APPENDIX 5

EEI
Engineering Solutions

UPDATED GEOTECHNICAL EVALUATION

Proposed 9-Acre Jefferson Avenue Multi-family Development

Lot 91 MB008/359 SD TR T L WC
APN 949-220-048
Vacant Land, Jefferson Avenue
Murrieta, Riverside County, California

EEI Project No. PWD-72978.4b

May 12, 2020
Revised June 05, 2020

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UPDATED GEOTECHNICAL EVALUATION

Prepared for:
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Temecula, Ca 92592

Project Site Location:

Lot 91 MB 008/359 SD TR TL W C
APN 949220048
Vacant Land, Jefferson Avenue
Murrieta, California

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Distribution: (1) Addressee via an electronic copy

1.0 INTRODUCTION

1.1 Purpose

The purpose of this Updated Geotechnical Evaluation Report is to provide geotechnical information to Pacific West Development, LP ("Client") regarding the subject property in the City of Murrieta, Riverside County, California. The information gathered in this Updated Geotechnical Evaluation is intended to provide the Client with an understanding of the physical conditions of site-specific subsurface soils, groundwater, and the regional geologic setting which could affect the cost or design of the proposed development at the subject property (Site Vicinity Map-**Figure 1**, Aerial Site Map-**Figure 2**).

This Updated Geotechnical Evaluation has been conducted in general accordance with accepted geotechnical engineering principles as well as in general conformance with the approved proposal and cost estimate for the project by EEI, dated February 7, 2020.

To supplement the existing subsurface data for the site, EEI conducted an additional onsite field exploration which consisted of the drilling, sampling and logging of two (2) hollow stem auger exploratory borings and five (5) Cone Penetration Test (CPT) soundings on April 18, 2020 and April 4, 2020 respectively. This report has been prepared for the sole use of Pacific West Development, LP (Client). Other parties, without the express written consent of EEI and Pacific West Development, LP should not rely upon this report.

1.2 Project Description

It is our understanding that the proposed development of the subject property will involve construction of seven (7) multi-story, multi-family residential structures with the associated appurtenant improvements, including a leasing office, clubhouse building, and swimming pool and carport structures. No other information regarding the proposed site development is known at this time.

No detailed grading plans were provided to EEI at the time of our preparation of this report. However, grading of the subject property is anticipated to include cuts and fills on the order of less five feet (exclusive of remedial grading). No foundation plans were provided to EEI at the time of preparation of this report. However, foundation loads assumed by EEI for the engineering analysis are up to 3,000 pounds per lineal foot and 50 kips for wall and column loads respectively.

1.3 Scope of Services

The scope of our services included:

- A review of the readily available data pertinent to the subject property and immediate vicinity, including published and unpublished geologic reports/maps and soils data for the area (**References**).
- Conducting a geotechnical reconnaissance of the subject property and nearby vicinity.
- Coordination with Underground Service Alert (USA) to identify the presence of underground utilities for clearance of the proposed exploratory boring locations.

- Drilling and logging of two (2) hollow stem auger (HSA) (8-inch diameter) exploratory borings and five (5) Cone Penetrometer Test (CPT) soundings **(Geotechnical Map, Figure 3).**

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

- The preparation of this report which presents our findings, conclusions, and recommendations for the proposed residential development.

2.0 BACKGROUND

2.1 Subject Property Description

The site has the legal description of Lot 91 MB008/359 SD TR T L W C (APN 949-220-048), in the City of Murrieta, Riverside County, California. Overall, the site is located on the north side of Jefferson Avenue approximately 900 feet southeast of Ivy Street (Site Location Map, **Figure 1**). The approximate geographic site coordinates are 33.5547° North latitude and 117.2017° West longitude. The predominantly undeveloped property consists of approximately 9-acres of relatively flat to moderately sloping terrain surrounded by the existing commercial parcels to the north and west, approximately 30-acres of vacant land to the east, and Jefferson Avenue to the south. At the time of our investigation, the site was covered with light to moderate vegetation growth consisting of low lying brush, grass and weeds. Additionally, an abandoned approximately 10 ft. by 15 ft., stone and grout pump house structure is present at the site and is located near Jefferson Avenue.

