GEOTECHNICAL ENGINEERING INVESTIGATION

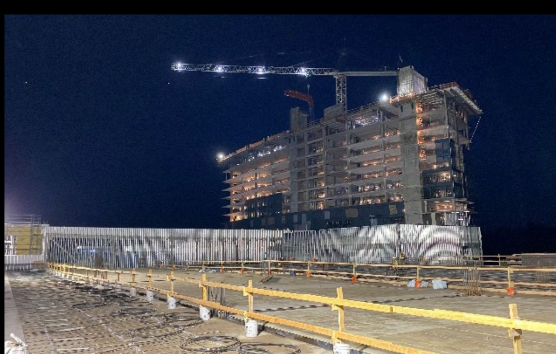
Larch Rd Geotechnical Report
10722 & 10792 W. Larch Rd
Tracy, CA 95304

CTE Job No 25-1558G
July 15, 2024

Prepared for:
Dylan Wooten
Schack & Company, Inc.
1025 Central
Tracy, CA 95376








Prepared By:
CTE Cal, Inc.

46716 Fremont Blvd
Fremont, CA 94538
P: 510.573.6362
F: 510.573.6684

3628 Madison Ave,
Suite 22
Sacramento, CA
95660
P: 916.331.6030
F: 916.331.6037

4230 Kiernan Ave,
Suite 150
Modesto, CA 95356
P: 209.543.1799
F: 209.342.7448

EXECUTIVE SUMMARY †

Site Description	Gated, empty, relatively flat dirt lot
Project Description	Subdividing two parcels into five lots consisting of four residential properties and one drainage basin.
Geological Hazards	Strong ground motions.

Grading Requirements

Excavation, scarification and compaction should be conducted as tabulated below.

Location	Over-Excavation	Scarification	Total Eng. Fill
Foundation Footprint	None recommended	12 inches below bottom of footing	12 inches
Pavement Areas	None recommended	12 inches below pavement base rock	12 inches

Scarified surfaces should be moisture conditioned and compacted in accordance with the requirements below.

Location	Material	Moisture	Compaction
Foundation Bearing	Native	Optimum +2%	90%
Foundation Bearing	Import Fill	Optimum	90%
Pavement Areas	Native	Optimum +2%	95%
Pavement Areas	Import Fill	Optimum	95%

Foundation Design

Type	Bearing Capacity	Minimum Width	Minimum Embedment	Bearing Capacity Increase with depth
Spread	2000 psf	24 inches	18 inches	100 psf/ft up to 2250 psf max
Continuous	2000 psf	12 inches	18 inches	100 psf/ft up to 2250 psf max

Lateral Load Parameters

Lateral Resistance		Equivalent Fluid Pressure		
Parameter	Value	Wall Type	Level Backfill	Sloped 2H:1V
Coefficient of Friction	0.2	Cantilever	40 pcf	60 pcf
Passive Resistance	200 psf/ft (1500 psfmax)	Restrained	60 pcf	80 pcf

Seismic and Pavement Design Parameters

Seismic Parameter	Value
Risk Category	II
Site Class	D
S_{DS}	0.781g
S_{D1}	0.745g
SDC	D

TI	R-Value	Asphalt Pavements		PCC Pavements	
		AC (in)	Class II AB (in)	PCC (in)	Class II AB (in)
4.0	16	3.0	5	4.0	4
5.0	16	3.0	8	4.0	4
6.0	16	4.0	10	5.0	4

†The results and recommendations presented in this Executive Summary should not be used for design or construction without detailed understanding of the full report. The document should be evaluated in full and all recommendations should be taken in the context of the report and standard engineering and construction practice in the region in which the project is located.

July 15, 2024 CTE Job No. 25-1558G

Attention: Dylan Wooten
Schack & Company, Inc.
1025 Central
Tracy, CA 95376
209-835-2178
dylan@schackandco.com

Subject: Geotechnical Engineering Investigation
Larch Rd Geotechnical Report
10722 & 10792 W. Larch Rd
Tracy, CA 95304

Dear Dylan Wooten,

In accordance with your request and authorization of CTE Cal Inc. (CTE) proposal, CTE has completed a geotechnical investigation at the above referenced project site. The attached report contains the results of our subsurface investigation, laboratory testing program, and engineering evaluation of the geotechnical and geological elements of the project site. Specifically, the report provides geotechnical engineering design parameters and construction recommendations for the design and development of the proposed project structures and site improvements.

Based on CTE's subsurface investigations, site materials testing, and our geotechnical and engineering evaluation, the project is considered feasible from a geotechnical standpoint provided the recommendations contained in the attached report are incorporated into the project design and construction. The recommendations contained are based on the assumption that CTE Cal will perform the required observation and inspection services. CTE Cal's experience has found that there are considerable cost savings and reduction of risk by retaining the Geotechnical Engineer of Record for construction observation services.

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.

Respectfully submitted,

CTE Cal, INC.

REGISTERED PROFESSIONAL ENGINEER
MICHAEL H KENNEDY
C 88971
CIVIL
STATE OF CALIFORNIA

Mike Kennedy, PE 88971
Senior Engineer

Selena Gray, EIT
Staff Engineer

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CTE CAL, INC.

1.0 INTRODUCTION AND SCOPE OF SERVICES

1.1 Introduction

The proposed development consists of subdividing two parcels into five lots located at 10722 and 10792 West Larch Road in Tracy, California.

This report presents the results of the Geotechnical Engineering Investigation, performed by CTE Cal, Inc. (CTE). The report provides conclusions and recommendations regarding the geotechnical design parameters and construction recommendations for the proposed development.

The investigation contained herein included surface and subsurface field explorations, laboratory testing of site soil deposits, geologic and seismic hazard evaluation of the project site, and engineering evaluation and analysis of the proposed project site and improvements. Based on the results of the investigation and analysis performed by CTE the project is considered feasible if the recommendations contained herein are incorporated into the design and construction of the project. References utilized in the investigation and analyses cited are presented in Appendix A.

1.2 Scope of Services

The scope of services provided for this 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.
- 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 AND PROJECT DESCRIPTION

The proposed project is to consist of subdividing two parcels into five lots consisting of four residential properties and one drainage basin and associated site improvements. The proposed project, totaling approximately 9.1± acres is to be constructed at 10722 and 10792 West Larch Road in Tracy, California. The project is currently bound by West Larch Road to the North, a residential property to the West, a hotel to the East, and Interstate Route No. 205 to the South.

At the time of our investigation, the project site consisted of a gated, empty, relatively flat dirt lot. Figure 1, Site Index Map, at the end of this report, shows the general location of the sites. Figure 2, Exploration Map, shows the configuration of the proposed project.

3.0 FIELD AND LABORATORY INVESTIGATIONS

3.1 Field Investigations

The field exploration program included performing a site reconnaissance and excavating 4 exploratory borings and 2 percolation test holes to determine the geometry and geotechnical characteristics of subsurface geologic deposits at the site areas proposed for new construction. Representative samples of the subsurface soil deposits were obtained from the soil boring for use in laboratory testing to determine the engineering properties and geotechnical parameters recommended for design. The borings (designated B-1 through B-4) and percolation test holes (designated P-1 through P-2) were excavated using a truck-mounted drill rig using the auger diameter and depth specified in Appendix B.

The field subsurface exploration program included performing Standard Penetration Tests (SPT) using a standard split barrel sampler (1.4-inch inside diameter, 2-inch outside diameter) which was operated in accordance with ASTM D-1586. The drive sampler was utilized to obtain samples of the subsurface soils at the depth intervals stated on the boring logs (see Appendix B) by driving the sampler into the bottom of the borehole with successive blows of a 140-pound auto hammer free-falling 30 inches. The number of blows required to drive the sampler three, six-inch intervals (18-inches total of sampler penetration) at each sampling location was recorded and the raw results of the drive sampler testing are shown on the boring logs (contained in Appendix B) in the column "Blows/6 inches". The standard penetration blow counts (N) were collected 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 based on the Unified Soil Classification System (ASTM D2487), sampler drive resistance, field testing, and visual observations. Exploration logs prepared for each of the borings provide soil descriptions, and blow count data. The boring logs are included in Appendix B which also contains the Boring Log Legend and Definition of Soil Terminology as shown on Plates BL1 and BL2, respectively. The location of the test boring is shown on Figure 2 at the end of this report.

Relatively undisturbed soil samples were obtained from the drive sampler during exploration activities. The samples were collected in capped, stainless steel sample tubes or placed in zip lock plastic bags. Bulk soil samples, if applicable, were recovered directly from drill cuttings or were obtained from surface deposits and placed in sample bags.

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 to original ground surface. Details of the soils encountered are shown on the Boring Logs which are presented in Appendix B.

3.1.1 Percolation Testing

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 two percolation test holes drilled at the locations shown on Figure-2. The percolation test holes were drilled from existing lot grade to a depth of 3-ft.

Groundwater was encountered onsite in the exploratory borings at depths ranging from approximately 11-12 feet BGS. These observations represent groundwater conditions at time of the field exploration 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 is indicated to be on the order of 5± feet below existing grade.

3.1.2 Percolation Testing Procedure

Upon completion of the percolation hole drilling, the sides of the hole were scored to remove any smeared soil surfaces, loose material was removed, and 2 inches of a coarse sand were added the bottom of percolation hole. A 3-inch diameter open-ended slotted drain pipe was then 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 of 6 inches above the sand 24 hours prior to test. During the 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 by adding water to a level of approximately 6± inches above the top of the 2 inches of sand 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 after each interval. Specific details are included on the attached percolation test data sheets located in Appendix-B.

3.1.3 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. Percolation test results are shown below in Table 3.1.3 Owing to variations in material type and depth, 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 soils encountered were described as low plastic clay. In general, the percolation rates are considered consistent with the soil types encountered at the site and the site location. The calculated conversion from percolation rate to infiltration rate is located in Appendix B. The resulting percolation rate in min/inch and Infiltration rates in gal/sf/day are listed in Table 3.1.3 below. The observed infiltration rates listed below do not include a safety factor.

TABLE 3.1.3
PERCOLATION RATES

TEST NUMBER	DEPTH (ft)	MATERIAL TYPE	PERCOLATION RATE (Min/In)	OBSERVED INFILITRATION RATE (Gal/ft^2/day)
P-1	3	Low Plastic Clay (CH)	120	1.1
P-2	3	Low Plastic Clay (CH)	120	1.1

3.2 Laboratory Testing Program

Laboratory tests were conducted on representative soil samples for classification purposes and to evaluate physical properties and engineering characteristics. Laboratory tests were conducted on representative soil samples collected from the borings. Geotechnical laboratory testing may include in situ moisture content, dry density, sieve analysis, relative fines content, Atterberg Limits, Expansion Index, R-Value testing and Consolidation testing, see Appendix C for specific tests performed. Test method descriptions and laboratory test results are presented in Appendix C.

4.0 GEOLOGY

4.1 General Geologic Setting

The site is approximately located at the northern end of the San Joaquin Valley. The San Joaquin Valley is bounded by the Sacramento Valley to the North, the Sierra Nevada Mountain Range to the East, the San Emigdio and Tehachapi Mountains to the South, and the Coastal Ranges to the West. The floor of San Joaquin Valley consists of layered marine and nonmarine sedimentary rock deposits. These deposits are predominantly Pleistocene-Holocene (2.6 million years ago to present) sedimentary rocks.

Based on geologic reconnaissance and observations made within the test borings, materials encountered during the investigation were considered to be consistent with Alluvial Fan Deposits (Qf) as shown on published regional geologic mapping "Geologic Map of the San Francisco-San Jose Quadrangle, California" (CDMG Regional Geologic Map 5A, Scale 1:250,000, 1991).

4.2 Generalized Soil Conditions

Soil materials encountered in our site explorations are consistent with the above referenced published geologic mapping. Soil materials encountered on site generally consisted of stiff to very stiff sandy low plastic clay (CL) from the surface to approximately 7.5 to 17.5 feet BGS underlain by medium to dense sands with and without fines (SC-SM, SP, and SC) to the maximum explored depth of 21.5 feet BGS.

Since the earth material profile described above is generalized, the reader is advised to consult the Test Boring Log contained in Appendix B, if determination of the earth material conditions at a specific depth and location are desired. The boring logs contain a more detailed earth material description regarding color, earth material type, and Unified Soil Classification System (USCS) symbol. It should be noted that earth material conditions cannot be fully determined by test borings and earth material sampling and testing. Hence, unexpected earth material conditions might be encountered during construction. If soil deposits encountered during construction vary substantially from materials encountered during the investigation, appropriate recommendations will be made during construction. Please contact CTE Cal if soil deposits encountered during construction vary substantially from materials encountered during the investigation.

4.3 Groundwater Conditions

Observations of groundwater conditions were made in the test borings at the time of field exploration. Groundwater was encountered in the boring locations at approximate depths of 11 to 12 feet BGS. Based on information contained on the California SGMA Data Viewer site (https://sgma.water.ca.gov/webgis/?appid=SGMADataViewer#gwlevels), ground water levels measured at nearby wells approximately 0.8-1.5 miles from the project site are typically between 3 to 10 feet BGS in the fall and spring seasons.

Groundwater levels can fluctuate on a seasonal basis due to changes in precipitation, irrigation, pumping, etc., and might increase above the levels determined by field exploration or obtained from nearby monitoring wells. With proper drainage groundwater is not expected to affect the proposed development. However, excavations below groundwater level will be impacted by seepage; therefore, we recommend grading and utility excavations be performed during dry season when ground water levels are lowest. CTE Cal does not perform waterproofing analysis and design. If the presence of groundwater is expected to affect the permanent structure, appropriate mitigation measures should be sought from a waterproofing specialist.

If construction is undertaken during wet-season/heavy-rains, saturated soils will not be 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 to achieve required soil compaction. Saturated soils resulting from significant precipitation events may need to be dried by aeration or an additive, such as lime, cement, or kiln dust added to stabilize the working surface and allow for proper soil compaction. Moisture conditioning (drying or wetting) of the engineered fill will likely be needed for the project. 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 the investigation it appears that geologic hazards at the site are primarily limited to those caused by violent shaking from earthquake generated ground motion waves. The subject site is not located within a seismic hazard zone for susceptibility to liquefaction or landslides. The subject site is not in an Alquist-Priolo special studies zone.

The underlying undisturbed soils encountered are considered adequate for support of moderately loaded structures with conventional shallow foundations. The soil conditions, groundwater level, and relatively short distances to several faults are significant geotechnical concerns that also control the selection of suitable foundation support for the proposed improvements. Design and construction recommendations presented herein have been developed based on the noted site conditions.

4.5 General Geologic Hazards Observation

Based on the site reconnaissance and review of the referenced literature, the site is not within a State of California-designated Alquist-Priolo Earthquake Fault Studies Zone (http://maps.conservation.ca.gov/cgs/fam/), and no known active fault traces shown on published hazard mapping underlie or project toward the site. According to the California Division of Mines and Geology, a fault is active if it displays evidence of activity in the last 11,000 years (Hart and Bryant, revised 2007). Therefore, the potential for surface rupture from displacement or fault movement directly beneath the proposed improvements is considered low.

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 faults are listed in Table 4.6. (U.S. Geological Survey (CGS), 2006, Quaternary fault and fold database for the United States, accessed 6/13/2024 USGS website (https://earthquake.usgs.gov/cfusion/hazfaults_2008_search/query_results.cfm)

TABLE 4.6 NEAR SITE FAULT PARAMETERS			
FAULT NAME	DISTANCE FROM SITE (MILES)	MAXIMUM EARTHQUAKE MAGNITUDE	SLIP RATE (MM/YR)
Great Valley 7	4.78	6.90	1.5
Greenville Connected	13.57	7.00	2
Mount Diablo Thrust	21.24	6.70	2
Calaveras;CN+CC+CS	26.78	7.03	n/a
Calaveras;CN	26.78	6.87	6
Calaveras;CN+CC	26.87	7.00	n/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 mapped areas are considered at risk of liquefaction-related ground failure during a seismic event based upon mapped surficial deposits. The project site is not currently mapped for potential liquefaction hazard by the CGS (refer to CGS website: (http://maps.conservation.ca.gov/cgs/informationwarehouse/index.html?map=regulatorymaps). Based on readily available published geologic information, there is no historical record of liquefaction occurring at the site.

Based on our explorations the near surface soil deposits at the site consist of stiff to very stiff sandy low plastic clay (CL) from the surface to approximately 7.5 to 17.5 feet BGS underlain by medium to dense sands with and without fines (SC-SM, SP, and SC) to the maximum explored depth of 21.5 feet BGS. Groundwater was encountered at depths between 11 to 12 feet BGS. Based on the site location, the relatively low intensity of ground shaking expected, and the consistency of the subsurface materials, the possibility of large differential settlements due to seismic dry sand settlement or liquefaction is considered low. Therefore, the potential for catastrophic building collapse due to a seismic liquefaction event are considered not significant.

4.8 Earthquake Induced Landsliding

Based on information available on the California Geological Survey (CGS) website (http://maps.conservation.ca.gov/cgs/lsi/) the subject site is not currently mapped within a State of California Seismic Hazard Zone for seismically induced landsliding. The project site is relatively flat lying, therefore the seismically induced landsliding hazard is considered low.

4.9 Tsunamis and Seiche Evaluation

Based on site location, elevation, and tsunami hazard mapping from the CGS website (http://maps.conservation.ca.gov/cgs/informationwarehouse/index.html?map=tsunami) the site is not in a tsunami inundation hazard zone. In addition, oscillatory waves (seiches) are considered unlikely due to the absence of large confined bodies of water in the site area.

4.10 Compressible and Expansive Soils

Earth materials encountered at the site within the zone of influence of the proposed foundation system are described as stiff to very stiff. These soils are not considered subject to significant compression under anticipated loads.

The near surface soils encountered during our investigation are also classified as low plastic and with low expansion potential and therefore would be expected to exhibit low volume change upon wetting or drying.

4.11 Soil Corrosion Potential

Assessment for potential of soil corrosion on construction materials was not included as part of this investigation. We recommend your corrosion engineer determine the potential corrosive characteristics of the on-site soils with respect to contact with the various underground materials that will be used for project construction.

4.12 Flooding Hazard Potential

Based on FEMA flood zone maps for San Joaquin County, California, Map No. 06077C0590F, (2009) to assess the potential for flooding of the site. Based on a review of the noted map, the site is in a designated zone, “Area of Minimal Flood Hazard - Zone X.

5.0 CONCLUSIONS AND RECOMMENDATIONS

We conclude that the proposed construction is feasible from a geotechnical standpoint, provided the recommendations in this report are incorporated into the design of the project. Recommendations for the design and construction of the proposed structures and associated improvements are included below.

5.1 Site Preparation

Project site stripping should include the demolition, removal and disposal of all asphalt and concrete debris, vegetation and other organic material in all proposed building pad and improvement areas. Loose, wet or otherwise unstable soil, including undocumented fill in the proposed improvement area should be excavated and evaluated by CTE for possible re-use as engineered fill or disposed of offsite. 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. CTE personnel shall observe and confirm that all asphalt and concrete debris, vegetation and other organic material, and unsuitable soil has been adequately removed in all proposed improvement areas.

5.1.1 Site Grading and Excavation Conditions

Site soils encountered within exploratory borings that are expected to be excavated, removed, or otherwise graded are expected to exhibit low resistance to excavation using standard grading equipment. Grading equipment should be selected by the Contractor based on experience with similar geologic deposits in the region, utilizing subsurface exploratory boring data provided in the appendices, and by using manufacturer's published criteria for equipment selection such as Caterpillar's "Handbook of Ripping".

5.2 Grading and Earthwork

CTE recommends that proposed structure areas should consist of engineered fill as specified in Table 5.2. Engineered fill is defined as native or imported earth material that has been verified by a CTE Cal representative to have the engineering characteristics specified herein, moisture conditioned (wetted or dried), graded/placed, and compacted under CTE Cal observation. The building pad engineered fill should extend to a minimum distance of at least 5 feet outside of all proposed structure areas wherever possible.

TABLE 5.2

GRADING AND EARTHWORK†

GRADING SCOPE	RECOMMENDATION
Overexcavation and/or Scarification	Excavation, scarification and compaction should be conducted as tabulated below.

Location	Over-Excavation	Scarification	Total Eng. Fill
Foundation Footprint	None recommended	12 inches below bottom of footing	12 inches
Pavement Areas	None recommended	12 inches below pavement base rock	12 inches

Scarified surfaces should be moisture conditioned and compacted in accordance with the requirements below.

Compaction and Moisture Requirements

Location	Material	Moisture	Compaction
Foundation Bearing	Native	Optimum +2%	90%
Foundation Bearing	Import Fill	Optimum	90%
Pavement Areas	Native	Optimum +2%	95%
Pavement Areas	Import Fill	Optimum	95%

Testing Frequency

Location	Frequency*
Building Pad	1 per 2500 square ft

TABLE 5.2 GRADING AND EARTHWORK†		
GRADING SCOPE	RECOMMENDATION	
	Utility Trenches	1 per 150 linear ft
	Pavement Areas	1 per 2500 square ft
	*Minimum one test per lift and one per day.	

