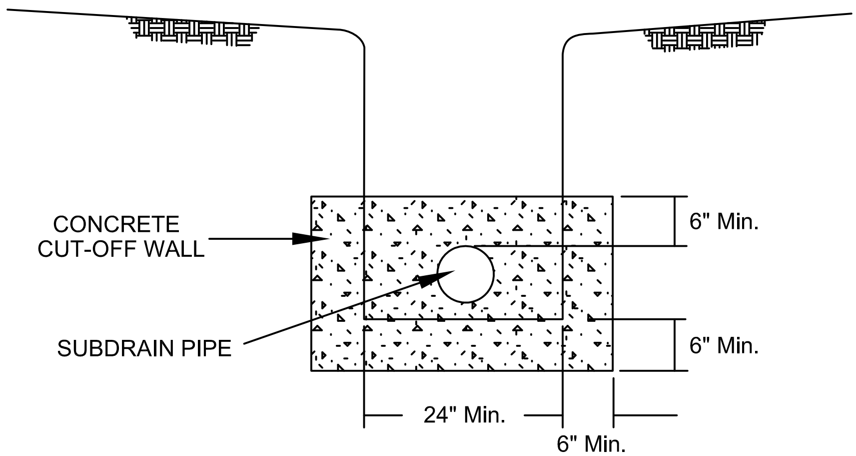
PIPE DIAMETER TO MEET THE FOLLOWING CRITERIA, SUBJECT TO FIELD REVIEW BASED ON ACTUAL GEOTECHNICAL CONDITIONS ENCOUNTERED DURING GRADING

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

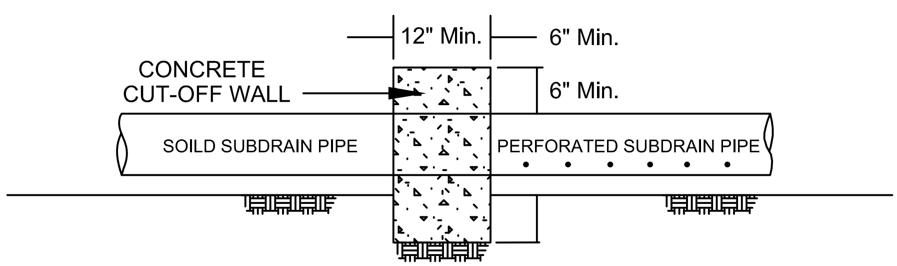
NOT TO SCALE

GEOFABRIC SUBDRAIN

FRONT VIEW



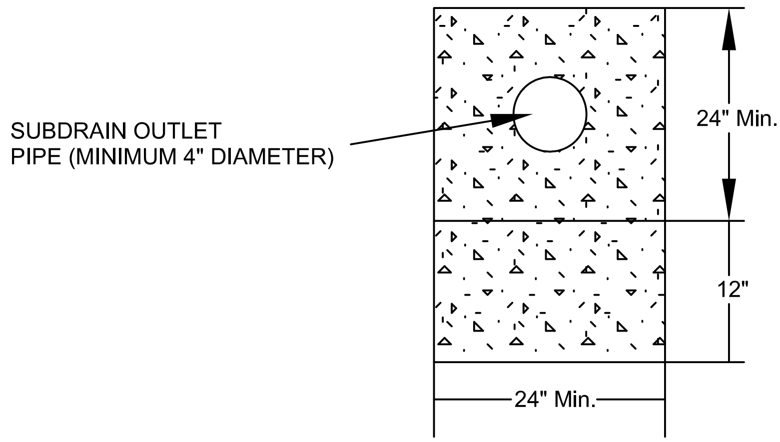
SIDE VIEW



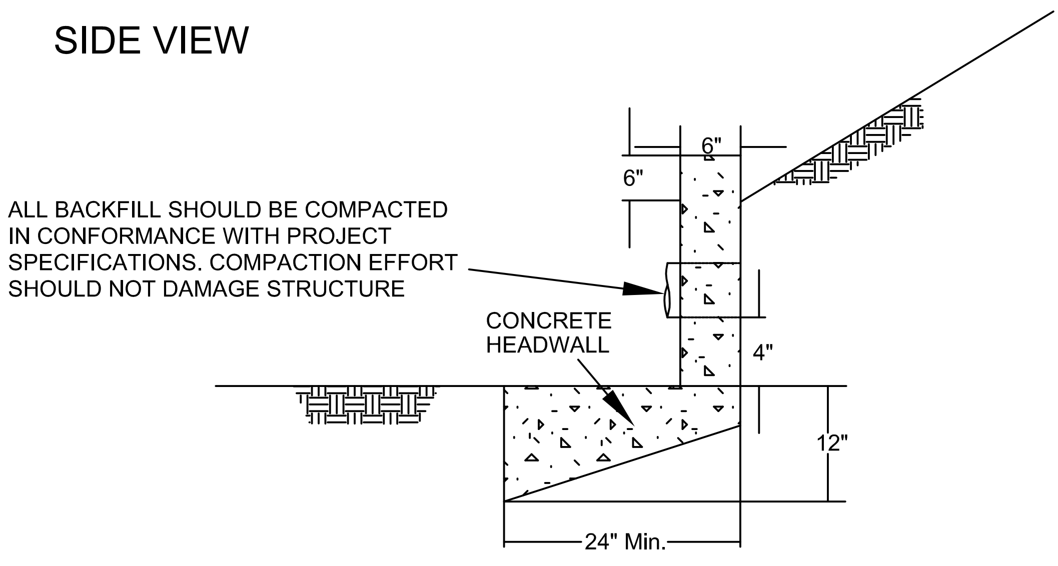
NOT TO SCALE

RECOMMENDED SUBDRAIN CUT-OFF WALL

FRONT VIEW



SIDE VIEW



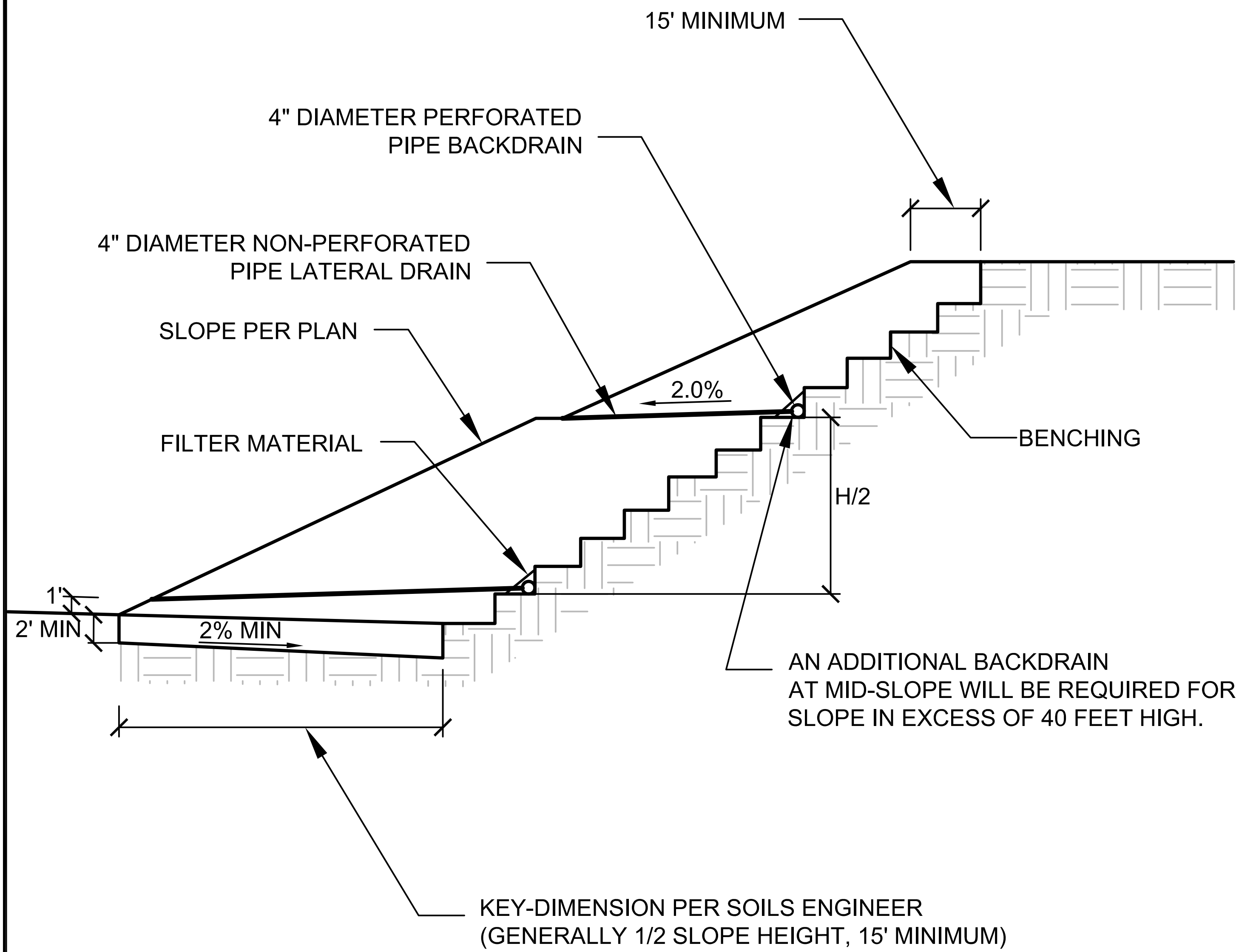
NOTE: HEADWALL SHOULD OUTLET AT TOE OF SLOPE OR INTO CONTROLLED SURFACE DRAINAGE DEVICE