2.2 Topography

The subject property is located on the United States Geological Survey (USGS), 7.5-Minute, Murrieta, CA Topographic Quadrangle (2018). The existing surface elevation at the subject property ranges from approximately 1100 to 1110 feet above mean sea level (amsl).

2.3 Previous Geotechnical Investigations

The Client provided EEI with copies of geotechnical investigations performed previously for the site proposed improvements by EnGEN Corporation (Dcember 7 & December 12, 2000) for our review.

The reported subsurface investigations consisted of drilling eight (8) small diameter hollow stem auger borings ranging in depth from 11.5 feet to 51.5 feet and excavating an approximately 500 foot long and 12 to 14 foot deep trench within the site area.

Generally, slope wash and alluvial deposits were encountered to be underlain by the bedrock of Pauba Formation within the borings and trench excavation. Maximum thickness of 47 feet of alluvial deposits were encounterd in the borings located within the low lying areas of the exterem northern portion of the site. Thin layers, less than approximately 3 foot thick, of alluvial /slope wash deposits underlain by bedrock of Pauba Formation were reported within the remaining portions of the site area.

Groundwater was encountered at approximately 23 feet below grade. The report has concluded that due to the presence of shallow groundwater and the granular nature of the alluvaial/slope wash deposits, the possibility of hazards due to liquefaction exists in the low lying area at the extreme northeasren portion of the site. Soils expansion was listed as very low to low, with an expansion index (EI) of 18. Generally, field data provided in these reports were consistent with EEI's subsurface investigation.

According to the existing published geological information, the southwestern portion of the site is partially located within the Alquist-Priolo (AP) Earthquake Fault Zone. EnGEN performed a fault hazard study (December 7, 2000). During this study, a 500 feet long trench was excavated on the site. The west end of the trench began at about 40 feet from the edge of the pavement along Jefferson Avenue. It was reported that the trench was approximately 25 feet wide at the top and 4 feet wide at the bottom. The trench ranged from 12 to 14 feet deep. The west and east ends of the trench were located approximately 232 feet and 187 feet from the northwest property line respectively.

No indication of active faulting (Holocene age, less than 11,000 before present) was found during this investigation. However, the EnGEN report recommended the establishment of a 50 foot-wide "Restricted Use Zone (RUZ)" from the edge of pavement on Jefferson Avenue for the proposed structures. Based on the results of our investigation and review of the available data, EEI concurs with the findings and recommendations of this fault hazard study.

3.0 FIELD EXPLORATION, SUBSURFACE CONDITIONS AND LABORATORY TESTING

3.1 Field Exploration

To supplement the existing subsurface data obtained during previous investigations at the site, EEI conducted an additional onsite field exploration which consisted of the drilling, sampling and logging of two (2) hollow stem auger exploratory borings and five (5) Cone Penetration Test (CPT) soundings on April 18, 2020 and April 4, 2020 respectively. Our exploratory borings (B-9-1 and B-9-2) were advanced to depths ranging from approximately 6 to 26.5 feet below the existing grade. The approximate locations of our borings are shown on **Figure 3**.

A truck mounted Ingersoll-Rand A-300 hollow stem auger (HSA) drilling rig was utilized to advance all four exploratory borings. Blow count (N) values were determined utilizing a 140-pound hammer, falling 30-inches onto a Standard Penetration Test (SPT) split-spoon sampler and a Modified California split-tube sampler. The blows per 6-inch increment required to advance the 18-inch long SPT and 18-inch long Modified California split-tube samplers were measured at various depth intervals are recorded on the boring logs, and are presented in **Appendix A** (Soil Classification Chart and Boring Logs).

Relatively "undisturbed "samples were collected in a 2.42-inch (inside diameter) California Modified split-tube sampler for visual examination and laboratory testing. Representative bulk samples were collected from the exploratory borings for appropriate laboratory testing. The soils were classified in accordance with the Unified Soil Classification System (ASTM D-2487).

Our CPT soundings (CPT-9-1 through CPT-9-5) were advanced to depths ranging from approximately 3.5 to 8 feet below grade and terminated due to refusal. The CPT soundings are presented in **Appendix A**.