†Requirements of local jurisdiction standards, and project specifications may require more restrictive moisture control and compaction requirements. These requirements should be followed when present.

The exposed over excavated surface should then be scarified, moisture conditioned and recompacted to the moisture and relative compaction as specified in Table 5.2. Moisture density relationship shall be established in accordance with ASTM D1557. The compaction percent listed in Table 5.2 shall be based on percent relative compaction when compared to the maximum dry density determined in accordance with ASTM D1557. Additional engineered fill, if required, shall then be placed in 8 inch loose lifts, moisture conditioned and compacted in accordance with Table 5.2. In certain conditions, local jurisdictions or additional project specifications may require more restrictive moisture control, compaction levels, or other grading requirements. The more restrictive requirements should be followed where a conflict may exist.

After stripping in pavement improvement areas is conducted, the stripped areas should be overexcavated below the proposed pavement subgrade and the excavated surface should then be scarified, moisture conditioned and recompacted to the moisture and relative compaction as specified in Table 5.2. Moisture-density relationship shall be established in accordance with ASTM D1557. Proof rolling with heavy equipment shall be performed with CTE Cal present to confirm that subgrade is compacted, stable and does not deflect under operating equipment loads where indicated necessary by CTE Cal. Additional engineered fill, if required, shall then be placed in 8-inch loose lifts, moisture conditioned and compacted in accordance with Table 5.2.

Soils generated from the site are expected to be acceptable for engineered fill provided the debris and organic materials are removed from the soils. The moisture content of the soil may be significantly higher or lower than the moisture range required for compaction. Import soils proposed for engineered fill should consist of soil deposits having an Expansion Index EI < 20 or liquid limit less than 30 (LL<30) and a plasticity index less than 12 (PI< 12), with no particles greater than 3 inches and 20 to 80% of the soil particles passing the #200 sieve. Imported fill meeting these requirements should be placed in 8 inch loose lifts, moisture conditioned and compacted to the moisture content and percent relative compaction stated in Table 5.2. A CTE representative should approve all imported soils prior to delivery to the site.

If unanticipated, unsuitable or unstable materials are encountered at the surface improvement subgrade or structure over-excavation such that proper compacted and stable materials cannot be obtained, over-excavations to remove such materials may be required. CTE shall inspect and approve all structure over-excavations, pavement and surface improvement subgrade areas to confirm that adequate soil conditions have been reached. CTE shall also observe and approve the scarification, moisture conditioning and recompaction of the excavated surfaces and the placement of all engineered fill.

5.3 Structure Foundation Recommendations

CTE anticipates it will be feasible to utilize reinforced continuous and isolated spread footing foundations to support the proposed structures at the subject site. It is recommended that these structure foundations be supported upon properly compacted engineered fill per the requirements stated in Section 5.2. Foundation dimensions and reinforcement should be based on the allowable soil bearing values stated in Table 5.3. Footing widths should be at least the widths stated in Table 5.3 or as determined by the project structural engineer. The footings should penetrate into and be embedded below building pad subgrade to the depth stated in Table 5.3. Allowable increases in bearing capacity, where appropriate, are calculated using additional embedment depth, in excess of the minimum requirement, up to the maximum allowable value.

Continuous perimeter spread footings should extend around the entire perimeter of the structures including all door openings if present. 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. The weight of the footing should be neglected in the above downward capacity calculations.

We recommend that all footings be reinforced as required by the structural engineer to provide structural continuity, to permit strong spanning of local irregularities, and to be rigid enough to accommodate potential differential static movements over the characteristic length as shown in Table 5.3. CTE recommends that at a minimum footing reinforcement should consist of at least code required minimum reinforcement with proper cover. Isolated spread footings should be reinforced in accordance with the requirements of the structural engineer.

Based on soil conditions observed at the site, the total static structure settlement is expected to be controlled by either static compression or consolidation. Dynamic settlement due to an earthquake event is in addition to the static settlement. See Table 5.3 for expected structure settlements.

The foundation excavations should be clean (i.e., free of all loose slough) and wetted prior to placing steel and concrete. Foundation excavations should be moisture conditioned to the moisture condition stated in Table 5.2 and verified by CTE no longer than 24 hours prior to foundation concrete placement and the placement of vapor barriers. 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. 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.

CTE's geotechnical engineer or his representative should observe soil conditions exposed in foundation excavations and shall test the foundation excavations to assure the proper moisture content has been achieved prior to footing placement. If the soil conditions encountered differ significantly from those presented in this report, then supplemental recommendations from CTE will be required.

TABLE 5.3

STRUCTURE FOUNDATION RECOMMENDATIONS

SYSTEM PARAMETERS

Type	Bearing Capacity	Minimum Width	Minimum Embedment	Bearing Capacity Increase with depth
Spread	2000 psf	24 inches	18 inches	100 psf/ft up to 2250 psf max
Continuous	2000 psf	12 inches	18 inches	100 psf/ft up to 2250 psf max

SETTLEMENT

Settlement Type	Settlement	Differential Settlement
Consolidation	N/A	N/A
Static Compression	1 inch	½ inch
Dynamic	½ inch	¼ inch
Total	1 ½ inch	¾ inch

Characteristic Length: 30 feet

5.4 Lateral Load Resistance

Foundation elements may be designed to resist lateral loads using the coefficient of friction or cohesion value as stated in Table 5.4 acting over the bearing contact area of the element. Total resistance equals the coefficient of friction times the dead load, or the cohesion strength times the contact area of the element base with the soil. The design passive resistance value stated in Table 5.4 may be used where lateral soil support is provided and protected from disturbance (ie below asphalt or concrete pavement, footing keyways, etc.). 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.

TABLE 5.4 LATERAL LOAD RESISTANCE	
Parameter	**Value**
Coefficient of Friction	0.2
Design Passive Resistance	200 psf/ft
Maximum Design Passive Resistance	1500 psf

Lateral load capacities for passive resistance can be increased by one-third for loading conditions that include wind or seismic.

5.5 Retaining Walls and Structures

Free draining retaining walls backfilled using select permeable onsite soils or import fill per the preceding section of this report, may be designed using the equivalent fluid weights given in the table below. These values are also considered suitable for permanent shoring, if proposed.

TABLE 5.5 EQUIVALENT FLUID UNIT WEIGHTS (pounds per cubic foot)		
WALL TYPE	LEVEL BACKFILL	SLOPE BACKFILL 2:1 (HORIZONTAL: VERTICAL)
Cantilevered Wall	40	60
Restrained Wall	60	80

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.

Temporary shoring, if used, should be designed in accordance with CalTrans Trenching and Shoring Manual, Chapter 7. Submit proposed temporary shoring design to CTE Cal for review and approval as part of the plan review process to confirm that design parameters used are in accordance with recommendations in this report.

The California Building Code Chapter 1807.2.2 requires structures assigned Seismic Design Category D, E, or F to include seismic lateral earth pressures on walls retaining more than 6 feet of retained soil vertically. 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 in previous Table)

P_K = Static Restrained Wall Thrust (given in previous Table)

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. Additional information for dynamic and static loading conditions for specific retaining structures can be provided on request from CTE.

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.

5.6 Foundation Setback

The bottoms of all utility trenches placed along the perimeter of the foundations 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.

Foundations should be offset from slopes by at least one-third the slope height. The offset from slopes steeper than 1H:1V should be increased by the projection of a 45° plane from the slope toe. The offset distance should be measured to the nearest edge of the foundation base. Deepening the footing is considered a valid way to achieve the required slope offset.

5.7 Concrete Slabs-On-Grade

Lightly loaded concrete slabs-on-ground placed beneath the structures should be designed for the anticipated loadings, but measure at least 4 inches in thickness. Concrete slabs exposed to vehicular traffic should measure at least 5 inches in thickness. Slab-on-grade reinforcement should consist of at least the minimum reinforcement required by ACI, placed at or above mid-slab height, but with proper cover. Control joints at appropriate spacing i.e. 12 feet each way 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).

All interior slab on grade areas shall be underlain by a capillary moisture break consisting of a 4” layer of ¾” minus crushed rock or Class 2 base. Prior to the installation of the capillary moisture barrier the existing subgrade shall be wetted to at least 3% above optimum moisture content as verified by a CTE representative no longer the 24 hours prior to concrete slab placement. All interior slab on grade located in moisture sensitive areas should be directly underlain by a minimum 15-mil thickness vapor retarder meeting the requirements of ASTM E1745 - Standard Specification for Plastic Water Vapor Retarders Used in Contact with Soil or

Granular Fill under Concrete Slabs, with all laps or penetrations sealed or taped. The vapor retarder should be installed above the capillary moisture break which in turn overlies the compacted building pad. The use of sand above the vapor retarder is not recommended. The concrete to be placed into the slab on grade shall have a water to cement ratio w/c $\leq$ 0.45 and be placed at a maximum slump of 4”.

CTE Cal does not practice in the field of moisture vapor transmission evaluation and mitigation. Therefore, we recommend that a qualified firm be engaged with to evaluate the general and specific moisture vapor transmission paths and any impact on the proposed construction. This firm should provide recommendations for mitigation of potential adverse impact of moisture vapor transmission on various components of the structure as deemed appropriate.

5.8 Seismic Design Criteria

Seismic design criteria are provided in Table 5.8 below based on CTE Cal’s understanding of the proposed improvement’s usage as reasonably determined by Table 1604.5 of the California Building Code. However, CTE Cal does not perform occupant load or usage analysis and the actual Risk Category may be different than that which is assumed herein. Should the Risk Category not align with that determined by the design professional responsible for such determinations, CTE Cal should be notified to provide updated seismic design criteria.

Soils that underlie the site are considered to be consistent with Site Class D materials. Site ground motion with 2% probability of exceedance in 50 years is presented in Table 5.8, below. The table is based on the United States Geological Survey’s (USGS) Probabilistic Seismic Design Maps through the third party interface ATC Hazards by Location Tool website (https://hazards.atcouncil.org/#/seismic?) for the site. The referenced USGS design maps are based on the design code reference document, ASCE 7-16 Standard with the 2022 California Building Code.

TABLE 5.8 SEISMIC GROUND MOTION VALUES		
PARAMETER	VALUE	REFERENCE
Risk Category[1]	II	CBC (2022) Table 1604.5
Site Class[2]	D	ASCE 7-16, Chapter 20
Mapped Spectral Response Acceleration Parameter, S_S	1.108g	ASCE 7-16 Figure 22-1
Mapped Spectral Response Acceleration Parameter, S_1	0.390g	ASCE 7-16 Figure 22-2
Seismic Coefficient, F_a	1.057	CBC (2022) Table 1613.2.3 (1)
Seismic Coefficient, F_v[3]	1.910	CBC (2022) Table 1613.2.3 (2) ASCE 7-16 Section 11.4.8
MCE Spectral Response Acceleration Parameter, S_{MS}	1.171g	ASCE 7-16 Section 11.4.4
MCE Spectral Response Acceleration Parameter, S_{M1}[4]	1.117g	ASCE 7-16 Section 11.4.8
Design Spectral Response Acceleration Parameter, S_{DS}	0.781g	ASCE 7-16 Section 11.4.5
Design Spectral Response Acceleration Parameter, S_{D1}[5]	0.745g	ASCE 7-16 Section 11.4.8
Mapped MCE Geometric Peak Ground Acceleration, PGA	0.461g	ASCE 7-16, Chapter 21
Mapped MCE Geometric Peak Ground Acceleration Adjusted for Site Class Effects, PGA_m	0.525g	ASCE 7-16, Chapter 11
Seismic Design Category	D	ASCE 7-16, Chapter 11

[1] Risk Category is based on an assumed nature of occupancy based on CTE Cal's understanding of project scope. Seismic design values may change based on actual occupancy type of the proposed construction.
[2] The 2022 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 21.5± feet, and this seismic site class definition considers soils below this depth to be consistent with the soils encountered at shallower depths.
[3] F_v is calculated in accordance with ASCE 7-16 Table 11.4-2 assuming that the exception for Site Class D with S_1 greater than or equal to 0.2 will be applied.
[4] S_{M1} is calculated in accordance with ASCE 7-16 Section 11.4.4
[5] S_{D1} is calculated in accordance with ASCE 7-16 Section 11.4.5

Per ASCE 7-16 Section 11.4.8 as modified by Supplement 3, a site-specific ground motion procedure would be required for the structure since the site falls under Class D, and the S_1 parameter is greater than or equal to 0.2. However, the Exception permits the use of the Code-Based ground motion values if the parameter S_{M1} determined by equation 11.4-2 is increased by 50% for all applications of S_{M1}. The resulting value of the S_{D1} parameter calculated in equation 11.4-4 should be used in all applications of S_{D1}.

5.9 Pavement Section Alternatives

It is understood asphaltic or concrete pavement is proposed for the site. The subgrade beneath all pavements should be moisture conditioned and compacted in accordance with Table 5.2 as per ASTM D1557. Pavements should be designed and constructed according to California Department of Transportation (Caltrans) standards by a registered design professional who assigns the required Traffic Index (TI) to the applicable locations at the project site. The pavement sections provided in the stamped civil drawings should be followed during construction. These values are provided as a guide for design and are not intended for reliance without such a design, stamped and approved by a registered design professional.

The pavement design sections listed below are based on Caltrans Highway Design Manual, using the recommended R-Value for subgrade soils as listed in Table 5.9, and on anticipated traffic indices as indicated below. If these assumptions are incorrect, then this office should be contacted to obtain further pavement recommendations.

TABLE 5.9
RECOMMENDED PAVEMENT SECTIONS

Traffic Area	Assumed Traffic Index	Subgrade “R”-Value	Asphalt Pavements		Portland Cement Concrete Pavements**	
			AC Thickness (inches)	Class II AB* Thickness (inches)	PCC Thickness (inches)	Class II AB* Thickness (inches)
Auto Parking Area	4.0	16	3.0	5	4.0	4
Truck Loading and Drive Areas	5.0	16	3.0	8	4.0	4
Truck Loading and Drive Areas	6.0	16	4.0	10	5.0	4

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

To significantly reduce concrete shrinkage cracking concrete pavements should be reinforced with code required minimum reinforcement placed at above mid-slab height, but with proper concrete cover, or as designed by the structural designer. Concrete pavements not supporting heavy traffic could be unreinforced provided they are constructed with expansion/contraction and/or construction joints spaced no greater than 24 times the pavement thickness, both ways, in nearly square patterns, and are detailed in general accordance with ACI Guidelines. Doweling of concrete pavements at critical pathways is also recommended.

Asphalt concrete paved areas should be designed, constructed, and maintained in accordance with, for example, the recommendations of the Asphalt Institute, or other widely recognized authority. Concrete paved areas should be designed and constructed in accordance with the recommendations of the American Concrete Institute or other widely recognized authority, particularly with regard to thickened edges, joints, and drainage. The Standard Specifications for Public Works construction (“Greenbook”) or Caltrans Standard Specifications may be referenced for pavement materials specifications. Where applicable, local jurisdictional or project specification requirements may be more restrictive than those presented herein. In such case, the more restrictive requirement should be followed.

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.

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.

Sources of foundation soil moisture reduction should be minimized as well, particularly in soils with expansion potential. Landscaping areas with vegetation with significant root networks can pull moisture from foundation soils, leading to shrinkage and possible settlement of foundations. Generally, large trees with deep root networks should be planted between half and one times their expected mature height from the foundation. However, much variability in moisture variation effects should be expected based on the tree species, which is outside the scope of this report. Further evaluation, should be requested from a qualified professional, such as an arborist, to evaluate a species for its potential effect on the structure's subsurface moisture and thereby the foundation system.

5.11 Construction Observation

The recommendations provided in this report are based on 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 Cal will provide the observation and testing services for the project. All field observations, inspections, and tests should be requested prior to the continuance of work such that elements are obstructed or difficult to access for any required corrections. To assure that the recommendations contained within this report are adhered to the following minimum inspection and testing services should be performed with regard to the geotechnical design of the project.

1. Continuous observation and testing to verify use of proper materials, densities and lift thicknesses during placement and compaction of compacted fill (including utility trenches)
2. Inspection of excavations for footings and other elements to verify excavations are extended to proper depth, have reached proper material, and that the material is adequate to achieve the design bearing capacity.
3. Continuous Inspection of Deep Foundation Elements (if applicable) upon fabrication or delivery to site, and during drilling, driving, and material placement
4. Inspection and testing of subgrade prior to the placement of fill or capillary moisture break materials to verify that the site has been properly prepared.
5. Pavement Class II Base inspection and testing prior to the placement of asphalt or concrete pavement.
6. Asphalt relative compaction testing during pavement placement. (Optional)

In furtherance of CTE Cal's desire to provide comprehensive recommendations as the Geotechnical Engineer of Record (GEOR) for this project and verify the implementation of this report's recommendations, CTE Cal will provide additional recommendations based on the site conditions observed during construction. If CTE Cal is not retained to perform field observation and inspection services, responsibility for the recommendations and conclusions made herein are transferred to the party performing such services. CTE Cal will notify the jurisdiction having authority that CTE Cal is no longer the GEOR on this project, and a letter of adoption of this responsibility should be requested from the other party.

5.12 Plan Review

CTE should review project grading, foundation, and temporary/permanent shoring plans before the start of earthworks to identify potential conflicts and to verify that the recommendations contained in the report are properly incorporated into design and are to be implemented in accordance with this report and construction standards of practice.

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 initiation of field 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 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.

Mike Kennedy, PE 88971
Project Engineer

REGISTERED PROFESSIONAL ENGINEER
MICHAEL H KENNEDY
C 88971
CIVIL
STATE OF CALIFORNIA

Selena Gray, EIT
Staff Engineer

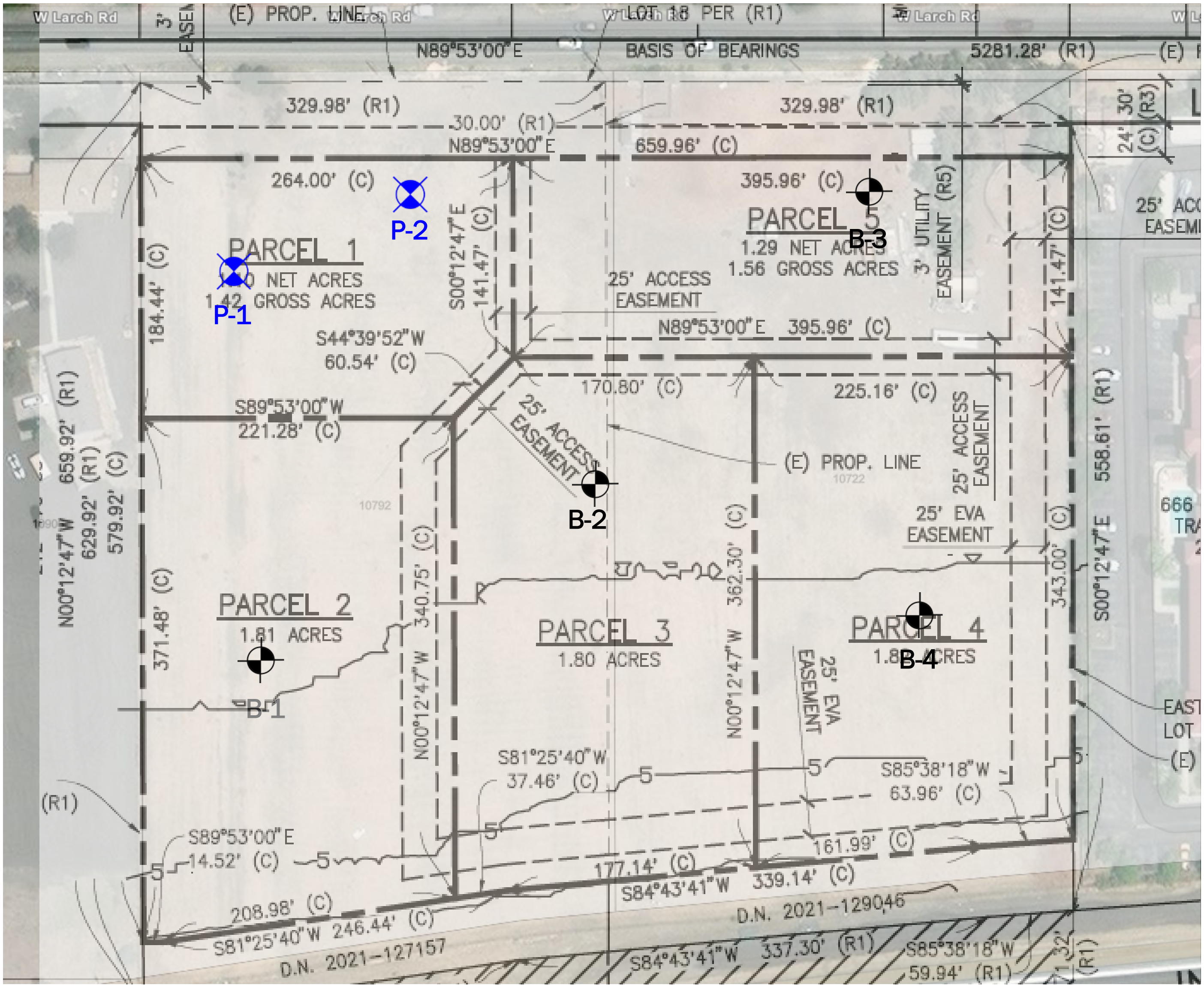
N

NOTES:

Location of site is approximate. Base map from Map Data ©2023 Google Earth.

