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June 6, 2019 CTE Job No. 25-0820G

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Subject: Geotechnical Investigation
Ceres Gateway Commercial Building Development
SW Quadrant Mitchell & Service Roads
Ceres, California

Mr. Ogden

As requested, we have performed a preliminary soil investigation for the referenced site. The attached report discusses the findings of our investigation activities and provides geotechnical recommendations for use during project design and construction. The project is considered feasible if the recommendations presented in this report are carried out.

If you have any questions regarding our findings or recommendations, please do not hesitate to contact this office. The opportunity to be of service is appreciated.

Respectively Submitted

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GEOTECHNICAL INVESTIGATION
CERES GATEWAY COMMERCIAL BUILDING DEVELOPMENT
SW QUADRANT MITCHELL & SERVICE ROADS
CERES, CALIFORNIA

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CTE JOB NO.: 25-0820G JUNE 6, 2019

TABLE OF CONTENTS

CTE CAL, INC.

1.0 INTRODUCTION AND SCOPE OF SERVICES

1.1 Introduction

This report presents the results of our geotechnical engineering investigation and provides conclusions and design criteria for the proposed improvements. It is our understanding that this project consists of the construction of a new commercial center, with associated improvements such as utilities, asphalt concrete parking lot, and access driveway.

Figure 1 shows the general location of the site. Figure 2 shows a plan view of the site with exploratory boring locations. A previous "Preliminary Geotechnical Investigation" report for the site was provided by this office (CTE # 25-0028G) on November 28, 2006. Ten borings and two percolation tests were performed during that study and information from those explorations is incorporated herein.

Our investigation included field exploration, laboratory testing, geologic hazard evaluation, and engineering analysis. Specific recommendations for excavations, fill placement, and foundation design for the proposed improvements are presented in this report. Cited references are presented in Appendix A.

1.2 Scope of Services

The scope of services provided for this preliminary investigation included:

- Review of readily available geologic reports and documents pertinent to the site area.
- Explorations to determine subsurface conditions to the depths influenced by the proposed construction.
- Perform storm water disposal soil suitability via percolation testing
- Laboratory testing of representative soil samples to provide data to evaluate the geotechnical design characteristics of the site foundation soils.
- Determination of the general geology and evaluation of potential geologic seismic hazards at the site.

- Preparation of this report describing the investigations performed and providing opinions/conclusions and geotechnical engineering recommendations for design and construction.

2.0 SITE LOCATION AND PROJECT DESCRIPTION

The site is located immediately east of U.S. State Highway 99 in Ceres, California as shown on the Site Index Map (Figure 1, attached). The property proposed for development is a triangular, 13.66 acre parcel at the southeast quadrant of the intersection between Mitchell Road and Service Road. At the time of our subsurface investigation the site was undeveloped and covered with a moderate growth of dry grass, Surface topography throughout the site is generally level, with exception of a 2± ft above ground mound of soil covering an existing irrigation pipe which extends roughly E-W across the central portion of the site.

Several existing palm trees were present near the center of the site which is assumed to be an area where a pre-existing residential structure were previously located. Based on historic aerial photography (Google Earth Pro Imagery, 2019), a pre-existing residential structures were also once located at the northwest and northeast corners of the site and were removed circa 2015 and 2016. Existing developed commercial properties (a church, auto wrecking yard and two gas stations) are located to the northeast and northwest of the project site and undeveloped agricultural land is present to the north, south, east and west of the site. Past use of the project site has been agricultural.

The property and proposed improvements are shown on a (draft) "Preliminary Site Plan Scheme #190327" by George Meu Associates (sheet AS-101, dated 8 April, 2019). The plan shows the new commercial center to include a hotel, restaurants, auto service and fueling stations, shops and parking lots. Storm water disposal drainage elements are understood to be proposed within parking lots areas. Flatwork, utilities, landscaping and other associated improvements are also expected to be constructed at the site as part of the project.

Percolation testing was performed by our office which provided design information for the proposed storm water drainage disposal system. The percolation testing conducted as well as the recommended infiltration rate are contained in Section 3.2.

The lot, proposed for new improvements, is relatively level and therefore only nominal grading is expected to be required to prepare the site. Preparation is expected to include removal the existing trees, any remnant foundations, old utilities, and, existing old fill present on the lot such that they will not conflict or negatively affect the service life of proposed improvements. Disturbed soils associated with such removals may require over-excavation and replacement with engineered fill as recommended herein. Recommendations for site grading and design of structure foundations and improvement have been provided below.

3.0 FIELD AND LABORATORY INVESTIGATION

3.1 Field Investigations

Field investigation, conducted on April 24 and 26, 2019, included site reconnaissance, mapping of surficial site deposits, excavation of five soil borings, and, six percolation test holes to assess the subsurface soil, groundwater and infiltration conditions at the site. The borings were drilled using a truck-mounted Simco 2400 SK-1 drill rig utilizing 4-inch diameter solid flight auger. The maximum explored depth of these borings was 25.0± feet below existing ground surface (begs).

The field subsurface exploration program included performing Standard Penetration Tests (SPT) using a standard split barrel (1.4-inch inside diameter, 2-inch outside diameter) sampler which was operated in accordance with ASTM D-1586. The sampler was utilized to obtain samples of the subsurface soils at depth intervals of 5-ft or less by driving the sampler into the bottom of the borehole with successive blows of an automatically tripped 140-pound hammer free-falling 30 inches.

The number of blows required to drive the sampler each six-inch interval (three intervals for 18 inches in total) of sampler penetration was recorded and are shown on the test boring logs (attached as Appendix B). The results of the drive sampler testing are shown on the boring logs in the column labeled "Blows/ 6 Inches". The standard penetration blow counts (N) were corrected and used during the geotechnical engineering evaluation and analysis to correlate soil strength and structure bearing characteristics.

Soils were logged in the field by a CTE Field Geologist and were classified according to the Unified Soil Classification System (ASTM D2487), sampler drive resistance, field testing, and visual observations. Exploration logs prepared for each of the borings provides soil descriptions, field in-situ test results, and blow count (N) data. The boring logs are included in Appendix B which contains the Boring Log Legend and Definition of Soil Terminology as shown on Plates BL1 and BL2, respectively. The location of the test borings are shown on Figure 2.

Relatively undisturbed soil samples were obtained in stainless steel sample tubes from the sampler and a bulk soil sample was recovered directly from drill cuttings. Soil samples were then transported to CTE's laboratory for further testing. Field descriptions within the boring logs have been modified, where appropriate, to reflect laboratory test results. Upon completion of drilling, the borings were backfilled from final boring depth up to original ground surface with soil cuttings.

3.2 Percolation Testing General

Our subsurface geotechnical investigation included conducting a site storm water disposal soil suitability evaluation via percolation testing. The evaluation included the drilling and testing of six percolation test holes drilled at the locations shown on Figure-2 (see P-1 thru P-6). The percolation test holes were drilled from existing lot grade to a depth of 8.0± feet (P-1), 8.0 feet. In addition as stated previously five subsurface borings were drilled, logged and sampled to a maximum depth of 25.0 ft below grade to access the subsurface soil and groundwater profile below the site.

Field investigation and subsurface exploration oversight was performed by an experienced geological engineer from this office. Soils were logged and field classified using the Unified Soil Classification (USC) System as to consistency, color, texture, and gradation, on the bases of drill action, drive sampler penetration and examination of soil samples and drill cuttings.

Soil materials encountered during our geotechnical subsurface drilling program generally consisted of loose and medium dense silty sands with locally, included layers of hard sandy silt from 5 to 8± ft in Boring-3 and 10 to 11± ft in Boring-4. Soil materials encountered within the percolation test holes were generally consistent with the silty sands encountered within the geotechnical borings. The material type presented on the percolation test data sheets represents the material type in which the percolation testing was conducted. Percolation data sheets are contained in Appendix-B.

Groundwater was not encountered within the percolation test holes or within the five borings drilled for our geotechnical investigation with Boring-5 achieving the maximum drilling depth of 25±ft. Groundwater was also not encountered within the maximum explored depth of 21.5 ft during our previous geotechnical investigation (CTE # 25-0028G, dated November 28, 2006).

These observations represent groundwater conditions at times of the field explorations and may not be indicative of other times, or at other locations. Groundwater conditions can vary with seasonal changes, local weather conditions, and, other factors. Groundwater depth in the vicinity of the site (Spring 2018) is indicated to be on the order of 45± feet below existing grade (https://gis.water.ca.gov/app/gicima/ and the highest groundwater dating back to spring of 2011 is indicated to be 31± feet below existing grade. Based on a groundwater depth of 31 feet and a maximum drainage element depth above 21 feet, groundwater will be at least 10 feet below the base of the proposed drainage element.

As stated previously, logs of subsurface borings and percolation data and test results are included in Appendix-B. Locations of the geotechnical test borings and percolation tests are shown on the attached "Exploration Location Map" (Figure-2). All test borings were backfilled to ground surface and surface restored to original condition upon completion of testing.

3.21 Percolation Testing Procedure

Upon completion of the percolation hole drilling, loose material was removed and a 3-inch diameter open-ended slotted drain pipe was installed to control potential sidewall caving of the test-hole. Pre-saturation of the soils to be tested was accomplished by filling each test hole with water to a level 12 inches above the bottom 17± to 21± hours prior testing. During testing a six inch (minimum) column of water "dissipated" from each of the percolation test holes within 30 minutes or less. Percolation testing was then performed immediately by adding water to a level of approximately 6±

inches above the top of the 2 inches of gravel placed at the base of each test hole. Recordings were made of the change (drop) in water level at regular time intervals and water level was refilled to 6-inches after each interval. Specific details are included on the attached percolation test data sheets located in Appendix-B.

3.22 Percolation and Infiltration Rates

The soil percolation rate is defined by the average time in minutes for a 1-inch column of water to "seep" into the soil. Percolation rate was calculated (in minutes per inch) by dividing the time (in minutes) by the change (drop) in water level (in inches). No correction factor was used in the calculation for boring diameter.

As shown below in Table 3.3 percolation test "P-1" achieved a steady percolation rate of 6.67 minutes/inch, "P-2" achieved a steady percolation rate of 1.74 minutes/inch, and "P-3" achieved a steady percolation rate of 4.00 minutes/inch and "P-4", "P-5" and "P-6" achieved a steady percolation rate of 10.0 minutes/inch.

In general, the percolation rates obtained are not considered inconsistent with those typical of the soil types encountered at the site and the site location. Based on percolation test results, as described above, the soil conditions at the site are considered suitable for a storm water disposal system in the vicinity of the proposed parking areas. Owing to variations in material grain size, type, and, depths, percolation rates would typically be expected to fluctuate somewhat across a site and are also dependent upon actual construction, depth, size, location and workmanship of the drainage element. The percolation test measures the length of time required for a quantity of water to infiltrate into the soil and is commonly referred to as the "percolation rate".

It should be noted that the percolation rate is related to, but not equal to, the infiltration rate. While an infiltration rate is a measure of the speed at which water progresses downward into the soil, the percolation rate measures not only the downward progression but the lateral progression through the soil as well. This reflects the fact that the surface area for infiltration testing would include only the horizontal surface while the percolation test includes both the bottom surface area and the sidewalls of the test hole.

The calculated conversion from percolation rate to infiltration rate is located in appendix B. The resulting percolation rates in min/inch and infiltration rates in gal/sf/day are listed in Table 3.3 below. The observed infiltration rates listed below do not include a safety factor.

TABLE 3.3				
TEST NUMBER	DEPTH (ft)	MATERIAL TYPE	PERCOLATION RATE (Min/In)	OBSERVED INFILITRATION RATE (Gal/ft^2/day)
P-1	8	SM (SILTY SAND)	6.67	29.9
P-2	8	SM (SILTY SAND)	1.74	167.4
P-3	8	SM (SILTY SAND)	4.00	53.9
P-4	8	SM (SILTY SAND)	10.00	19.2
P-5	8	SM (SILTY SAND)	10.00	19.2
P-6	8	SM (SILTY SAND)	10.00	19.2

3.3 Laboratory Investigations

Laboratory tests were conducted on representative soil samples for classification purposes and to evaluate physical properties and engineering characteristics. Laboratory tests were conducted to determine Moisture Content, Dry Density, and, R-value (ASTM D2844). Laboratory results and test methodologies are included in Appendix C.

4.0 GEOLOGY

4.1 General Geologic Setting

The site is located within the City of Ceres (central Stanislaus County), from a geomorphologic standpoint this area is within California's Central San Joaquin Valley. The most significant geologic process affecting the site/vicinity is the accumulation of deposits from numerous alluvial fans associated with creeks, streams and rivers originating at higher elevations within the Sierra Foothills and Mountain Ranges which carry the sediment into the relatively flat, broad valley.

The San Joaquin Valley has been filled with hundreds of feet of erosional sediments, ranging in age from Pleistocene to Holocene. Recent alluvial deposits generally consist of poorly sorted silts and fine sands with less extensive lenses of medium to coarse grained sands and gravel. Lacustrine deposits occur along the axis of the valley, and consist of clays, silts, and fine sands. These alluvial units overlie Pliocene-Pleistocene continental clastic deposits, which in turn lie over older continental and marine deposits. A pre-Tertiary basement complex of granitic and metamorphic rocks unconformably underlies the entire area.

Based on the USGS Geologic Map of the San Francisco-San Jose quadrangle, California (CDMG Regional Geologic Map 5A, Scale 1:250,000, 1991) geologic units at the site/vicinity consist of Quaternary Alluvial Fan Complex deposits (Holocene and Upper Pleistocene) of the Modesto Formation. These deposits are described as undeformed, generally unweathered, unconsolidated, poorly to moderately sorted and bedded coarse sandy gravel and gravelly coarse sand as stream terraces and valley fills and at fan heads, grading downstream to sorted and bedded silt, clay, and fine sand on lower fans.

4.2 Generalized Soil Conditions

Soil materials encountered in our site explorations are considered consistent with alluvial fan deposits as described on published geologic mapping (discussed above). As encountered in our explorations, native soils consisted of alluvium comprised of loose and medium dense silty sands, and, locally, included a layer of hard silt from 5 to 8± ft in Boring-3 and 10 to 11± ft in Boring-4. The loose sands are considered "weak", unconsolidated, and will require reprocessing to adequately provide support for the proposed buildings and other improvements. Depth of recommended reprocessing is discussed below.

4.3 Groundwater Conditions

Groundwater conditions within the test borings were evaluated at the time of field exploration and groundwater was not encountered within the maximum explored depth (25.0±ft) of any of the borings excavated on April 24, 2019. Groundwater was also not encountered within the borings performed on 9/7/06 for our previous subsurface exploration at the site (21.5±ft maximum depth). These observations represent groundwater conditions at the time of the field exploration and may not be indicative of other times, or at other locations.

Groundwater conditions can change with varying seasonal and weather conditions, and other factors. Groundwater depth in the vicinity of the site (Spring 2018) is indicated to be on the order of 45± feet below existing grade (https://gis.water.ca.gov/app/gicima/ and the highest groundwater dating back to spring of 2011 is indicated to be 31± feet below existing grade. Groundwater is not expected to affect construction of the proposed structure or other improvements.

Wet weather construction methods should be anticipated if construction is scheduled to occur during the rainy season. During periods of appreciable precipitation, localized higher groundwater and/or perched water situations should be expected which could produce locally or widespread saturated surface soils. In addition, if construction is undertaken during wet-season/heavy-rains, saturated soils are not expected to be acceptable for grading or compaction and could hamper progress due to limited equipment mobility and/or inability to achieve appropriate moisture content and required soil compaction.

Saturated soils, if present, may need to be dried by extensive aeration or chemically modified through the addition of lime, cement, or kiln dust added to stabilize the working surface. Appropriate erosion

control and permanent site surface drainage elements per the latest California Building Code should be designed and implemented as per the project civil engineer.

4.4 Geologic Hazards

Based on our explorations and research, the most significant geotechnical condition which could affect the proposed structures is the potential for strong shaking from a potential earthquake and potential for effects from strong ground motion that could cause local compression of the loose soils present at the site. California Building Code and recommendations below should be conformed to and confirmed during grading and construction. Engineered fill materials constructed as described below are considered adequate for support of moderately loaded structures using conventional shallow foundations. Design and construction recommendations presented herein have been developed based on the noted site conditions.

4.5 General Geologic Hazards Observation

Based on our site reconnaissance, evidence from our explorations, and a review of appropriate geologic literature, it is our opinion that the site is not located on any known fault traces (http://maps.conservation.ca.gov/cgs/informationwarehouse/index.html?map=regulatorymaps). The site does not lie within a State of California- "Fault-Rupture Hazard Zone" (DMG, 2000) and State of California- "Seismic Hazard Zone" mapping is currently not planned for the site/vicinity (http://gmw.consrv.ca.gov/shmp/html/pdf_maps_no.html).

The potential for fault rupture or damage from fault displacement or fault movement directly below the site or near to the site is considered to be low. However, the site is located within an area where shaking from earthquake generated ground motion waves should be considered likely.

4.6 Local and Regional Faulting

The California Geological Survey (CGS) and the United States Geological Survey (USGS) broadly group faults as "Class A" or "Class B". 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 typically have a fault-slip rate less than 5 mm/yr. The nearest known Class A fault is the Calaveras Fault located approximately 41.17 miles from the site and the nearest known Class B fault is the Great Valley 7 located approximately 14.92 miles from the site (U.S. Geological Survey (CGS), 2006, Quaternary fault and fold database for the United States, accessed 6/6/19, from USGS web site: https://earthquake.usgs.gov/cfusion/hazfaults_2008_search/query_results.cfm). Information for other principal regional faults is included in Table 1, below.

TABLE 1 NEAR SITE FAULT PARAMETERS			
FAULT NAME	DISTANCE FROM SITE (MILES)	MAXIMUM EARTHQUAKE MAGNITUDE	CLASSIFICATION
GREAT VALLEY 7	14.92	6.9	B
GREAT VALLEY 8	15.62	6.8	B
ORTIGILITA	28.36	7.1	B
GREAT VALLEY 9	32.93	6.8	B
GREENVILLE CONNECTED	33.48	7.0	B
CALAVERAS CN + CC	41.17	7.0	A
CALAVERAS CC + CS	45.17	6.5	A

4.7 Liquefaction and Seismic Settlement Evaluation

Liquefaction occurs when saturated fine-grained sands and/or silts lose their physical strength temporarily 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.