3.2 Laboratory Testing

Selected samples obtained from our borings were tested to evaluate pertinent soil classification and engineering properties and enable development of geotechnical conclusions and recommendations. The laboratory tests consisted of:

- Fines Content
- Atterberg Limits
- Direct Shear
- Corrosivity
- Expansion Index
- Proctor

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

4.0 GEOLOGIC SETTING AND SUBSURFACE CONDITIONS

4.1 Geologic Setting

The subject property lies within the Peninsular Ranges Geomorphic Province of southern California. This province consists of a series of ranges separated by northwest trending valleys; sub parallel to branches of the San Andreas Fault Zone (CGS, 2002). The Peninsular Ranges Geomorphic Province, one of the largest geomorphic units in western North America, extends from the Transverse Ranges Geomorphic Province and the Los Angeles Basin, south to Baja California. It is bound on the west by the Pacific Ocean, on the south by the Gulf of California and on the east by the Colorado Desert Province. The Peninsular Ranges are essentially a series of northwest-southeast oriented fault blocks (CGS, 2002).

More regionally speaking, the subject property is located in the Northern Peninsular Range on the southern sector of the structural unit known as the Perris Block. The Perris Block is bounded on the northeast by the San Jacinto Fault Zone, on the southwest by the Elsinore Fault Zone, and on the north by the Cucamonga Fault Zone. The southern boundary of the Perris Block is not as distinct, but is believed to coincide with a complex group of faults trending southeast from the Murrieta area. The Peninsular Ranges are characterized by large Mesozoic age intrusive rock masses flanked by volcanic, metasedimentary, and sedimentary rocks. Various thicknesses of colluvial/alluvial sediments derived from the erosion of the elevated portions of the region fill the low lying areas. Alluvium and Pauba Formation bedrock materials underlie the site.

4.2 Site Geology

Information obtained from our subsurface investigation as well previous site investigations indicate that the subject site is underlain by variable thicknesses of alluvial deposits on top of the bedrock of the Pauba Formation.

4.2.1 Alluvial Deposits

Up to 47 feet of alluvial deposits were encounterd in the borings located within the low lying areas of the exterem northern portion of the site. Thin layers, less than approximately 5 foot thick, of alluvial /slope wash deposits underlain by bedrock of Pauba Formation were reported within the remaining portions of the site area. Alluvial deposits generally consisted of brown, reddish brown silty sands and silty sandy clays, moist to very moist, and range in consistency from soft and loose to medium stiff and medium dense with traces of caliche.

4.2.2 Pauba Formation

Pauba Formation bedrock was encountered immediately underneath the alluvial deposits at the site. However, Pauba Formation bedrock was also locally encountered at the surface within the eastern portion of the site area. The Pauba Formation bedrock was generally observed to consist of interbedded layers of yellowish brown to orangish brown siltstone, claystone and sandstone units that were stiff to hard, massive to thinly bedded, poorly to moderately indurated and highly weathered, moist to very moist with traces of caliche.

4.3 Groundwater

Groundwater was encountered during our subsurface investigation as well previous investigations at the site at approximate depths of 21 to 23 feet below the existing ground surface. Based on our review of relevant existing literature, we anticipate the depth of groundwater to be on the order of 25 feet within the immediate area.

5.0 GEOLOGIC HAZARDS

5.1 Seismic Design Values

The subject property, like most of southern California, will be subject to strong ground shaking during major earthquakes. The site is partially located within an Alquist- Priolo Earthquake Fault Zone for the Elsinore Fault Zone. The Elsinore Fault Zone is thought to be located immediately southwest of Jefferson Avenue.

EEI utilized seismic design criteria provided in accordance with the California Building Code (CBC, 2019) and ASCE 7-16. Final selection of the appropriate seismic design coefficients should be made by the project structural engineer based on the local laws and ordinances, expected building response, and desired level of conservatism. The site coefficients and adjusted maximum considered earthquake spectral response accelerations in accordance with ASCE 7-16 are presented in **Table 1**.