CTE CAL, INC.
4230 Kiernan Avenue, Suite 150
Modesto, CA 95356
P: 209.543.1799
www.ctecal.com

INDEX MAP	CTE JOB NO.	
Geotechnical Investigation Larch Rd Geotechnical Report 10722 & 10792 W. Larch Rd Tracy, CA 95304	25-1558G	
	SCALE N.T.S.	
	DATE 5/15/2024	FIGURE 1



KEY MAP

N

NOTES:

Location of site is approximate. Base map from Map Data ©2023 Google Earth.

CTE CAL, INC.
4230 Kiernan Avenue, Suite 150
Modesto, CA 95356
P: 209.543.1799
www.ctecal.com

EXPLORATION LOCATION MAP Geotechnical Investigation Larch Rd Geotechnical Report 10722 & 10792 W. Larch Rd Tracy, CA 95304	CTE JOB NO. 25-1558G	
	SCALE N.T.S.	
	DATE 5/15/2024	FIGURE 2

APPENDIX A

REFERENCES CITED

1. ACI Design Manual, Section 318, Chapter 4.

2. ASCE/SEI 7-16, 2019, “Minimum Design Loads For Buildings and Other Structures”.

3. ASTM D1557, “Test Method for Laboratory Compaction Characteristics of Soil Using Modified Effort,” Volume 04.08.

4. ATC Hazards by Location website (https://hazards.atcouncil.org/#/seismic?) which utilizes USGS hazard data, reference 2016 ASCE 7 Standard.

5. California Building Standards Commission, “Title 24 - California Building Code 2022”

6. California Department of Water Recourses website: (http://wwwdwr.water.ca.gov/waterdatalibrary/index.cfm)

7. California Division of Mines and Geology (CDMG), “Geologic Map of the San Francisco-San Jose Quadrangle” (1:250,000; 1991), compiled by Wagner, D.L., Bortugno, E.J., and McJunkin, R.D.

8. California Geologic Survey (CGS) website for geologic hazards: (https://maps.conservation.ca.gov/cgs/EQZApp/app/)

9. California SGMA Data Viewer Website: (https://sgma.water.ca.gov/webgis/?appid=SGMADataViewer#gwlevels)

10. Caterpillar, 2000. “Handbook of Ripping, Twelfth Edition.”

11. FEMA Flood Map Service Center; “San Joaquin County All Jurisdictions”, Flood Zone Map No. 06077C0590F, October 2009.

12. Google Earth aerial imagery.

13. Hart, Earl W., Revised 2007, "Fault-Rupture Hazard Zones in California, Alquist Priolo, Special Studies Zones Act of 1972,” California Division of Mines and Geology, Special Publication 42.

14. Jennings, Charles W., “Fault Map of California”, 2010, CGS.

APPENDIX B

DEFINITION OF TERMS, LEGEND, BORING LOGS, & PERCOLATION TEST DATA

CTE CAL, INC.

SACRAMENTO	FREMONT	MODESTO
3628 Madison Ave, Ste. 22	46716 Fremont Blvd	4230 Kiernan Ave, Ste. 150
Sacramento, CA 95660	Fremont, CA 94538	Modesto, CA 95356
Ph: (916) 331-6030	Ph: (510) 573-6362	Ph: (209) 543-1799
Fax: (916) 331-6037	info@ctecal.com	Fax: (209) 543-1775

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

PENETRATION RESISTANCE AND PROPERTIES BASED ON THE SPT (PECK ET AL.)

	SPT (N) Blows/ft	Relative Density		SPT (N) Blows/ft	Consistency
Sands	0-4	Very Loose	Clays	<2	Very Soft
	4-10	Loose		2-4	Soft
	10-30	Medium		4-8	Medium
	30-50	Dense		8-15	Stiff
	Over 50	Very Dense		15-30	Very Stiff
				Over 30	Hard

ADDITIONAL TESTS

(OTHER THAN TEST PIT AND BORING LOG COLUMN HEADINGS)

MAX- Maximum Dry Density	PM- Permeability	PP- Pocket Penetrometer
GS- Grain Size Distribution	SG- Specific Gravity	FC- Fines Content
SE- Sand Equivalent	HA- Hydrometer Analysis	DS- Direct Shear
EI- Expansion Index	AL- Atterberg Limits	UC- Unconfined Compression
CHM- Sulfate & Chloride, pH, Resistivity	RV- R-Value	MD- Moisture/Density
COR - Corrosivity	CN- Consolidation	M- Moisture
SD- Sample Disturbed	CP- Collapse Potential	SC- Swell Compression
REM- Remolded	HC- Hydrocollapse	OI- Organic Impurities

FIGURE: BL1

CTE CAL, INC.

SACRAMENTO
3628 Madison Ave, Ste. 22
Sacramento, CA 95660
Ph: (916) 331-6030
Fax: (916) 331-6037

FREMONT
46716 Fremont Blvd
Fremont, CA 94538
Ph: (510) 573-6362
info@ctecal.com

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

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

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

FIGURE: BL2

CTE CAL, INC.

SACRAMENTO
3628 Madison Ave., Ste. 22
Sacramento, CA 95660
Ph: (916) 331 - 6030
Fax: (916) 331 - 6037

FREMONT
46716 Fremont Blvd.
Fremont, CA 94538
Ph: (510) 573 - 6362
Fax: (510) 573 - 6684

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

PROJECT:	Larch Rd Subdivision	DRILLER:	West Coast Exploration	SHEET:	1 of 4
CTE JOB NO:	25-1558G	DRILL METHOD:	Solid Stem Auger	DRILL DATE:	5/31/2024
LOGGED BY:	BR	SAMPLE METHOD(S):	SPT & Bulk	ELEV: EGS	DEPTH: 20 ft

BORING: B-1

Depth (Feet)	Bulk Sample	Driven Type	Blowcount Per 6" (N Count)	U.S.C.S. Symbol	Graphic Log	DESCRIPTION	Dry Density (pcf)	Moisture (%)	Laboratory Tests
0									
2.5			6 7 10 (17)	CL		Very Stiff Sandy Low-Plastic CLAY (CL) Brown, Wet		17	FC=76%
5			7 8 8 (16)	CL		Very Stiff Sandy Low-Plastic CLAY (CL) Brown, Wet			EI=34
10			5 8 14 (22)	CL		Very Stiff Sandy Low-Plastic CLAY (CL) Brown, Saturated			PL=17 LL=29 PI =12
15			5 7 8 (15)	CL		Very Stiff Sandy Low-Plastic CLAY (CL) Brown, Saturated			
20			7 9 9 (18)	SC		Medium Dense Clayey SAND (SC) Brown, Saturated			
25									

Boring Terminated @ 20 ft
Groundwater Encountered @ 12 ft

Boring Backfilled on 5/31/24

CTE CAL, INC.

SACRAMENTO
3628 Madison Ave., Ste. 22
Sacramento, CA 95660
Ph: (916) 331 - 6030
Fax: (916) 331 - 6037

FREMONT
46716 Fremont Blvd.
Fremont, CA 94538
Ph: (510) 573 - 6362
Fax: (510) 573 - 6684

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

PROJECT: Larch Rd Subdivision	DRILLER: West Coast Exploration		SHEET: 2 of 4
CTE JOB NO: 25-1558G	DRILL METHOD: Solid Stem Auger		DRILL DATE: 5/31/2024
LOGGED BY: BR	SAMPLE METHOD(S): SPT & Bulk	ELEV: EGS	DEPTH: 15 ft

BORING: B-2

Depth (Feet)	Bulk Sample	Driven Type	Blowcount Per 6" (N Count)	U.S.C.S. Symbol	Graphic Log	DESCRIPTION	Dry Density (pcf)	Moisture (%)	Laboratory Tests
0									
	X			CL		Sandy Low-Plastic CLAY (CL) Brown, Wet			R-Value=16
		X	6 7 7 (14)	CL		Stiff Sandy Low-Plastic CLAY (CL) Brown, Wet			PL=15 LL=40 PI =25
5		X	10 7 7 (14)	CL		Stiff Sandy Low-Plastic CLAY (CL) Brown, Wet			EI=34
10		X	10 14 14 (28)	CL		Very Stiff Sandy Low-Plastic CLAY (CL) Brown, Saturated		18	FC=52%
15		X	14 20 22 (42)	SP		Dense Poorly Graded SAND (SP) Brown, Saturated			
						Boring Terminated @ 15 ft Groundwater Encountered @ 12 ft			
20									
25									

Boring Backfilled on 5/31/24

CTE CAL, INC.

SACRAMENTO
3628 Madison Ave., Ste. 22
Sacramento, CA 95660
Ph: (916) 331 - 6030
Fax: (916) 331 - 6037

FREMONT
46716 Fremont Blvd.
Fremont, CA 94538
Ph: (510) 573 - 6362
Fax: (510) 573 - 6684

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

PROJECT: Larch Rd Subdivision | DRILLER: West Coast Exploration | SHEET: 3 of 4
CTE JOB NO: 25-1558G | DRILL METHOD: Solid Stem Auger | DRILL DATE: 5/31/2024
LOGGED BY: BR | SAMPLE METHOD(S): SPT & Bulk | ELEV: EGS | DEPTH: 10 ft

BORING: B-3

Depth (Feet)	Bulk Sample	Driven Type	Blowcount Per 6" (N Count)	U.S.C.S. Symbol	Graphic Log	DESCRIPTION	Dry Density (pcf)	Moisture (%)	Laboratory Tests
0									
2.5		SPT	7 8 10 (18)	CL		Very Stiff Sandy Low-Plastic CLAY (CL) Brown, Wet			
5		SPT	10 10 10 (20)	CL		Very Stiff Sandy Low-Plastic CLAY (CL) Brown, Wet		17	FC=60%
10		SPT	12 14 14 (28)	CL		Very Stiff Sandy Low-Plastic CLAY (CL) Brown, Wet			
						Boring Terminated @ 10 ft Groundwater Encountered @ 11 ft			
15									
20									
25									

Boring Backfilled on 5/31/24

CTE CAL, INC.

SACRAMENTO
3628 Madison Ave., Ste. 22
Sacramento, CA 95660
Ph: (916) 331 - 6030
Fax: (916) 331 - 6037

FREMONT
46716 Fremont Blvd.
Fremont, CA 94538
Ph: (510) 573 - 6362
Fax: (510) 573 - 6684

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

PROJECT:	Larch Rd Subdivision	DRILLER:	West Coast Exploration	SHEET:	4 of 4
CTE JOB NO:	25-1558G	DRILL METHOD:	Solid Stem Auger	DRILL DATE:	5/31/2024
LOGGED BY:	BR	SAMPLE METHOD(S):	SPT & Bulk	ELEV: EGS	DEPTH: 10 ft

BORING: B-4

Depth (Feet)	Bulk Sample	Driven Type	Blowcount Per 6" (N Count)	U.S.C.S. Symbol	Graphic Log	DESCRIPTION	Dry Density (pcf)	Moisture (%)	Laboratory Tests
0									
			5 5 7 (12)	CL		Stiff Sandy Low-Plastic CLAY (CL) Brown, Wet			
5			8 8 10 (18)	CL		Very Stiff Sandy Low-Plastic CLAY (CL) Brown, Wet			
10			5 7 8 (15)	SC-SM		Medium Dense Silty Clayey SAND (SC-SM) Brown, Wet		23	FC=36%
						Boring Terminated @ 10 ft No Groundwater Encountered			
15									
20									
25									

Boring Backfilled on 5/31/24

CTE CAL, INC. 3628 MADISON AVENUE, SUITE #22 | SACRAMENTO, CA 95660 | 916.331.6030 | FAX 916.331.6037

PERCOLATION TEST DATA SHEET

PROJECT: arch Rd Geotechnical Repor	PROJECT No: 25-1558G	TEST DATE:	6/3/2024
Test Hole No: P-1	Tested By: DB	DRILL DATE:	5/31/2024
Depth of Test Hole, D_t: 3'	USCS Classification:	Dark Brown, Dry, Low Plastic CLAY (CL)	
Test Hole Dimensions (Inches)			
Diameter (if round)= 4"			

PRE SATURATION

Trial No.	Start Time	Stop Time	Time Interval (min)	Initial Depth to Water (in)	Final Depth to Water (in)	Change in Water Level (in)	Comments:
1							Pre-Saturated for 24-Hours prior to testing
2							

TEST MEASUREMENTS

Trial No.	Start Time	Stop Time	Δt Time Interval (min)	D_o Initial Depth to Water (in)	D_f Final Depth to Water (in)	ΔD Change in Water Level (in)	Percolation Rate (min./in.)
1	9:22	9:52	30	6.00	5.75	0.25	120.0
2	9:52	10:22	30	6.00	5.75	0.25	120.0
3	10:22	10:52	30	6.00	5.75	0.25	120.0
4	10:52	11:22	30	6.00	5.75	0.25	120.0
5	11:22	11:52	30	6.00	5.75	0.25	120.0
6	11:52	12:22	30	6.00	5.75	0.25	120.0
7	12:22	12:52	30	6.00	5.75	0.25	120.0
8	12:52	1:22	30	6.00	5.75	0.25	120.0
9							
10							
11							
12							
13							
14							
15							
16							
17							
18							
19							
20							

Comments: Final Steady Percolation Rate = 120 min/inch
See attachment for conversion to gal/sf per day
Test hole backfilled with cuttings 6/3/2024

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PERCOLATION TEST DATA SHEET

PROJECT: arch Rd Geotechnical Repor	PROJECT No: 25-1558G	TEST DATE:	6/3/2024
Test Hole No: P-2	Tested By: DB	DRILL DATE:	5/31/2024
Depth of Test Hole, D_t: 3'	USCS Classification:	Dark Brown, Dry, Low plastic CLAY (CL)	
Test Hole Dimensions (Inches)			
Diameter (if round)= 4"			

PRE SATURATION

Trial No.	Start Time	Stop Time	Time Interval (min)	Initial Depth to Water (in)	Final Depth to Water (in)	Change in Water Level (in)	Comments:
1							Pre-Saturated for 24-Hours prior to
2							testing. Observed remaining water

TEST MEASUREMENTS

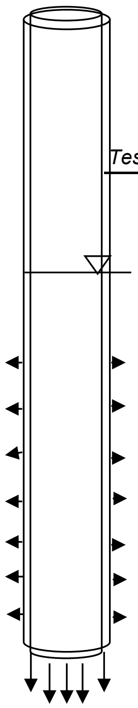
Trial No.	Start Time	Stop Time	Δt Time Interval (min)	D_o Initial Depth to Water (in)	D_f Final Depth to Water (in)	ΔD Change in Water Level (in)	Percolation Rate (min./in.)
1	1:00	1:30	30	6.00	5.750	0.250	120.0
2							
3							
4							
5							
6							
7							
8							
9							
10							
11							
12							
13							
14							
15							
16							
17							
18							
19							
20							

Comments: Final Steady Percolation Rate = 120 min/inch
See attachment for conversion to gal/sf per day
Test hole backfilled with cuttings 6/3/2024

CTE# 25-1558G

INFILTRATION RATE PER PORCHET METHOD

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



Percolation Data at the Final Interval

Test No.	*Radius*	*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})*
P-1:	2.0	30.00	6.00	5.75	0.25	5.88
P-2:	2.0	30.00	6.00	5.75	0.25	5.88

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

P-1: It = (0.25 in)(60 min/hr)(2 in) / (30 min) (2 in + 2(5.88 in))= 0.07 in/hr
P-2: It = (0.25 in)(60 min/hr)(2 in) / (30 min) (2 in + 2(5.88 in))= 0.07 in/hr

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

P-1= ((0.07)(24)(7.48))/12= 1.1 gal/sf/day
P-2= ((0.07)(24)(7.48))/12= 1.1 gal/sf/day

APPENDIX C

LABORATORY METHODS AND RESULTS

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

Natural Moisture Content

The procedure of ASTM D2216 was used to measure the moisture content of representative samples.

Classification

Soils were classified visually according to the Unified Soil Classification System. Visual classifications were supplemented by laboratory testing of selected samples according to ASTM D2487.

Atterberg Limits

The procedure of ASTM D4318 was used to measure the liquid limit, plastic limit and plasticity index of representative samples.

Material Finer than No. 200 Sieve

Particle-size analyses were performed on selected representative samples according to ASTM D1140.

R-Value

The procedure of ASTM D2844 was performed to determine the potential strength of subgrade and base materials for use in road pavements.

Expansion Index

The ASTM D4829 procedure was used on selected samples to determine the expansion potential.

Sieve Analysis

The ASTM D6913 procedure was used to determine the particle size distribution of selected samples.

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Laboratory Test Method For Material Finer Than 75µm (# 200) Sieve In Soil By Washing (ASTM D1140)

Project Name: Larch Rd

Project No.: 25-1558G

Sample Description: Boring Samples at Depth

Date: 5/31/2024

Sampled By: Bradley R.

Lab #: 6865

Method Used: ■ A ☐ B

Borehole	B1	B2	B3	B4	0	0	0
Depth (ft)	2'	10'	5'	10'	0	0	0
Tare (g)	379.6	376.8	379.0	440.00	0.00	0.00	0.00
Tare+ Moist Sample (g)	1168.3	1387.2	1338.0	1518.40	0.00	0.00	0.00
Tare+Dry **before** wash (g)	1054.6	1231.8	1199.1	1316.10	0.00	0.00	0.00
Soak Time (min)	10+	10+	10+	10+	10+	10+	10+
Tare+Dry **after** wash (g)	540.6	783.7	707.4	999.3	328.0	496.5	417.4
Moisture (%)	16.8%	18.2%	16.9%	23.1%			
Soil Loss (g)	514.0	448.1	491.7	316.8			
Finer Than #200 Sieve (%)	**76%**	**52%**	**60%**	**36%**	**-**	**-**	**-**

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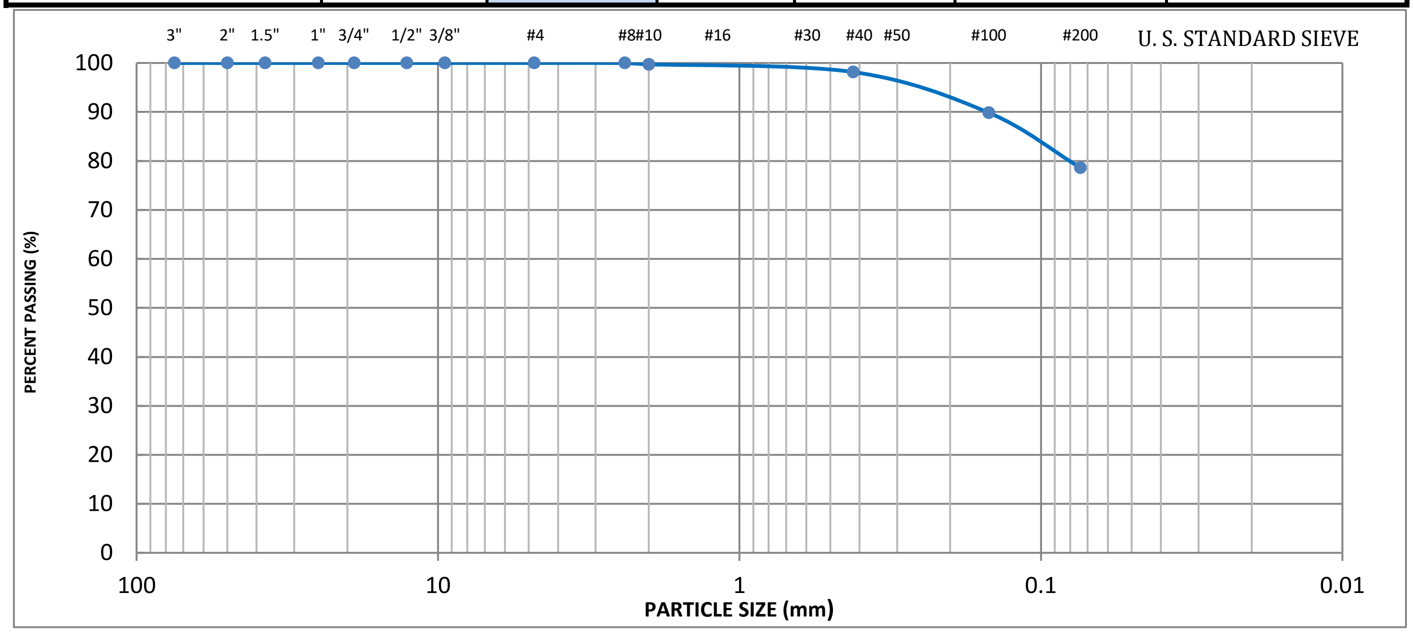
Laboratory Test Method For Sieve Analysis (ASTM C136)