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

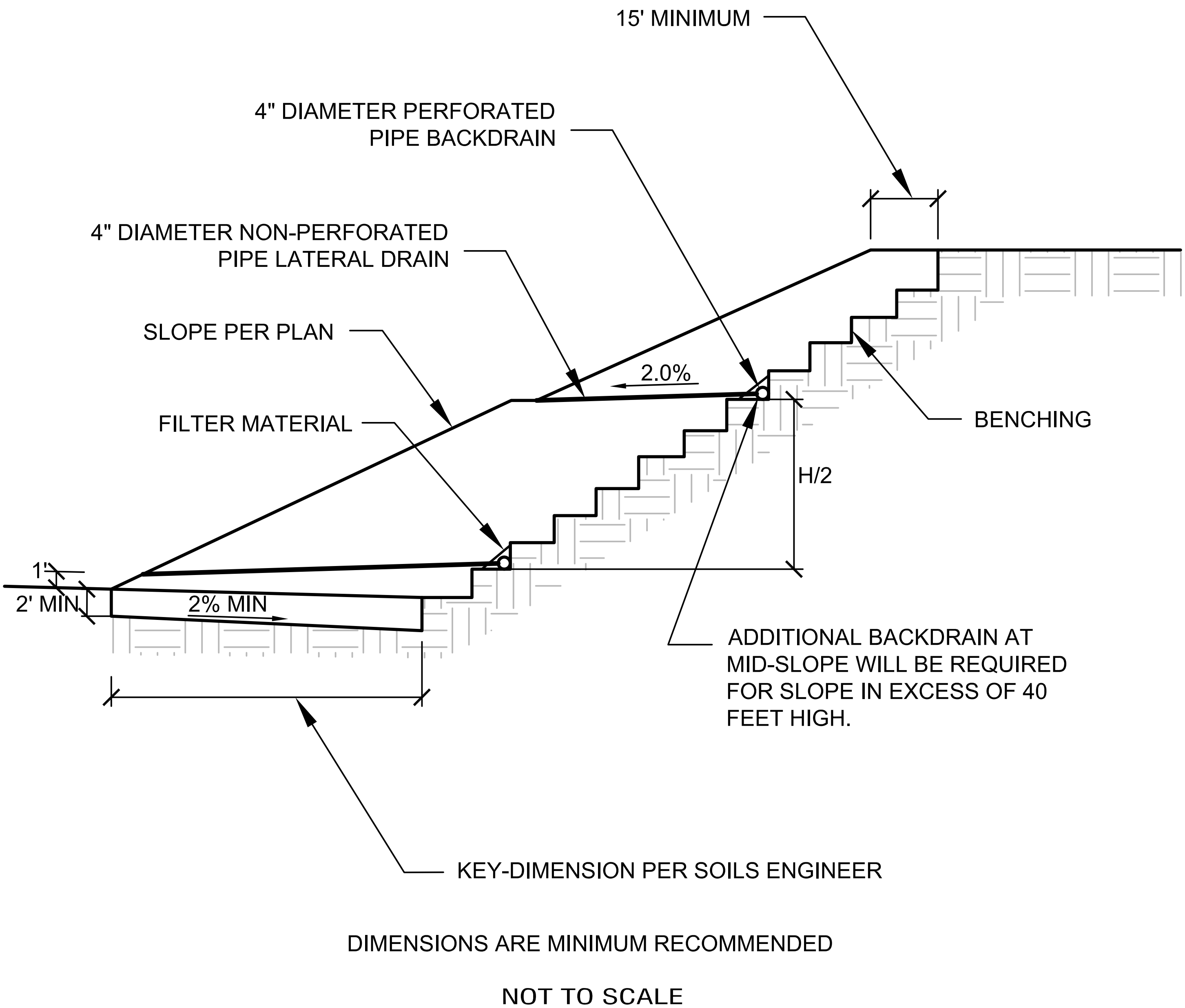
TYPICAL SUBDRAIN OUTLET HEADWALL DETAIL



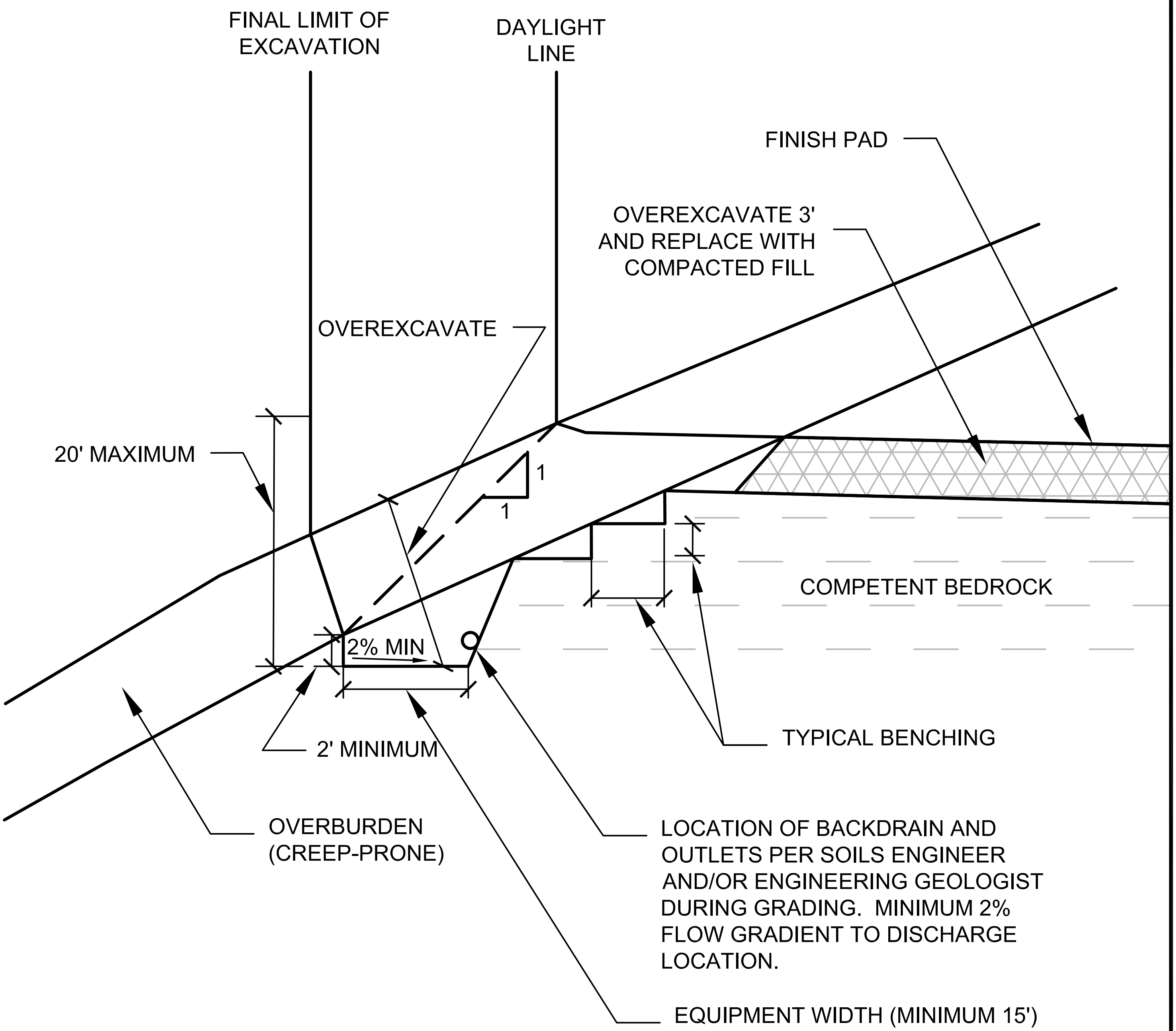
DIMENSIONS ARE MINIMUM RECOMMENDED

NOT TO SCALE

TYPICAL SLOPE STABILIZATION FILL DETAIL

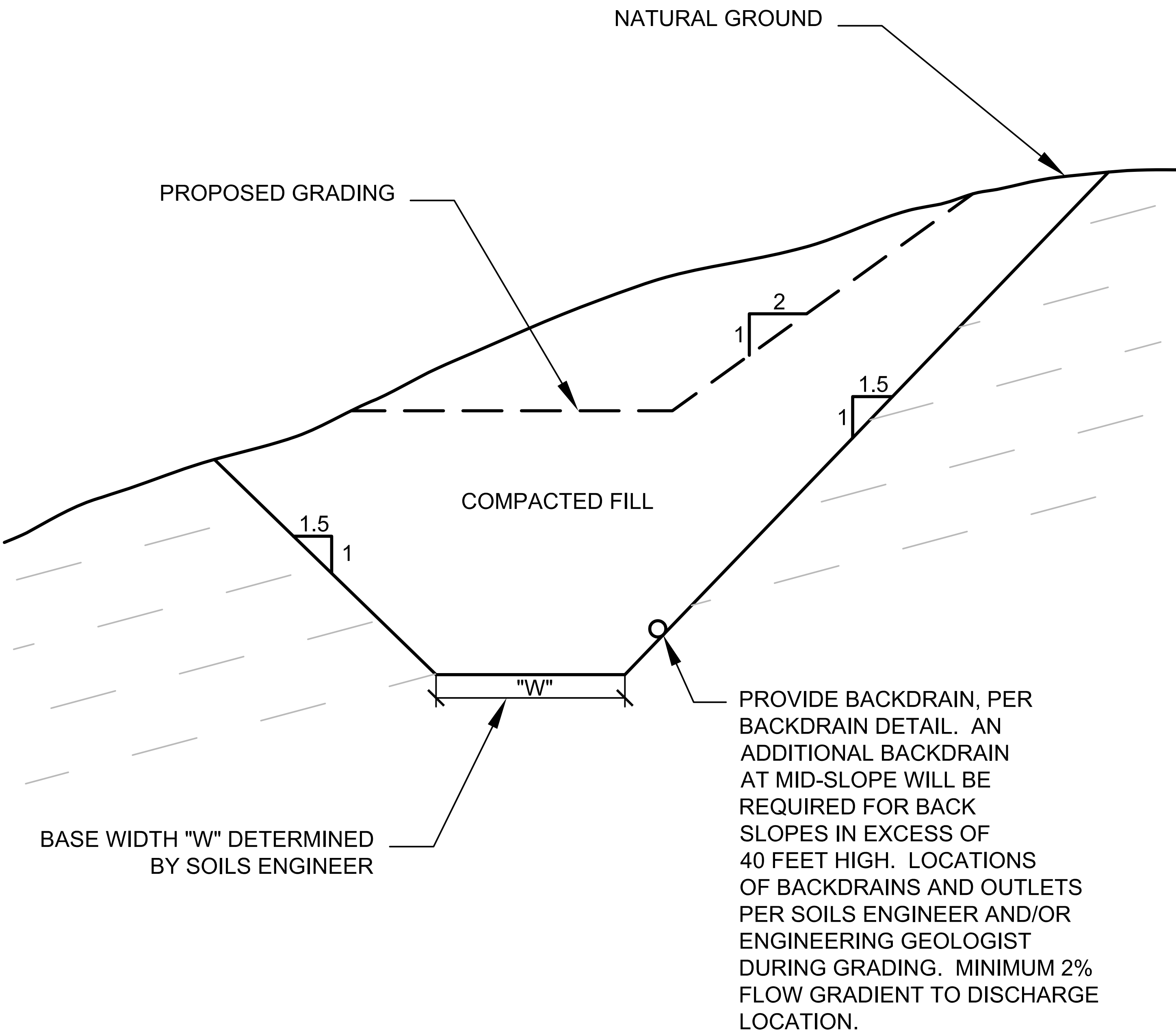


TYPICAL BUTTRESS FILL DETAIL