Table 1
ASCE 7-16 Seismic Design Values

Parameter	Value
Site Coordinates	Latitude 33.5547° Longitude -117.2017°
Mapped Spectral Acceleration Value at Short Period: $\mathbf{S_s}$	1.620g.
Mapped Spectral Acceleration Value at 1-Second Period: $\mathbf{S_1}$	0.608g.
Site Soil Classification	D
Short Period Site Coefficient: $\mathbf{F_a}$	1.00
1-Second Period Site Coefficient: $\mathbf{F_v}$	1.70
Adjusted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration at Short Period: $\mathbf{S_{MS}}$	1.620g.
Adjusted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration at 1-Second Period: $\mathbf{S_{M1}}$	1.034g.
Design Spectral Response Acceleration at Short Periods: $\mathbf{S_{DS}}$	1.080g.
Design Spectral Response Acceleration at 1-Second Period: $\mathbf{S_{D1}}$	0.689g.
Peak Ground Acceleration Adjusted For Site Class Effects: $\mathbf{PGA_M}$	0.793g.

It should be realized that the purpose of the seismic design utilizing the above parameters is to safeguard against major structural failures and loss of life, but not to prevent damage altogether. Even if the structural engineer provides designs in accordance with the applicable codes for seismic design, the possibility of damage cannot be ruled out if moderate to strong shaking occurs as a result of large earthquake. This is the case for essentially all structures in Southern California.

5.2 Faulting and Surface Rupture

According to existing published geological information, the southwestern portion of the site is partially located within an Alquist-Priolo Earthquake Fault zone. EnGEN performed a fault hazard study (December 7, 2000). During this study a 500 feet long trench was excavated on the site. The west end of the trench began at about 40 feet from the edge of the pavement along Jefferson Avenue. It was reported that the trench was approximately 25 feet wide at the top and 4 feet wide at the bottom. The trench ranged from 12 to 14 feet deep. The west and east ends of the trench were located approximately 232 feet and 187 feet from the northwest property line respectively.

No indication of active faulting (Holocene age, less than 11,000 before present) was found during this investigation. However, the EnGEN report recommended the establishment of a 50 foot-wide "Restricted Use Zone (RUZ)" from the edge of pavement on Jefferson Avenue for the proposed structures.

The closest major faults that are likely to affect the subject site are listed below in **Table 2.**

TABLE 2
Nearby Active Faults

Fault	Distance in Miles (Kilometers)[1]	Maximum Magnitude[1]
Elsinore;W+GI+T+J	0.05 (0.08)	7.77
Elsinore;W+GI+T	0.05 (0.08)	7.48
Elsinore;T	0.05 (0.08)	7.07
Elsinore;T+J	0.05 (0.08)	7.54
Elsinore;T+J+CM	0.05 (0.08)	7.64
Elsinore;GI+T+J+CM	0.05 (0.08)	7.74
Elsinore;GI+T	0.05 (0.08)	7.29

5.3 Landslides and Slope Stability

Seismically induced landslides and other slope failures are common occurrences during or soon after earthquakes. However, due to the presence of the very low on-site gradient, the potential for seismically induced landsliding to occur is very low.

5.4 Liquefaction and Dynamic Settlement

Liquefaction is a sudden loss of strength of saturated, cohesionless soil caused by cyclic loading (e.g., earthquake shaking). Generally, liquefaction occurs in predominantly poorly consolidated granular soil where the groundwater depth is less than 50 feet.

Review of the State of California Seismic Hazard Map for the Murrieta Quadrangle indicates that the subject property is not situated within a mapped Liquefaction Zone. However, groundwater table at the site is on the order of 25 feet below grade, and Up to 47 feet of relatively loose and generally granular alluvial deposits were encounterd within the north-northeastern portion of the site. Based on the results of this and previous investigations at the site it is our opinion that the potential of liquefaction in the the north- northeastern portion of the site is considered likely. Therefore, remedial measures to alleviate and/or minimize the effect of liquefaction on the propsed improvements within the northern portion of the site are necessary.

The approximate limits of the liquefiable area are delineated on the Geotechnical Map **(Figure 3).**

5.5 Flooding

The subject property is not located within a Tsunami Evacuation Area; therefore, damage due to tsunami is considered low.