The California Geological Survey (CGS) has designated certain areas within California as potential liquefaction hazard zones. These are areas considered at risk of liquefaction-related ground failure during a seismic event. The project site is not currently mapped for potential liquefaction hazard by the CGS (refer to CGS website: (http://gmw.consrv.ca.gov/shmp/html/pdf_maps_no.html). Based on readily available published geologic information, there is no historical record of liquefaction occurring at the site.

Subsurface information obtained during our study was limited to a depth of 25.0 ft; within such depth, the site is predominately underlain by loose and medium dense silty sand (SM) and locally hard silt (ML) materials. In addition, our explorations indicate no free groundwater was encountered within the upper 25.0± feet and research using the Groundwater Information Center Interactive Map Application (https://gis.water.ca.gov/app/gicima/) indicates groundwater to be on the order of 45± feet (Spring 2018) below existing grade and high historic groundwater level dating back to spring of 2011 is indicated to be 31± feet below existing grade.

To determine the potential for liquefaction below the groundwater level indicated above, subsurface drilling, sampling and data acquisition would be required to a depth of 50± ft. Drilling and sampling to a depth of 50± ft was beyond the agreed scope of services for this limited study.

Based on the information discussed above, the maximum accelerations anticipated for the site, and, no previous record of liquefaction occurrence at the site/vicinity, it is our opinion the potential for liquefaction at the project site is likely relatively low. However, owing to the limited depth of information required for our study, it cannot be precluded that potential for liquefaction may exist below the depth of groundwater. If determining the potential for liquefaction is considered important and further definition is desirable, additional subsurface exploration (to 50± ft depth) and appropriate addendum to this report can be provided by this office (upon request with revised scope of work).

4.8 Earthquake Induced Landsliding

Based on information available on the California Geological Survey (CGS) website the site is not currently within a State of California Seismic Hazard Zone for seismically induced landsliding. In addition, the site and approximately 1 mile of surrounding terrain within the valley is relatively flat-lying; therefore, seismically induced landsliding and/or other (gravity) landslides are not considered a significant hazard at the site.

4.9 Tsunamis and Seiche Evaluation

The site is not located within an inundation area as defined by the State of California Emergency Management Agency (Interactive Map Access). In addition, the site is located inland within the Central Valley over 65 miles from the Pacific Ocean and at over 95± feet above MSL. Based on this geometric relationship, the potential for tsunami damage at the site is considered negligible. Damage caused by oscillatory waves (Seiche) is not considered likely as the site is not near any significant bodies of water.

4.10 Compressible and Expansive Soils

Compressible materials consisting of surficial organic material, loose soils, undocumented fills, debris, rubble, old foundations, rubbish, etc. are considered unsuitable materials for support of structures and improvements. In addition, any disturbed soils associated with such removals should be entirely removed such that firm intact native soils are present at base of excavations. In general, where obstructions, old foundations and disturbed soils are removed, residual excavations should be backfilled with compacted with engineered fill to appropriate grade, as recommended below.

Near surface soil deposits at the site consist of loose to medium dense silty sands and sandy silts. As such near surface soil materials are not expected to provide uniform support of structures and surface improvements unless remediated through over-excavation and recompaction as recommended in Sections 5.1 and 5.2. Based on the materials encountered the near surface site soils are not

considered to be potentially expansive. By definition, the site soils are non-expansive and the potential for post construction shrinkage and swelling is not considered a risk at the site.

5.0 CONCLUSIONS AND RECOMMENDATIONS

We conclude that the proposed construction on the site is feasible from a geotechnical standpoint, provided the recommendations in this report are incorporated into the design and construction of the project. The most significant geotechnical condition which could affect the proposed development is (1) the inconsistent nature of the site surficial deposits to uniformly support the proposed structures and improvements (2) the potential for strong shaking from a potential earthquake (3) the possible presence of old foundations associated with pre-existing structures, which may require removal and over-excavation of disturbed subgrade soils below, are considered significant considerations.

To minimize the potential presence of old fill, old debris, loose soils, or, other unsuitable bearing materials below the proposed structures or surface improvements, recommendations have been provided below which should be utilized during earthwork operations. Specific recommendations for site grading, design and construction of the proposed facility and associated improvements are included below.

5.1 Site Preparation

Prior to earthwork, all areas to be improved should be stripped of any vegetation and organic materials and cleared of surface obstructions, existing improvements, and/or old pre-existing foundations. The vegetation, organic materials and debris from the clearing operation should be removed from the site or processed for use in re-vegetation operations.

Any unsuitable soil, fill, old foundations, root-balls, septic systems, underground utilities, and/or existing obstructions encountered or observed during grading that extend below the limits of excavation should be entirely removed to competent material or as designated on the plans (whichever is deeper) and replaced with properly compacted engineered fill. Utilities that extend into the construction area and are scheduled to be abandoned should be properly capped at the perimeter of the construction zone or moved as directed in the plans.

In order to provide uniform structure foundation support and reduce the potential for post construction movement and distress of structures and improvements CTE recommends that overexcavation of site soils be performed to a depth of 2 feet below existing grade or to a depth of at least 1 foot below the proposed footing base and surface improvement subgrade whichever is greater. The recommended overexcavation should extend a minimum of five-feet horizontally beyond proposed building footprints and improvement limits if possible.

Upon the completion of overexcavation, the exposed subgrade should be verified by our representative to consist of firm, "intact", relatively undisturbed native soils. If such soils are not available at this level, additional removals may be required.

5.2 Grading and Earthwork

CTE should continuously observe the grading and earthwork operations for this project. Such observations are essential to identify field conditions that differ from those predicted by this investigation, to adjust designs to actual field conditions, and to verify that the grading is in overall accordance with the recommendations presented in this report. The anticipated site excavations should generally be accomplished with heavy-duty construction equipment under normal conditions.

The geotechnical consultant should verify that the proper site preparation and required over-excavation have been completed prior to fill placement. Areas to receive fill or improvements should be then be scarified, properly moisture conditioned recompacted. Fill and backfill should be compacted to a minimum of 90 percent relative compaction as evaluated by ASTM D1557 at moisture content at least 2 % (percent) above optimum. The optimum lift thickness for backfill soil will be dependent 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 done in overall conformance with geotechnical recommendations and local ordinances. Existing soils derived from on-site are considered suitable for reuse on the site provided they are screened of organic materials and materials greater than three inches in maximum dimension, moisture conditioned and compacted as indicated below. If imported fill or "non-expansive import" is proposed beneath structures, pavements and walks, it should have an expansion index less than or equal to 30 (per UBC 18-I-B) with no more than 35 percent passing the No. 200 sieve. All imported fill materials should be evaluated by the soils engineer to determine soil parameters (e.g. expansion index, maximum dry density, gradation etc.) and adequacy before placement on the site.

5.3 Structure Foundation Recommendations

Continuous and isolated spread footings are considered suitable for use at this site to support the proposed structures. All structure footings should be founded entirely in engineered fill as recommended herein. CTE's geotechnical engineer or his representative should observe soil conditions exposed in foundation excavations. If the soil conditions encountered differ significantly from those presented in this report, supplemental recommendations will be required.

Foundation dimensions and reinforcement should be based on allowable bearing values of 2,000 pounds per square foot (psf) for spread footings of at least 12-inches in width penetrating into and embedded below rough pad soil grade at least 18 inches deep below the lowest adjacent subgrade. The allowable foundation bearing pressures apply to dead loads plus design live load conditions. The design bearing pressure may be increased by one-third when considering total loads that include short duration wind or seismic conditions. The weight of the foundation concrete below grade may be neglected in dead load computations.

We recommend that all footings be reinforced as required by the structural designer to provide structural continuity, to permit strong spanning of local irregularities and to be rigid enough to accommodate potential differential movements estimated at about one-half inch over 30 linear feet. Based on the conditions observed at the site, the total structure settlement is expected to be on the order of one inch (1") for static compression and one inches (1") for dynamic settlement in the event a large seismic event occurs on a nearby earthquake. Differential settlements on the order of 0.5 inches and 0.5 inches are recommended for static and dynamic settlements, respectively.
The foundation excavations should be clean (i.e., free of all loose slough), firm, and moist prior to placing steel and concrete. Foundation excavations should be moisture conditioned to at least 2% over optimum moisture content and recompacted to 90% relative compaction if required. CTE shall inspect , test and approve the base of all footing excavations.

The concrete for the foundation should not be placed against a dry excavation surface. Concrete should be pumped or placed by means of a tremie or elephant's trunk to avoid aggregate segregation and earth contamination. Concrete should not be chuted against the excavation sidewalls for excavations over five feet deep. Rebar reinforcement should be properly supported with proper clearances maintained during concrete placement. The concrete should be properly vibrated to mitigate formation of voids and to promote bonding of the concrete to steel reinforcing. These recommendations are predicated upon CTE's representative observing the bearing materials as well as the manner of concrete placement.

5.4 Lateral Load Resistance

Shallow footings may be designed to resist lateral loads using a coefficient of friction of 0.30 (total frictional resistance equals the coefficient of friction times the dead load). A design passive resistance value of 200 pounds per square foot per foot of depth may be used. The allowable lateral resistance can be taken as the sum of the frictional resistance and the passive resistance, provided the passive resistance does not exceed two-thirds of the total allowable resistance.

5.5 Foundation Setback

The bottoms of utility trenches placed along the perimeter of the foundation should be above an imaginary plane that projects at a 45 degree angle down from the lowest outermost edge of the foundation. Where trenches pass through the plane the trench should be installed perpendicular to the face of the foundation for a distance of at least the depth of the foundation. Deepening of affected foundation is considered an effective means of attaining the prescribed setbacks.

5.6 Concrete Slabs-On-Grade

For buildings utilizing shallow spread foundations, lightly loaded concrete slabs should be designed for the anticipated loadings, but measure at least 4.0 inches in thickness. Minimum slab-on-grade reinforcement should consist of # 4 reinforcing bars placed on 24-inch centers, each way, at above mid-slab height, but with proper cover. Unless a structural slab is utilized, building slabs-on-grade subject to automobile traffic or equipment loading should be at least 5.0 inches thick or designed based on loading per the project structural engineer. All interior concrete slab on grade shall be installed above a 4" thick capillary moisture break which in turn overlies the compacted building pad. The capillary moisture break material shall consist of ¾ inch minus crushed rock or class 2 base.

All interior slab on grade located in moisture sensitive areas should be directly underlain by a minimum 10-mil thickness vapor retarder with all laps or penetrations sealed or taped. The vapor retarder should be installed directly over the capillary moisture break. The use of sand above the vapor retarder is not recommended. The concrete to be placed into the conventional slab on grade shall have a water to cement ratio w/c $\leq$ 0.45 and shall be placed at a maximum slump of 4" +/-.

The structural engineer/architect and slab installation contractor should refer to ACI 302 and ACI 360 for procedures and cautions regarding the use and placement of a vapor barrier. In areas of exposed concrete, control joints should be saw-cut into the slab after concrete placement in accordance with ACI Design Manual, Section 302.1R-37 8.3.12 (tooled control joints are not recommended). To control the width of cracking, continuous slab reinforcement should be considered in exposed concrete slabs.

5.7 Earth Pressures and Retaining Walls

Although not anticipated to be constructed at this site free draining retaining walls backfilled using generally select granular soils, may be designed using the equivalent fluid weights given in the table 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	50
RESTRAINED WALL	50	70

Traffic surcharges on retaining walls should generally be equal to 1/3 of the vertical load of the traffic located within ten lateral feet of wall. 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 5.8)

P_K = Static Restrained Wall Thrust (given previously Table 5.8)

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 = ½ Peak Ground Acceleration = ½ $(S_{DS}/2.5)$

H = Total Height of the Wall

γ = Total Unit Weight of Soil ≈ 125 pounds per cubic foot

The increment of dynamic thrust in both cases should be based on a trapezoidal distribution (essentially an inverted triangle), with a line of action located at 0.6H above the bottom of the wall. The values above assume non-expansive backfill and free-draining conditions. Measures should be taken to prevent moisture buildup behind all retaining walls. Drainage measures should include free-draining backfill materials and sloped, perforated drains. These drains should discharge to an appropriate off-site location. Waterproofing should be as specified by the project architect.

5.8 Seismic Design Criteria

In general accordance with the 2016 CBC, Table 1613.3.5. CBC Site Class is based on the average characteristics of the upper 100 feet of the subsurface profile. The 2016 CBC requires a site soil profile determination extending to a depth of 100 feet for seismic site classification. Borings for this study extended to a maximum depth of 25± feet, and therefore the seismic site class definition considers soils below 25 feet in depth to be consistent with the medium dense soils encountered at shallower depths.

Therefore soils that underlie the site are considered to be consistent with Site Class D materials. Site ground motion with 10% probability of exceedance in 50 years are presented in Table 5.8, below. The table is based on United States Geological Survey's (USGS) Probabilistic Seismic Design Maps webpage (online http://earthquake.usgs.gov/designmaps/us/application.php) for the site coordinates 37.579005°N latitude and -120.940422°W longitude.

The referenced USGS design maps are based seismic ground motion values determined using the USGS Ground Motion Parameter Calculator which is based on the 2016 California Building Code (CBC) and design code reference document, ASCE 7-10 (with 2013 errata) Standard.

SEISMIC GROUND MOTION VALUES		
PARAMETER	***VALUE***	***CBC REFERENCE (2016)***
Site Class[1]	***D[2]***	***ASCE 7, Chapter 20***
Mapped Spectral Response Acceleration Parameter, S_S	***0.924g***	***Figure 1613.3.1 (1)***
Mapped Spectral Response Acceleration Parameter, S_1	***0.337g***	***Figure 1613.3.1 (2)***
Seismic Coefficient, F_a	***1.131***	***Table 1613.3.3 (1)***
Seismic Coefficient, F_v	***1.726***	***Table 1613.3.3 (2)***
MCE Spectral Response Acceleration Parameter, S_{MS}	***1.044g***	***Section 1613.3.3***
MCE Spectral Response Acceleration Parameter, S_{M1}	***0.582g***	***Section 1613.3.3***

Design Spectral Response Acceleration Parameter, S_{DS}	***0.696g***	***Section 1613.3.4***
Design Spectral Response Acceleration Parameter, S_{D1}	***0.388g***	***Section 1613.3.4***
Mapped MCE Geometric Peak Ground Acceleration, PGA_m	***0.396g***	***ASCE 7, Chapter 11***
Seismic Design Category	***D***	***ASCE 7, Chapter 11***

[1]In general accordance with the 2016 CBC based on the more severe design category in accordance with Table 1613.3.5(1) or 1613.3.5(2) for buildings in Risk Categories I, II, and III.

5.9 Exterior Flatwork

To reduce the potential for distress to exterior flatwork caused by minor settlement of foundation soils, we recommend that such flatwork be installed with crack-control joints at appropriate spacing as designed by the project architect. Flatwork, which should be installed with crack control joints, includes driveways, sidewalks, and architectural features. All subgrade should be prepared according to the earthwork recommendations previously given before placing concrete. Positive drainage should be established and maintained adjacent to all flatwork.

5.10 Drainage

Foundation and concrete-slab-on grade performance depends greatly on how well the runoff waters drain from the site. This is true both during construction and over the entire life of the structure. The ground surface around structures should be graded so that water flows rapidly away from the structures without ponding. The surface gradient needed to do this depends on the landscaping type. In general, the pavements and flowerbeds within five feet of buildings should slope away at gradients of at least two percent. Densely vegetated areas should have minimum gradients of five percent away from buildings if doing so is practical.

Planters should be constructed so that water from them will not seep into the foundation areas or beneath slabs and pavement. In any event, the site maintenance personnel should be instructed to limit irrigation to the minimum actually necessary to sustain the landscaping plants properly. Should excessive irrigation, waterline breaks, or unusually high rainfall occur, saturated zones and groundwater may develop. Consequently, the site should be graded so that water drains away readily without saturating the foundation or landscaped areas or cascading over slope faces.

A potential source of water, such as water pipes, drains, and the like should be frequently examined for signs of leakage or damage. Any such leakage or damage should be repaired promptly. The project Civil Engineers should thoroughly evaluate the on-site drainage and make provisions as necessary to keep surface waters from affecting the site.

Generally, CTE recommends against allowing water to infiltrate building pads or adjacent to slopes. We understand that some agencies are encouraging the use of storm-water cleansing devices. Use of such devices tends to increase the possibility of high groundwater and slope instability. If storm-water cleansing devices must be used, then we recommend that they be underlain by an impervious barrier and the infiltrate be collected via subsurface piping and discharged off site.
Utility trenches are a common source of water infiltration and migration. All utility trenches that penetrate beneath the building perimeter should be effectively sealed to restrict water intrusion and flow through the trenches that could migrate below the building. We recommend constructing an effective "trench plug" that extends at least 2 feet out from the face of the building exterior and beneath the perimeter footing.

Utility plug material should consist of concrete or low plastic clay (Plasticity Index <15) compacted to 90 percent relative compaction (per ASTM D 1557) at a water content at slightly above the soil's optimum water content. The concrete or the clay fill should be placed to completely surround the utility line; and, if used, the clay should be placed and compacted in accordance with recommendations in this report.

5.11 Vehicular Pavements and Site Improvements

Recommended pavement sections for auto drive/parking, truck drive/loading and city street areas are presented in the table below. Two options are presented below. Option 1 is for construction of asphaltic concrete pavements and Option 2 is for construction of full-depth concrete pavements. The preliminary pavement sections presented are based on a Resistance "R"- Values obtained from a sample of site soils and our experience with the soil types in the vicinity of the site.

All Class II aggregate base should meet or exceed Caltrans Standard Specifications (including Minimum R-Value=78). For onsite design it is assumed that the upper 12 inches of subgrade and all base materials are properly compacted to 95% relative compaction at above optimum moisture content. For city streets designed based on Caltrans Standard Specifications, structural section materials (AC, AB & subgrade) should be properly compacted to 95% relative compaction within 30-inches (minimum) below finished pavement grade.