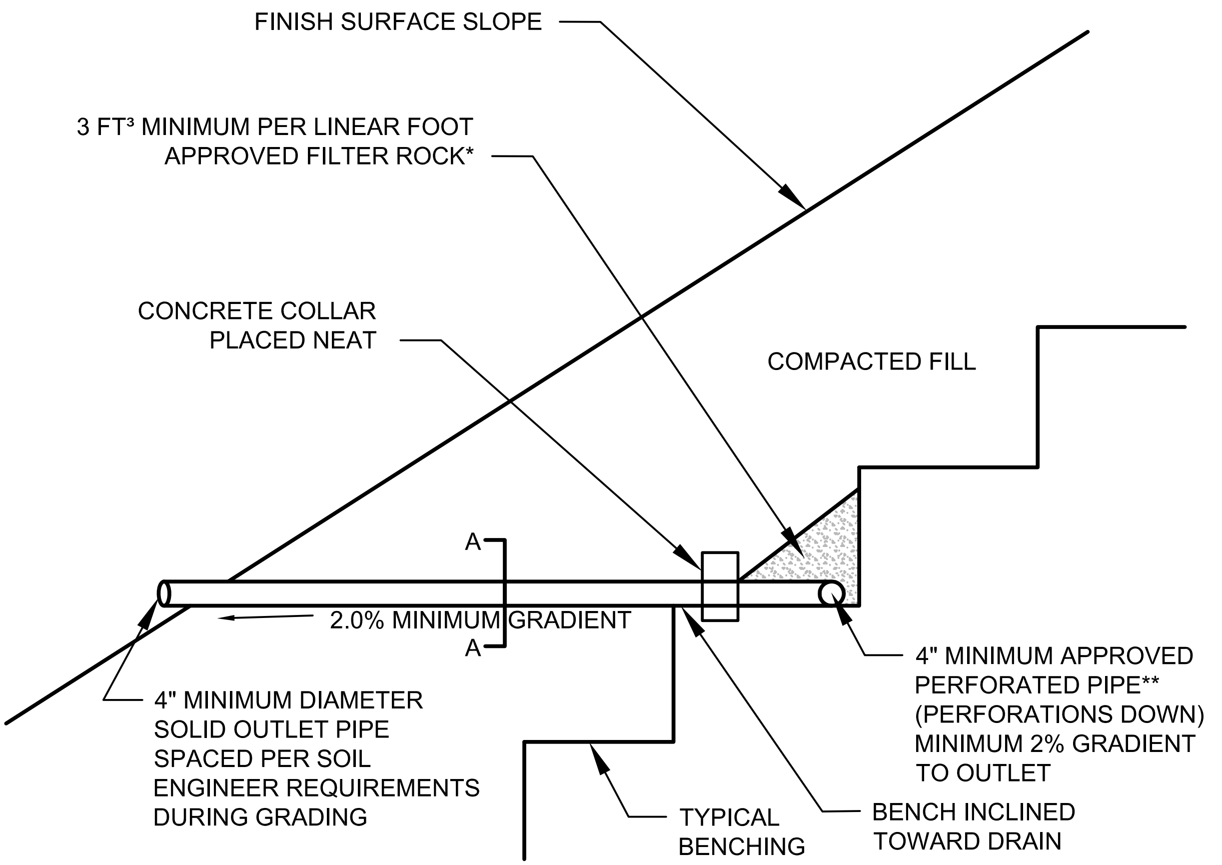
NOT TO SCALE

DAYLIGHT SHEAR KEY DETAIL

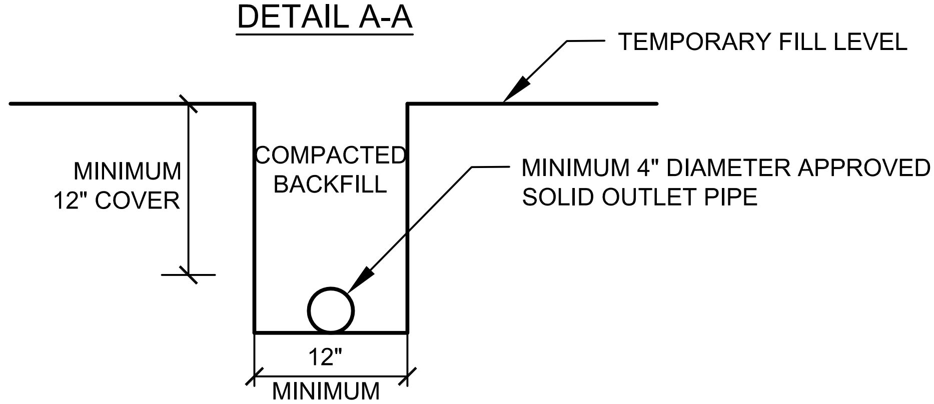


NOT TO SCALE

TYPICAL SHEAR KEY DETAIL



DETAIL A-A



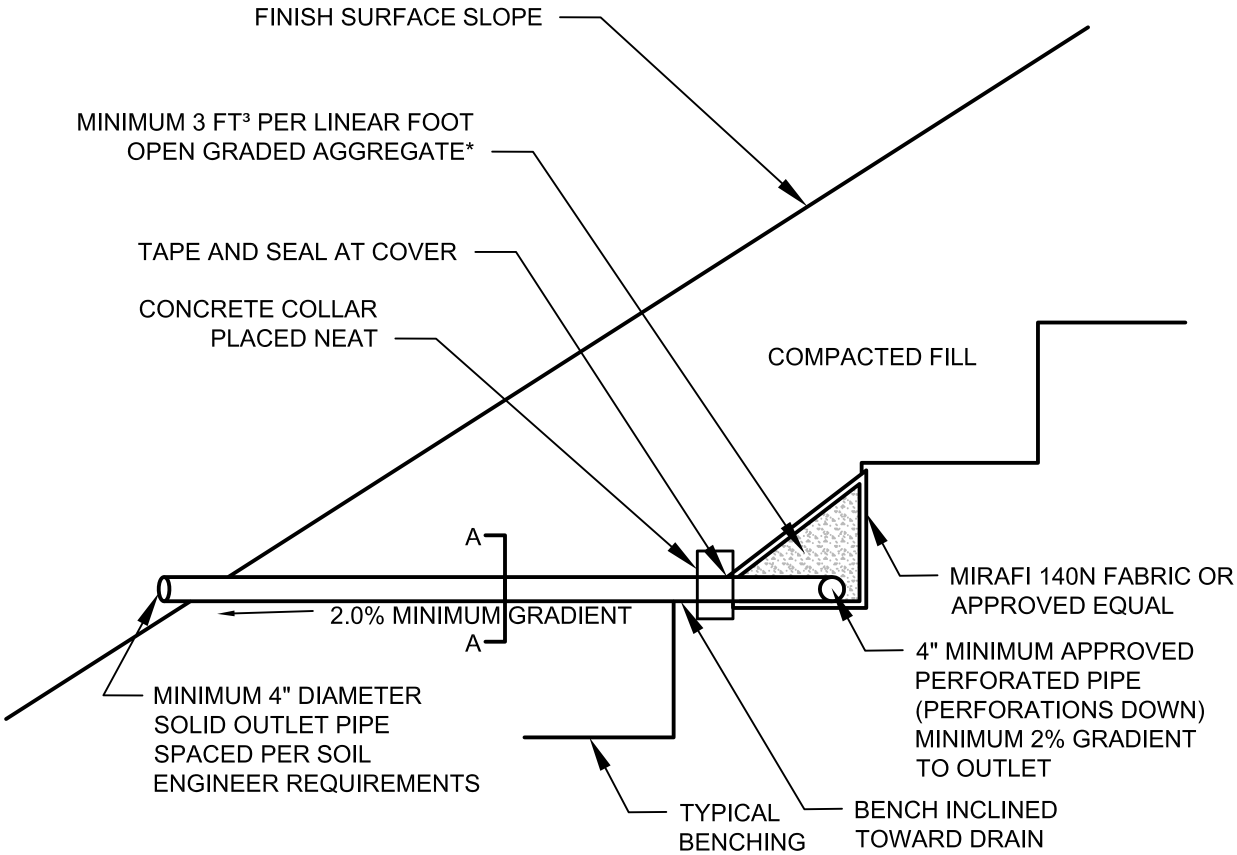
**APPROVED PIPE TYPE:
SCHEDULE 40 POLYVINYL CHLORIDE (P.V.C.) OR APPROVED EQUAL.
MINIMUM CRUSH STRENGTH 1000 PSI