EEI reviewed the Federal Emergency Management Agency (FEMA) Flood Hazard Map online database (FEMA, 2008) to determine if the subject property was located within an area designated as a Flood Hazard Zone. According to the Flood Insurance Rate Map (FIRM), Map No. 06065C2715G, effective August 8, 2008, the subject property is located within an area of minimal flood hazard, identified as Flood Zone X.

Additionally; the potential for earthquake-induced flooding at the site, caused by the failure of dams or other water-retaining structures as a result of earthquakes is considered very low. The risk of seiches affecting the site during a nearby seismic event is also considered low.

5.6 Expansive Soil and Subsidence

Underlying soil/bedrock at the site possess low expansive characteristics. The expansion potential of these materials is not considered to pose a hazard for the proposed site development.

6.0 CONCLUSIONS

Based on our field exploration, laboratory testing and engineering and geologic analysis, it is our opinion that the subject property is suitable for the proposed development from the geotechnical engineering and geologic viewpoint provided the recommendations presented in our geotechnical report are incorporated into the design and construction phase of the project. However, there are existing geotechnical conditions associated with the property that will warrant mitigation and/or consideration during planning stages. If site plans and/or the proposed building location are revised, additional field studies may be warranted to address proposed site-specific conditions. The main geotechnical conclusions for the project are presented in the following text.

- Drilling and logging of two (2) hollow stem auger (HSA) (8” diameter) exploratory borings (B-1-9 and B-2-9) and five CPT soundings to supplement the existing subsurface data obtained during previous site investigations. Our borings/CPT soundings were advanced to depths ranging from approximately 3.5 to 26.5 feet below the existing ground surface. Subsurface materials encountered in our exploratory borings/CPT soundings consisted of 3 to 23 feet of alluvial deposits underlain by the bedrock of Pauba formation. However, previous investigation at the site encountered up to 47 feet of alluvial deposits underlain by the bedrock of Pauba Formation. Drilling refusal was not encountered in any of the two exploratory borings. However, all of the CPT soundings were terminated due to refusal.

- Groundwater was encountered in our exploratory boring B-9-1 at 21 feet below the existing grade at the time of our subsurface exploration. Groundwater was encountered during previous investigation at the site at approximate depths of 23 feet below the existing ground surface. Based on our review of relevant existing literature, we anticipate the depth of groundwater to be on the order of 25 feet within the immediate area.

- According to the existing published geological information, the southwestern portion of the site is partially located within the Alquist-Priolo (AP) Earthquake Fault zone. EnGEN performed a fault hazard study (December 7, 2000). During this study a 500 feet long trench was excavated on the site. The west end of the trench began at about 40 feet from the edge of the pavement along Jefferson Avenue. The west and east ends of the trench were located approximately 232 feet and 187 feet from the northwest property line respectively.

- No indication of active (Holocene-Age) faulting was found during this investigation. However, this report recommended establishment of a 50 foot wide “Restricted Use Zone (RUZ)” from the edge of pavement on Jefferson Avenue for the proposed structures. EEI concurs with the findings and recommendations of this fault hazard study.

- Liquefaction occurs when loose, saturated sands and silts are subjected to strong ground shaking. The strong ground shaking causes pore-water pressure to raise, soils to lose their shear strength and become liquid; potentially resulting in large total and differential ground surface settlements as well as possible lateral spreading during an earthquake.

  Due to the near surface presence of relatively loose and granular alluvial materials and shallow groundwater underlying the site, it is our opinion that the potential for liquefaction to occur at the northern portion of the subject property is very likely. Therefore, remedial measures to alleviate and/or minimize the effect of liquefaction on the propsed improvements within the northern portion of the site are necessary.

- Mitigation to preclude or reduce the risk of damage resulting from liquefaction could add significantly to the cost of the project. The decision regarding the extent of mitigation measures employed must be made by the owner considering the costs of the measures relative to the risk of damage and the importance of the structure. Detailed design criteria for alternative mitigation measures are beyond the scope of this investigation. Pending completion of the evaluation of the alternatives by the design team and the owner, this report will provide grading and foundation design criteria which based on our judgement will provide the most reasonable balance between cost and mitigation.