TABLE 5
RECOMMENDED PAVEMENT THICKNESS

Traffic Area	Assumed Traffic Index	Subgrade "R"-Value	Option 1: Asphalt Pavements		Option 2: PCC Concrete Pavements (inches)
			AC Thickness (inches)	Class II AB Thickness (inches)	
Auto Drive /Parking	5.0	50+	3.0	4.0	6.0
Truck Drive & Loading	6.0	50+	3.5	4.0	6.0
City Streets	7.0	50+	4.0	5.0	N.A.

Please note that these pavement sections may not be acceptable for city or public street repair or improvements. The Traffic Indexes (TI's) used in the calculations of pavement sections was assumed, sections for other TI's can be provided if desired from data in-hand upon your request.

5.12 Construction Observation

The recommendations provided in this report are based limited subsurface information observed, at locations, and within, exploratory borings performed for this project and preliminary concept design proposed construction as of the date of publication. The interpolated subsurface conditions, on which this report relies, should be checked in the field during construction to verify conditions described herein are as anticipated. Any changes which occur to preliminary information provided to this office as of the date of this publication, this office should be notified and afforded an opportunity to update information provided in this report.

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 earthworks should be observed and tested to verify that grading activity has been performed according to the recommendations contained within this report. The project engineer should evaluate all footing trenches before reinforcing steel placement.

5.13 Plan Review

CTE should review project grading and foundation plans before the start of earthworks to identify potential conflicts and to verify that the recommendations contained in the report are to be implemented.

6.0 LIMITATIONS OF INVESTIGATION

As indicated, the recommendations presented herein are based on the field exploration, laboratory testing and our geologic and engineering analysis. Following completion of testing, these recommendations will be confirmed and or modified, if necessary, based on the materials exposed and re-worked during grading.

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

Our conclusions and recommendations are based on an analysis of the observed conditions. If conditions different from those described in this report are encountered, our office should be notified and additional recommendations, if required, will be provided upon request. We appreciate this opportunity to be of service on this project. If you have any questions regarding this report, please do not hesitate to contact the undersigned.

We appreciate the opportunity to be of service on this project. Should you have any questions or need further information please do not hesitate to contact this office.

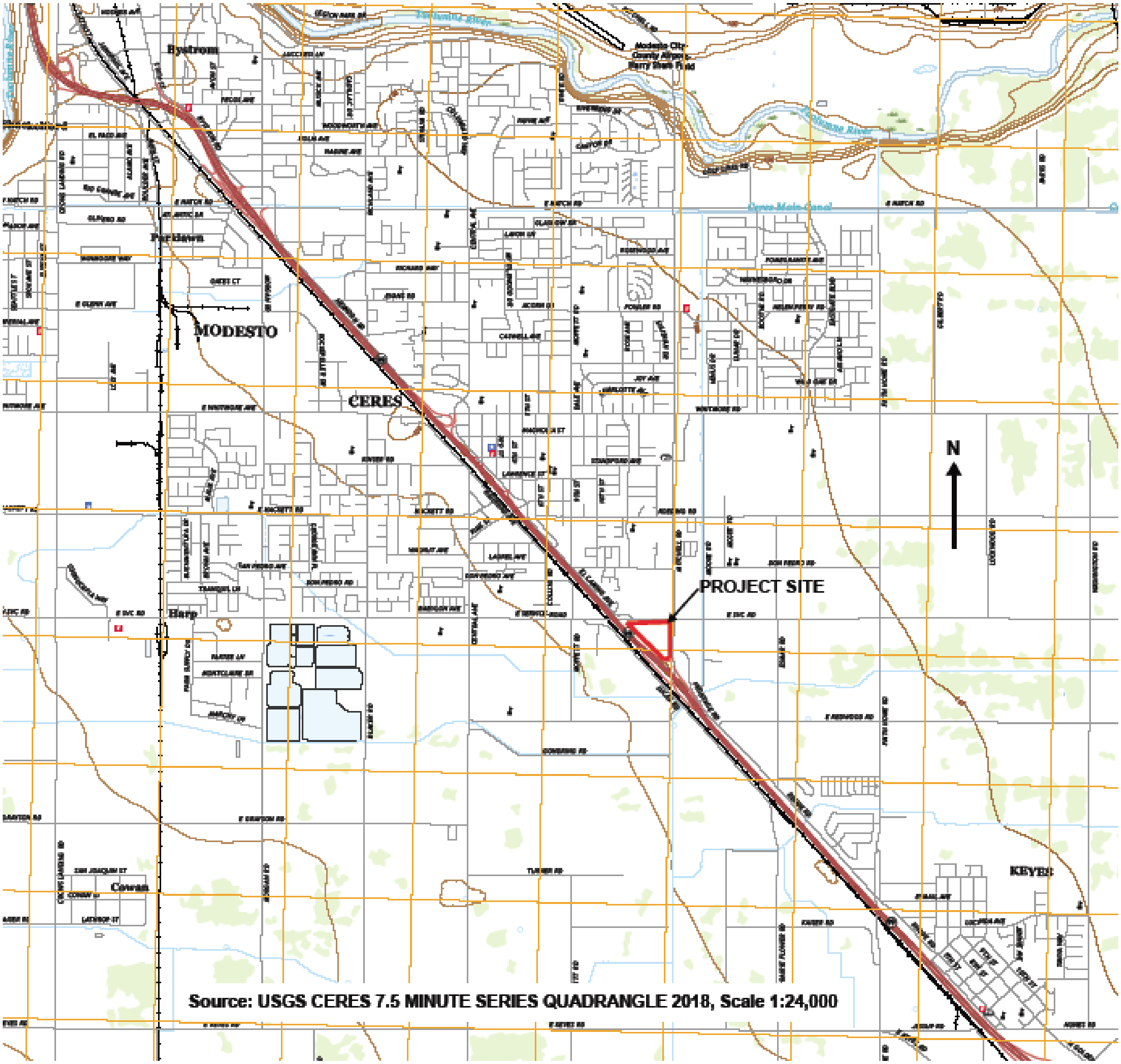
Respectfully submitted,

CTE CAL, INC.

Rod Ballard GE 2173
Principal Engineer

Michael Kennedy PE 88971
Project Engineer

T. Alan Krause
Staff Geological Engineer



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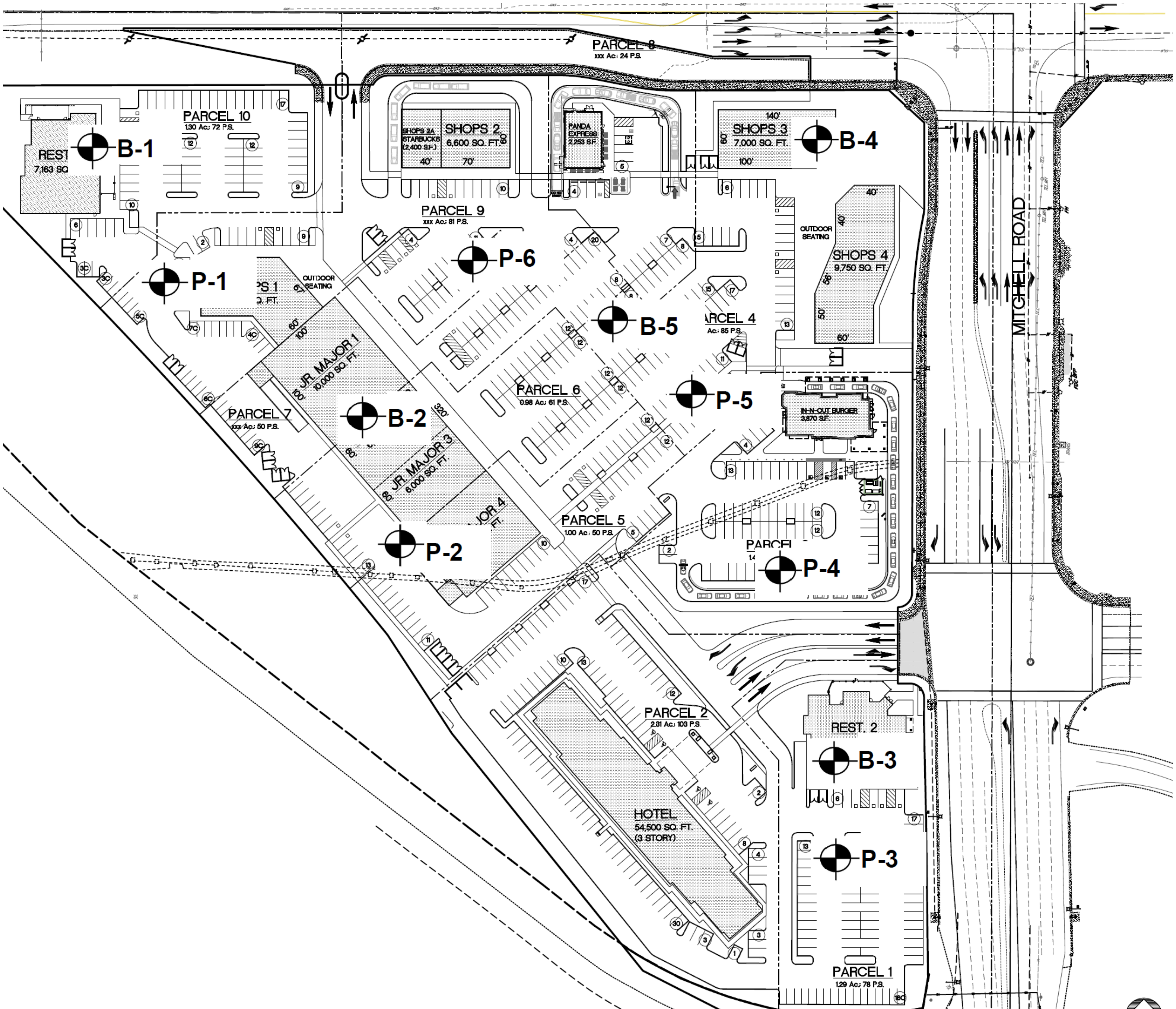
SITE INDEX MAP CERES GATEWAY MITCHELL ROAD AT SERVICE ROAD CERES, CALIFORNIA	CTE JOB NO. 25-0820G	
	SCALE N.T.S.	
	DATE 05/24/19	FIGURE 1



-CTE CAL Inc. Exploration & Percolation Test Locations 4/24/19 & 4/26/19

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EXPLORATION MAP CERES GATEWAY MITCHELL ROAD AT SERVICE ROAD CERES, CALIFORNIA	CTE JOB NO. 25-0820G	
	SCALE N.T.S.	
	DATE 05/24/19	FIGURE 2

APPENDIX A

REFERENCES CITED

REFERENCES CITED

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2. California Division of Mines and Geology, CD 2000-003 “Digital Images of Official Maps of Alquist-Priolo Earthquake Fault Zones of California, Southern Region,” compiled by Martin and Ross.

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6. U.S. Geological Survey (CGS), 2006, Quaternary fault and fold database for the United States, accessed 9/28/17, from USGS web site: http//earthquake.usgs.gov/hazards/qfaults/.

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APPENDIX B

FIELD EXPLORATION METHODS AND BORING LOGS
PERCOLATION TEST REPORTS
PERCOLATION TO INFILTRATION RATE CONVERSION

CTE CAL
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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:	DRILLER:	SHEET: of
CTE JOB NO:	DRILL METHOD:	DRILLING DATE:
LOGGED BY:	SAMPLE METHOD:	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:	Ceres Gateway Center	DRILLER:	West Coast Exploration	SHEET:	1 of 1
CTE JOB NO:	25-0820G	DRILL METHOD:	4" Auger	DRILLING DATE	4/24/19
LOGGED BY:	A. Krause	SAMPLE METHOD	SPT	ELEVATION	EGS

BORING: B-1

Depth (Feet)	Bulk Sample	Driven Type	Blows/6 Inches	Dry Density (pcf)	Moisture (%)	U.S.C.S. Symbol	Graphic Log	DESCRIPTION	Laboratory Tests
0									
		X	3 5 5			SM		Medium dense, light brown, damp, silty fine SAND	
5		X	4 4 4			SM		As Above, Loose	
10		X	4 6 6			SM		As Above, Medium dense	
								Total depth = 11.5 feet No Free Ground water encountered Boring Grout Backfilled 4/24/19	
15									
20									
25									

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PROJECT:	Ceres Gateway Center	DRILLER:	West Coast Exploration	SHEET:	1 of 1
CTE JOB NO:	25-0820G	DRILL METHOD:	4" Auger	DRILLING DATE	4/24/19
LOGGED BY:	A. Krause	SAMPLE METHOD	SPT	ELEVATION	EGS

BORING: B-2

Depth (Feet)	Bulk Sample	Driven Type	Blows/6 Inches	Dry Density (pcf)	Moisture (%)	U.S.C.S. Symbol	Graphic Log	DESCRIPTION	Laboratory Tests
0									
			4 4 4			SM/ SP		Loose, light brown, damp, silty to poorly graded fine SAND	
5			5 5 6			SM		Medium dense, light brown silty fine -medium SAND	
10			7 7 10			SM		As Above	
								Total depth = 11.5 feet No Free Ground water encountered Boring Grout Backfilled 4/24/19	
15									
20									
25									

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PROJECT:	Ceres Gateway Center	DRILLER:	West Coast Exploration	SHEET:	1 of 1
CTE JOB NO:	25-0820G	DRILL METHOD:	4" Auger	DRILLING DATE	4/24/19
LOGGED BY:	A. Krause	SAMPLE METHOD	SPT	ELEVATION	EGS

BORING: B-3

Depth (Feet)	Bulk Sample	Driven Type	Blows/6 Inches	Dry Density (pcf)	Moisture (%)	U.S.C.S. Symbol	Graphic Log	DESCRIPTION	Laboratory Tests
0									
		X	3 3 4			SM		Loose, light brown, damp, silty fine SAND	
5		X	10 50			SM		As Above, Medium dense	
						ML		Hard, light tan, non-plastic SILT	
10		X	8 10 10			SM		Medium dense, orange brown to gray, silty fine-medium SAND	
								Total depth = 11.5 feet No Free Ground water encountered Boring Grout Backfilled 4/24/19	
15									
20									
25									

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PROJECT:	Ceres Gateway Center	DRILLER:	West Coast Exploration	SHEET:	1 of 1
CTE JOB NO:	25-0820G	DRILL METHOD:	4" Auger	DRILLING DATE	4/24/19
LOGGED BY:	A. Krause	SAMPLE METHOD	SPT	ELEVATION	EGS

BORING: B-4

Depth (Feet)	Bulk Sample / Driven Type	Blows/6 Inches	Dry Density (pcf)	Moisture (%)	U.S.C.S. Symbol	DESCRIPTION	Laboratory Tests
0							
	Driven	2 2 5			SM	Loose, light brown, damp, silty fine SAND	
5	Driven	5 6 6			SM	As Above, Medium dense	
10	Driven	30			SM	As Above, Medium dense	
		50			ML	Hard, light tan, non-plastic SILT	
						Total depth = 11.0 feet No Free Ground water encountered Boring Grout Backfilled 4/24/19	
15							
20							
25							

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PROJECT:	Ceres Gateway Center	DRILLER:	West Coast Exploration	SHEET:	1 of 1
CTE JOB NO:	25-0820G	DRILL METHOD:	4" Auger	DRILLING DATE	4/24/19
LOGGED BY:	A. Krause	SAMPLE METHOD	SPT	ELEVATION	EGS

BORING: B-5

Depth (Feet)	Bulk Sample	Driven Type	Blows/6 Inches	Dry Density (pcf)	Moisture (%)	U.S.C.S. Symbol	Graphic Log	DESCRIPTION	Laboratory Tests
0		X				SM		Medium dense, light brown, silty fine SAND	
5						SM		Same As Above	
10						SM		Medium dense, orange brown to gray, silty fine-medium SAND	
15						SM		Same As Above	
20						SM		Same As Above, locally interbedded with thin layers very fine sandy silt	
						SM		Same As Above	
25									

Total Depth=25.0; No Free Groundwater Encountered; Boring Backfilled 4/24/19

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PERCOLATION TEST DATA SHEET

PROJECT: Ceres Gateway	PROJECT No: 25-0820G	TEST DATE:	4/26/2019
TEST HOLE NO.: **P-1**	TESTED BY: A. Krause	DRILL DATE:	4/24/2019
DEPTH OF TEST HOLE (ft), Dt= 8'	USCS CLASSIFICATION: Med dense, light brown silty fine SAND (SM)		
DIAMETER (inches)= 6.0			

PRE SATURATION

Trial No.	Start Time	Stop Time	Time Interval (min)	Initial Depth of Water (in)	Final Depth of Water (in)	Change in Water Level (in)	Comments:
1	2:00 AM	10:54 AM	20 hr 54 min	12.00	0.00	12.00	Started Test, See Below
2	10:54 AM	11:24 AM	30	6.00	0.00	6.00	

TEST MEASUREMENTS

Trial No.	Start Time	Stop Time	Δt Time Interval (min)	Do Initial Depth of Water (in)	Df Final Depth of Water (in)	ΔD Change in Water Level (in)	Percolation Rate (min./in.)
1	11:24	11:34	10.0	6.00	4.25	1.75	5.71
2	11:34	11:44	10.0	6.00	4.50	1.50	6.67
3	11:44	11:54	10.0	6.00	4.50	1.50	6.67
4	11:54	12:04	10.0	6.00	4.50	1.50	6.67
5	12:04	12:14	10.0	6.00	4.50	1.50	6.67
6	12:14	12:24	10.0	6.00	4.50	1.50	6.67
7							
8							
9							
10							
11							
12							
13							
14							
15							
16							
17							
18							
19							
20							
21							
22							
23							

Comments: Final Steady Percolation Rate = 6.67 Min/Inch
See attachment for conversion to gal/sf per day

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PERCOLATION TEST DATA SHEET

PROJECT:	Ceres Gateway	PROJECT No:	25-0820G	TEST DATE:	4/26/2019
TEST HOLE NO.:	**P-2**	TESTED BY:	A. Krause	DRILL DATE:	4/24/2019
DEPTH OF TEST HOLE (ft), Dt=	8'	USCS CLASSIFICATION:	Med dense, light brown silty fine SAND (SM)		
DIAMETER (inches)=	6.0				