*FILTER ROCK TO MEET FOLLOWING SPECIFICATIONS OR APPROVED EQUAL:

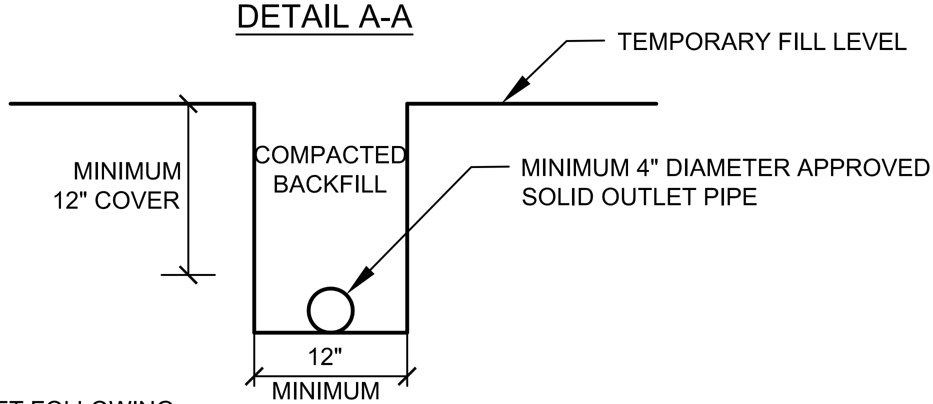
SIEVE SIZE	PERCENTAGE PASSING
1"	100
¾"	90-100
⅜"	40-100
NO. 4	25-40
NO. 30	5-15
NO. 50	0-7
NO. 200	0-3

NOT TO SCALE

TYPICAL BACKDRAIN DETAIL



DETAIL A-A

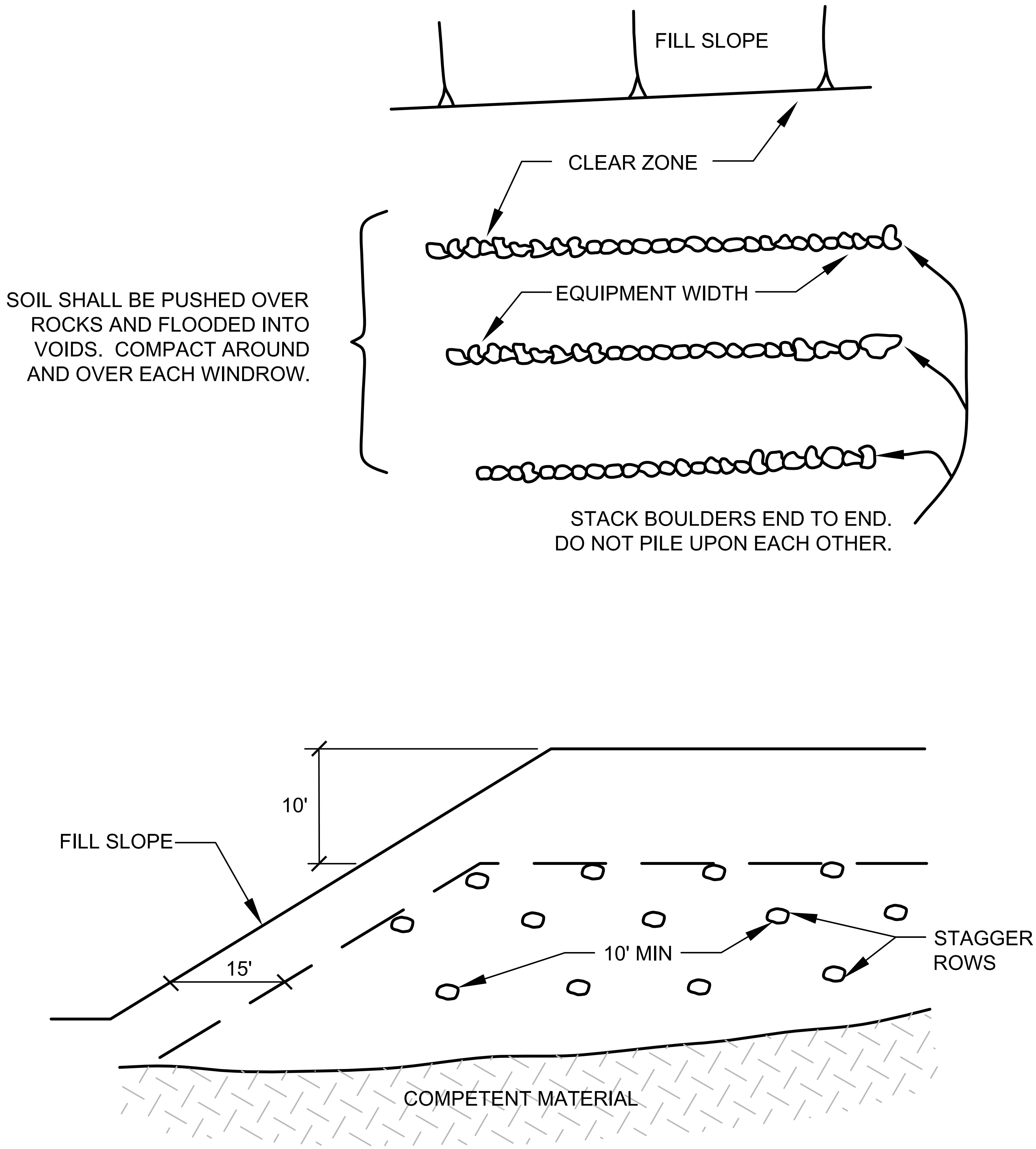


*NOTE: AGGREGATE TO MEET FOLLOWING SPECIFICATIONS OR APPROVED EQUAL:

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

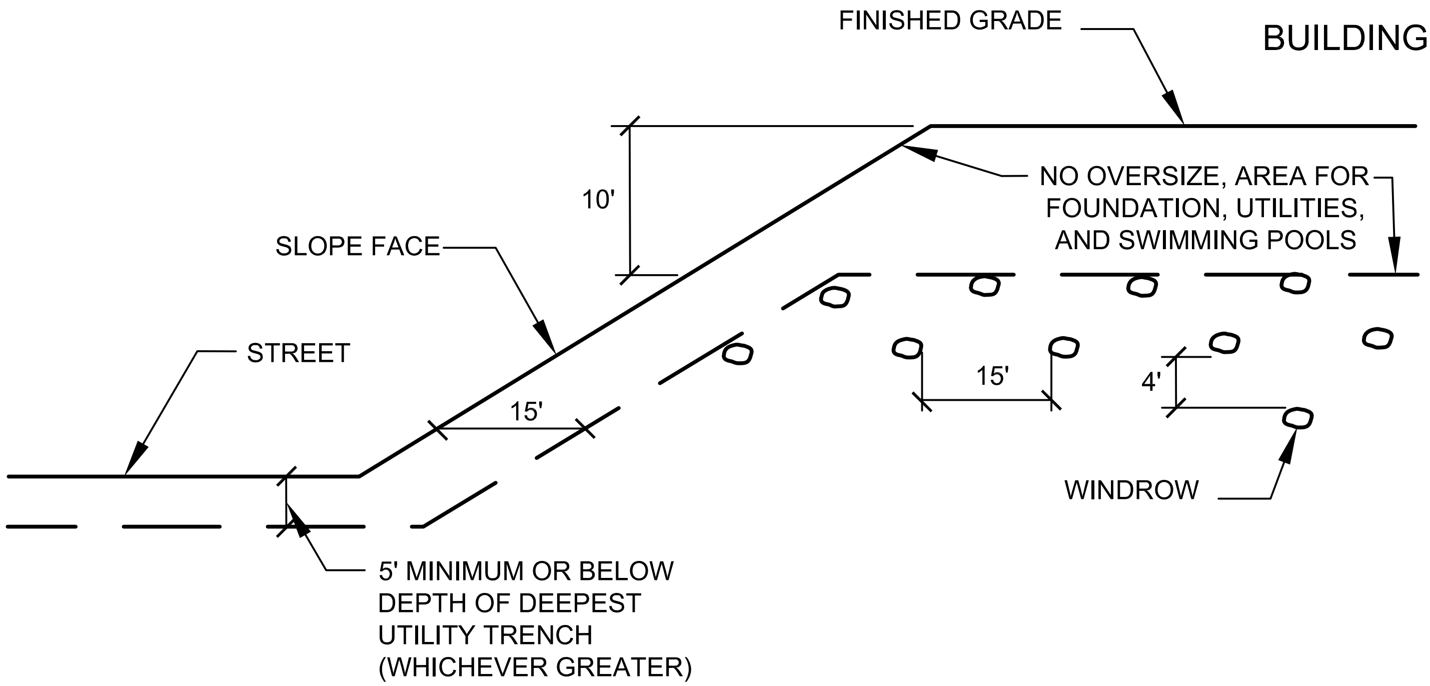
NOT TO SCALE

BACKDRAIN DETAIL (GEOFRABIC)

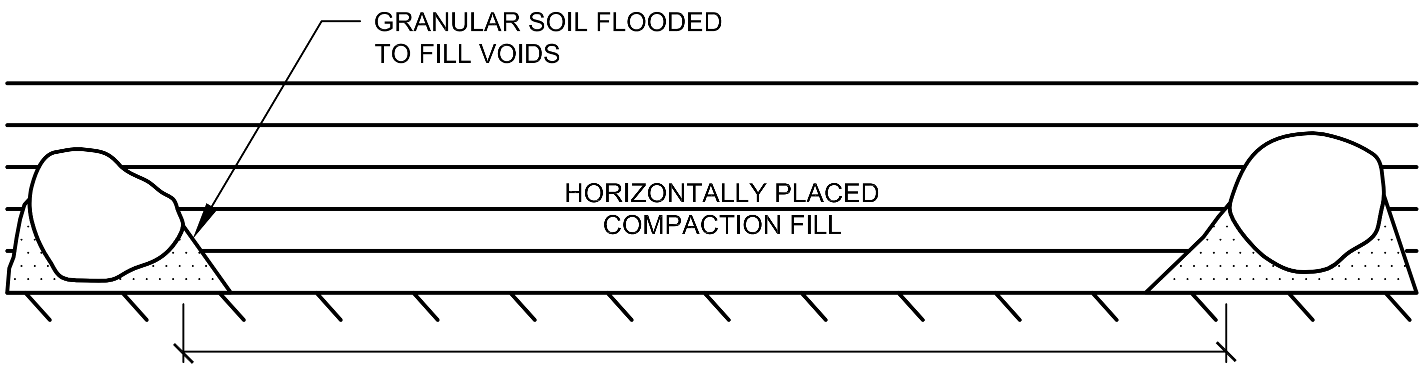


NOT TO SCALE

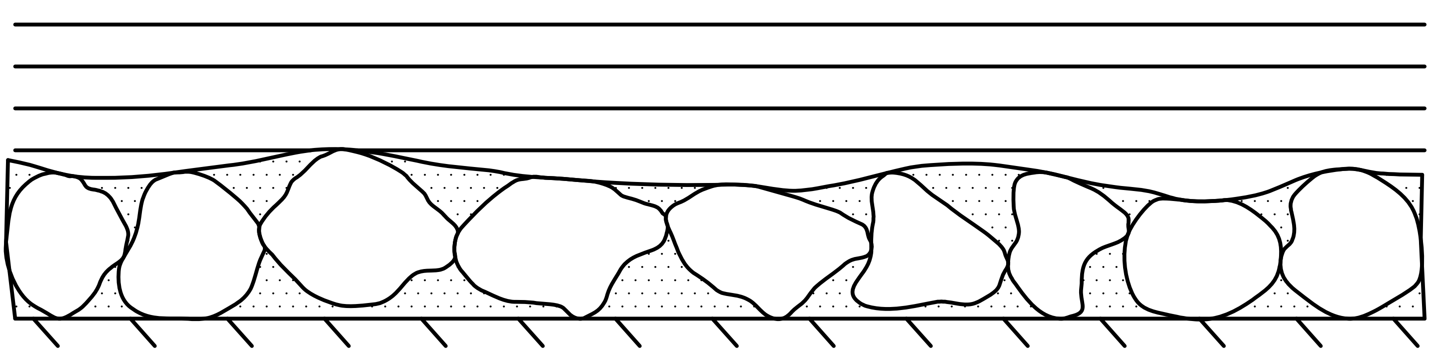
ROCK DISPOSAL DETAIL



TYPICAL WINDROW DETAIL (EDGE VIEW)