- Suitable alternative mitigative measures to minimize the effect of liquefaction on the proposed improvements could include Remedial Grading and Rigid Shallow Foundations; Driven Piles; and insitue densification methods such as Vibroflotation, Vibro-Compaction, Vibro-Piers, Dynamic Deep Compaction and Compaction Grouting. However; these methods require mobilization of special equipments and will probably not to be economical for a project of this size. Pending completion of the evaluation of alternatives by the design team and the owner, we judge that combination of a “Remedial Grading” and utilization of a “Rigid Shallow Foundation System” will provide the most reasonable balance between cost and mitigation. Accordingly, recommended design criteria for this option are presented in the following sections.

- The existing onsite soils are unsuitable for the support of any engineered fill, structures or buildings in their current condition. Remedial grading operation at the site should include removal of the surficial loose alluvial deposits through out the entire site. These removals should extend to at least five feet below the proposed bottom of the foundation system within the designated liqefiable area; and to the contact with the firm bedrock of the Pauba Formation within the remaing portions of the site. A minimum of 5 feet of removal and recompaction should be anticipated.

- The subject property and immediate vicinity are relatively flat with very low relief with no slopes present; therefore, the potential for slope instability and lateral spreading is very low.

- Underlying soil/bedrock at the site possess low expansive characteristics. The expansion potential of these materials is not considered to pose a hazard for the proposed site development.

- The existing onsite fill soils/natural deposits are excavatable with conventional construction equipment.

- The onsite subsurface materials appear to be suitable for use as a structural fill provided that they are moisture conditioned (as needed) and meet EEI's recommendations for size and organic content and are properly compacted.

- Based on our analysis we judge that combination of remedial grading and "Rigid Conventional Shallow Foundation system" for support of the proposed structures will provide the most reasonable balance between the cost and liquefaction mitigation within the designated liqefiable area. A conventional shallow foundation system in conjunction with a concrete slab-on-grade floor appears to be suitable for support of the improvements within the remaining portion of the site.

7.0 GRADING RECOMMENDATIONS

7.1 General

The proposed site development should be constructed in general conformance with the guidelines presented herein, as well as the California Building Code (CBC 2019) and the requirements of local jurisdictions. Additionally, general Earthwork and Grading Guidelines are provided herein as **Appendix C.**

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

The recommendations presented herein are based on the preliminary information provided to us regarding site development. EEI should be provided with grading and foundation plans once they are available so that we can determine if the recommendations provided in this report remain applicable.

7.2 Site Preparation and Grading

When grading is conducted, it should be performed in accordance with good construction practice, applicable Code requirements, and the following recommendations.

Debris and other deleterious material, such as organic soils, tree rootballs and/or environmentally impacted earth materials (if any) should be removed from the subject property prior to the start of grading. Areas to receive fill should be properly scarified and/or benched in accordance with current industry standards of practice and guidelines specified in the CBC (2019) and the requirements of the local jurisdiction.

Abandoned trenches should be properly backfilled and tested. If unanticipated subsurface improvements (utility lines, septic systems, wells, utilities, etc.) are encountered during earthwork operations, the Geotechnical Engineer should be informed and appropriate remedial recommendations would then be provided.

Based on the observed subsurface conditions, we anticipate that the onsite alluvial soils and bedrock deposits can generally be excavated with conventional heavy earth moving equipment in good operating condition. The existing alluvial and bedrock materials appear to be suitable for use as structural fill provided they are free of any deleterious material, oversized materials larger than 6-inches in largest dimension and are properly moisture conditioned (as needed) and re-compacted to at least 90 percent of the maximum dry density (based on ASTM D1557).