Project Name:	Larch Rd		Date:	5/31/2024
Project No.:	25-1558G		Sampled By:	Bradley R.
Sample Description:	B1 at 2'		Lab #:	6865

Total Dry Weight of Sample Before Wash: 675.0

Total Dry Weight of Sample After Wash: 161.0

Sieve Openings		Mass Retained (g)		Percent Retained		Percent Passing	Specification Requirements
mm.	Inches	Individual	Cumulative	Individual	Cumulative	Cumulative	
75.0	3.0	0.0		0.0	0.0	100.0	
50.0	2.0	0.0		0.0	0.0	100.0	
37.5	1.5	0.0		0.0	0.0	100.0	
25.0	1.0	0.0		0.0	0.0	100.0	
19.0	3/4"	0.0		0.0	0.0	100.0	
12.7	1/2"	0.0		0.0	0.0	100.0	
9.5	3/8"	0.0		0.0	0.0	100.0	
4.8	#4	0.0	0.0	0.0	0.0	100.0	
2.4	#8	0.1	0.1	0.0	0.0	100.0	
2.0	#10	1.7	1.8	0.3	0.3	99.7	
0.420	#40	10.6	12.4	1.6	1.8	98.2	
0.149	#100	56.1	68.5	8.3	10.1	89.9	
0.074	#200	75.8	144.3	11.2	21.4	78.6	
Pan		16.7	161.0		23.9		



D_{10} (mm)	#NAME?	C_U	#NAME?	LL	
D_{30} (mm)	#NAME?	C_C	#NAME?	PI	
D_{50} (mm)	#NAME?	% Gravel	0.0	Fines Classification	
D_{60} (mm)	#NAME?	% Sand	21.4		
		%Fines	78.6		

Soil Type:

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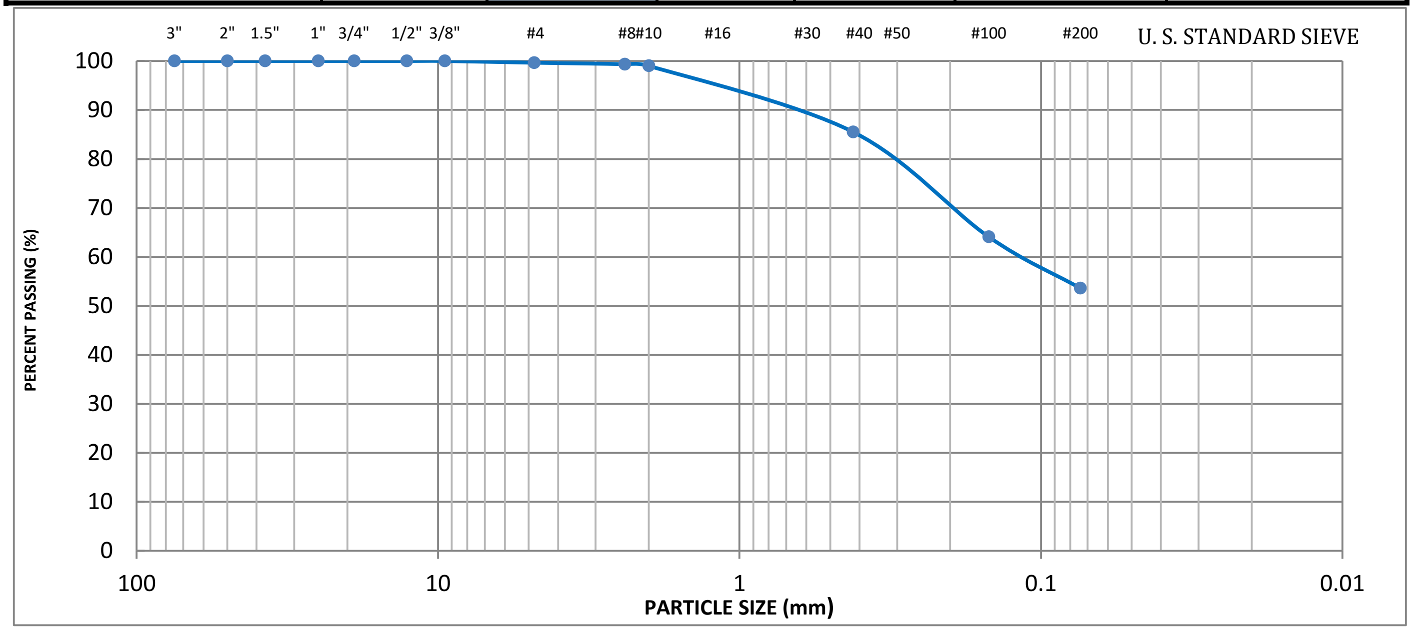
Laboratory Test Method For Sieve Analysis (ASTM C136)

Project Name:	Larch Rd	Date:	5/31/2024
Project No.:	25-1558G	Sampled By:	Bradley R.
Sample Description:	B2 at 10'	Lab #:	6865

Total Dry Weight of Sample Before Wash: 855.0

Total Dry Weight of Sample After Wash: 406.9

Sieve Openings		Mass Retained (g)		Percent Retained		Percent Passing	Specification Requirements
mm.	Inches	Individual	Cumulative	Individual	Cumulative	Cumulative	
75.0	3.0	0.0		0.0	0.0	100.0	
50.0	2.0	0.0		0.0	0.0	100.0	
37.5	1.5	0.0		0.0	0.0	100.0	
25.0	1.0	0.0		0.0	0.0	100.0	
19.0	3/4"	0.0		0.0	0.0	100.0	
12.7	1/2"	0.0		0.0	0.0	100.0	
9.5	3/8"	0.0	0.0	0.0	0.0	100.0	
4.8	#4	3.0	3.0	0.4	0.4	99.6	
2.4	#8	2.6	5.6	0.3	0.7	99.3	
2.0	#10	2.9	8.5	0.3	1.0	99.0	
0.420	#40	115.3	123.8	13.5	14.5	85.5	
0.149	#100	183.2	307.0	21.4	35.9	64.1	
0.074	#200	89.6	396.6	10.5	46.4	53.6	
Pan		9.4	406.0		47.5		



D_{10} (mm)	#NAME?	C_U	#NAME?	LL	
D_{30} (mm)	#NAME?	C_C	#NAME?	PI	
D_{50} (mm)	#NAME?	% Gravel	0.4	Fines Classification	
D_{60} (mm)	#NAME?	% Sand	46.0		
		%Fines	53.6		

Soil Type:

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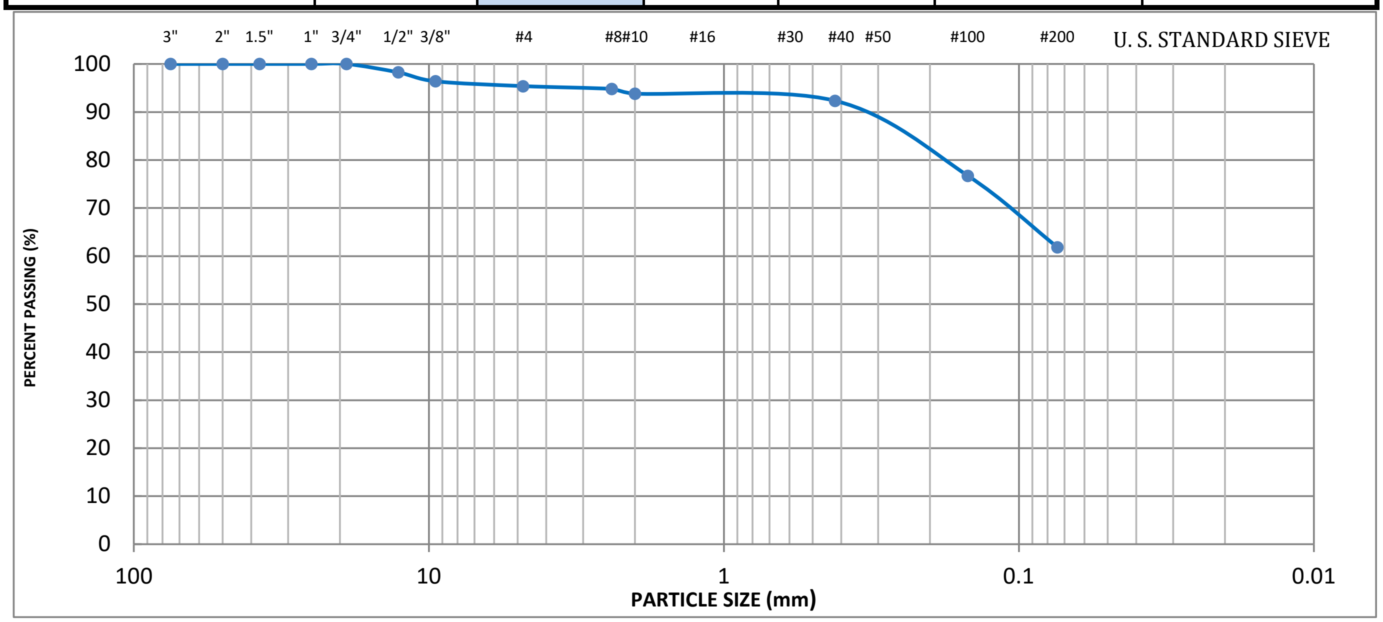
Laboratory Test Method For Sieve Analysis (ASTM C136)

Project Name:	Larch Rd		Date:	5/31/2024
Project No.:	25-1558G		Sampled By:	Bradley R.
Sample Description:	B3 at 5'		Lab #:	6865

Total Dry Weight of Sample Before Wash:	820.1
Total Dry Weight of Sample After Wash:	328.4

Sieve Openings		Mass Retained (g)		Percent Retained		Percent Passing	Specification Requirements
mm.	Inches	Individual	Cumulative	Individual	Cumulative	Cumulative	
75.0	3.0	0.0		0.0	0.0	100.0	
50.0	2.0	0.0		0.0	0.0	100.0	
37.5	1.5	0.0		0.0	0.0	100.0	
25.0	1.0	0.0		0.0	0.0	100.0	
19.0	3/4"	0.0	0.0	0.0	0.0	100.0	
12.7	1/2"	14.3	14.3	1.7	1.7	98.3	
9.5	3/8"	15.3	29.6	1.9	3.6	96.4	
4.8	#4	8.2	37.8	1.0	4.6	95.4	
2.4	#8	4.9	42.7	0.6	5.2	94.8	
2.0	#10	8.0	50.7	1.0	6.2	93.8	
0.420	#40	12.1	62.8	1.5	7.7	92.3	
0.149	#100	128.0	190.8	15.6	23.3	76.7	
0.074	#200	122.2	313.0	14.9	38.2	61.8	
Pan		15.0	328.0		40.0		



D_{10} (mm)	#NAME?	C_U	#NAME?	LL	
D_{30} (mm)	#NAME?	C_C	#NAME?	PI	
D_{50} (mm)	#NAME?	% Gravel	4.6	Fines Classification	
D_{60} (mm)	#NAME?	% Sand	33.6		
		%Fines	61.8		

Soil Type:

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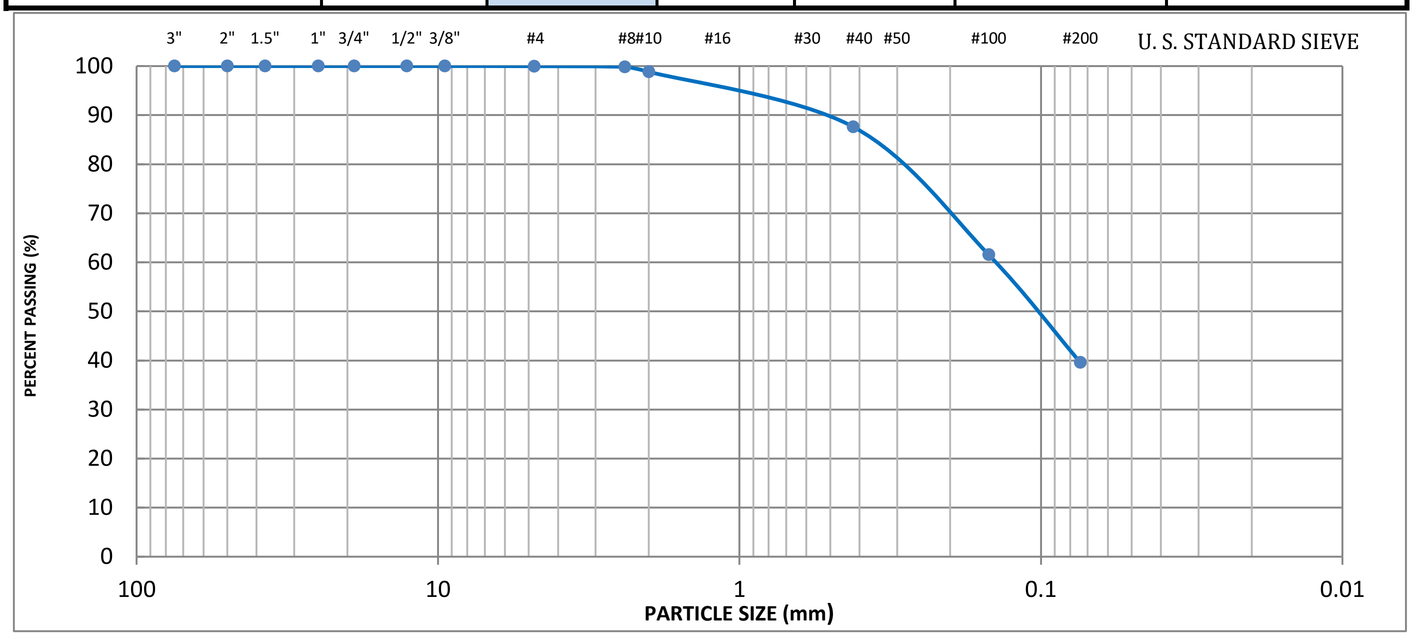
Laboratory Test Method For Sieve Analysis (ASTM C136)

Project Name:	Larch Rd	Date:	5/31/2024
Project No.:	25-1558G	Sampled By:	Bradley R.
Sample Description:	B4 at 10'	Lab #:	6865

Total Dry Weight of Sample Before Wash: 876.1
Total Dry Weight of Sample After Wash: 559.3

Sieve Openings		Mass Retained (g)		Percent Retained		Percent Passing	Specification Requirements
mm.	Inches	Individual	Cumulative	Individual	Cumulative	Cumulative	
75.0	3.0	0.0		0.0	0.0	100.0	
50.0	2.0	0.0		0.0	0.0	100.0	
37.5	1.5	0.0		0.0	0.0	100.0	
25.0	1.0	0.0		0.0	0.0	100.0	
19.0	3/4"	0.0		0.0	0.0	100.0	
12.7	1/2"	0.0		0.0	0.0	100.0	
9.5	3/8"	0.0	0.0	0.0	0.0	100.0	
4.8	#4	0.2	0.2	0.0	0.0	100.0	
2.4	#8	1.4	1.6	0.2	0.2	99.8	
2.0	#10	8.6	10.2	1.0	1.2	98.8	
0.420	#40	98.1	108.3	11.2	12.4	87.6	
0.149	#100	228.7	337.0	26.1	38.5	61.5	
0.074	#200	192.4	529.4	22.0	60.4	39.6	
Pan		29.6	559.0		63.8		



D_{10} (mm)	#NAME?	C_U	#NAME?	LL	
D_{30} (mm)	#NAME?	C_C	#NAME?	PI	
D_{50} (mm)	#NAME?	% Gravel	0.0	Fines Classification	
D_{60} (mm)	#NAME?	% Sand	60.4		
		%Fines	39.6		

Soil Type: **SM**

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Laboratory Test Method For Atterberg Limits (ASTM D4318)

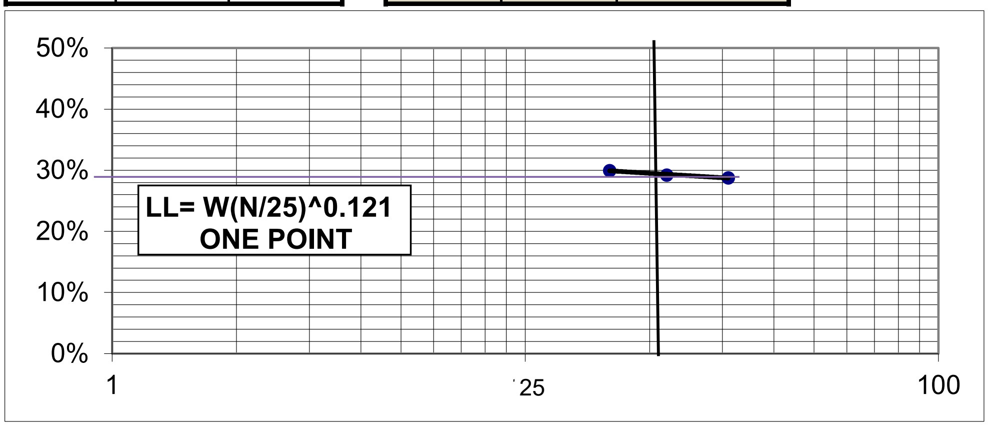
Project Name:	Larch Rd	Date:	5/31/2024
Project No.:	25-1558G	Sampled By:	Bradley R.
Sample Description:	B1 @ 10'	Lab #:	6865

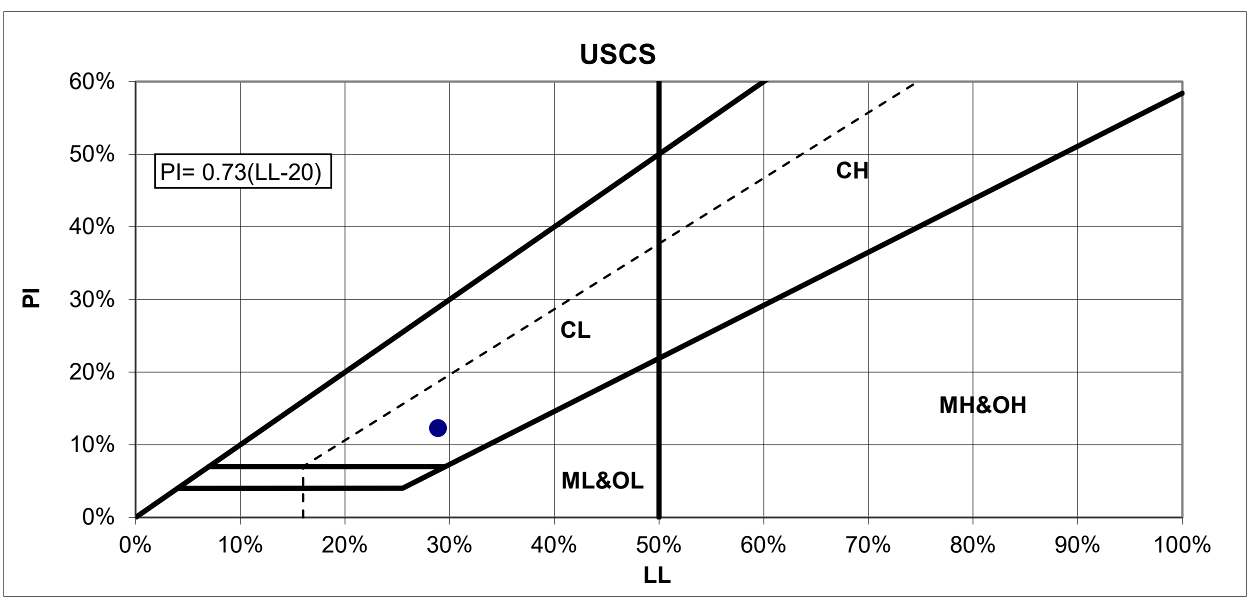
	LIQUID LIMITS			PLASTIC LIMIT	
TARE (g)	20.61	20.98	20.76	13.95	20.79
TARE+WET (g)	30.89	31.59	29.98	19.95	26.79
TARE+DRY (g)	28.52	29.19	27.92	19.09	25.94
WATER	2.37	2.40	2.06	0.86	0.85
# BLOWS	16	22	31		
% MOIST	**29.96%**	**29.23%**	**28.77%**	**16.73%**	**16.50%**

Method used		Dry
	X	Moist
Plastic Limit	X	Hand rolled
		Mach.rolling device
Liquid Limit Apparatus	X	Manual
		Mechanical
Casagrande Grooving tool	X	Metal
		Plastic

ONE POINT		
28.4%	**28.8%**	**29.5%**

LL	PL	PI
29%	**17%**	**12%**





Soil Type: CL

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Laboratory Test Method For Atterberg Limits (ASTM D4318)