PRE SATURATION

Trial No.	Start Time	Stop Time	Time Interval (min)	Initial Depth of Water (in)	Final Depth of Water (in)	Change in Water Level (in)	Comments:
1	2:15 AM	10:47 AM	20 hr 32 min	12.00	0.00	12.00	Started Test, See Below
2	10:47 AM	11:05 AM	18	6.00	0.00	6.00	

TEST MEASUREMENTS

Trial No.	Start Time	Stop Time	Δt Time Interval (min)	Do Initial Depth of Water (in)	Df Final Depth of Water (in)	ΔD Change in Water Level (in)	Percolation Rate (min./in.)
1	11:05	11:15	10.0	6.00	0.00	6.00	1.67
2	11:15	11:25	10.0	6.00	0.00	6.00	1.67
3	11:25	11:35	10.0	6.00	0.25	5.75	1.74
4	11:35	11:45	10.0	6.00	0.25	5.75	1.74
5	11:45	11:55	10.0	6.00	0.25	5.75	1.74
6	11:55	12:05	10.0	6.00	0.25	5.75	1.74
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23							

Comments: Final Steady Percolation Rate = 1.74 Min/Inch

See attachment for conversion to gal/sf per day

CTE CAL, INC. 4230 Kiernan Ave., Ste. 150 | Modesto, CA 95356 | Ph: (209) 543 - 1799 | Fax: (209) 543-1775

PERCOLATION TEST DATA SHEET

PROJECT:	Ceres Gateway	PROJECT No:	25-0820G	TEST DATE:	4/26/2019
TEST HOLE NO.:	**P-3**	TESTED BY:	A. Krause	DRILL DATE:	4/24/2019
DEPTH OF TEST HOLE (ft), Dt=	8'	USCS CLASSIFICATION:	Med dense, light brown silty fine SAND (SM)		
DIAMETER (inches)=	6.0				

PRE SATURATION

Trial No.	Start Time	Stop Time	Time Interval (min)	Initial Depth of Water (in)	Final Depth of Water (in)	Change in Water Level (in)	Comments:
1	2:50 AM	10:30 AM	19 hr 40 min	12.00	0.00	12.00	Started Test, See Below
2	10:30 AM	11:00 AM	30	6.00	0.00	6.00	

TEST MEASUREMENTS

Trial No.	Start Time	Stop Time	Δt Time Interval (min)	Do Initial Depth of Water (in)	Df Final Depth of Water (in)	ΔD Change in Water Level (in)	Percolation Rate (min./in.)
1	11:00	11:10	10.0	6.00	2.50	3.50	2.86
2	11:10	11:20	10.0	6.00	3.50	2.50	4.00
3	11:20	11:30	10.0	6.00	3.50	2.50	4.00
4	11:30	11:40	10.0	6.00	3.50	2.50	4.00
5	11:40	11:50	10.0	6.00	3.50	2.50	4.00
6	11:50	12:00	10.0	6.00	3.50	2.50	4.00
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Comments: Final Steady Percolation Rate = 4.00 Min/Inch
See attachment for conversion to gal/sf per day

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PERCOLATION TEST DATA SHEET

PROJECT: Ceres Gateway	PROJECT No: 25-0820G	TEST DATE:	4/26/2019
TEST HOLE NO.: **P-4**	TESTED BY: A. Krause	DRILL DATE:	4/24/2019
DEPTH OF TEST HOLE (ft), Dt= 8'	USCS CLASSIFICATION: Med dense, light brown silty fine SAND (SM)		
DIAMETER (inches)= 6.0			

PRE SATURATION

Trial No.	Start Time	Stop Time	Time Interval (min)	Initial Depth of Water (in)	Final Depth of Water (in)	Change in Water Level (in)	Comments:
1	3:00 PM	10:07 AM	19 hr 7 min	12.00	0.00	12.00	Started Test, See Below
2	10:07 AM	10:37 AM	30	6.00	0.00	6.00	

TEST MEASUREMENTS

Trial No.	Start Time	Stop Time	Δt Time Interval (min)	Do Initial Depth of Water (in)	Df Final Depth of Water (in)	ΔD Change in Water Level (in)	Percolation Rate (min./in.)
1	10:37	10:47	10.0	6.00	4.00	2.00	5.00
2	10:47	10:57	10.0	6.00	3.75	2.25	4.44
3	10:57	11:07	10.0	6.00	4.00	2.00	5.00
4	11:07	11:17	10.0	6.00	4.25	1.75	5.71
5	11:17	11:30	10.0	6.00	4.75	1.25	8.00
6	11:30	11:40	10.0	6.00	5.00	1.00	10.00
7	11:40	11:50	10.0	6.00	4.50	1.50	6.67
8	11:50	12:00	10.0	6.00	5.00	1.00	10.00
9	12:00	12:10	10.0	6.00	5.00	1.00	10.00
10	12:10	12:20	10.0	6.00	5.00	1.00	10.00
11	12:20	12:30	10.0	6.00	5.00	1.00	10.00
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Comments: Final Steady Percolation Rate = 10.00 Min/Inch
See attachment for conversion to gal/sf per day

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PERCOLATION TEST DATA SHEET

PROJECT:	Ceres Gateway	PROJECT No:	25-0820G	TEST DATE:	4/26/2019
TEST HOLE NO.:	**P-5**	TESTED BY:	A. Krause	DRILL DATE:	4/24/2019
DEPTH OF TEST HOLE (ft), Dt=	8'	USCS CLASSIFICATION:	Med dense, light brown silty fine SAND (SM)		
DIAMETER (inches)=	6.0				

PRE SATURATION

Trial No.	Start Time	Stop Time	Time Interval (min)	Initial Depth of Water (in)	Final Depth of Water (in)	Change in Water Level (in)	Comments:
1	3:20 AM	10:00 AM	18 hr 40 min	12.00	0.00	12.00	Started Test, See Below
2	10:00 AM	10:30 AM	30	6.00	0.00	6.00	

TEST MEASUREMENTS

Trial No.	Start Time	Stop Time	Δt Time Interval (min)	Do Initial Depth of Water (in)	Df Final Depth of Water (in)	ΔD Change in Water Level (in)	Percolation Rate (min./in.)
1	10:30	10:40	10.0	6.00	4.50	1.50	6.67
2	10:40	10:50	10.0	6.00	4.50	1.50	6.67
3	10:50	11:00	10.0	6.00	5.00	1.00	10.00
4	11:00	11:10	10.0	6.00	4.25	1.75	5.71
5	11:10	11:20	10.0	6.00	5.25	0.75	13.33
6	11:20	11:30	10.0	6.00	4.25	1.75	5.71
7	11:35	11:45	10.0	6.00	5.00	1.00	10.00
	11:45	11:55	10.0	6.00	5.00	1.00	10.00
9	11:55	12:05	10.0	6.00	5.00	1.00	10.00
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Comments: Final Steady Percolation Rate = 10.00 Min/Inch
See attachment for conversion to gal/sf per day

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PERCOLATION TEST DATA SHEET

PROJECT: Ceres Gateway	PROJECT No: 25-0820G	TEST DATE:	4/26/2019
TEST HOLE NO.: **P-6**	TESTED BY: A. Krause	DRILL DATE:	4/24/2019
DEPTH OF TEST HOLE (ft), Dt= 8'	USCS CLASSIFICATION: Med dense, light brown silty fine SAND (SM)		
DIAMETER (inches)= 6.0			

PRE SATURATION

Trial No.	Start Time	Stop Time	Time Interval (min)	Initial Depth of Water (in)	Final Depth of Water (in)	Change in Water Level (in)	Comments:
1	4:00 AM	9:30 AM	17 hr 30 min	12.00	0.00	12.00	Started Test, See Below
2	9:54 AM	10:24 AM	30	6.00	0.00	6.00	

TEST MEASUREMENTS

Trial No.	Start Time	Stop Time	Δt Time Interval (min)	Do Initial Depth of Water (in)	Df Final Depth of Water (in)	ΔD Change in Water Level (in)	Percolation Rate (min./in.)
1	10:24	10:34	10.0	6.00	4.50	1.50	6.67
2	10:34	10:44	10.0	6.00	4.50	1.50	6.67
3	10:44	10:54	10.0	6.00	4.00	2.00	5.00
4	10:55	11:05	10.0	6.00	5.00	1.00	10.00
5	11:05	11:15	10.0	6.00	5.00	1.00	10.00
6	11:15	11:25	10.0	6.00	5.00	1.00	10.00
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Comments: Final Steady Percolation Rate = 10.00 Min/Inch
See attachment for conversion to gal/sf per day

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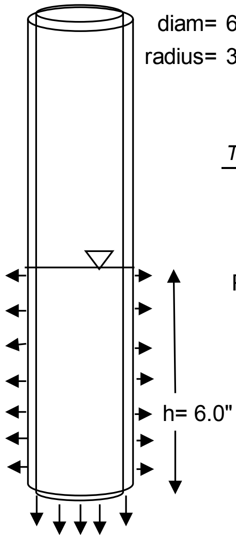
CTE# 25-0820G

INFILTRATION RATE PER PORCHET METHOD

Reference: "Riverside County-Low Impact Development BMP Design Handbook" (Page 20)

diam= 6"
radius= 3"



Percolation Data at the Final Interval

Test No.	*Time Interval (Δt)*	*Initial Depth of Water in inches (D_o)*	*Final Depth of Water in inches (D_f)*	*Change in Height of Water in inches (ΔH)*	*Average Head Over Time Interval in inches (H_{avg})*
P1:	10.00	6.00	4.50	1.50	5.25
P2:	10.00	6.00	0.25	5.75	3.13
P3:	10.00	6.00	3.50	2.50	4.75
P4,P5,P6:	10.00	6.00	5.00	1.00	5.50

Infiltration Rate $It=(\Delta H\ 60\ r)/\Delta t(r+2H_{avg})$

P1:	It = (1.50 in)(60 min/hr)(3 in) / (10 min) (3 in + 2(5.25 in))=	2.00	in/hr
P2:	It = (5.75 in)(60 min/hr)(3 in) / (10 min) (3 in + 2(3.13 in))=	11.19	in/hr
P3:	It = (2.50 in)(60 min/hr)(3 in) / (10 min) (3 in + 2(4.75 in))=	3.60	in/hr
P4,P5,P6:	It = (1.00 in)(60 min/hr)(3 in) / (10 min) (3 in + 2(5.50 in))=	1.29	in/hr

Infiltration Rate in gal/sf/day = (It in/hr)(24 hr/day)(7.48 gal / cf)(ft/12 in)

P1 =	29.92	gal/sf/day
P2 =	167.39	gal/sf/day
P3 =	53.86	gal/sf/day
P4,P5,P6 =	19.23	gal/sf/day

APPENDIX C

LABORATORY METHODS AND RESULTS

CTE CAL, INC. 3628 Madison Avenue Ste. 22 | Sacramento, CA 95660 | Ph: (916) 331-6030 | Fax: (916) 331-6037

Material Finer than #200 Sieve

ASTM D-1140

Project Name: Ceres Gateway

Date Received: 4/26/2019

Project #: 25-0820G

Sampled By: Alan

Lab #: 4912

Sample ID:	B1	B1	B2	B2	B3	B3	B4	B4		
Depth:	1'	5'	1'	5'	1'	5'	1'	5'		
Classification	SM	SM	SP-SM	SP-SM	SM	SM	SM	SM		
Wet Weight	294.3	302.5	279.9	286.5	298.5	311.5	291.4	282.8		
Dry Weight (Before Wash)	279.5	285.0	266.7	268.2	277.2	278.5	277.6	265.5		
Dry Weight (After Wash)	230.7	238.9	237.2	237.7	209.0	181.9	226.7	201.2		
Tare	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0		
Soil Loss	48.8	46.1	29.5	30.5	68.2	96.6	50.9	64.3		
Moisture %	5.3%	6.1%	4.9%	6.8%	7.7%	11.8%	5.0%	6.5%		
Percent Passing # 200 Sieve	17.5%	16.2%	11.1%	11.4%	24.6%	34.7%	18.3%	24.2%		

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CIVIL ENGINEERING | GEOTECHNICAL | ENVIRONMENTAL | CONSTRUCTION INSPECTION AND TESTING

MOISTURE & DENSITY TEST

Client: Ceres Gateway Crt LLC

Sample Date: 4/26/2019

Project Name: Ceres Gateway

Lab Number: 4912

Project Number: 25-0820G

Sample No.	B1	B1	B2	B2
DEPTH FT	1'	5'	1'	5'
SAMPLE HT	6.00	6.00	6.00	5.95
TUBE DIA.	1.41	1.41	1.41	1.41
SOIL+RING	423.2	423.5	409.1	408.5
RING	128.9	121.0	129.2	122.0
SOIL WT., g	294.3	302.5	279.9	286.5
SOIL, LB	0.64881	0.66689	0.61706	0.63161
VOL. SOIL	0.00542	0.00542	0.00542	0.00538
WET DENS	119.7	123.0	113.8	117.5
SOIL WET	294.3	302.5	279.9	286.5
SOIL DRY	279.5	285.0	266.7	268.2
% MOIST	5.3%	6.1%	4.9%	6.8%
DRY DENS	113.7	115.9	108.4	110.0

Reviewed By: Kristin Kohls

Date: May 9, 2019

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CIVIL ENGINEERING | GEOTECHNICAL | ENVIRONMENTAL | CONSTRUCTION INSPECTION AND TESTING

MOISTURE & DENSITY TEST

Client: Ceres Gateway Crt LLC **Sample Date:** 4/26/2019

Project Name: Ceres Gateway **Lab Number:** 4912

Project Number: 25-0820G

Sample No.	B3	B3	B4	B4
DEPTH FT	1'	5'	1'	5'
SAMPLE HT	5.95	6.00	6.00	5.95
TUBE DIA.	1.41	1.41	1.41	1.41
SOIL+RING	420.6	432.5	412.2	410.2
RING	122.1	121.0	120.8	127.4
SOIL WT., g	298.5	311.5	291.4	282.8
SOIL, LB	0.65807	0.68673	0.64242	0.62346
VOL. SOIL	0.00538	0.00542	0.00542	0.00538
WET DENS	122.4	126.7	118.5	116.0
SOIL WET	298.5	311.5	291.4	282.8
SOIL DRY	277.2	278.5	277.6	265.5
% MOIST	7.7%	11.8%	5.0%	6.5%
DRY DENS	113.7	113.2	112.9	108.9

Reviewed By: Kristin Kohls Date: May 9, 2019

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CONSTRUCTION INSPECTION AND TESTING | GEOTECHNICAL | ENVIRONMENTAL | CIVIL ENGINEERING

REPORT OF RESISTANCE 'R' VALUE-EXPANSION PRESSURE

Job Name: Ceres Gateway
Job No. 25-0820G
Lab No. 4912
Sample No. B5 0-2'

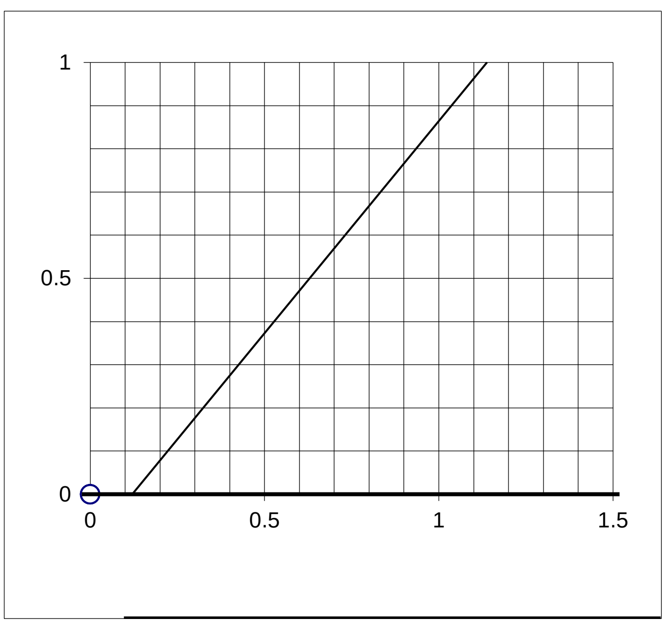
Date: 4/26/2019
Submitted By: Alan
Tested/ Calc.By: kk
Type of Material: Fine Silty Sand

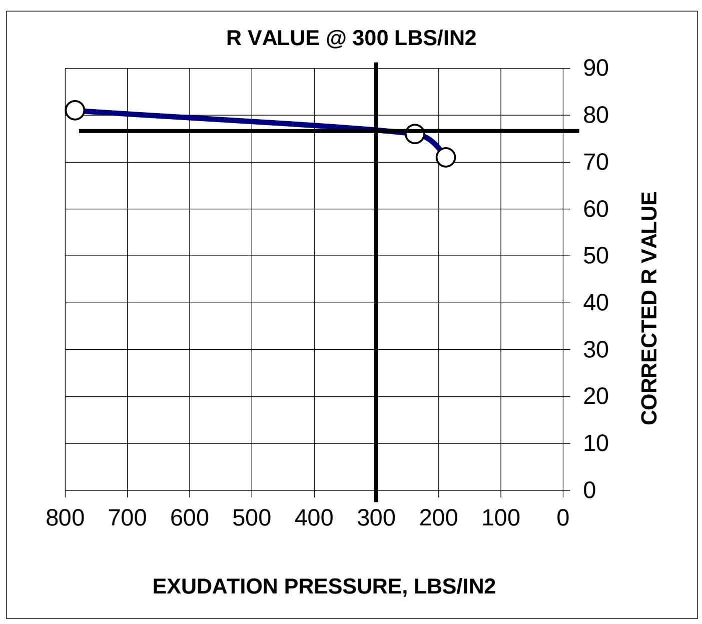
Test Procedure: ASTM D2844

Specimen/ Mold No.	209	I	14
Compactor Air Pressure, - ft.lbs.	350	350	350
Initial Moisture, - %	1.5%	1.5%	1.5%
Sample Size - g	1200	1200	1200
Water Added, - ml	84	96	108
Moisture at Compaction, - %	8.5%	9.5%	10.5%
Wt. Of Briquette and Mold, - g	3238.9	3189.1	3291.4
Wt. Of Mold, - g	2090	2095	2097.1
Wt. Of Briquitte, - g	1148.9	1094.1	1194.3
Height of Briquette, - in	2.55	2.49	2.7
Dry Density, - pcf	125.7	121.5	121.2
Stabilometer PH @ 2000 lbs	22	25	29
Displacement	3.54	4.2	4.9
R' Value	81	76	69
Corrected 'R' Value	81	76	71
Exudation Pressure, - lbs	9800	2970	2353
Exudation Pressure, - psi	784	238	188
Stabilometer Thickness - ft	0	0	0
Expansion - in.	0	0	0
Expansion Pressure - Pascals	0.00	0.00	0.00
Expansion Press, Thick-ft	0	0	0