If import soils are planned, the earthwork contractor should ensure that all proposed fill materials are approved by the Geotechnical Engineer prior to use. Representative soil samples should be made available for testing at least ten (10) working days prior to hauling to the property to allow for laboratory tests. Import fill soils (if planned), should conform to the following specifications:

7.2.1 Import Fill and Select Backfill Material

The import fill and select backfill material should be free of perishable material and should meet the following criteria:

a.	Maximum particle size	1 inch
b.	Maximum Liquid Limit (LL)	5%
c.	Maximum Plasticity Index (PI)	0%
d.	Maximum percentage passing No. 200 sieve	25%
e.	Minimum sand equivalent	30
d.	Maximum Expansive Index (EI) (ASTM D-2849)	30

7.3 Remedial Earthwork

7.3.1 Designated Liquefiable Area

As was mentioned previously in this report, there is a high potential for liquefaction within the northern portion of the site area during earthquake. Therefore; remedial grading measures to preclude or reduce the risk of damage resulting from liquefaction in this area of the site should be considered. Remedial grading operation within this portion of the site should include removal of the existing alluvial deposits to at least five feet below the bottom elevations of the propsed foundation system and replacement as properly compacted fill soil.

When excavations deeper than five feet are made, temporary construction slopes should be no steeper than 1:1 (horizontal to vertivcal). Temporary construction slopes, sheeting and bracing should be provided by the contractor, as necessary, to protect workers in the excavation.

Following removal of the unsuitable materials, the bottom of the resulting excavation(s) should be observed by a representative of EEI to check that unsuitable materials have been sufficiently removed. It should be understood that based on the observations of our field representative, localized deeper removals may be recommended.

7.3.2 Remaining (Nonliqefiable) Portions of the Site Area

Approximately 3 to 5 foot thick of alluvium/slope wash deposits underlain by the bedrock of Pauba Formation were encountered in the remaining portions of the site. Remedial grading in these areas should included removal of the alluvium/slope wash soils to the contact with the firm underlying bedrock deposits and replacing as properly compacted fill. A minimum of 5 foot removal and recompaction should be anticipated.

7.3.3 Transitional Areas

To minimize the risk of differential settlement, entire footing system for each of the proposed structures should be founded in uniform material. Therefore; transitional zones between different materials at the site should be removed and replaced as properly compacted soil. Depth of removal should extend minimum of 3 feet below the bottom Elevation of the footing system.

7.4 Earthwork Operations

Prior to the start of grading operations, utility lines within the project area, if any, should be located and marked in the field so they can be rerouted or protected during the site development. All debris and perishable material should be removed from the site.

The area of site preparation should extend at least five feet beyond any proposed improvements (e.g., building foot print, appurtenant structures, sidewalks, walkways, pavement areas, etc.). Any remnants of past construction debris, perishable materials, and existing soft and disturbed slope wash/alluvial deposits should be excavated to contact with the firm underlying bedrock and / or natural alluvial deposits. When excavations deeper than five feet are made, temporary construction slopes should be no steeper than 1:1 (horizontal to vertical). Temporary construction slopes, sheeting and bracing and/or temporary shoring should be provided by the contractor, as necessary, to protect workers in the excavation. Where excavations undermine existing improvements, temporary structural support should be provided to reduce risk of damage resulting from undercutting. Permanent cut and fill slopes should not be constructed steeper than 2:1.

Where fill is to be placed, the upper 6 to 8 inches of surface exposed by the excavation should be scarified, moisture-conditioned to 2 percent to 4 percent over optimum moisture content, and compacted to minimum 90 percent relative compaction[1]. The fill soil should then be placed in layers less than 8 inches in loose thickness and moisture conditioned to 2 to 4 percent over optimum moisture content, and compacted to minimum 90 percent relative compaction. If localized areas of relatively loose soil prevent proper compaction, over-excavation and re-compaction will be necessary. The onsite soils are generally suitable for use as compacted fill and trench backfill.

[1] Relative compaction refers to the in-place dry density of soil expressed as a percentage of the maximum dry density of the same material, as determined by the ASTM (D1557) test method. Optimum moisture content corresponding to the maximum dry density, as determined by the ASTM (D1557) test method.

7.5 Yielding Subgrade Conditions

The soils encountered at the subject property can exhibit "pumping" or yielding if they become saturated. This can often occur in response to periods of significant precipitation, such as during the winter rainy season. If this occurs and in order to help stabilize the yielding subgrade soils within the bottom of the removal areas, the contractor can consider the placement of stabilization fabric or geo-grid over the yielding areas, depending on the relative severity.