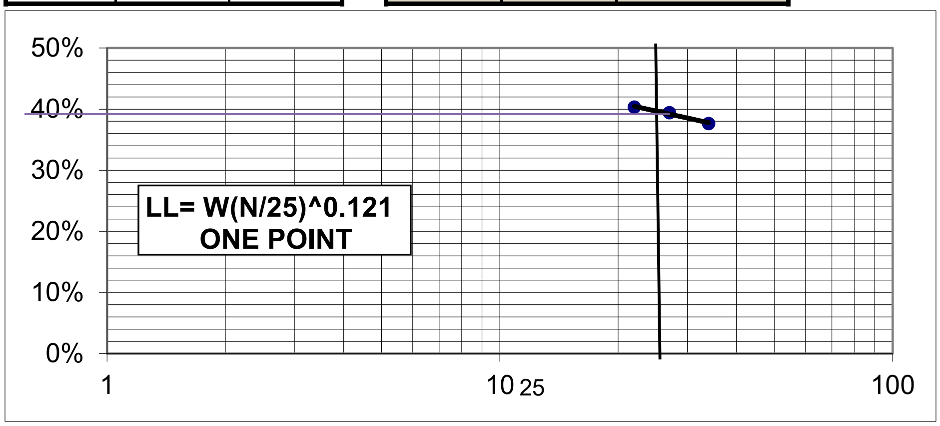
Project Name:	Larch Rd	Date:	5/31/2024
Project No.:	25-1558G	Sampled By:	Bradley R.
Sample Description:	B2 @ 2'	Lab #:	6865

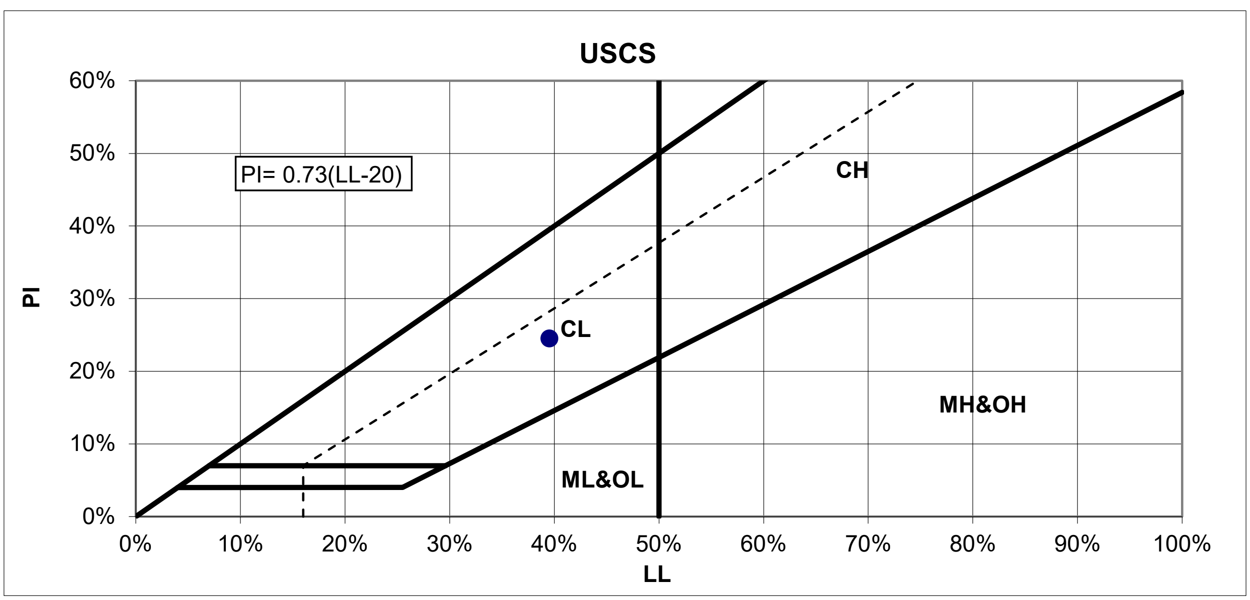
	LIQUID LIMITS			PLASTIC LIMIT	
TARE (g)	20.99	13.54	13.15	22.02	21.18
TARE+WET (g)	29.13	21.64	23.20	31.70	28.75
TARE+DRY (g)	26.79	19.35	20.45	30.40	27.79
WATER	2.34	2.29	2.75	1.30	0.96
# BLOWS	22	27	34		
% MOIST	**40.34%**	**39.41%**	**37.67%**	**15.51%**	**14.52%**

Method used		
		Dry
	X	Moist
Plastic Limit	X	Hand rolled
		Mach.rolling device
Liquid Limit Apparatus	X	Manual
		Mechanical
Casagrande Grooving tool	X	Metal
		Plastic

ONE POINT		
39.7%	**39.8%**	**39.1%**

LL	PL	PI
40%	**15%**	**25%**





Soil Type: CL

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Laboratory Test Method For Expansion Index (ASTM D4829)

Project Name: Larch Rd
Project No.: 25-1558G
Sample Description: B1@5' and B2@5'

Date: 5/31/2024
Sampled By: Bradley R.
Lab #: 6865

		Initial	Saturated
WET WEIGHT	(g)	458.7	435.4
DRY WEIGHT	(g)	417.1	364.3
% MOISTURE	(%)	10.0%	19.5%
WEIGHT OF RING & SOIL	(g)	**768.3**	**803.1**
WEIGHT OF RING	(g)	367.7	367.7
WEIGHT OF SOIL	(g)	400.6	435.4
WEIGHT OF SOIL	(lbs.)	0.8832	0.9599
VOLUME OF SOIL	(cf)	0.00730	0.00754
WET DENSITY	(pcf)	121.0	127.4
DRY DENSITY	(pcf)	110.0	106.5
% SATURATION	(%)	**51.8%**	**90.8%**

EXPANSION READING

DATE	TIME:		INCH		
		INITIAL READING	0.039	**VERY LOW**	**0-20**
				LOW	**21-50**
				MEDIUM	**51 -90**
		FINAL READING	0.071	**HIGH**	**91-130**
		EXPANSION INDEX =	32.6	**VERY HIGH**	**130>**

EXPANSION INDEX50 = **34**

NOTES:
1.- 2.67 SP. GR. = 1/2.7= 0.3704
2.- % SATURATION MUST BE 50% +/- 2%

EI at saturation between 48-52%

Measured EI: 33
Measured Saturation: 51.8

EI at 50% Saturation: **34**

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

REPORT OF RESISTANCE 'R' VALUE-EXPANSION PRESSURE
ASTM D2844

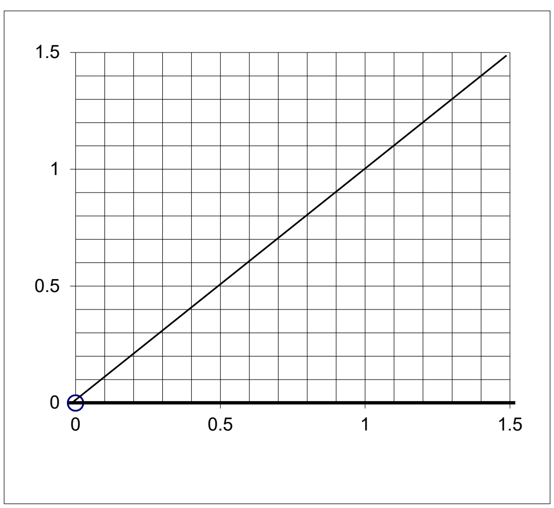
Project Name:	Larch Rd	**Date:**	5/31/2024
Project #:	25-1558G	**Sampled By:**	Bradley R.
Sample Description:	B-2 @ 2-5'	**Lab #:**	6865
		Type of Material:	Clayey

Test Procedure: ASTM D2844

Specimen/ Mold No.	16	I	H
Compactor Air Pressure, - ft.lbs.	50	50	50
Initial Moisture, - %	17.9%	17.9%	17.9%
Sample Size - g	1100.9	1100.3	1100
Water Added, - ml	30	15	0
Moisture at Compaction, - %	20.7%	19.3%	17.9%
Wt. Of Briquette and Mold, - g	3164.5	3140.6	3136
Wt. Of Mold, - g	2071.8	2068.4	2070.8
Wt. Of Briquitte, - g	1092.7	1072.2	1065.2
Height of Briquette, - in	2.40	2.40	2.43
Dry Density, - pcf	114.3	113.4	112.6
Stabilometer PH @ 2000 lbs	128	120	114
Displacement	4.11	3.70	3.09
R' Value	13	18	24
Corrected 'R' Value	13	17	22
Exudation Pressure, - lbs	2410	4030	6890
Exudation Pressure, - psi	193	322	551
Stabilometer Thickness - ft	0	0	0
Expansion - in.	0	0	0
Expansion Pressure - Pascals	0	0	0
Expansion Press, Thick-ft	0	0	0

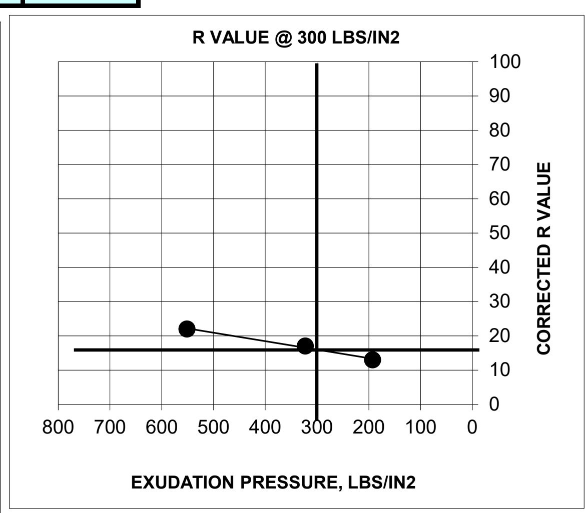
R-value 16

TI	
Expansion	N/A



Cover Thickness by Expansion Pressure-Feet

Expansion From Graph:	N/A



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

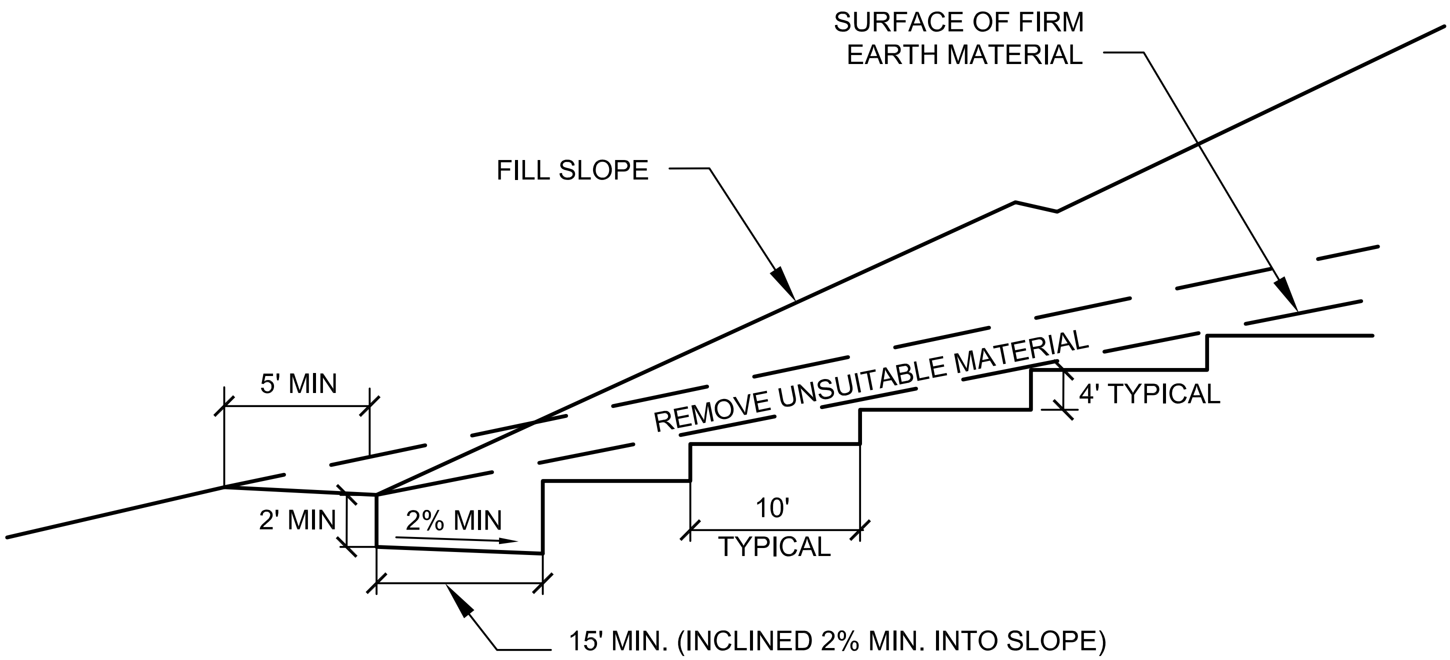
As a precautionary measure, plastic sheeting should be readily available, or kept on hand, to protect all slope areas from saturation by periods of heavy or prolonged rainfall. This measure is strongly recommended, beginning with the period prior to landscape planting.

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.

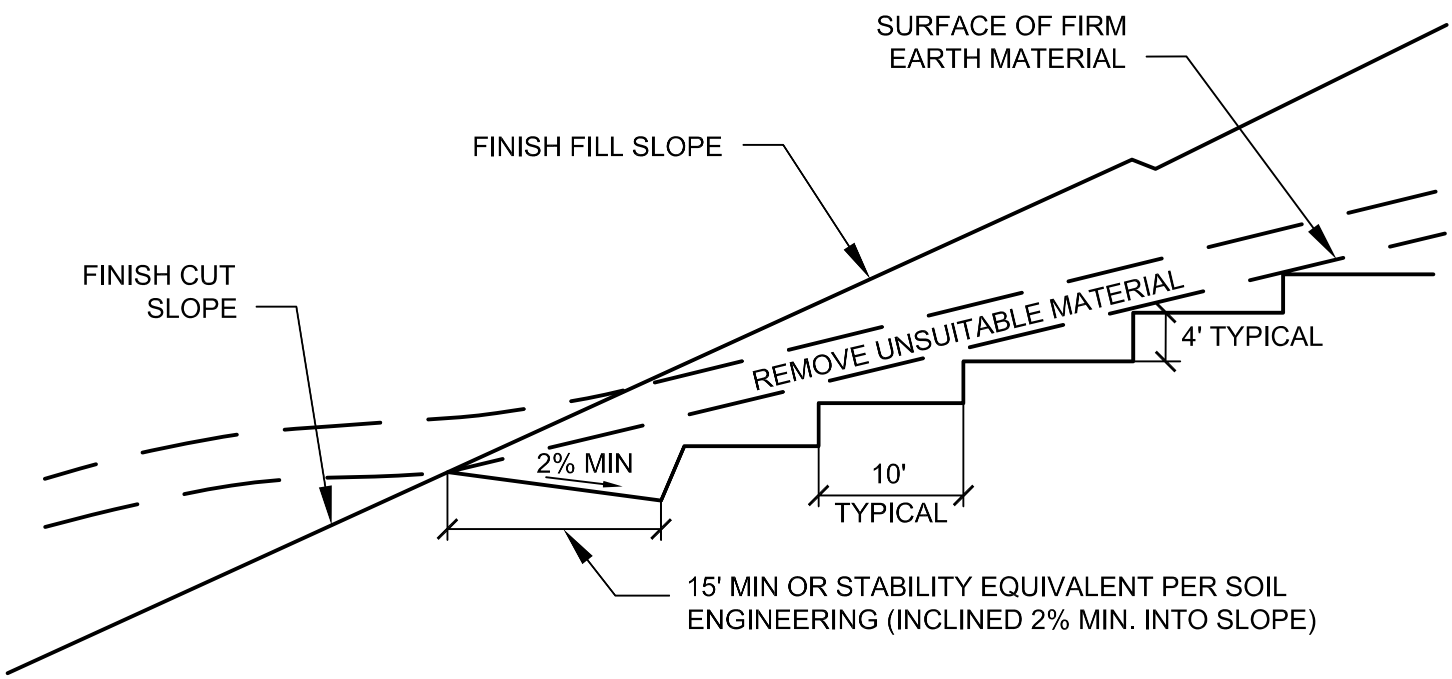
If slope failures occur as a result of exposure to period of heavy rainfall, the failure areas and currently unaffected areas should be covered with plastic sheeting to protect against additional saturation.