R-value 78

TI	
Expansion	





Cover Thickness by Expansion Pressure-Feet

Expansion From Graph:	

APPENDIX D

STANDARD SPECIFICATIONS FOR GRADING

Section 1 - General

CTE, Cal, Inc. (CTE) 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

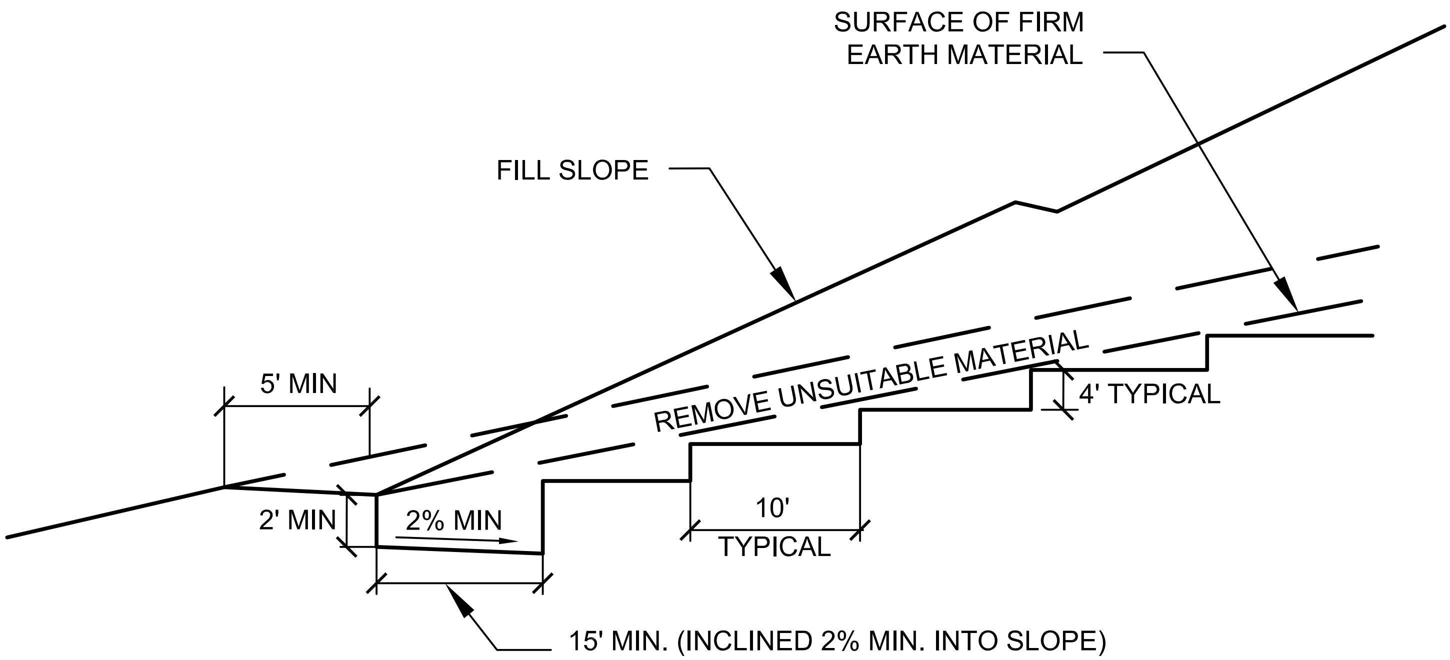
As a precautionary measure, plastic sheeting should be readily available, or kept on hand, to protect all slope areas from saturation by periods of heavy or prolonged rainfall. This measure is strongly recommended, beginning with the period prior to landscape planting.

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.

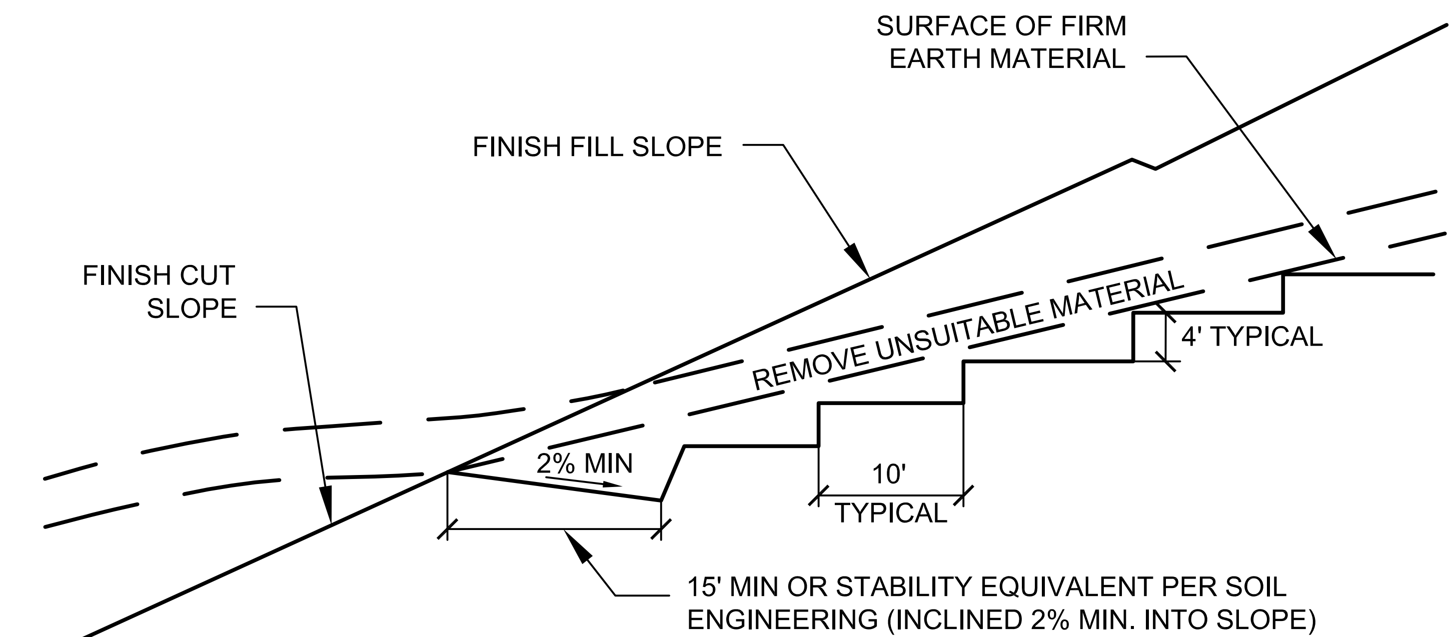
If slope failures occur as a result of exposure to period of heavy rainfall, the failure areas and currently unaffected areas should be covered with plastic sheeting to protect against additional saturation.

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

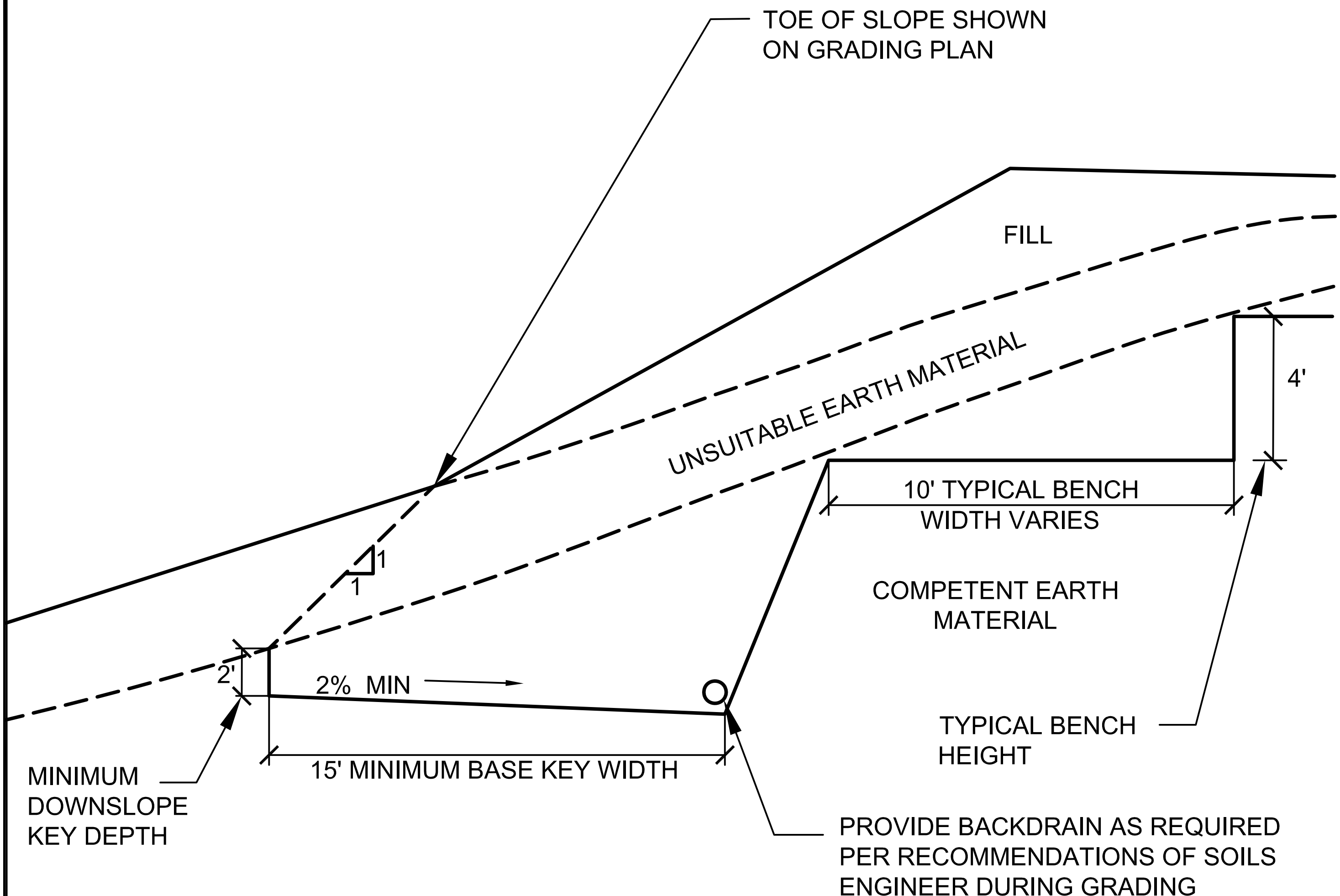


BENCHING FILL OVER CUT



NOT TO SCALE

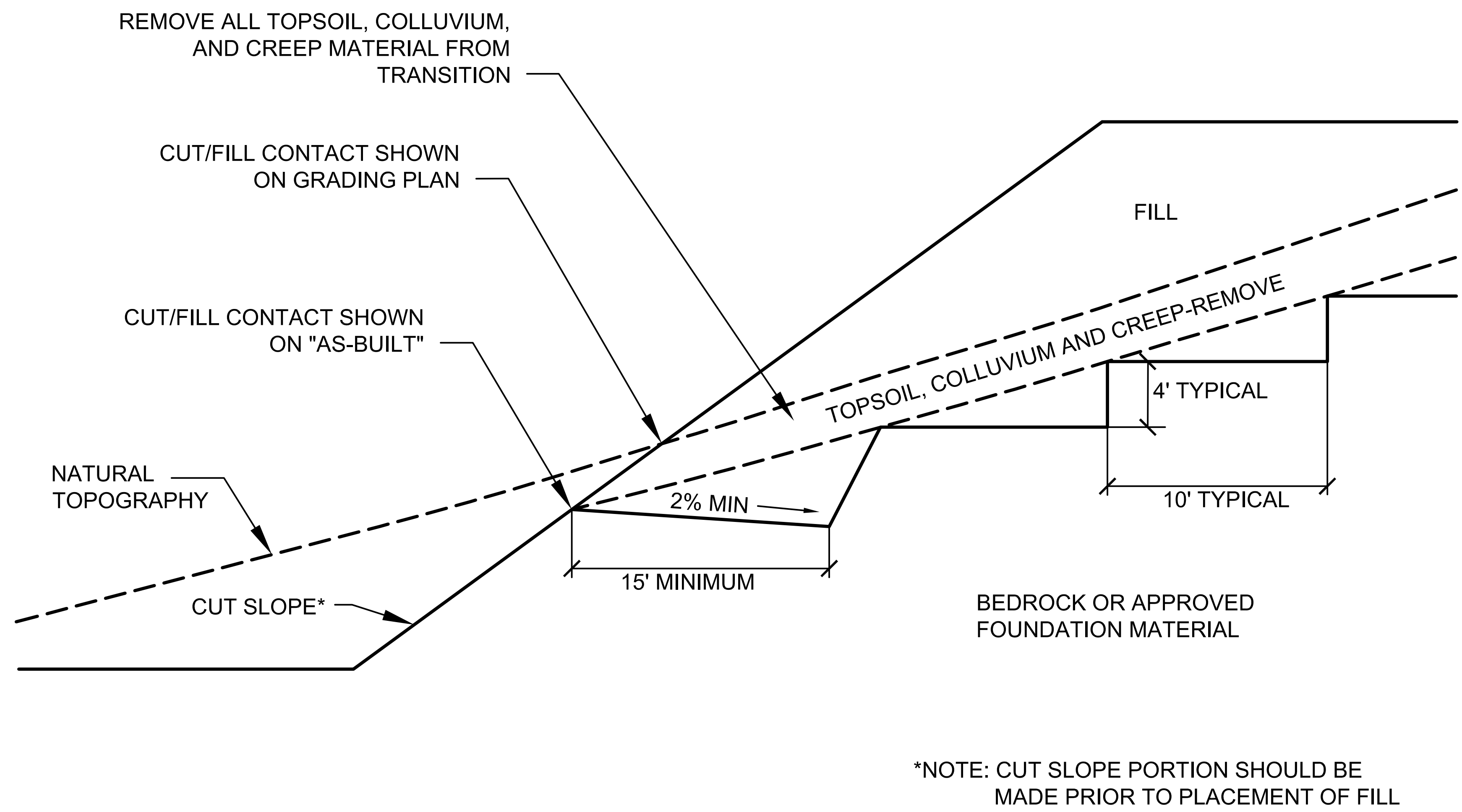
BENCHING FOR COMPACTED FILL DETAIL



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

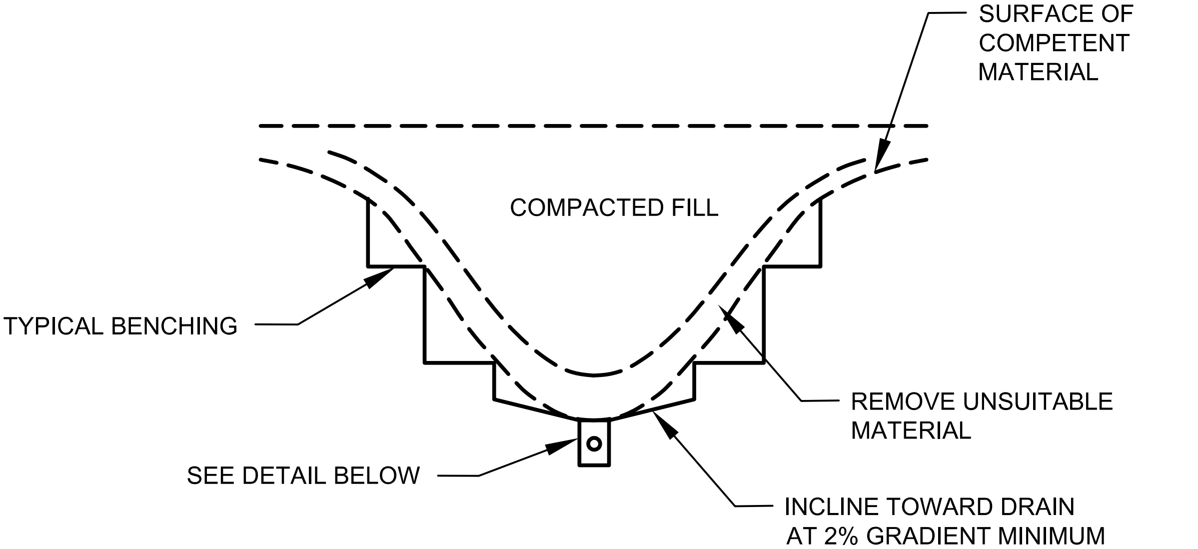
NOT TO SCALE

FILL SLOPE ABOVE NATURAL GROUND DETAIL

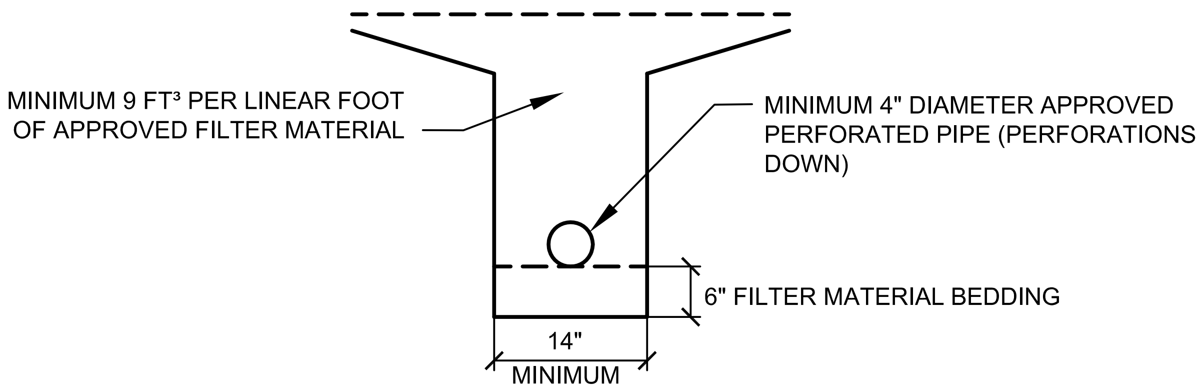


NOT TO SCALE

FILL SLOPE ABOVE CUT SLOPE DETAIL



DETAIL



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. 30	18-33
NO. 8	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