PROFILE VIEW

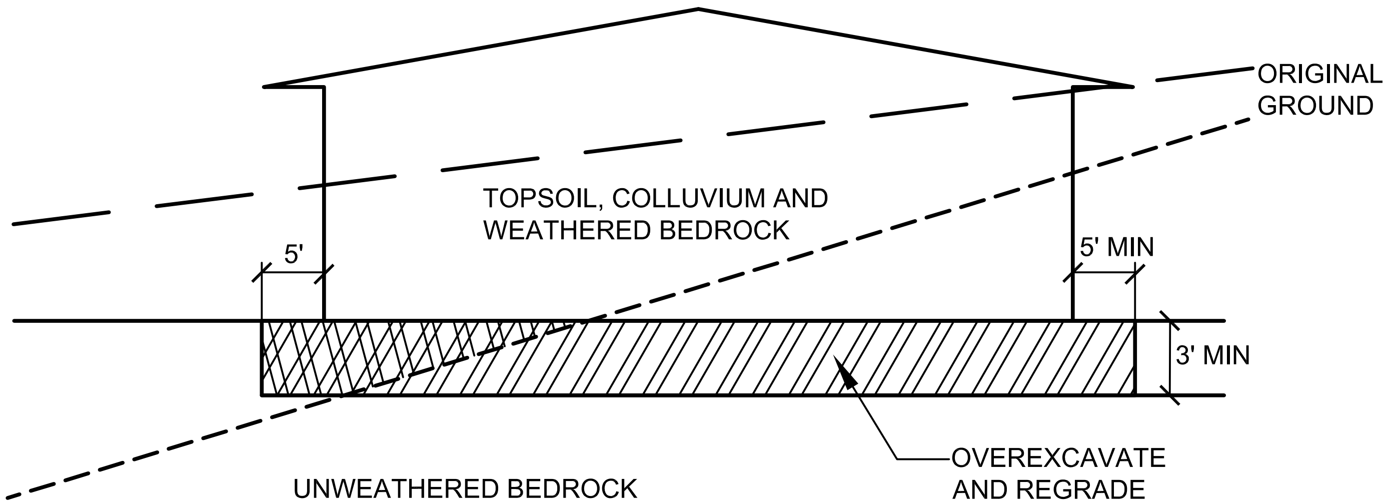


NOT TO SCALE

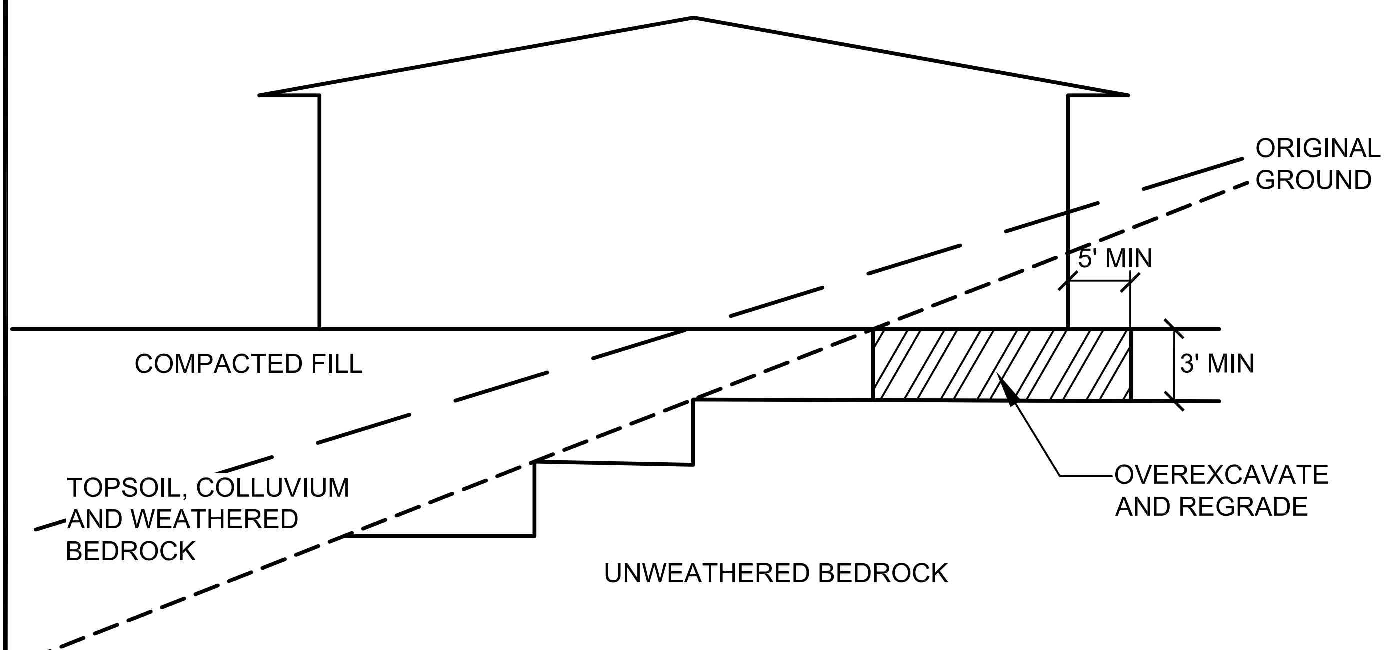
ROCK DISPOSAL DETAIL

GENERAL GRADING RECOMMENDATIONS

CUT LOT



CUT/FILL LOT (TRANSITION)



NOT TO SCALE

TRANSITION LOT DETAIL

<u>APPENDIX E</u>

PERCOLATION TESTING AND INFILTRATION RATES

APPENDIX E

PERCOLATION TEST RESULTS AND CALCULATED INFILTRATION RATES

This Appendix E provides calculated infiltration rates based upon the percolation test results placed within 50 feet of the proposed BMP basins. Appendix E presents a revised calculated infiltration rate comparative to that issued in the October 3, 2016 report. Current project plans call for three BMP basins. As such six percolation tests hole were placed and logged. Additionally, two soil borings were placed within 50 feet and extended to at least 10 feet below each basin bottom excavation for an evaluation of the potential presence of groundwater at those locations. This report generally follows the County of San Diego storm water infiltration design requirements of February 2016 Appendix C and Appendix D "Geotechnical and Groundwater Investigation Requirements: and Appendix D "Approved Infiltration Rate Assessment Methods for Selection of Storm Water BMPs.".Geotechnical information including soil boring locations and distribution of geologic units is shown on attached Figure 2, Geotechnical Map. Soil boring including logs of the percolation test holes are provided in Appendix B.

The proposed infiltration basin locations and associated depths are as provided by REC Consultants (REC). CTE understands the project is in the design phase of development, and a site specific evaluation of infiltration basin infiltration is necessary as per Appendix C of the County of San Diego "Geotechnical and Groundwater Investigation Requirements" dated February 2016. Associated Worksheets C.4-1 and D.5-1 are attached at the end of this Appendix E.

2.0 PERCOLATION TESTS AND CALCULATED INFILTRATION RATES

2.1 Percolation Tests

Six percolation tests were performed in general accordance with Appendix D of the County of San Diego Storm Water Design requirements Based on visual and tactile identification, the percolation tests adjacent to BMP Basin 1 (P-5 and P-6) were performed in weathered dense to very dense Cretaceous Santiago Peak Volcanic that excavated as dry to slightly moist, olive gray, silty fine to coarse sand. Percolation tests adjacent to BMP Basin 2 (P-1 and P-2) and BMP Basin 3(P-3 and P-4) were performed in medium dense to dense Quaternary Undivided Alluvium and Colluvium that excavated as dry to slightly moist, red brown, clayey fine to medium sand with gravel and cobbles. The measured percolation rates are shown on following Table 2.1.

	TABLE 2.1 PERCOLATION RATES						
Boring/Depth (inches) P-1/99	Time	Time Change (minutes)	Initial Water Level (inches)	Final Water Level (inches)	Water Level Change (inches)	Percolation Rate	
						Inches/ Hour	Inches/ Minute
	0900	Initial	78	Initial	Initial		
	0930	30	78	80.75	2.75		
	1000	60	77.875	79.5	1.625		
	1030	90	78	79.75	1.75		
	1100	120	78.125	79.5	1.375		
	1130	150	78.125	79.375	1.25		
	1200	180	77.75	78.875	1.125		
	1230	210	78	79.25	1.25		
	1300	240	77.875	79.125	1.25		
	1330	270	77.25	78.875	1.125		
	1400	300	77.875	79	1.125		
	1430	330	78.125	79.375	1.25		
	1500	360	78.125	79.25	1.125	2.2497	0.0375
Boring/Depth (inches) P-2/93.5	Time	Time Change (minutes)	Initial Water Level (inches)	Final Water Level (inches)	Water Level Change (inches)	Percolation Rate	
						Inches/ Hour	Inches/ Minute
	0905	Initial	72	Initial	Initial		
	0935	30	72	77.5	5.5		
	1005	60	72.25	76.125	3.875		
	1035	90	71.75	74.25	2.5		
	1105	120	72.125	74	1.875		
	1135	150	72	73.25	1.25		
	1205	180	72.125	73.5	1.375		
	1235	210	72.25	73.5	1.25		
	1305	240	72.25	73.625	1.25		
	1335	270	72	73.375	1.375		
	1405	300	71.75	73	1.25		
	1435	330	71.875	73.125	1.25		
	1505	360	72.125	73.375	1.25	2.500	0.0417
Boring/Depth (inches) P-3/46.5	Time	Time Change (minutes)	Water Level (inches)	Final Water Level (inches)	Water Level Change (inches)	Percolation Rate	
						Inches/ Hour	Inches/ Minute
	0910	Initial	24	Initial	Initial		
	0940	30	24	28.75	4.75		
	1010	60	24.25	27.75	3.5		
	1040	90	24	27	3		
	1110	120	23.75	26.75	3		
	1140	150	23.75	26.875	3.125		
	1210	180	23.625	26.625	3		

	1240	210	23.375	26.375	3		
	1310	240	23.375	25.75	2.375		
	1340	270	23.75	26.625	2.875		
	1410	300	24	26	2		
	1440	330	23.5	25.75	2.25		
	1510	360	23.75	25.625	1.875	3.7500	0.0625
Boring/Depth (inches) P-4/46	Time	Time Change (minutes)	Water Level (inches)	Final Water Level (inches)	Water Level Change (inches)	Percolation Rate	
						Inches/ Hour	Inches/ Minute
	0915	Initial	24	Initial	Initial		
	0945	30	24	27.75	3.75		
	1015	60	23.25	27.25	4		
	1045	90	24.25	27.75	3.5		
	1115	120	23.875	27.75	3.875		
	1145	150	23.875	26.875	3		
	1215	180	24	26.75	2.75		
	1245	210	24.125	26.5	2.375		
	1315	240	23.75	26.125	2.375		
	1345	270	24	27.375	3.375		
	1415	300	24	26.25	2.25		
	1445	330	24.25	27.125	2.875		
	1515	360	24	26.625	2.625	5.2500	0.0875
Boring/Depth (inches) P-5/97.25	Time	Time Change (minutes)	Water Level (inches)	Final Water Level (inches)	Water Level Change (inches)	Percolation Rate	
						Inches/ Hour	Inches/ Minute
	0912	Initial	74.875	Initial	Initial		
	0942	30	74.875	78.625	3.75		
	10.12	60	74.625	76.375	1.75		
	10.42	90	74.8125	76.375	1.375		
	1112	120	74.25	75.5625	1.3125		
	1142	150	74.875	75.9375	1.0625		
	1212	180	74.875	76.125	1.25		
	1242	210	74.875	76	1.125		
	1312	240	74.5	75.5625	1.0625		
	1342	270	74.875	75.9375	1.0625		
	1412	300	74.875	76	1.125		
	1442	330	74.875	76.125	1.25		
	1512	360	73.4375	74.875	1.4375	2.8750	0.0497
Boring/Depth (inches) P-6/108.75	Time	Time Change (minutes)	Water Level (inches)	Final Water Level (inches)	Water Level Change (inches)	Percolation Rate	
						Inches/ Hour	Inches/ Minute
	0917	Initial	85.25	Initial	Initial		
	0947	30	85.25	89.0625	3.8125		
	1017	60	84.375	87.8125	3.4375		

	1047	90	84.4375	87.375	2.9375		
	1117	120	83.8125	87.1875	3.375		
	1147	150	85.25	87.8125	2.5625		
	1217	180	84.9375	87.4375	2.625		
	1247	210	85.25	87.875	2.625		
	1317	240	84.9375	87.5	2.5625		
	1347	270	84.5625	87.5625	3		
	1417	300	85.75	87.5625	1.8125		
	1447	330	85.4375	88.1875	2.75		
	1517	360	84.75	87.25	2.5	5.0000	0.0833

NOTES Water Level as measured from the top of the hole.
The borehole had an eight inch diameter.
The percolation test holes were presoaked approximately 24 hours prior to the tests.
Weather was clear and warm during the percolation test.