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

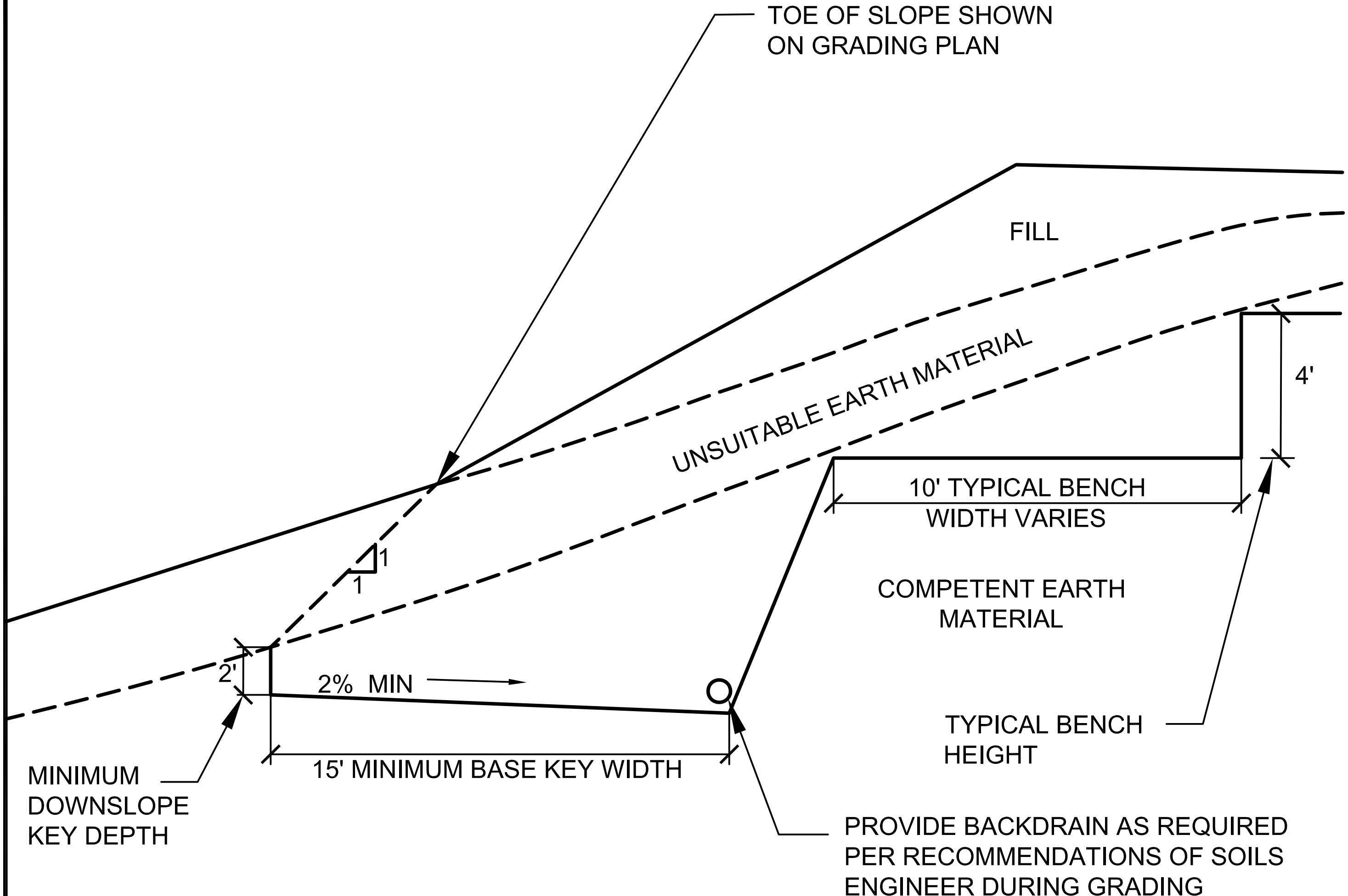


BENCHING FILL OVER CUT



NOT TO SCALE

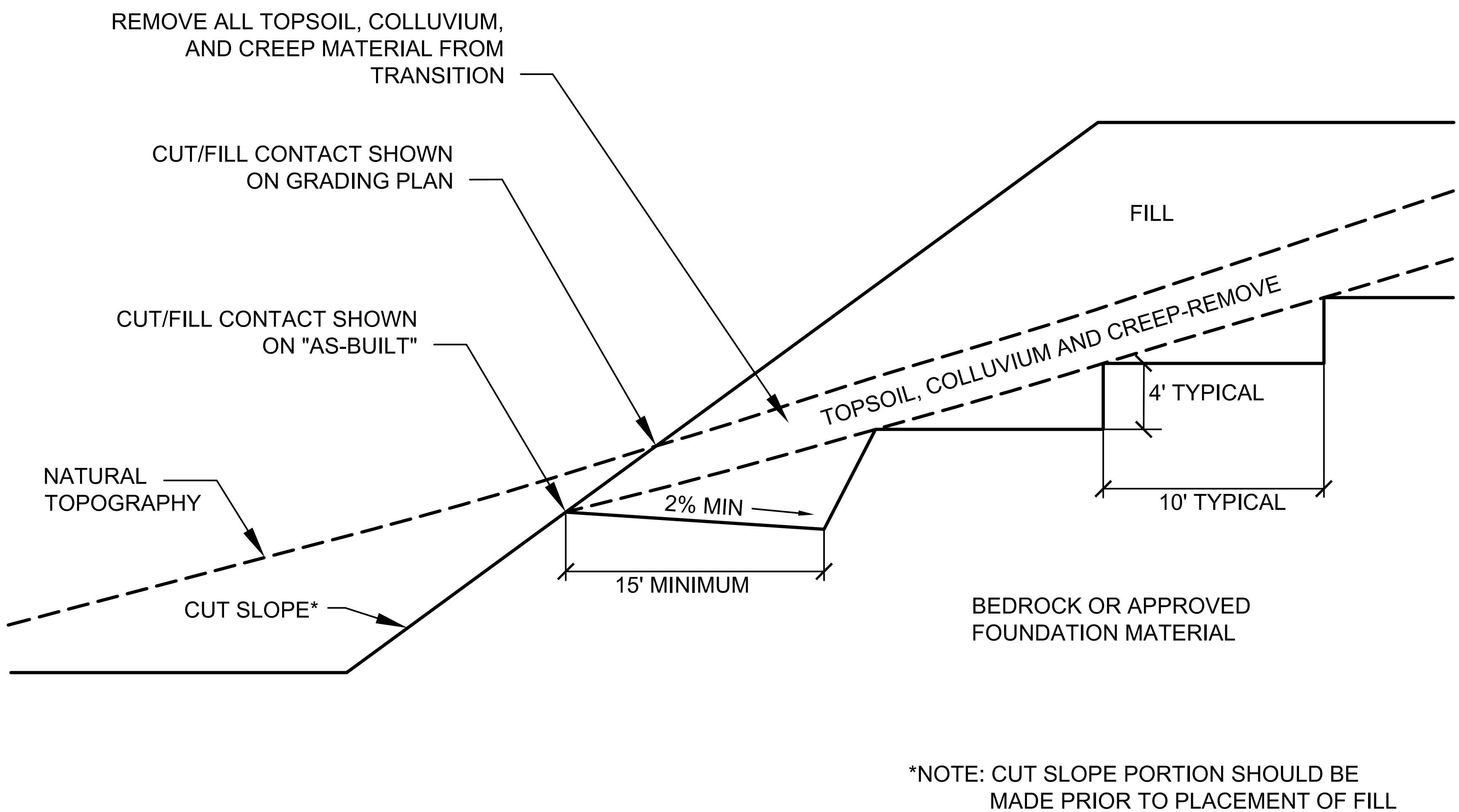
BENCHING FOR COMPACTED FILL DETAIL



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

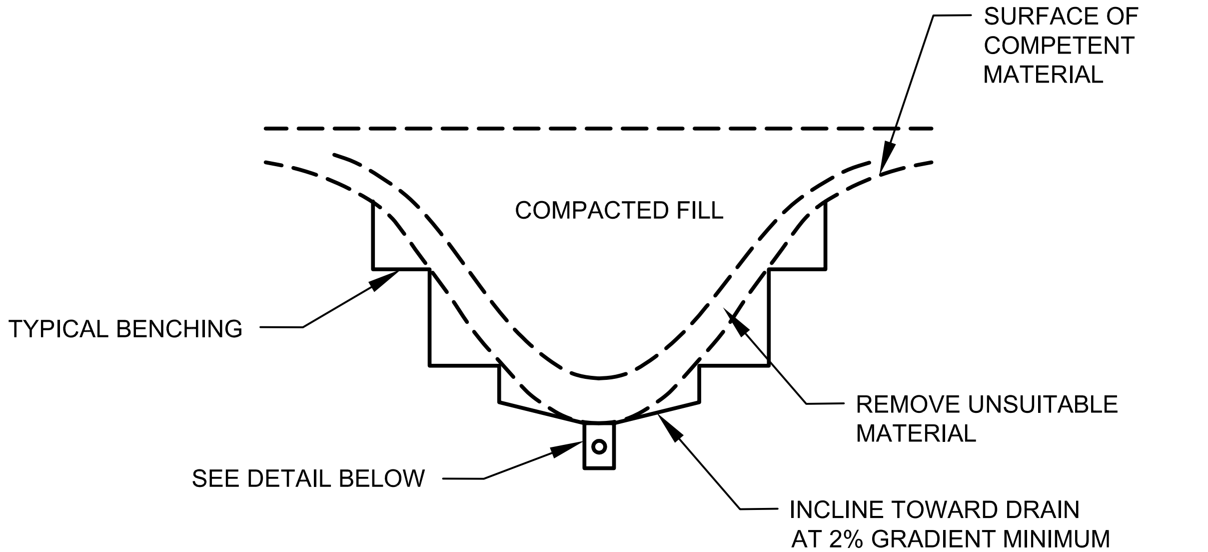
NOT TO SCALE

FILL SLOPE ABOVE NATURAL GROUND DETAIL

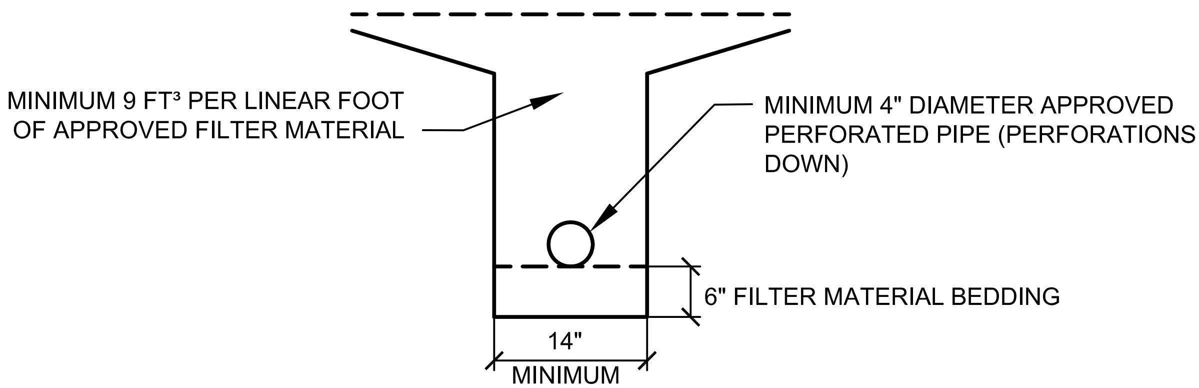


NOT TO SCALE

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

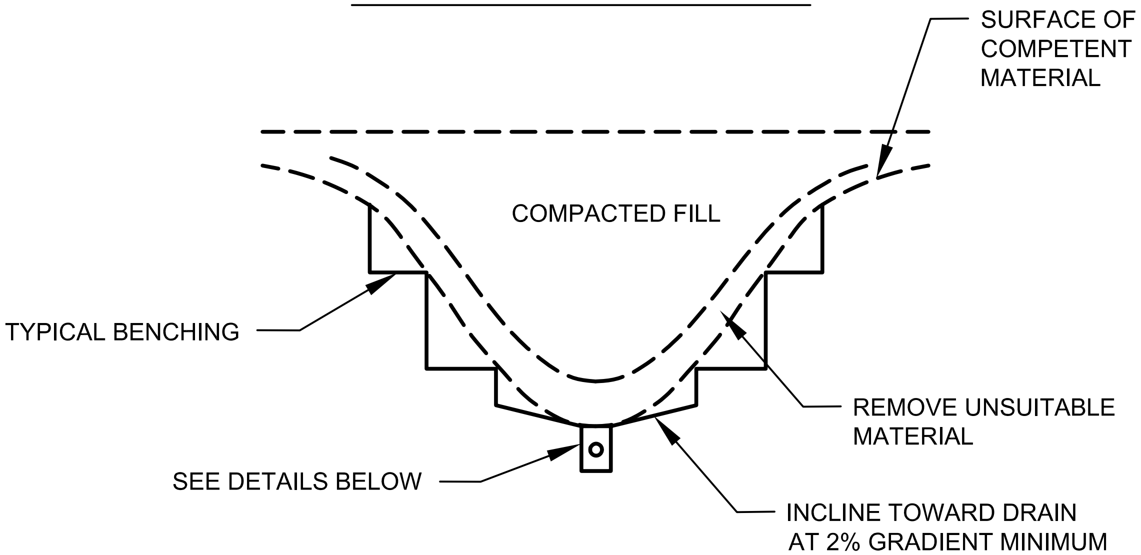
PIPE DIAMETER TO MEET THE FOLLOWING CRITERIA, SUBJECT TO FIELD REVIEW BASED ON ACTUAL GEOTECHNICAL CONDITIONS ENCOUNTERED DURING GRADING

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

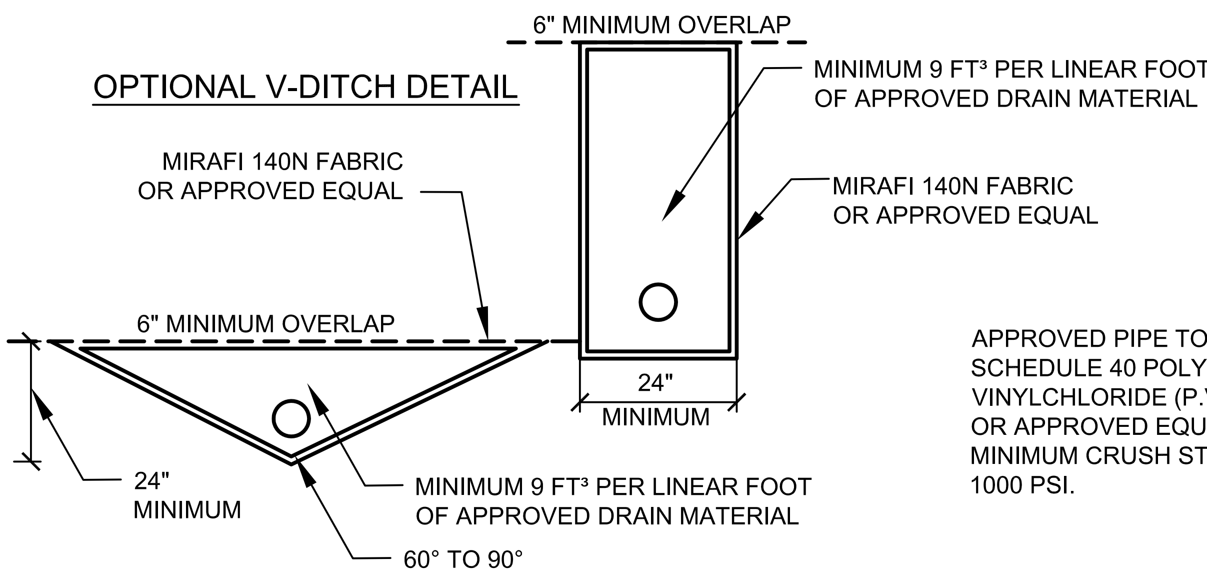
NOT TO SCALE

TYPICAL CANYON SUBDRAIN DETAIL

CANYON SUBDRAIN DETAILS



TRENCH DETAILS



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

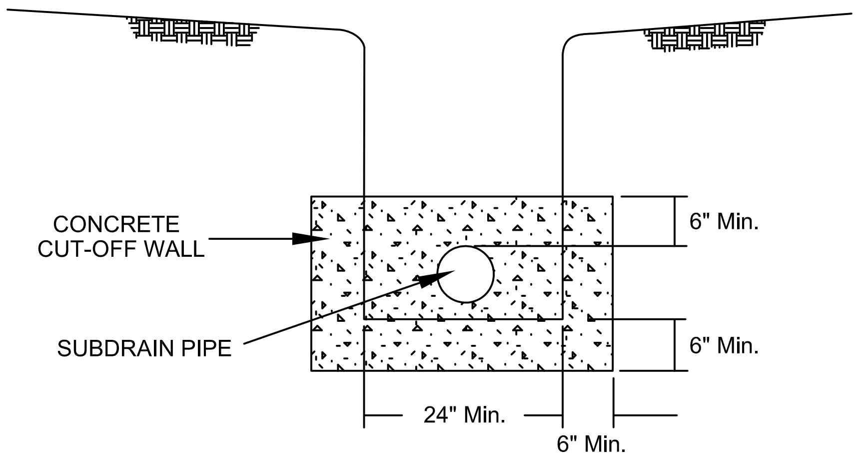
PIPE DIAMETER TO MEET THE FOLLOWING CRITERIA, SUBJECT TO FIELD REVIEW BASED ON ACTUAL GEOTECHNICAL CONDITIONS ENCOUNTERED DURING GRADING

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

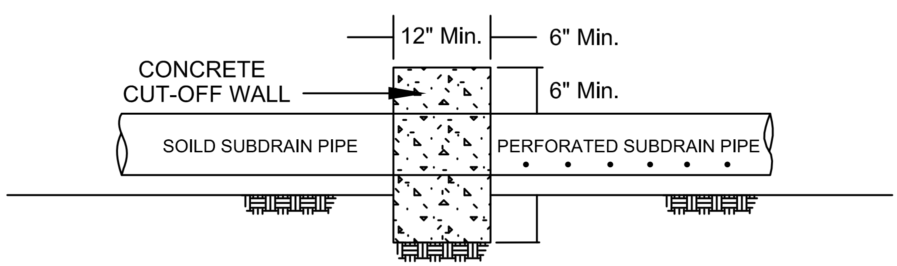
NOT TO SCALE

GEOFABRIC SUBDRAIN

FRONT VIEW



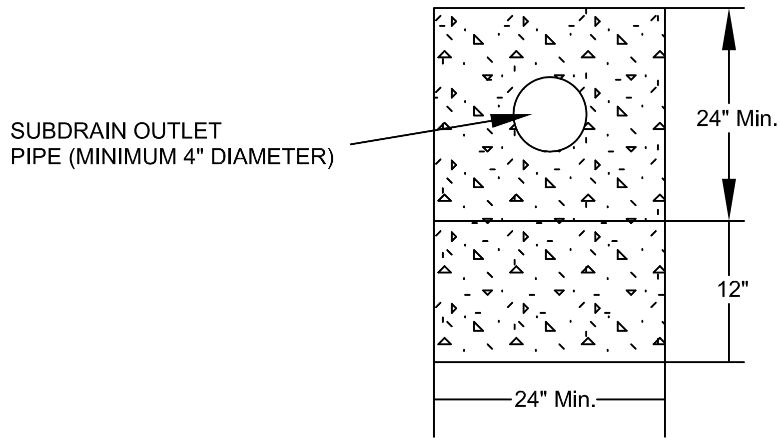
SIDE VIEW



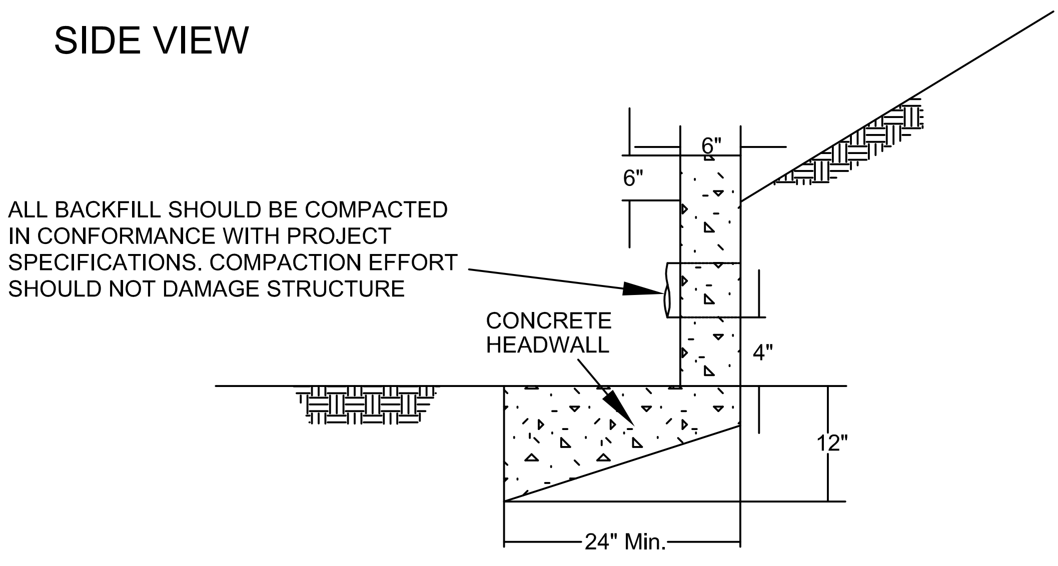
NOT TO SCALE

RECOMMENDED SUBDRAIN CUT-OFF WALL

FRONT VIEW



SIDE VIEW



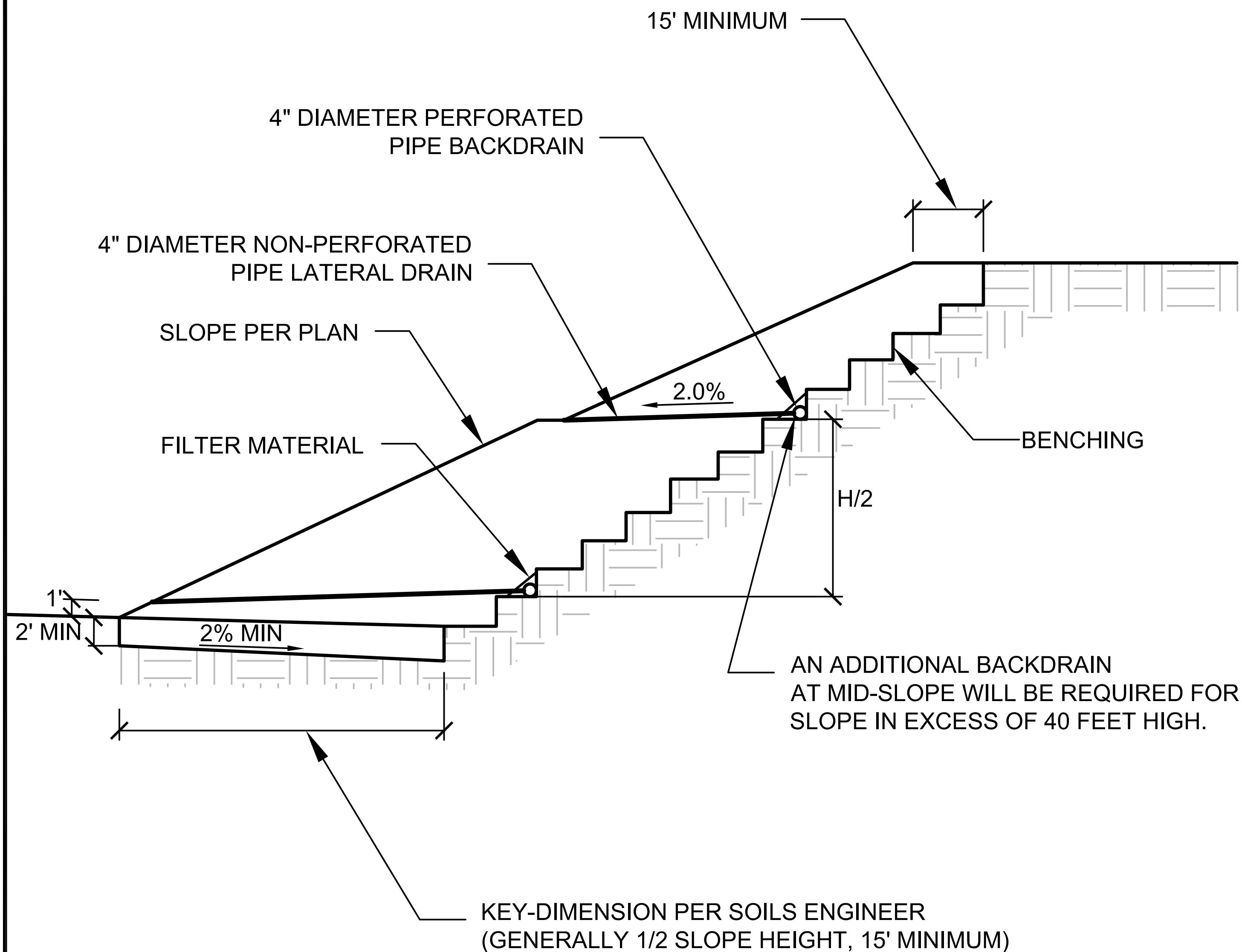
NOTE: HEADWALL SHOULD OUTLET AT TOE OF SLOPE OR INTO CONTROLLED SURFACE DRAINAGE DEVICE