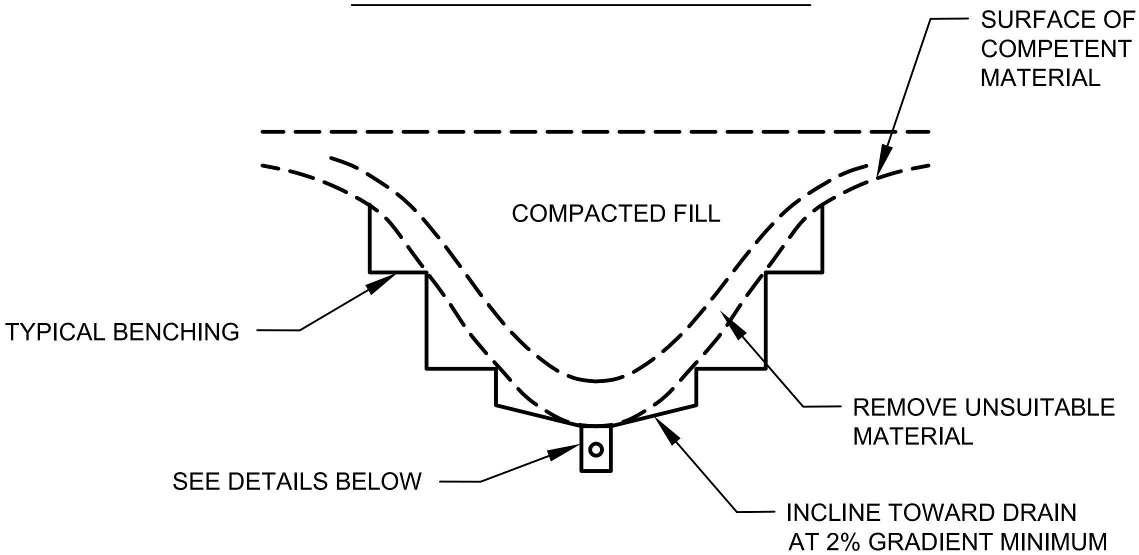
PIPE DIAMETER TO MEET THE FOLLOWING CRITERIA, SUBJECT TO FIELD REVIEW BASED ON ACTUAL GEOTECHNICAL CONDITIONS ENCOUNTERED DURING GRADING

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

NOT TO SCALE

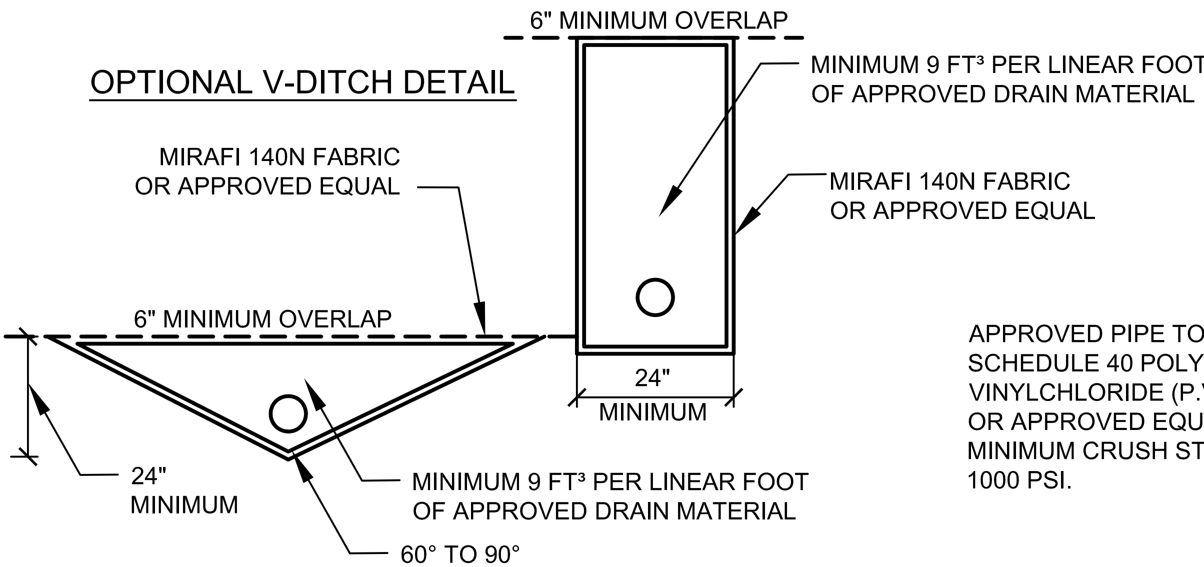
TYPICAL CANYON SUBDRAIN DETAIL

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

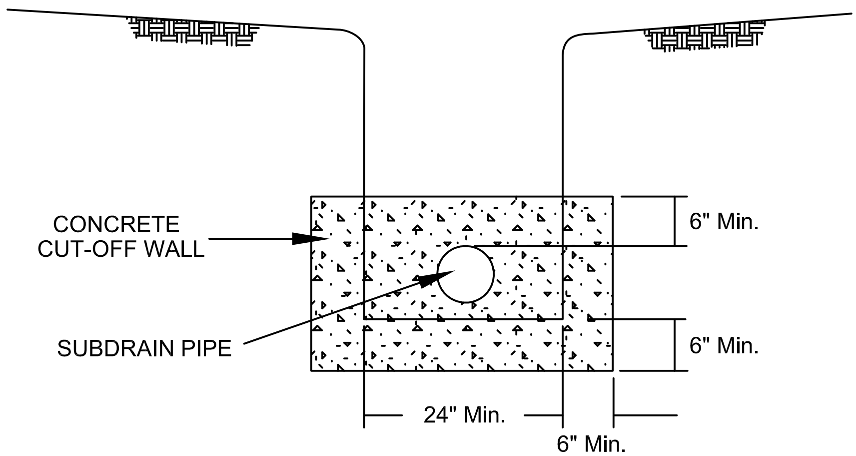
PIPE DIAMETER TO MEET THE FOLLOWING CRITERIA, SUBJECT TO FIELD REVIEW BASED ON ACTUAL GEOTECHNICAL CONDITIONS ENCOUNTERED DURING GRADING

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

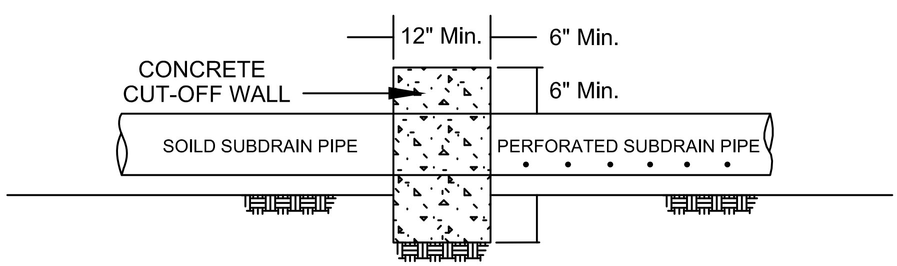
NOT TO SCALE

GEOFABRIC SUBDRAIN

FRONT VIEW



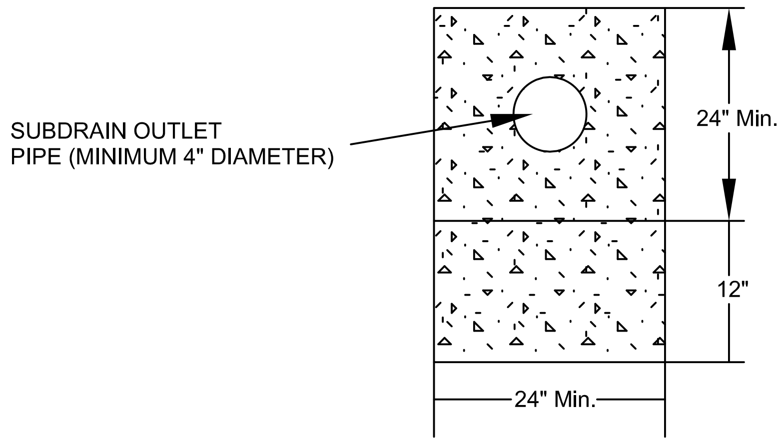
SIDE VIEW



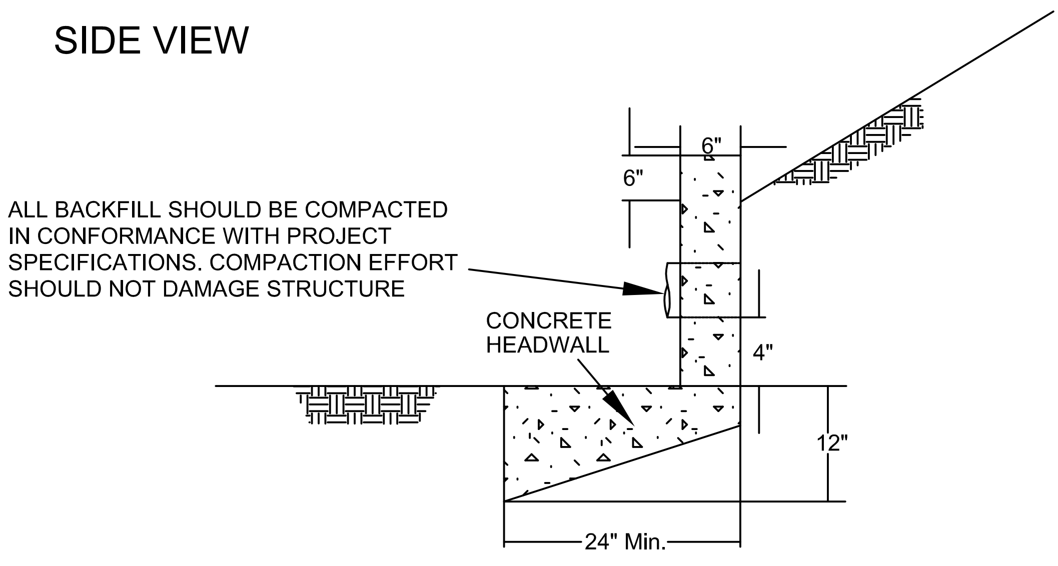
NOT TO SCALE

RECOMMENDED SUBDRAIN CUT-OFF WALL

FRONT VIEW



SIDE VIEW



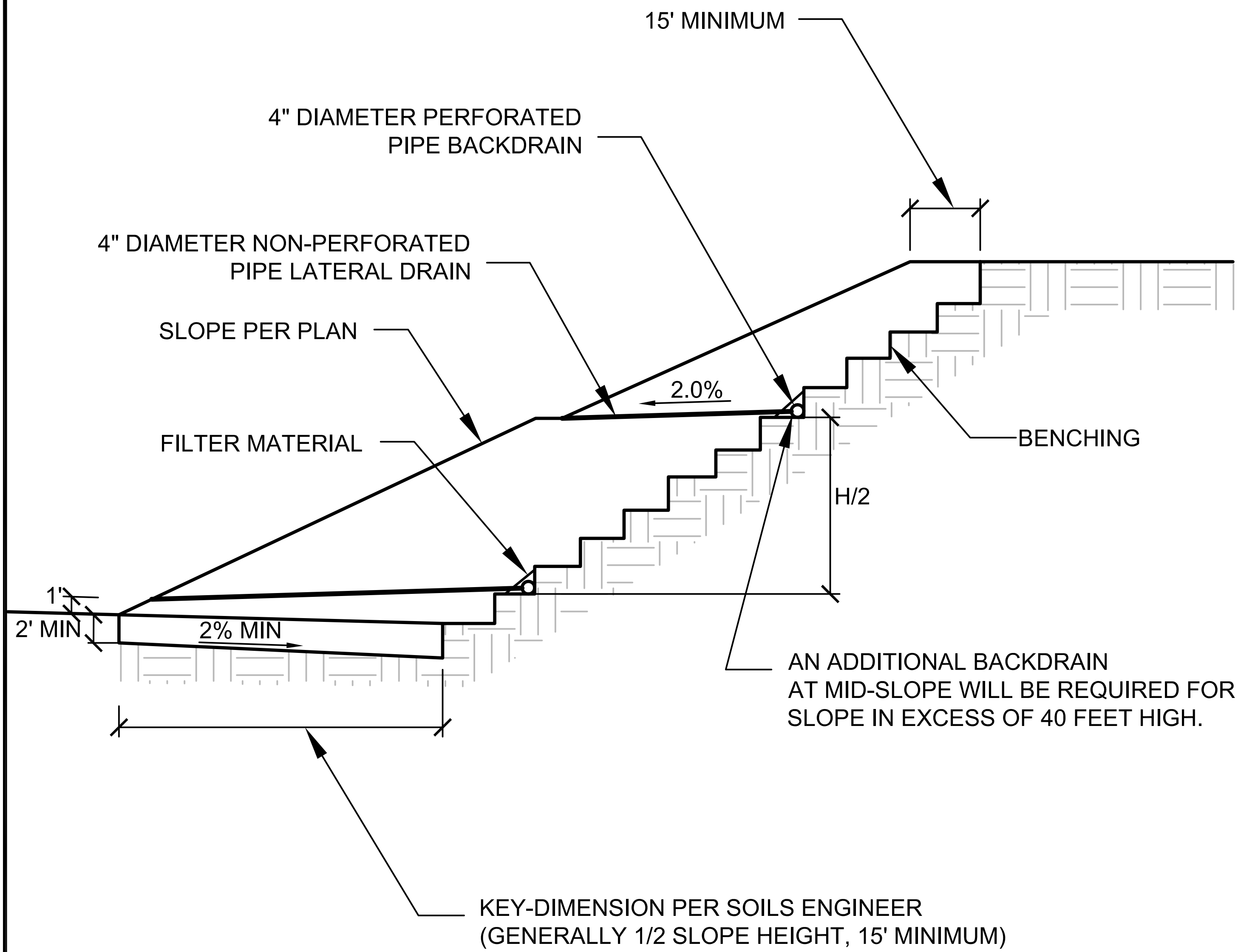
NOTE: HEADWALL SHOULD OUTLET AT TOE OF SLOPE OR INTO CONTROLLED SURFACE DRAINAGE DEVICE