As per the County of San Diego BMP design documents (February 2016) infiltration rates are to be evaluated through Porchet Method. CTE utilized the Porchet Method through guidance of the County of Riverside (2011). The intent of the infiltration rate is to take into account bias inherent in percolation test bore hole sidewall infiltration as would not occur at a basin bottom where such sidewalls are not present.

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

It= {(change H 60 r) / [change t(r+2Hav)]}
Where:
Change t=time interval
Df=final depth to water
r=test hole radius
change t=60 minutes
Do=initial depth to water
Dt=total depth of test hole
Ho=Dt – Do is initial height of water at selected time interval
Hf=Dt-Df- is the final height of water at the selected time interval
Change H=is the change in height over the time interval
Hav=(Ho+Hf) / 2 is the average head height over the time interval

Given the measurement values of Table 2.1, the calculated infiltration rates without Safety Factor are as follows.

2.2 BMP Basin 1, Percolation Tests 5 and 6

Proposed BMP Basin 1 is located near the northwestern margin of the site, adjacent to Campo Road. P-5 is located adjacent to the eastern margin of the proposed basin. P-6 is located adjacent to the western margin of the proposed basin. The finished surface of the proposed BMP Basin 1 is at 514 feetabove MSL. The percolation tests were conducted at the approximate elevation of the proposed basin excavation bottom. The calculated infiltration rates uncorrected for safey factor for BMP Basin 1 are as follows.

BMP Basin 1, P-5

Given (units in inches)

Df=74.875
Do=73.4375
Dt=97.25
r=4
change t=30 minutes

Calculated Infiltration Rate=0.229141 inches/hour

BMP Basin 1, P-6

Given (units in inches)

Df=87.25
Do=84.75
Dt=108.75
r=4
change t=30 minutes

Calculated Infiltration Rate=0.40404 inches/hour

The infiltration rates associated with BMP Basin 1 are calculated to be 0.229141 inches/hour and 0.40404 inches/hour at P-5 and P-6, respectively. CTE recommends the lower uncorrected for safety factor calculated infiltration rate value of 0.229141 inches per hour be conservatively utilized for consideration of applied safety factor as presented in following Section 4.

2.3 BMP Basin 2, Percolation Tests 1 and 2

Proposed BMP Basin 2 is located near the southern corner of the site, adjacent to Campo Road. P-1 is located adjacent to the northwestern margin of the proposed basin. P-2 is located adjacent to the eastern margin of the proposed basin. The finished surface of the proposed BMP Basin 2 is at 476.5 feet above MSL. The percolation tests were conducted at the approximate elevation of the proposed BMP basin excavation bottom. The calculated infiltration rates for BMP Basin 2 are as follows.

BMP Basin 2, P-1

Given (units in inches)

Df=79.25
Do=78.125
Dt=99
r=4
change t=30 minutes

Calculated Infiltration Rate=0.201681 inches/hour

BMP Basin 2, P-2

Given (units in inches)

Df=73.375
Do=72.125
Dt=93.5
r=4
change t=30 minutes

Calculated Infiltration Rate=0.21978 inches/hour

The infiltration rates associated with BMP Basin 2 are calculated to be 0.201681 inches/hour and 0.21978 inches/hour at P-1 and P-2, respectively . CTE recommends the lower uncorrected for safety factor calculated infiltration rate value of 0.201681 inches per hout should conservatively be utilized for consideration of applied safety factor as presented in following Section 4.

2.4 BMP Basin 3, Percolation Tests 3 and 4

Proposed BMP Basin 3 is located adjacent to property line along southeastern margin of the site. P-3 is located adjacent to the southern margin of the proposed basin. P-2 is located adjacent to the northern margin of the proposed basin. The finished surface of the proposed BMP Basin 2 is at 492 feetabove MSL. The percolation tests were conducted at the approximate elevation of the proposed basin excavation bottom. The calculated infiltration rates for BMP Basin 3 are as follows.

BMP Basin 3, P-3

Given (units in inches)

Df=25.625
Do=23.75
Dt=46.5
r=4
change t=30 minutes

Calculated Infiltration Rate=0.314961 inches/hour

BMP Basin 3, P-3

Given (units in inches)

Df=26.625
Do=24
Dt=46
r=4
change t=30 minutes

Calculated Infiltration Rate=0.46281 inches/hour

The infiltration rates associated with BMP Basin 3 are calculated to be 0.314961 inches/hour and 0.46281 inches/hour at P-3 and P-4, respectively. CTE recommends the lower uncorrected for safety factor calculated infiltration rate value of 0.314961 inches per hout should conservatively be utilized for consideration of applied safety factor as presented in following Section 4.

3.0 GEOTECHNICAL ASSESSMENT

Following is a geotechnical assessment with respect to implementation of bio-filtration basins at the subject site.

1. Infiltrate is anticipated to flow downslope to the southwest toward Campo Road where numerous underground utilities are present. Lining the sides of these BMP with impermeable geofabric is recommended to mitigate lateral migration of infiltrate. The lining should extend to the maximum depth of utility trench and foundation excavations within 100 feet of the proposed basins.

2. Based upon geotechnical information as presented by this Preliminary Geotechnical Report, groundwater depth is at least 10 feet below existing ground surface. As such, infiltrate from the proposed bio-basins is not anticipated to impact groundwater. However, due to the medium dense to dense characteristics of underlying soils groundwater is anticipated to mound beneath the BMP basins. The potential adverse effects of mounding is anticipated to be minimized by installation of an impermeable liner on the sidewalls of the proposed basins as presented in Item 1 above.

3. A “blue line” stream is about 150 feet downslope of the property across Campo Road to the southwest. It is considered unlikely that infiltrate from the subject site would enter this stream as such flow due to the distance from the site and recommended impermeable lining of the basin sidewalls as recommended in Item 1 above.

4. CTE, as a geotechnical consultant, is unaware of water rights with respect to the site and nearby properties to include those down slope of the site.
5. There is no known contamination on this site and up gradient properties to be adversely affected by the proposed bio-filtration basins.

6. Due to the medium dense to dense underlying native soils and future compacted fill soil bio-basin infiltrate is unlikely to develop the adverse conditions of settlement, and liquefaction for the facility as proposed.

7. Laboratory test results indicate the site soils have a very low to low expansion potential that in combination with processing, moisture conditioning of necessary fill soils indicate infiltrate is unlikely to produce the adverse effects of expansive soils. Recommendations of Item 1 above to install an impermeable liner on the basin sidewalls should be implemented to minimize the potential adverse effects of moisture intrusion into the site low expansion soils.

8. Position of the BMP basins and anticipated depth of infiltrate with respect to proposed existing and proposed slopes is favorable with respect to slope stability.

9. CTE did not observe water wells on the site during field explorations. Additionally, CTE contacted the County of San Diego, Department of Environmental Health, Land and Water Quality Division on October 5, 2016 to search for water wells on APN 506-140-06 and APN 506-140-07 (subject site); APN 5016-140-10 (property to southwest across Campo Road); and, APN 506-140-11 (east adjacent Skyline Wesleyan Church property). This inquiry found that no water wells were listed for the site and property to the southwest. However, a water well is registered on the east adjacent Skyline Wesleyan Church. This water well is over 100 feet from the proposed BMP basins. As such, the proposed BMP basins are not anticipated to impact water wells that are within 100 feet of them (BMP basins)

4.0 WORKSHEETS

The County of San Diego Appendix C (February 2016) required Worksheets C.4.1 "Categorization of Infiltration Feasibility Condition" and D.5-1 "Factor of Safety and Design Infiltration Rate Worksheet" as completed are attached with this Appendix E.

5.0 RECOMMENDED CALCULATED INFILTRATION RATES

A Minimum Safety Factor of 2 should be applied in accordance with attached Worksheet D.5-1 "Factor of Safety and Design Infiltration Rate Worksheet" to calculate the infiltration values of BMP Basin 1, BMP Basin 2, and BMP Basin 3. The lowest of two uncorrected calculated infiltration rates for each basin are utilized to provide CTE's recommended calculated infiltration rate to include a reduction through a safety factor of 2 as follows:

BMP Basin 1: 0.1146 inches/hour
BMP Basin 2: 0.1008 inches/hour
BMP Basin 3: 0.1575 inches/hour

The project basin designer may increase the utilized safety factor through reference to attached Worksheet D.5-1 “Redundancy” and “Level of Pretreatment” as these topics are pertinent to their project scope.

6.0 CLOSING

This report has been prepared as per County of San Diego storm water control requirements. CTE does not accept any liabilities toward preparation of the County of San Diego required design requirements. It is noted that implementation of the information provided herein is subject to interpretation and approval of the County of San Diego who has adopted the subject storm water requirements. CTE does not accept the rationale of the subject County of San Diego storm water design requirements other that preparation of this report as required by the County of San Diego storm water design documents.