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

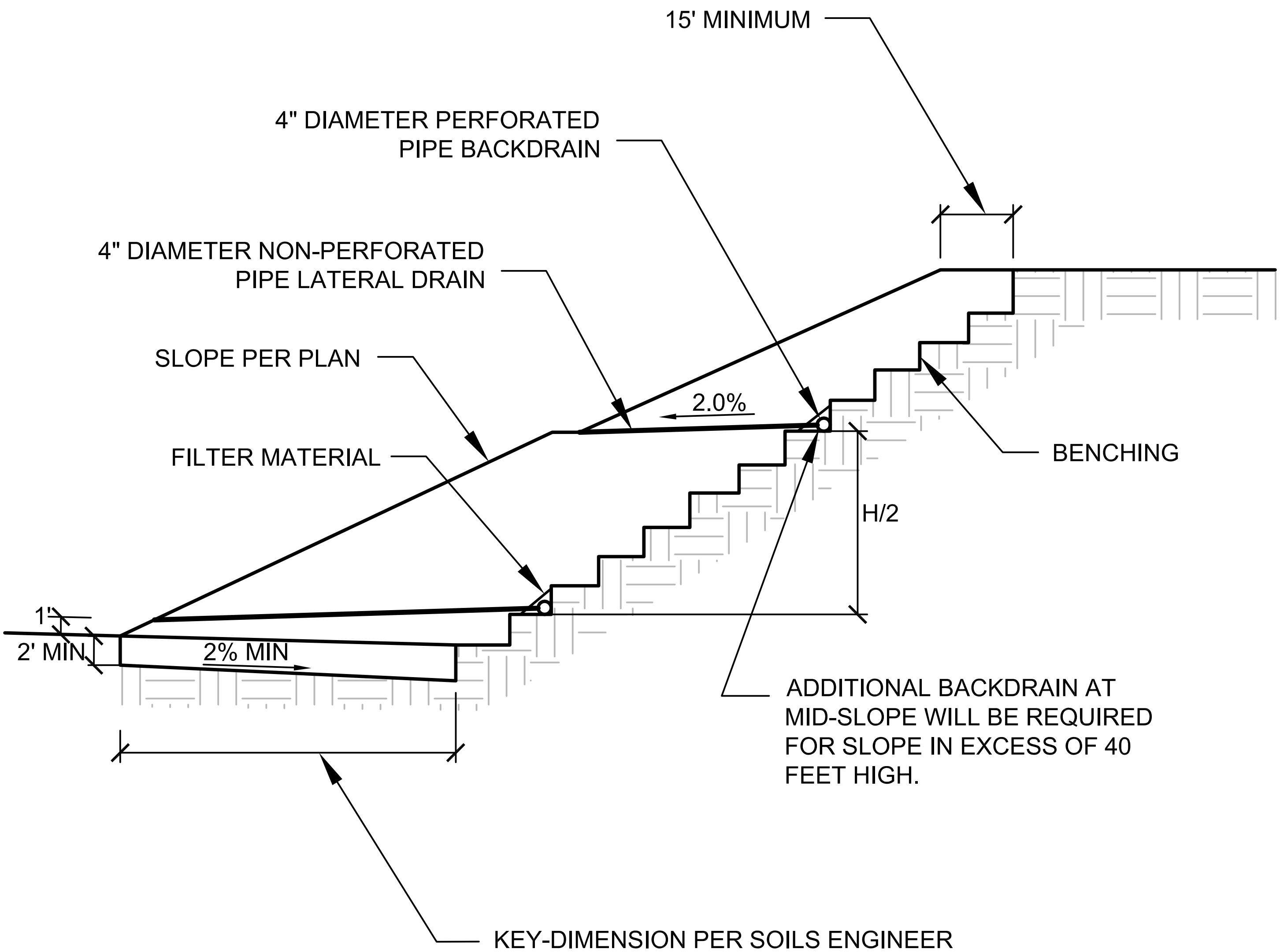
TYPICAL SUBDRAIN OUTLET HEADWALL DETAIL



DIMENSIONS ARE MINIMUM RECOMMENDED

NOT TO SCALE

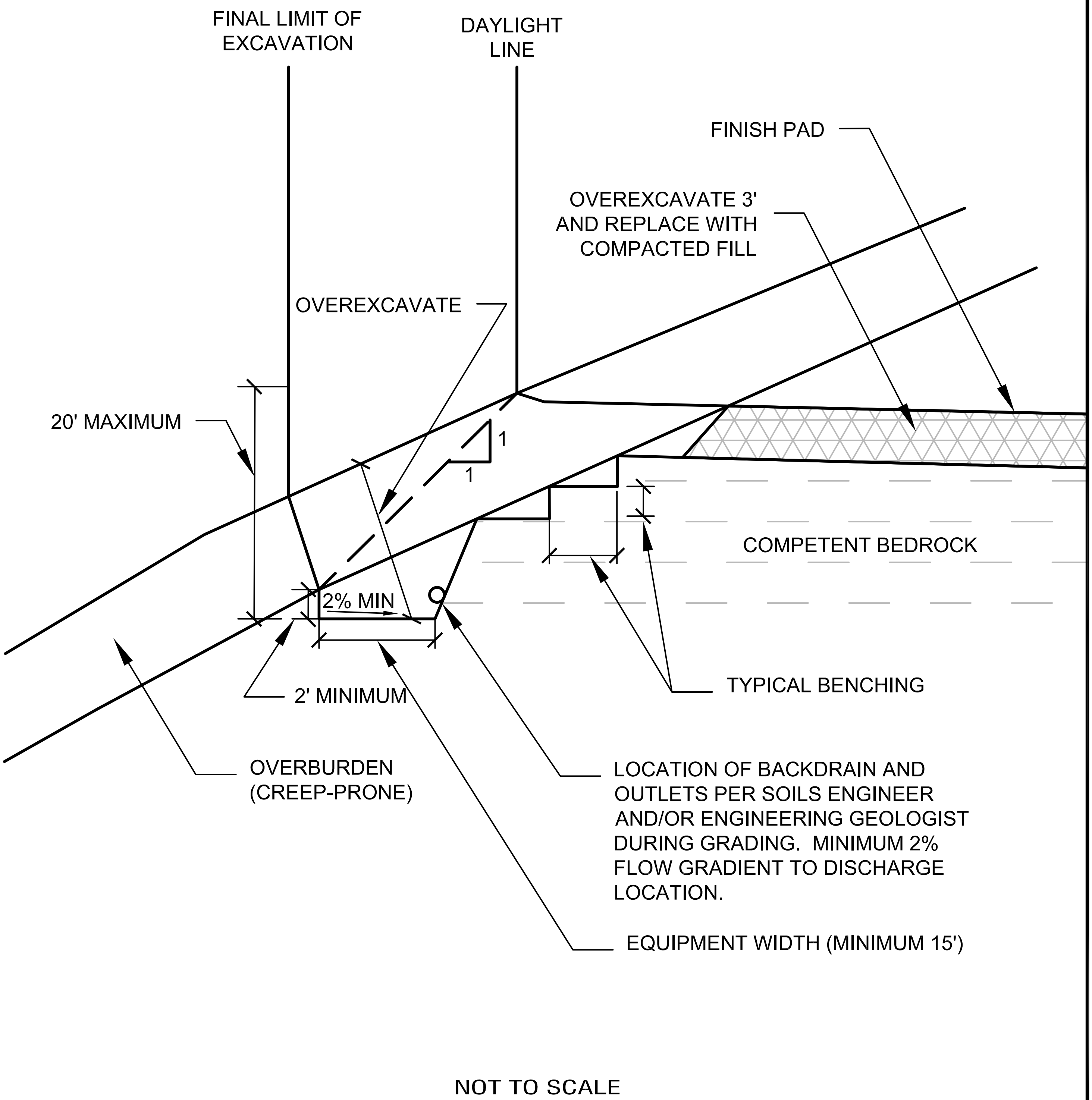
TYPICAL SLOPE STABILIZATION FILL DETAIL



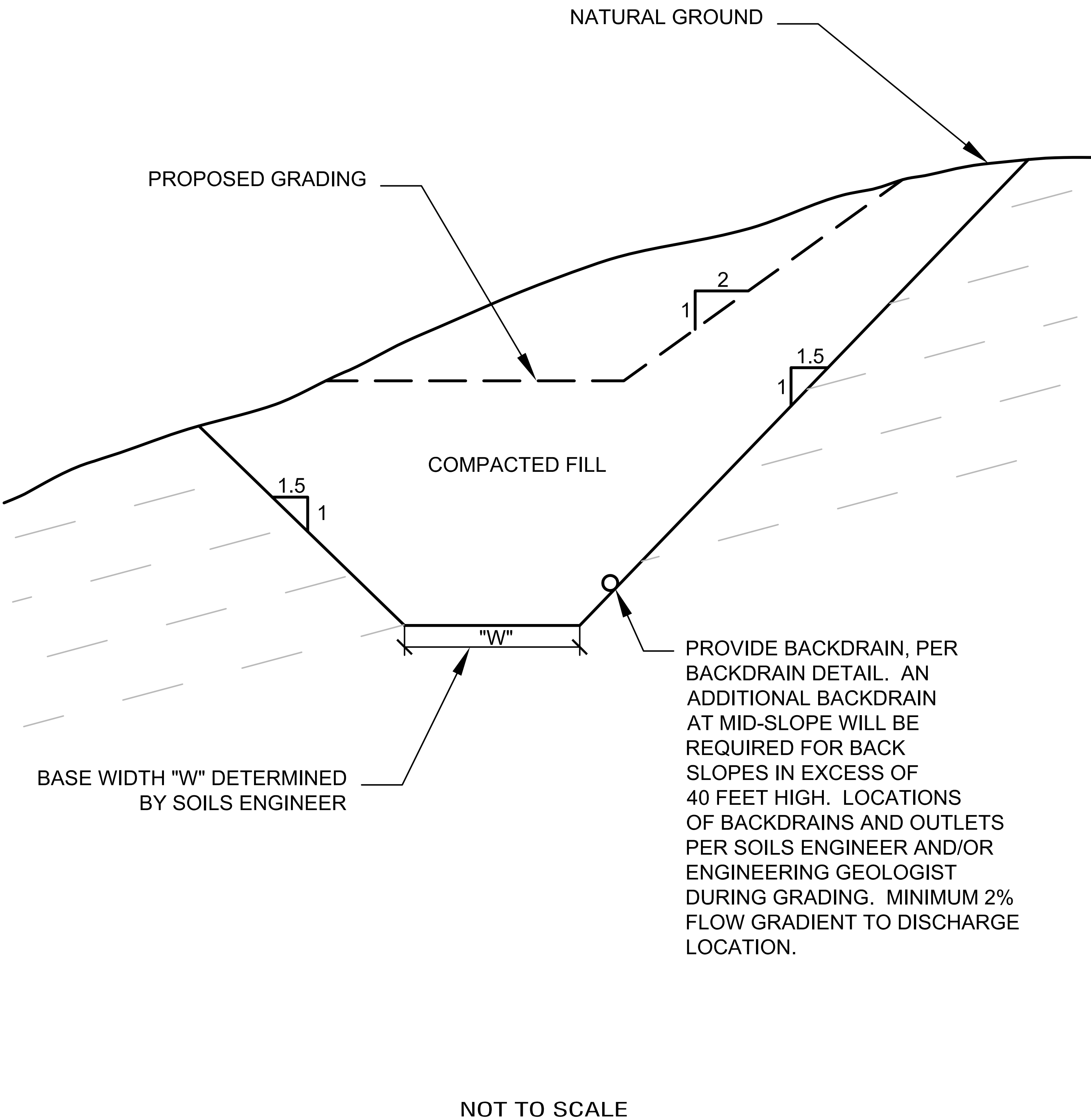
DIMENSIONS ARE MINIMUM RECOMMENDED

NOT TO SCALE

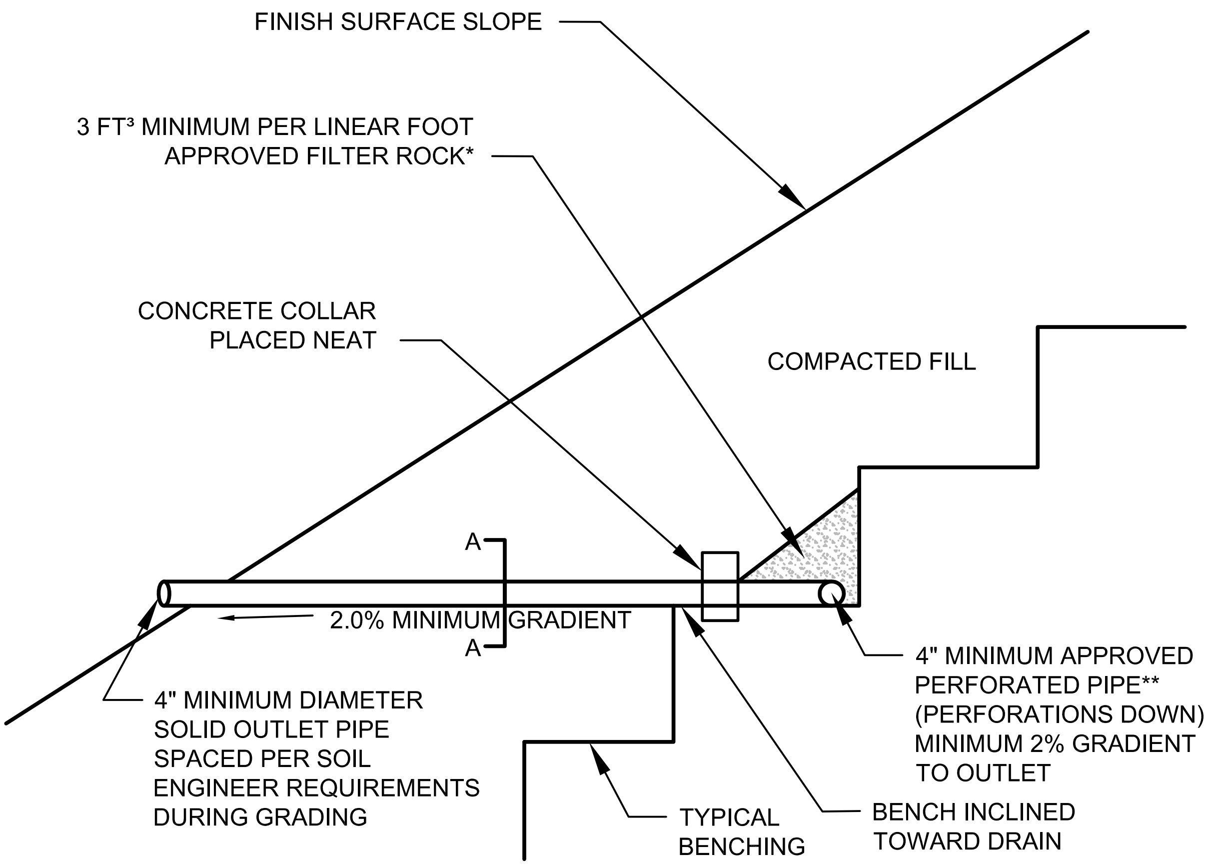
TYPICAL BUTTRESS FILL DETAIL



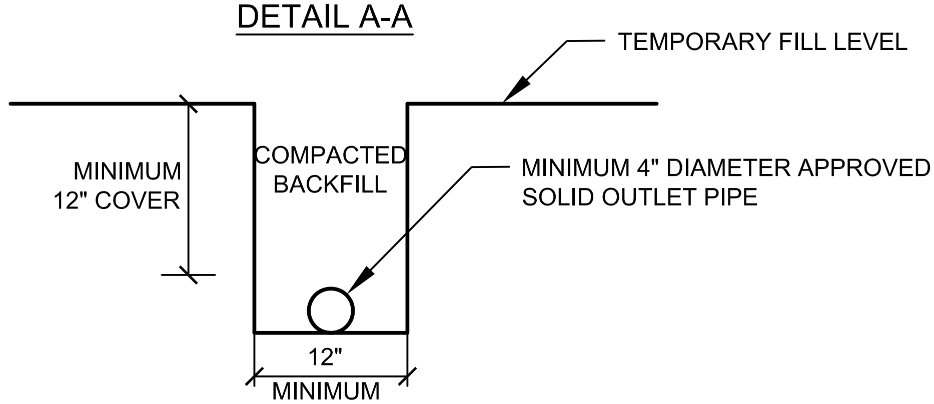
DAYLIGHT SHEAR KEY DETAIL



TYPICAL SHEAR KEY DETAIL



DETAIL A-A



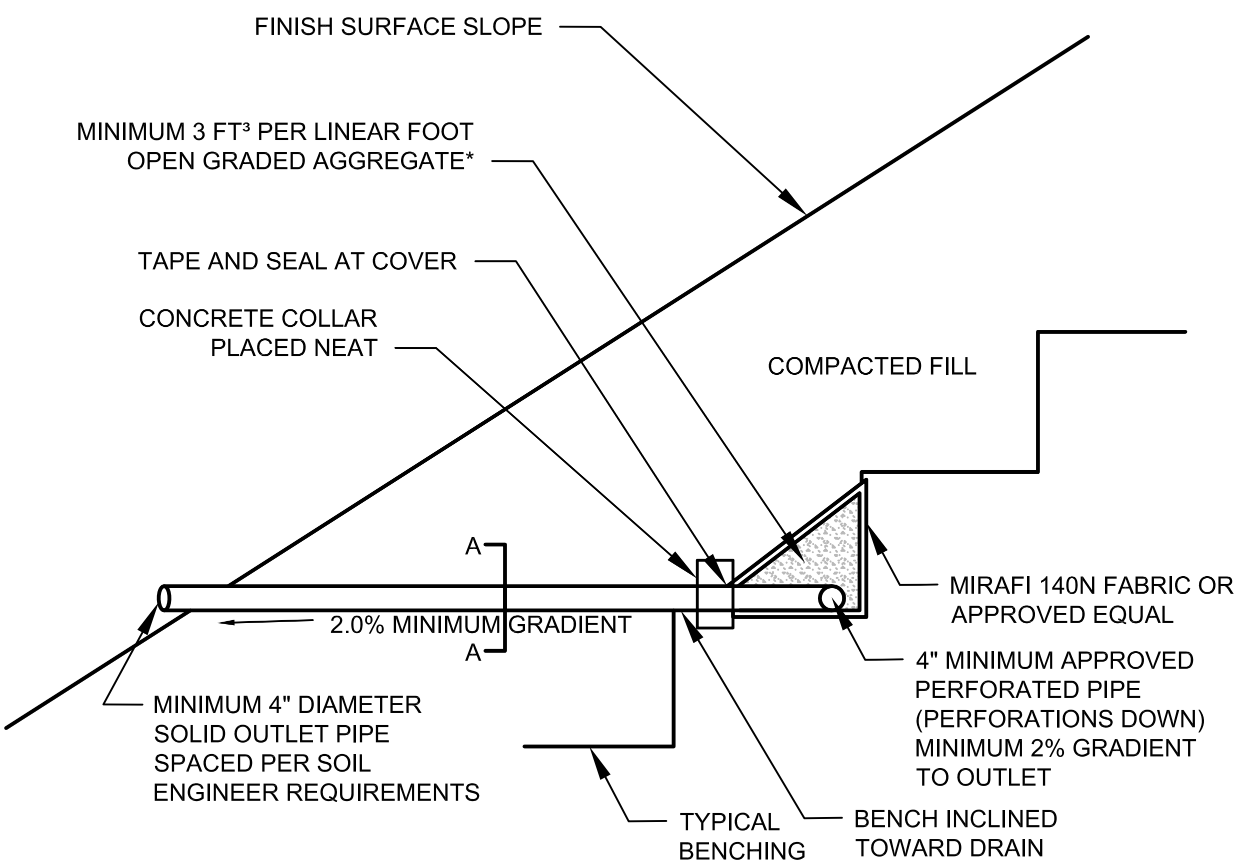
**APPROVED PIPE TYPE:
SCHEDULE 40 POLYVINYL CHLORIDE
(P.V.C.) OR APPROVED EQUAL.
MINIMUM CRUSH STRENGTH 1000 PSI

*FILTER ROCK TO MEET FOLLOWING
SPECIFICATIONS OR APPROVED EQUAL:

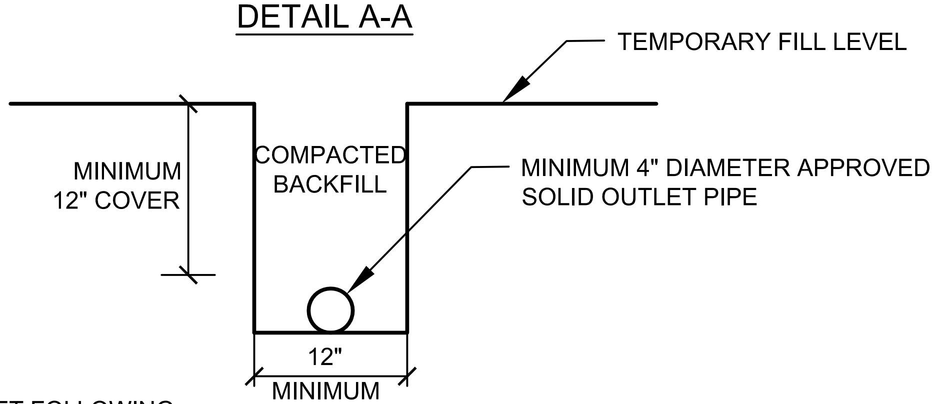
SIEVE SIZE	PERCENTAGE PASSING
1"	100
¾"	90-100
⅜"	40-100
NO. 4	25-40
NO. 30	5-15
NO. 50	0-7
NO. 200	0-3