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

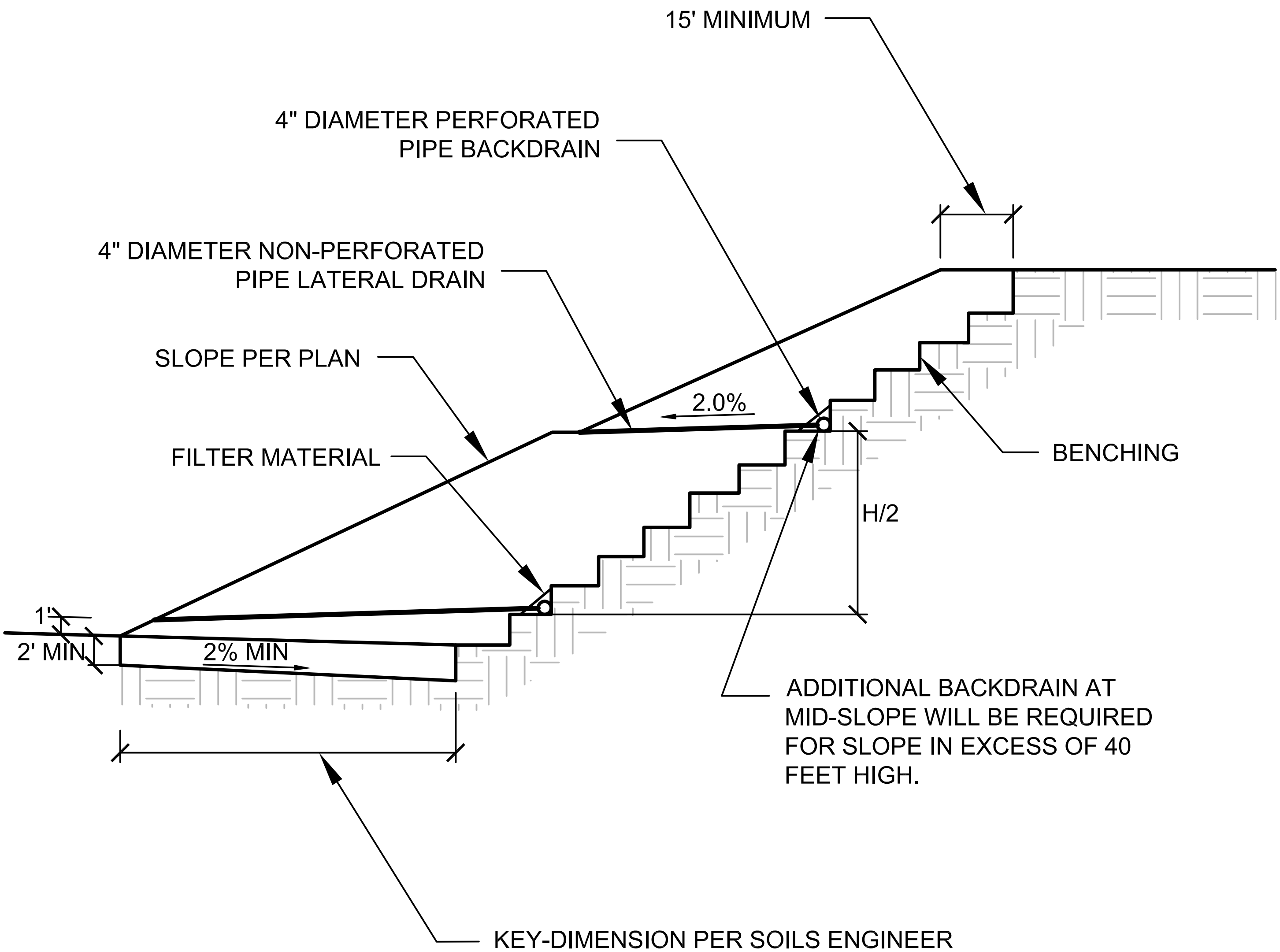
TYPICAL SUBDRAIN OUTLET HEADWALL DETAIL



DIMENSIONS ARE MINIMUM RECOMMENDED

NOT TO SCALE

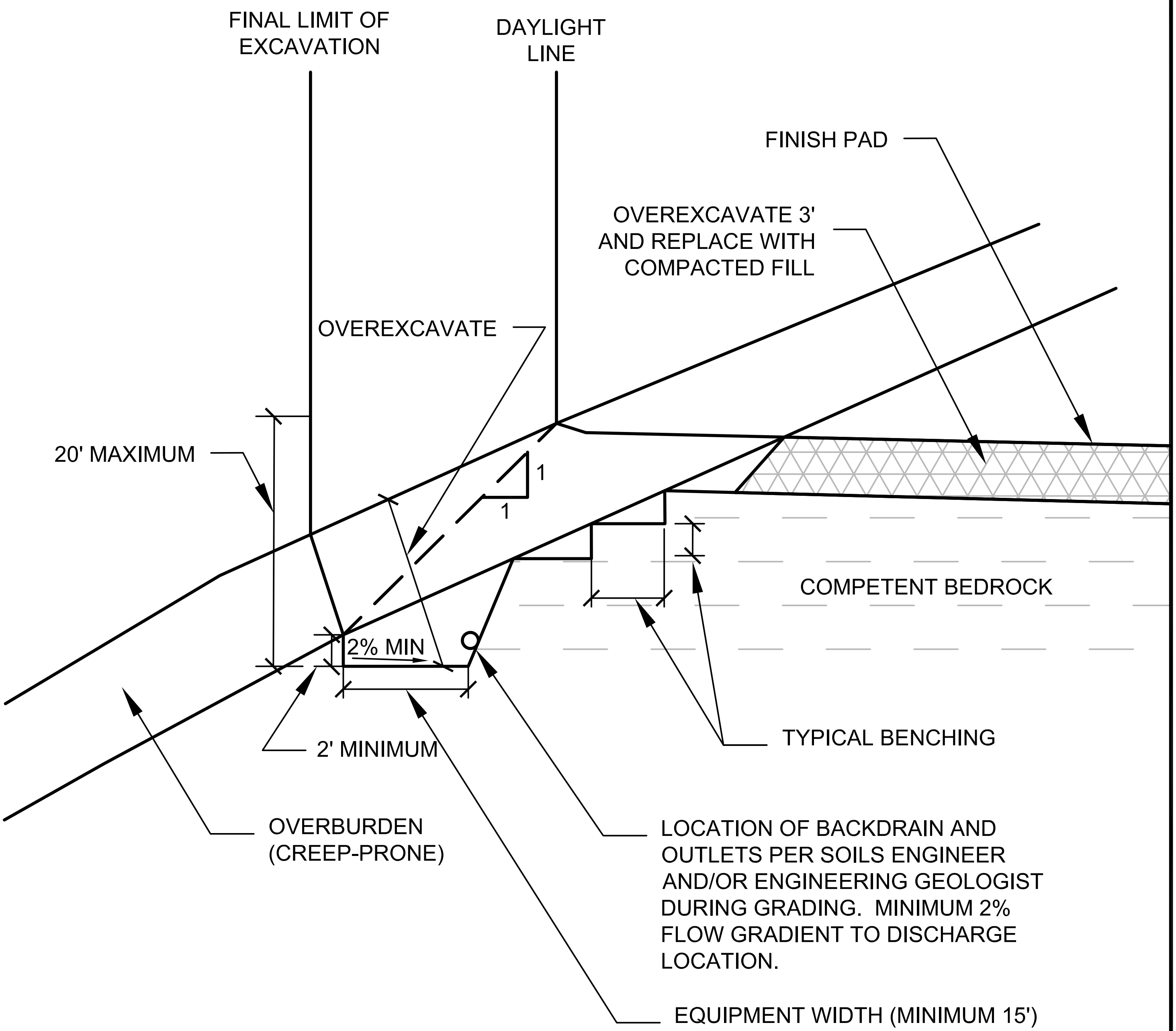
TYPICAL SLOPE STABILIZATION FILL DETAIL



DIMENSIONS ARE MINIMUM RECOMMENDED

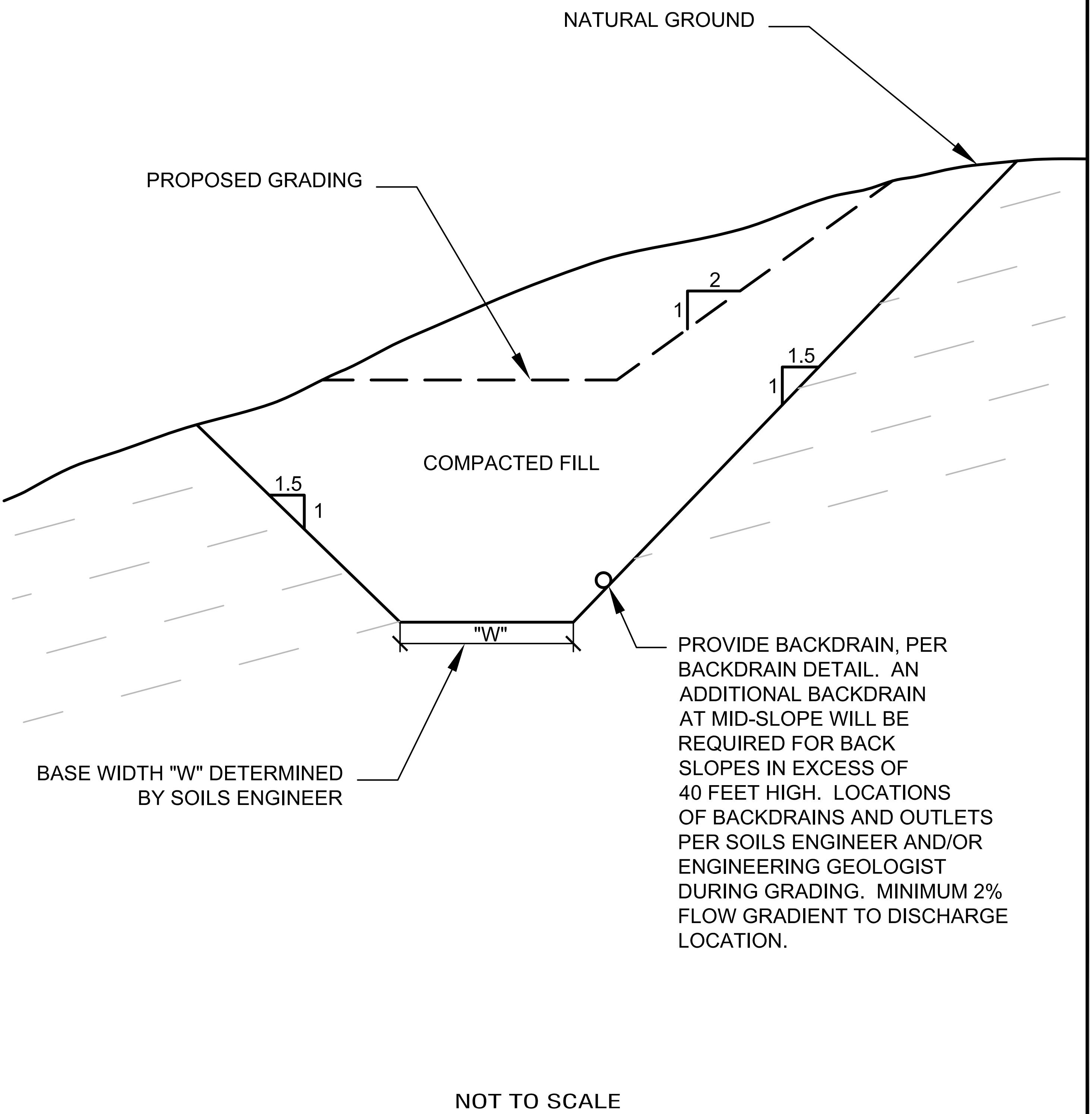
NOT TO SCALE

TYPICAL BUTTRESS FILL DETAIL

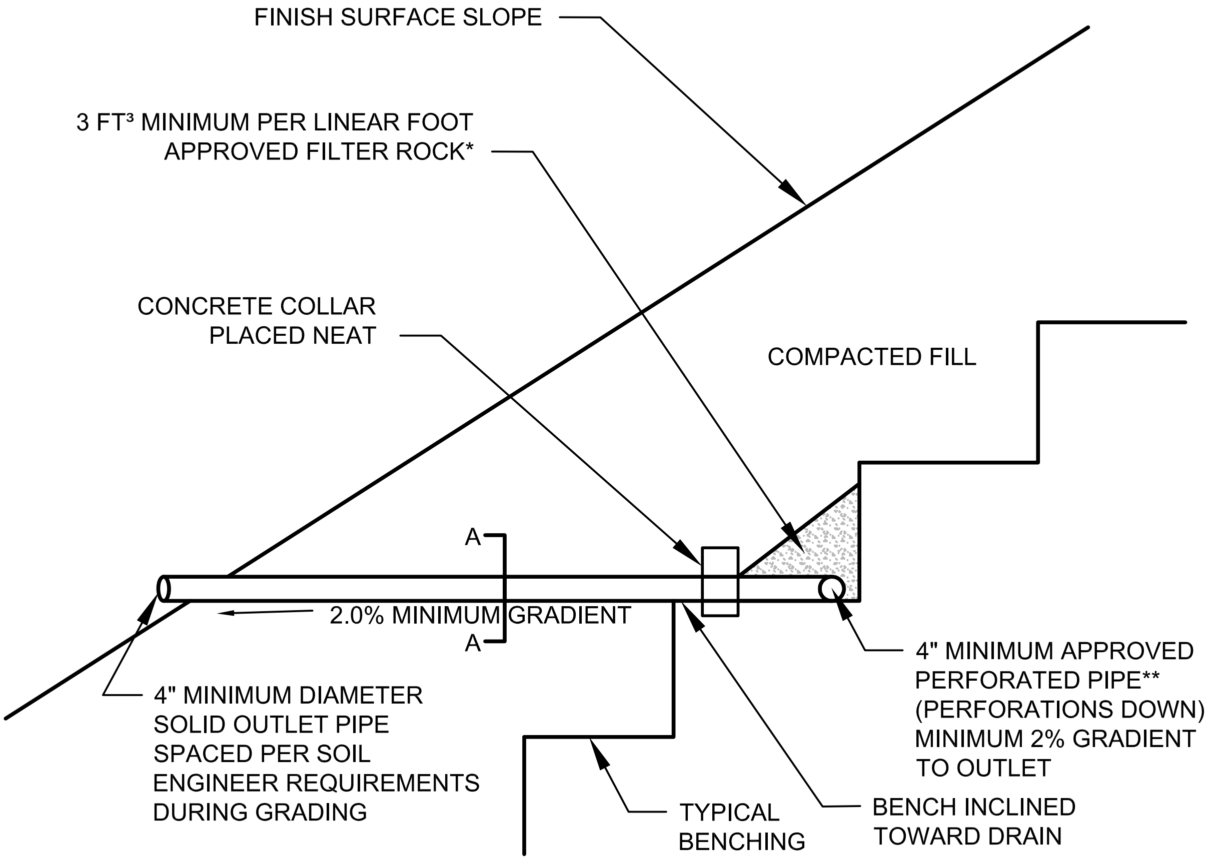


NOT TO SCALE

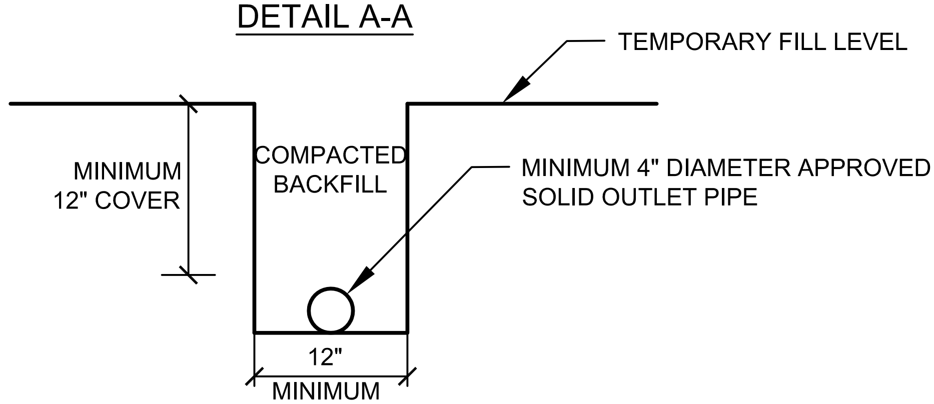
DAYLIGHT SHEAR KEY DETAIL



TYPICAL SHEAR KEY DETAIL



DETAIL A-A



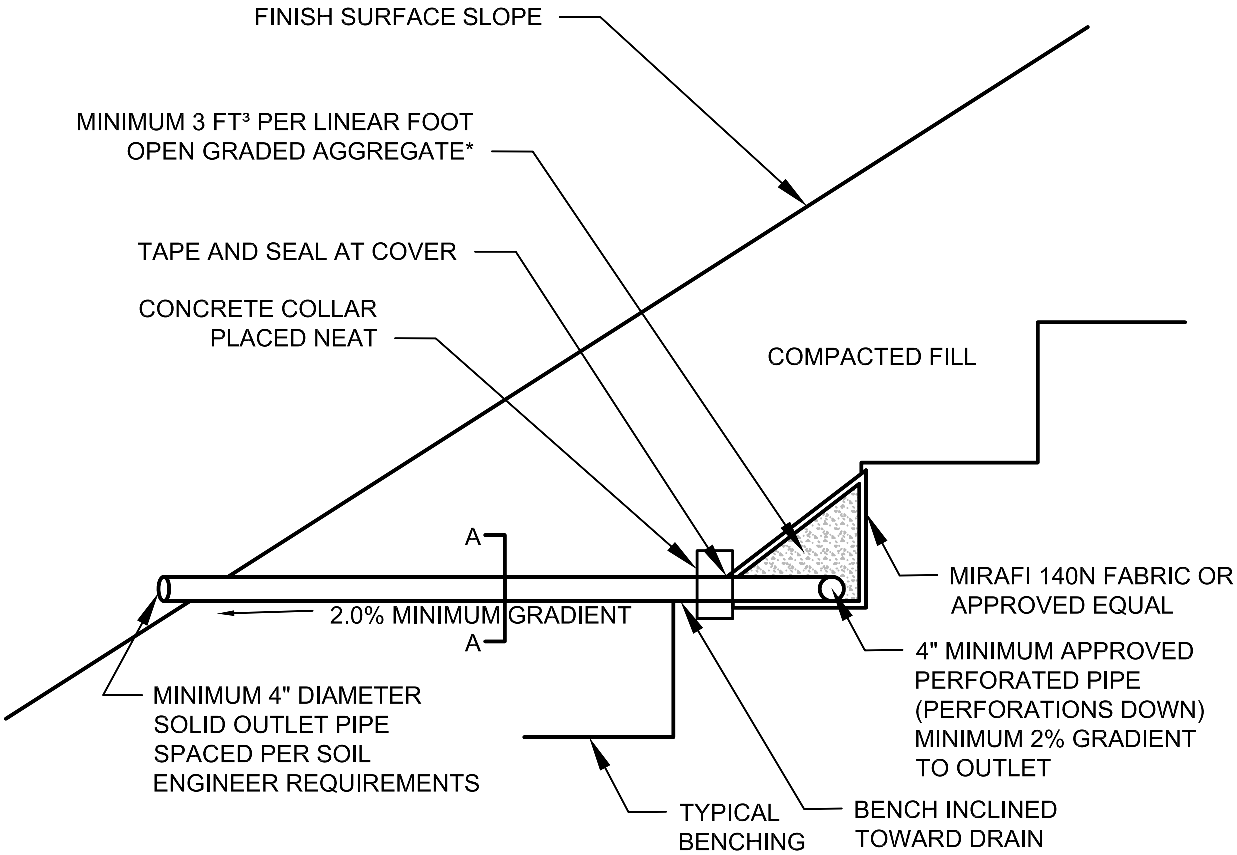
*FILTER ROCK TO MEET FOLLOWING SPECIFICATIONS OR APPROVED EQUAL:

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

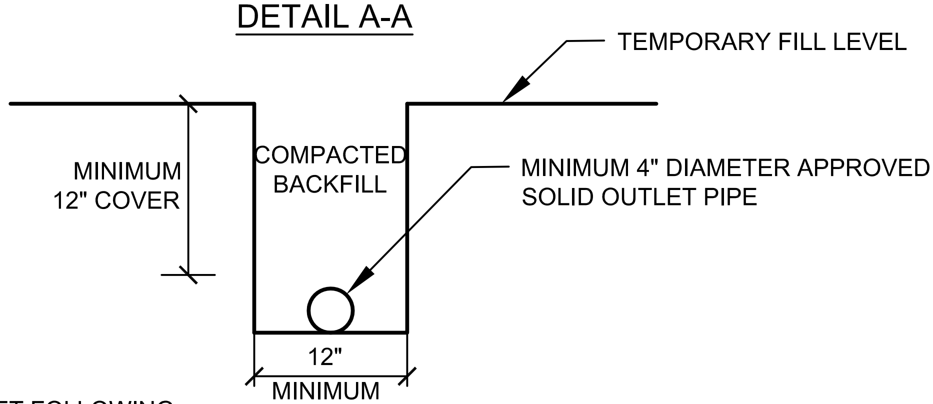
**APPROVED PIPE TYPE:
SCHEDULE 40 POLYVINYL CHLORIDE (P.V.C.) OR APPROVED EQUAL.
MINIMUM CRUSH STRENGTH 1000 PSI