Worksheet C.4-1: Categorization of Infiltration Feasibility Condition

<table>
<tr><th colspan="3">Categorization of Infiltration Feasibility Condition</th><th colspan="2">Worksheet C.4-1</th></tr>
<tr><td colspan="5">Part 1 - Full Infiltration Feasibility Screening Criteria
Would infiltration of the full design volume be feasible from a physical perspective without any undesirable consequences that cannot be reasonably mitigated?</td></tr>
<tr><td>Criteria</td><td colspan="2">Screening Question</td><td>Yes</td><td>No</td></tr>
<tr><td>1</td><td colspan="2">Is the estimated reliable infiltration rate below proposed facility locations greater than 0.5 inches per hour? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2 and Appendix D.</td><td></td><td>X</td></tr>
<tr><td colspan="5">Provide basis: No, calculated infiltration rates for all three proposed basins were less than 0.5 inches per hour. Review the CTE document "Preliminary Geotechnical Report, Proposed Skyline Retirement Center" dated October 3, 2016 for subsurface conditions, applicable maps and cross sections, and exploration logs. Appendix E of the Preliminary Geotechnical Report provides percolation rates and infiltration rate calculations.

Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.</td></tr>
<tr><td>2</td><td colspan="2">Can infiltration greater than 0.5 inches per hour be allowed without increasing risk of geotechnical hazards (slope stability, groundwater mounding, utilities, or other factors) that cannot be mitigated to an acceptable level? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2.</td><td>X</td><td></td></tr>
<tr><td colspan="5">Provide basis: Infiltrate from proposed basins is anticipate to move downslope to the southwest where major electrical utilities such as electrical, cable service, and natural gas are located. Infiltrate over time could impact these infrastructure facilities. As such, the sidewalls of the basin should be lined at a minimum of three feet or the depth of the deepest utility or foundation excavation within 100 feet of basin to minimize such potential adverse impacts. The basin bottom should remain unlined.

Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.</td></tr>
</table>

Worksheet C.4-1 Page 2 of 4			
Criteria	Screening Question	Yes	No
3	Can infiltration greater than 0.5 inches per hour be allowed without increasing risk of groundwater contamination (shallow water table, storm water pollutants or other factors) that cannot be mitigated to an acceptable level? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.	X	
Provide basis: Calculated infiltration is less than 0.5 inches per hour. Groundwater is anticipated to be deeper than at least 10 feet below the bottom of planned basins bottoms based upon test borings placed within 50 feet of the basins (see the Preliminary Geotechnical Report dated October 3, 2016 for boring logs). The site and up-gradient properties are not known contaminated sites according to Geotracker, a State of California on line resource for listings of regulated contaminated sites. Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.			
4	Can infiltration greater than 0.5 inches per hour be allowed without causing potential water balance issues such as change of seasonality of ephemeral streams or increased discharge of contaminated groundwater to surface waters? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.	X	
Provide basis: Calculated infiltration rates are less than 0.5 inches per hour. However, it is anticipated that any amount of infiltration at the site would not increase the risk of changing the seasonality of ephemeral streams or increase the risk of contaminating surface waters than currently exists. A blue line stream is approximately 150 feet southwest of the site across Campo Road. Potential impacts of the proposed basins to the blue line creek are low due to distance in combination with construction of the recommended lining of basin sidewalls to the maximum depth of adjacent utility trench and foundation excavations within 100 feet of the basins. The site and up-gradient properties are not known contaminated sites according to Geotracker, a State of California on line resource for listings of regulated contaminated sites. As such there is minimal potential contamination impacts to the blue line creek with installation of the proposed basins. Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.			
Part 1 Result*	If all answers to rows 1 - 4 are "Yes" a full infiltration design is potentially feasible. The feasibility screening category is Full Infiltration If any answer from row 1-4 is "No", infiltration may be possible to some extent but would not generally be feasible or desirable to achieve a "full infiltration" design. Proceed to Part 2	NO	

*To be completed using gathered site information and best professional judgment considering the definition of MEP in the MS4 Permit. Additional testing and/or studies may be required by City Engineer to substantiate findings.

Worksheet C.4-1 Page 3 of 4			
Part 2 – Partial Infiltration vs. No Infiltration Feasibility Screening Criteria Would infiltration of water in any appreciable amount be physically feasible without any negative consequences that cannot be reasonably mitigated?			
Criteria	Screening Question	Yes	No
5	Do soil and geologic conditions allow for infiltration in any appreciable rate or volume? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2 and Appendix D.	X	
Provide basis: On site calculated infiltration rates are less than 0.5 inches/hour. See Appendix E of the October 3, 2016 Preliminary Geotechnical Report to which this Worksheet is attached. The recommended infiltration rates including a safety factor of 2 per Worksheet D.5-1 are: BMP Basin 1: 0.1146 inches/hour BMP Basin 2: 0.1008 inches/hour BMP Basin 3: 0.1575 inches/hour As such there was infiltration in all three basins. The determination of "appreciable" is a function of interpretation by the County of San Diego and project designers. CTE has stated "Yes" simply because infiltration has been recorded at the site. Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability and why it was not feasible to mitigate low infiltration rates.			
6	Can Infiltration in any appreciable quantity be allowed without increasing risk of geotechnical hazards (slope stability, groundwater mounding, utilities, or other factors) that cannot be mitigated to an acceptable level? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2.	X	
Provide basis: See Question 2, Part 1. Potential adverse geotechnical impacts to geotechnical hazards may be minimized by installation of an impermeable liner on the sidewalls of the proposed BMP basins. Such impermeable liners should extend to the maximum depth of all utility infrastructure and foundations excavations within 100 feet of the closest approximation to the BMP basins. Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability and why it was not feasible to mitigate low infiltration rates.			

Worksheet C.4-1 Page 4 of 4			
Criteria	Screening Question	Yes	No
7	Can Infiltration in any appreciable quantity be allowed without posing significant risk for groundwater related concerns (shallow water table, storm water pollutants or other factors)? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.	X	
Provide basis: Groundwater is not present within at least 10 feet of the bottom of the BMP basins. The site and upslope properties are not known contaminated sites based upon reference to Geotracker, an on line source for regulatory listed known contaminated properties.Mounding and lateral infiltration of infiltrate is to be mitigated by recommended lining of BMP basin sidewalls with an impereable geotextile. The impermeable liner should extend to the maximum depth of utility infrastructure and foundation excavations for these facilities within 100 feet of the closest approximation to a BMP basin. Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability and why it was not feasible to mitigate low infiltration rates.			
8	Can infiltration be allowed without violating downstream water rights? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.	X	
Provide basis: To CTE's knowledge there is no downstream water rights violation as the site infiltrate is anticipated to remain within or relatively close to the property. Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability and why it was not feasible to mitigate low infiltration rates.			
Part 2 Result*	If all answers from row 1-4 are yes then partial infiltration design is potentially feasible. The feasibility screening category is Partial Infiltration. If any answer from row 5-8 is no, then infiltration of any volume is considered to be infeasible within the drainage area. The feasibility screening category is No Infiltration.	YES	

*To be completed using gathered site information and best professional judgment considering the definition of MEP in the MS4 Permit. Additional testing and/or studies may be required by City Engineer to substantiate findings

Worksheet D.5-1: Factor of Safety and Design Infiltration Rate Worksheet

<table>
<tr><th colspan="4">Factor of Safety and Design Infiltration Rate Worksheet</th><th colspan="2">Worksheet D.5-1</th></tr>
<tr><td colspan="2">Factor Category</td><td>Factor Description</td><td>Assigned Weight (w)</td><td>Factor Value (v)</td><td>Product (p)
p = w x v</td></tr>
<tr><td rowspan="5">A</td><td rowspan="5">Suitability Assessment</td><td>Soil assessment methods</td><td>0.25</td><td>1</td><td>0.25</td></tr>
<tr><td>Predominant soil texture</td><td>0.25</td><td>1</td><td>0.25</td></tr>
<tr><td>Site soil variability</td><td>0.25</td><td>1</td><td>0.25</td></tr>
<tr><td>Depth to groundwater / impervious layer</td><td>0.25</td><td>1</td><td>0.25</td></tr>
<tr><td colspan="3">Suitability Assessment Safety Factor, $S_A = \Sigma p$</td><td>1.0</td></tr>
<tr><td rowspan="4">B</td><td rowspan="4">Design</td><td>Level of pretreatment/ expected sediment loads</td><td>0.5</td><td>1</td><td>0.5</td></tr>
<tr><td>Redundancy/resiliency</td><td>0.25</td><td>1</td><td>0.25</td></tr>
<tr><td>Compaction during construction</td><td>0.25</td><td>2</td><td>0.5</td></tr>
<tr><td colspan="3">Design Safety Factor, $S_B = \Sigma p$</td><td>1.25</td></tr>
<tr><td colspan="5">Combined Safety Factor, $S_{total} = S_A \times S_B$</td><td>1.25 as modified below</td></tr>
<tr><td colspan="5">Observed Infiltration Rate, inch/hr, $K_{observed}$
(corrected for test-specific bias)</td><td>See Below.</td></tr>
<tr><td colspan="5">Design Infiltration Rate, in/hr, $K_{design} = K_{observed} / S_{total}$</td><td>See Below</td></tr>
<tr><td colspan="6">Supporting Data</td></tr>
<tr><td colspan="6">Briefly describe infiltration test and provide reference to test forms:
The minimum allowed safety factor as per the reference County guidelines (February 2016) is 2. As such the lowest of two calculated infiltration rates are: BMP Basin 1=0.1146 in/hr, BMP Basin 2=0.1008 in/hr, BMP Basin 3=0.1575 in/hr.</td></tr>
</table>