NOT TO SCALE

TYPICAL BACKDRAIN DETAIL



DETAIL A-A

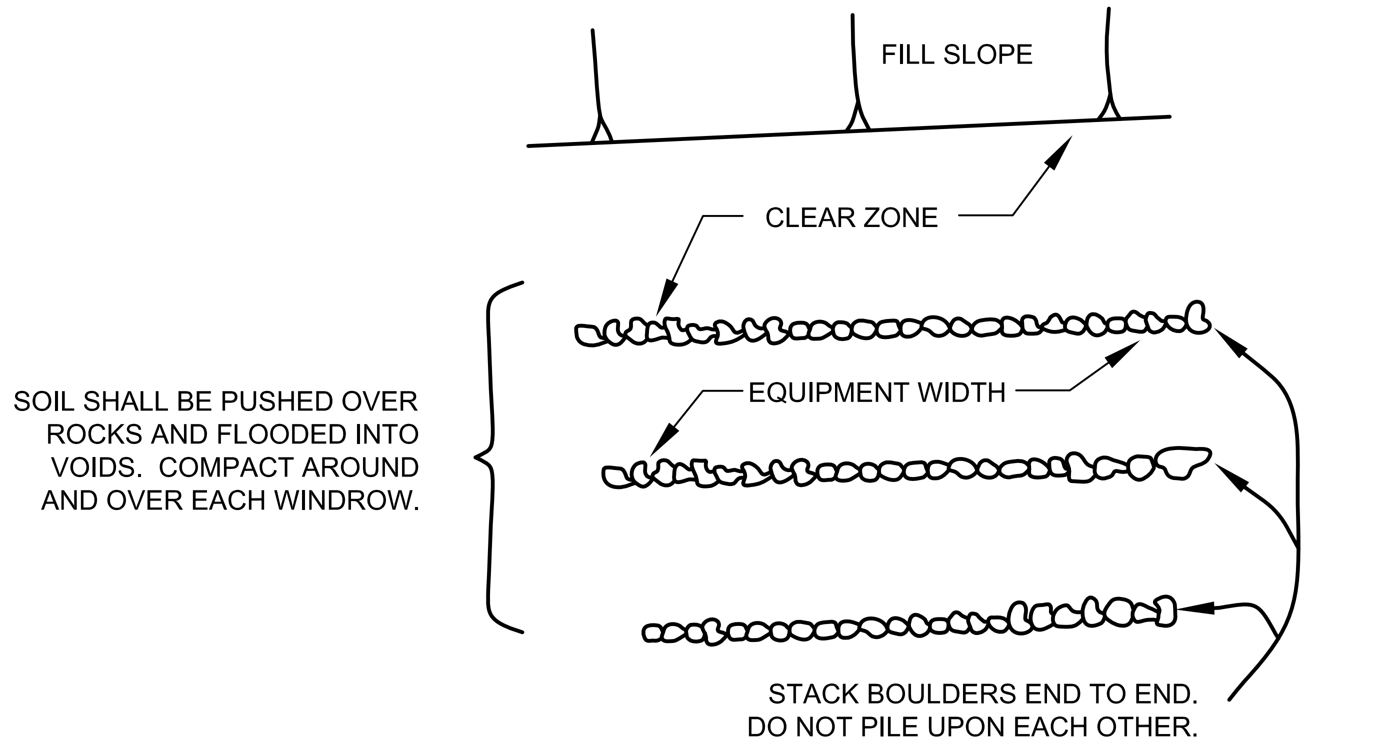


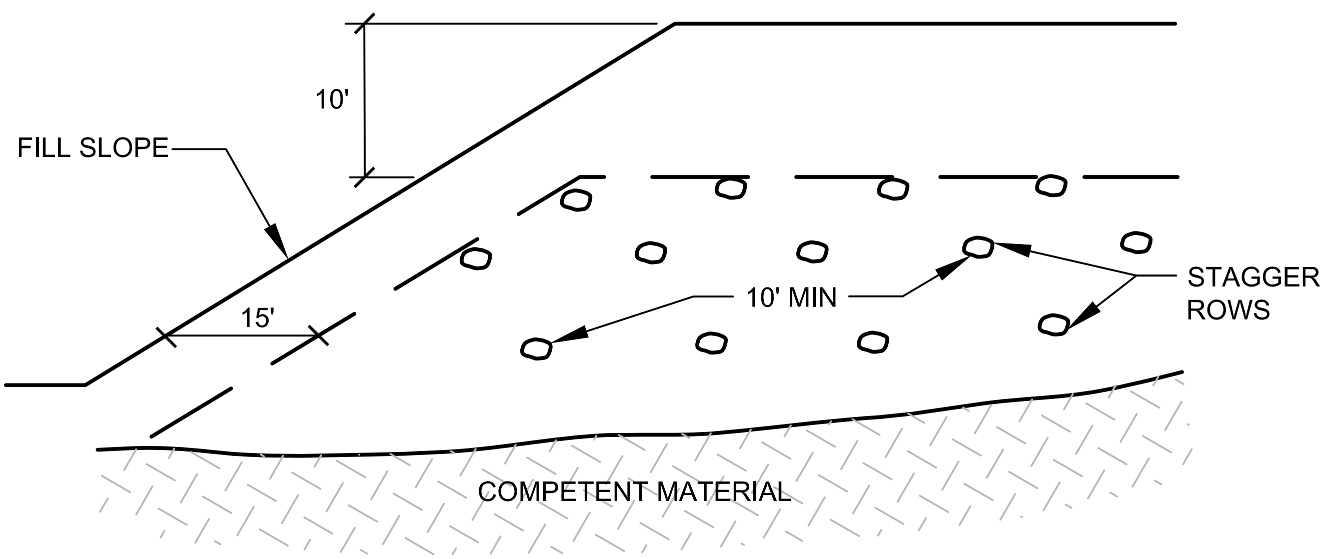
*NOTE: AGGREGATE TO MEET FOLLOWING SPECIFICATIONS OR APPROVED EQUAL:

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

NOT TO SCALE

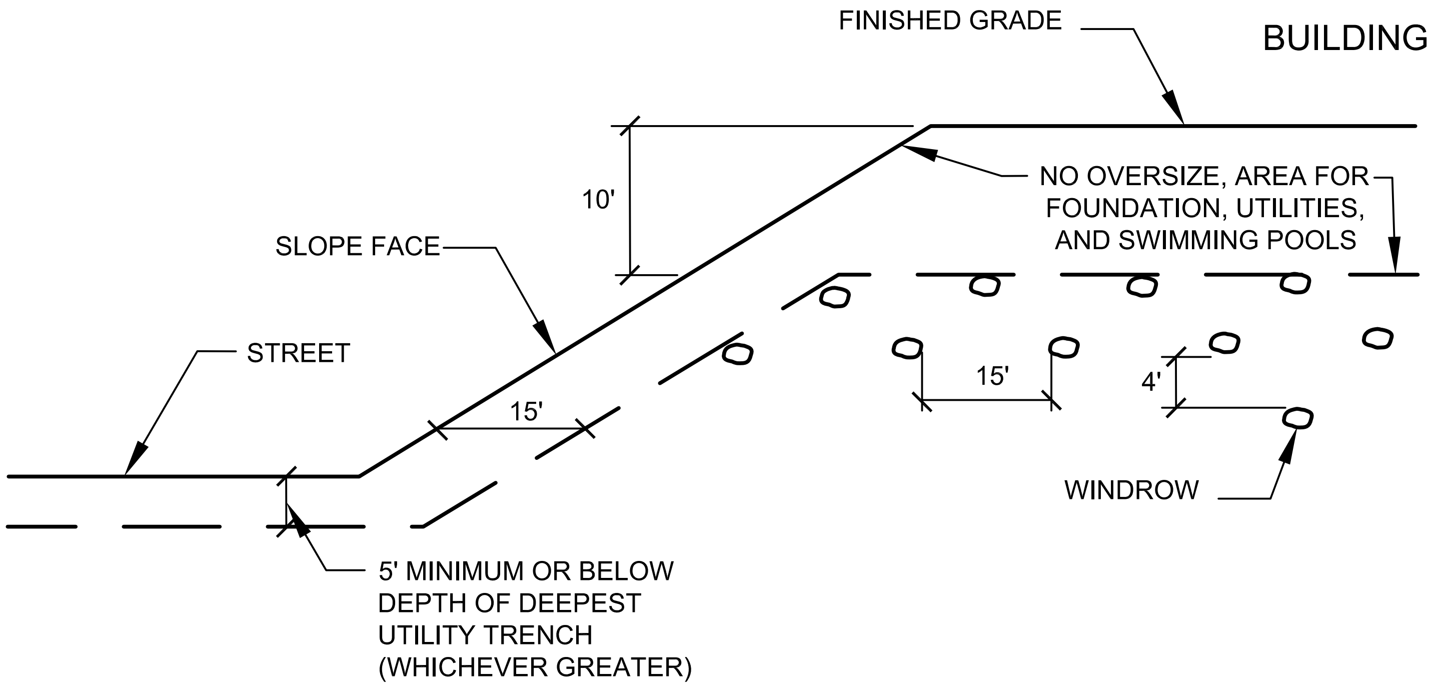
BACKDRAIN DETAIL (GEOFRABIC)



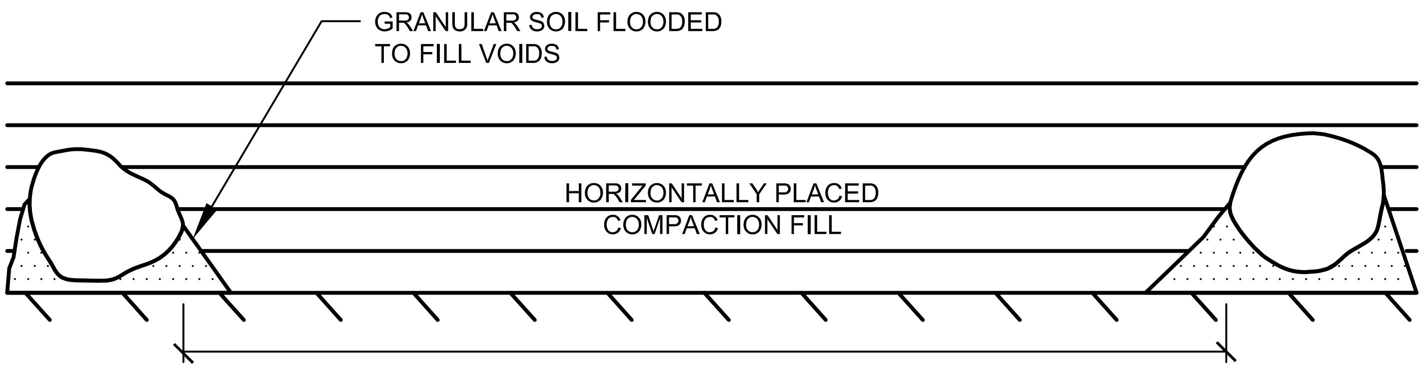


NOT TO SCALE

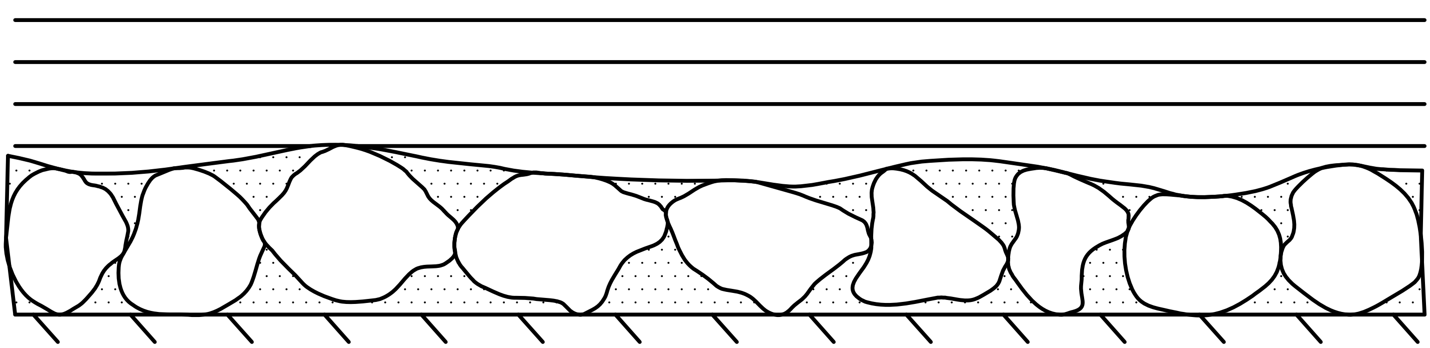
ROCK DISPOSAL DETAIL



TYPICAL WINDROW DETAIL (EDGE VIEW)



PROFILE VIEW

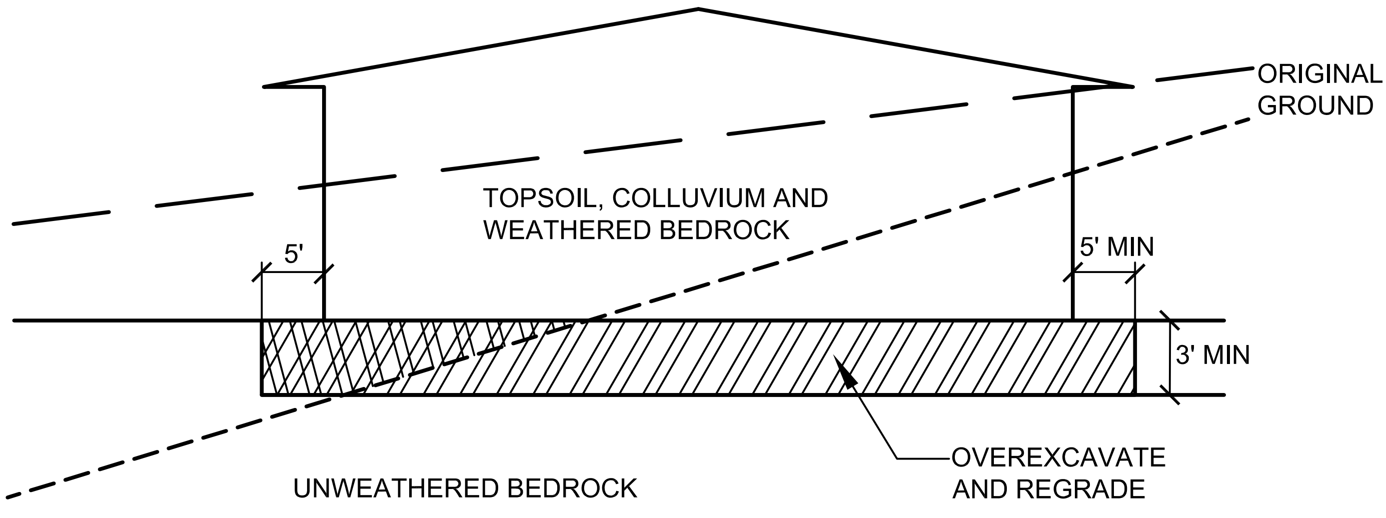


NOT TO SCALE

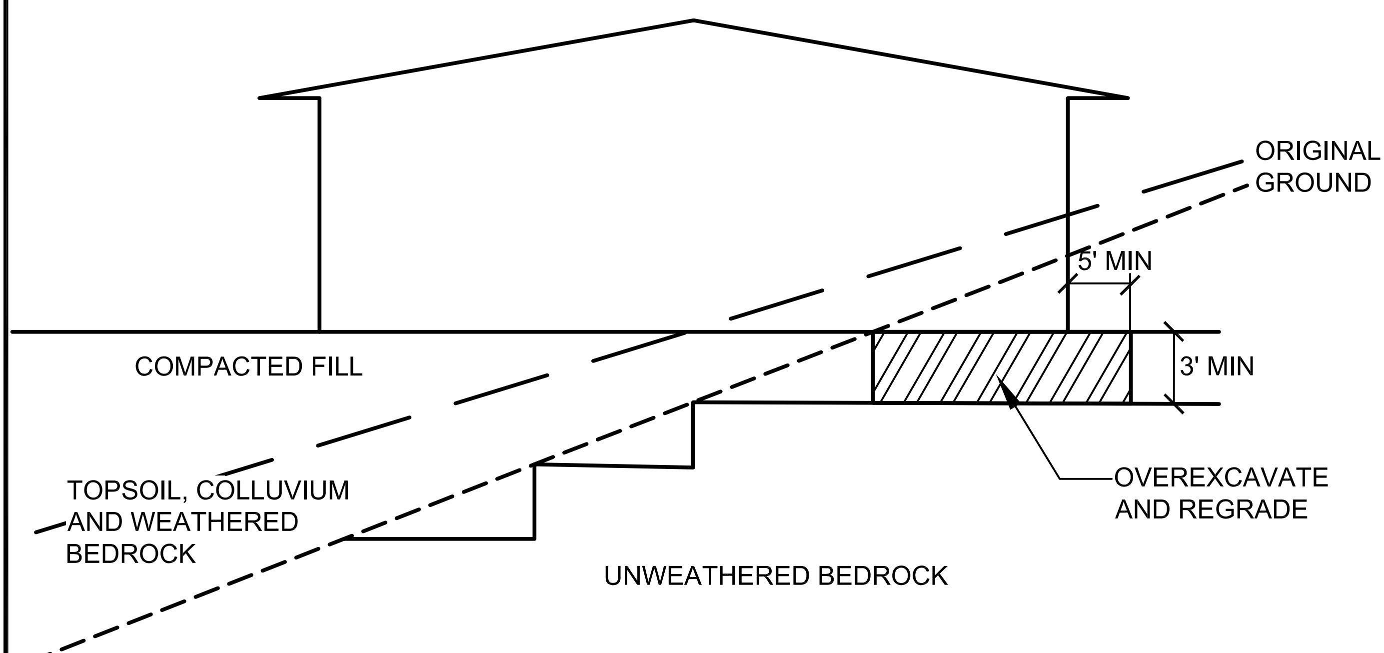
ROCK DISPOSAL DETAIL

GENERAL GRADING RECOMMENDATIONS

CUT LOT



CUT/FILL LOT (TRANSITION)



NOT TO SCALE

TRANSITION LOT DETAIL

APPENDIX E

US SEISMIC DESIGN VALUES

ATC Hazards by Location

Search Information

Address:	10792 W Larch Rd, Tracy, CA 95304, USA
Coordinates:	37.764117, -121.439058
Elevation:	10 ft
Timestamp:	2024-07-11T18:11:38.735Z
Hazard Type:	Seismic
Reference Document:	ASCE7-16
Risk Category:	II
Site Class:	D

Basic Parameters

Name	Value	Description
S_S	1.109	MCE_R ground motion (period=0.2s)
S_1	0.39	MCE_R ground motion (period=1.0s)
S_{MS}	1.171	Site-modified spectral acceleration value
S_{M1}	* null	Site-modified spectral acceleration value
S_{DS}	0.781	Numeric seismic design value at 0.2s SA
S_{D1}	* null	Numeric seismic design value at 1.0s SA

* See Section 11.4.8

▾Additional Information

Name	Value	Description
SDC	* null	Seismic design category
F_a	1.057	Site amplification factor at 0.2s
F_v	* null	Site amplification factor at 1.0s
CR_S	0.931	Coefficient of risk (0.2s)
CR_1	0.936	Coefficient of risk (1.0s)
PGA	0.461	MCE_G peak ground acceleration
F_{PGA}	1.139	Site amplification factor at PGA
PGA_M	0.525	Site modified peak ground acceleration
T_L	8	Long-period transition period (s)
SsRT	1.109	Probabilistic risk-targeted ground motion (0.2s)
SsUH	1.19	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
SsD	1.5	Factored deterministic acceleration value (0.2s)
S1RT	0.39	Probabilistic risk-targeted ground motion (1.0s)
S1UH	0.417	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
S1D	0.6	Factored deterministic acceleration value (1.0s)
PGAd	0.523	Factored deterministic acceleration value (PGA)

* See Section 11.4.8

The results indicated here DO NOT reflect any state or local amendments to the values or any delineation lines made during the building code adoption process. Users should confirm any output obtained from this tool with the local Authority Having Jurisdiction before proceeding with design.

Please note that the ATC Hazards by Location website will not be updated to support ASCE 7-22. Find out why.

Disclaimer

Hazard loads are provided by the U.S. Geological Survey Seismic Design Web Services.