NOT TO SCALE

TYPICAL BACKDRAIN DETAIL



DETAIL A-A

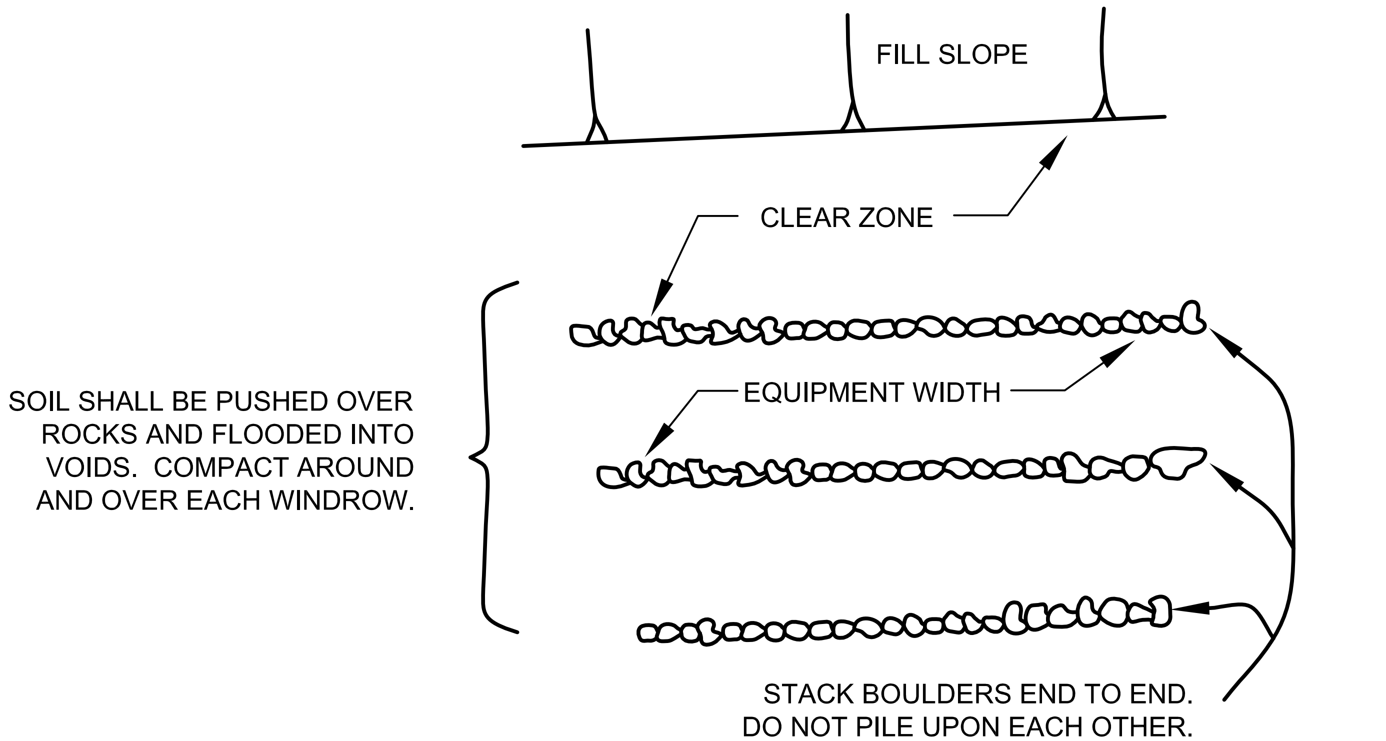


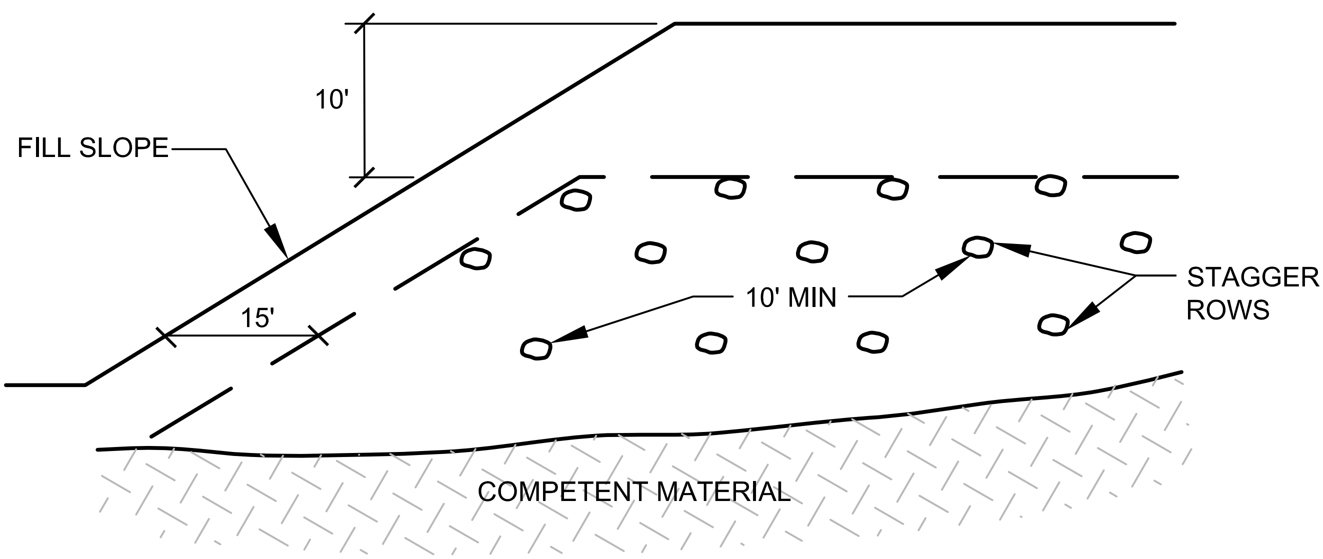
*NOTE: AGGREGATE TO MEET FOLLOWING
SPECIFICATIONS OR APPROVED EQUAL:

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

NOT TO SCALE

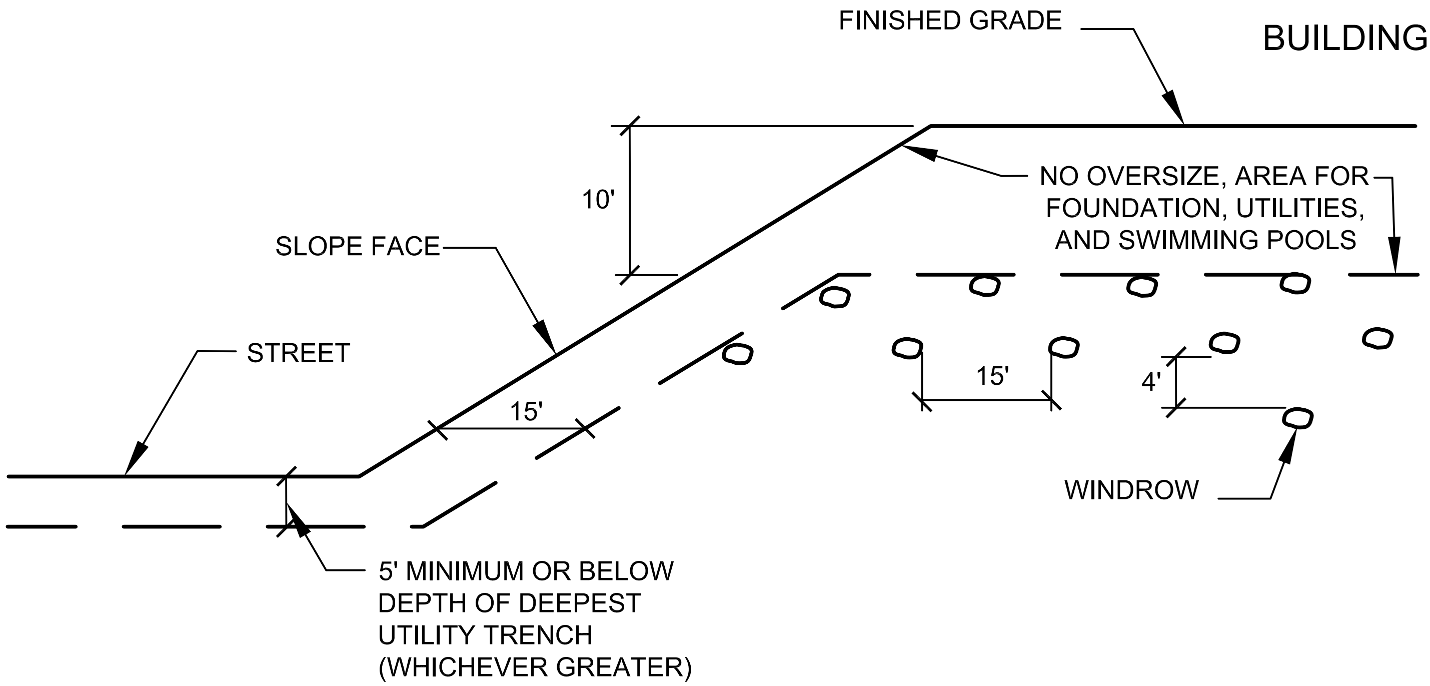
BACKDRAIN DETAIL (GEOFRABIC)



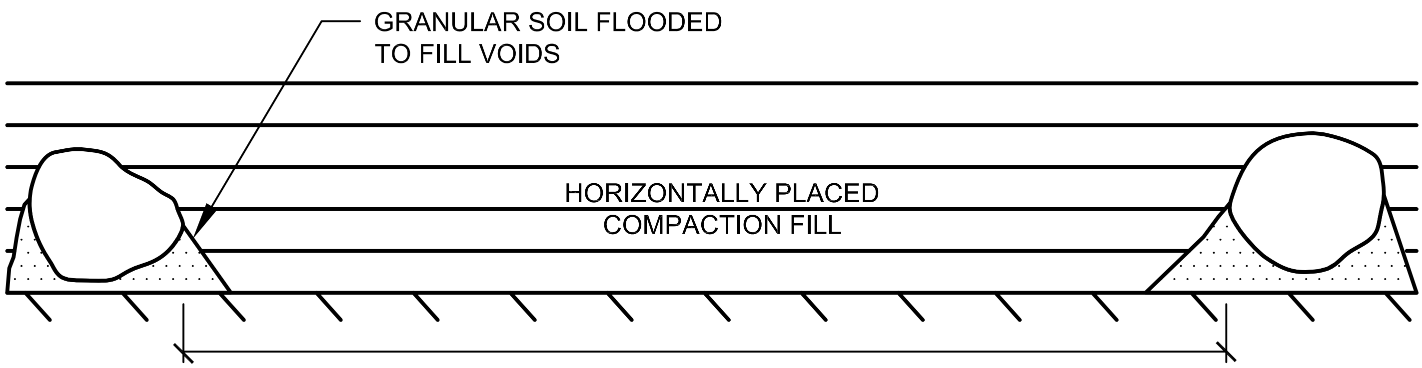


NOT TO SCALE

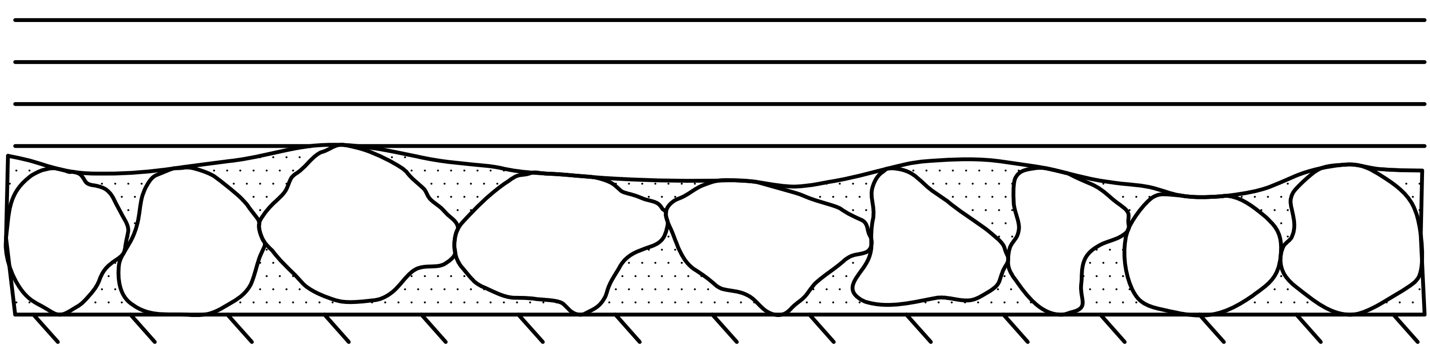
ROCK DISPOSAL DETAIL



TYPICAL WINDROW DETAIL (EDGE VIEW)



PROFILE VIEW

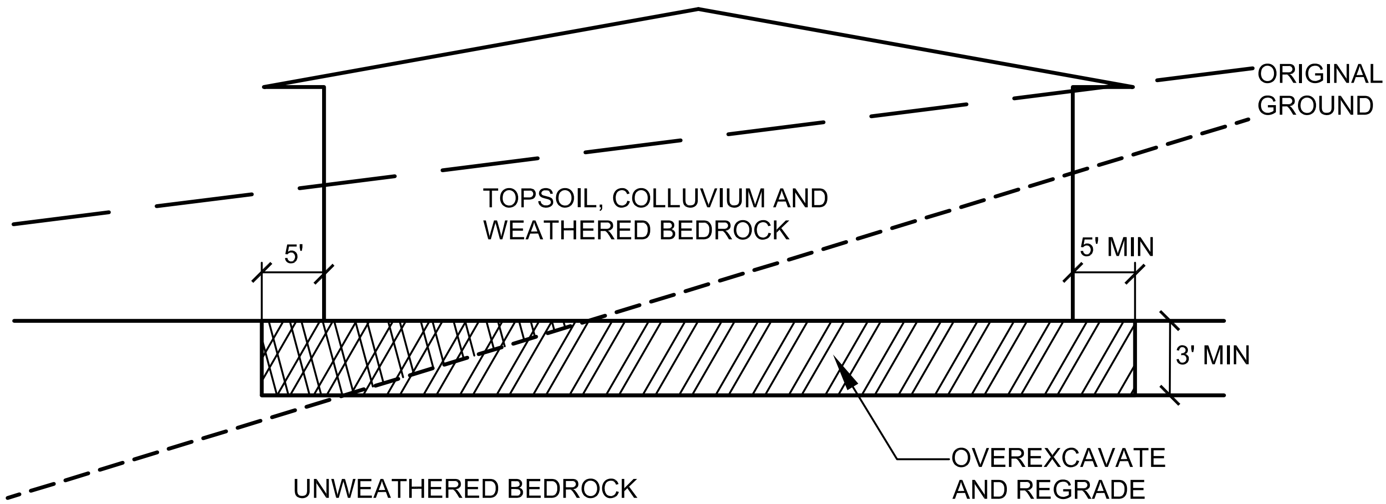


NOT TO SCALE

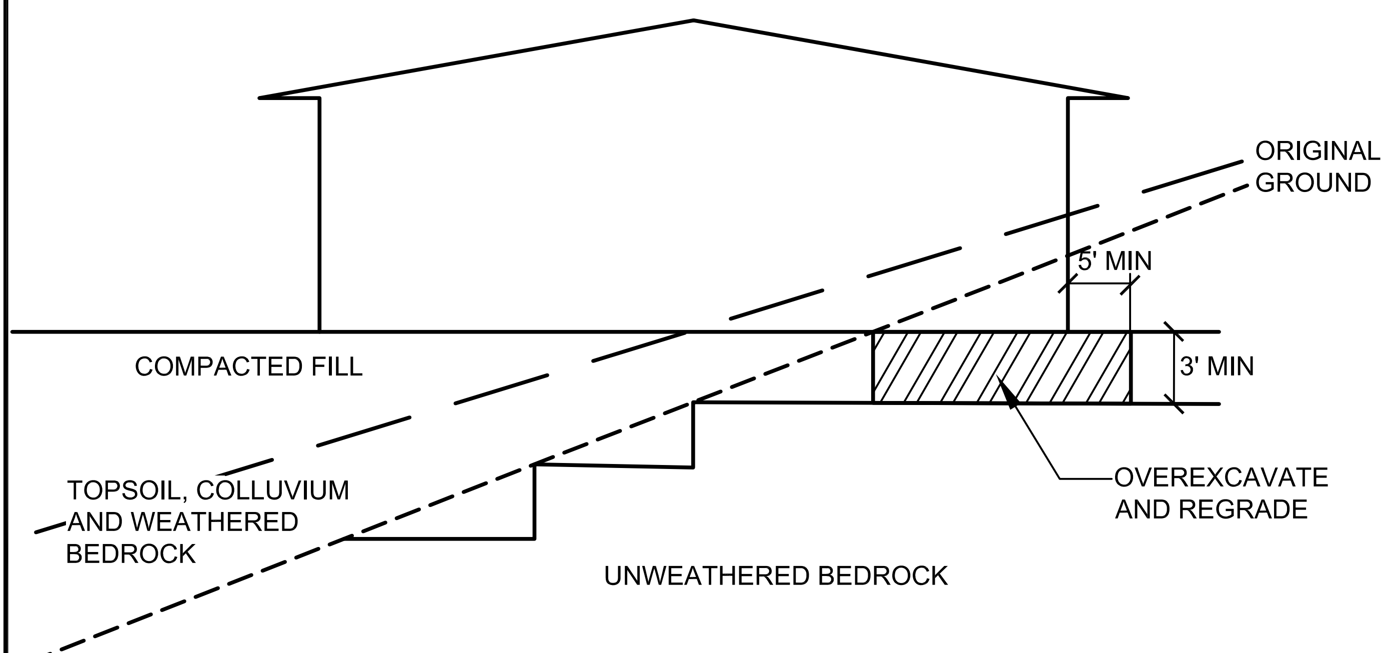
ROCK DISPOSAL DETAIL

GENERAL GRADING RECOMMENDATIONS

CUT LOT



CUT/FILL LOT (TRANSITION)



NOT TO SCALE

TRANSITION LOT DETAIL