Mirafi 600X (or approved equivalent) stabilization fabric may be used for areas with low to moderate yielding conditions. Geo-grid such as Tensar TX-5 (or approved equivalent) may be used for areas with moderate to severe yielding conditions. Uniform sized, ¾- to 2-inch crushed rock, should be placed over the stabilization fabric or geo-grid. A 12-inch thick section of crushed rock will typically be necessary to stabilize yielding ground.

A filter fabric should be placed over the crushed rock/gravel to prevent migration of fines into the gravel and subsequent settlement of the overlying fill. Fill soils, which should be placed and compacted in accordance with the recommendations presented herein, should then be placed over the filter fabric until design finish grades are reached. The crushed rock/gravel and stabilization fabric or geo-grid should extend at least 5 feet laterally beyond the limits of the yielding areas. These operations should be performed under the observation and testing of a representative of EEI in order to evaluate the effectiveness of these measures and to provide additional recommendations for mitigation, as necessary.

After preparation of the subgrade by removal and replacement with compacted fill, we do not anticipate that any significant subgrade yielding will occur except for normal settlement due to the applied loads.

7.6 Shrinkage and Bulking

Several factors will impact earthwork balancing on the subject property, including shrinkage, bulking, subsidence, trench spoils from utilities and footing excavations, and final pavement section thickness as well as the accuracy of topography. Shrinkage, bulking and subsidence are primarily dependent upon the degree of compactive effort achieved during construction. Shrinkage, bulking and subsidence should be considered by the project civil engineer relative to final site balancing. It is recommended that the site development be planned to include an area that could be raised or lowered to accommodate final site balancing.

7.7 Temporary Site Excavations

It is anticipated that excavations in the onsite materials can be achieved with conventional earthwork equipment in good working order. Temporary excavations within the alluvial materials (considered to be a Type B soil per OSHA guidelines) should be stable at 1.5H: 1V inclinations for short durations during construction, and where cuts do not exceed 10 feet in height. Some sloughing of surface soils should be anticipated. Temporary excavations 4 feet deep or less can be made vertically.

The faces of temporary slopes should be inspected daily by the contractor's competent person before personnel are allowed to enter the excavation. Any zones of potential instability, sloughing or raveling should be brought to the attention of the Engineer and corrective action implemented before personnel begin working in the excavation.

Excavated soils should not be stockpiled behind temporary excavations within a distance equal to the depth of the excavation. EEI should be notified if other surcharge loads are anticipated so that lateral load criteria can be developed for the specific situation. If temporary slopes are to be maintained during the rainy season, berms are recommended along the tops of slopes to prevent runoff water from entering the excavation and eroding the slope faces.

7.8 Slopes

Permanent slopes should be constructed at an inclination of 2:1 H: V or flatter. Faces of fill slopes should be compacted either by rolling with a sheep-foot roller or other suitable equipment, or by overfilling and cutting back to design grade. All slopes are susceptible to surficial slope failure and erosion. Water should not be allowed to flow over the top of slopes. Additionally, slopes should be planted with vegetation that will reduce the potential for erosion.

8.0 Foundation Recommendations

8.1 General

The foundation recommendations provided herein are based on the proposed development Information provided by the Client. EEI should be provided with Grading and Foundation Plans once they are available so that we can determine if the recommendations provided in this report remains applicable. Recommendations by the project's Structural Engineer or Architect may exceed the following minimum recommendations. However; if analyses by the Structural Engineer result in less critical details than are provided herein as minimums, the minimums presented herein should be adopted.

As was mentioned previously in this report, there is a potential for liquefaction of the onsite low density natural alluvial soils during earthquake within the north-northeastern portion of the site exist. Mitigation to preclude or reduce the risk of damage resulting from liquefaction could add significantly to the cost of the project. The decision regarding the extent of mitigation measures employed must be made by the owner considering the costs of the measures relative to the risk of damage and the importance of the structure. Suitable alternative mitigative measures to minimize the effect of liquefaction on the proposed improvements could include Remedial Grading and Rigid Shallow Foundations; Driven Piles; and insitue densification methods such as Vibroflotation, Vibro-Compaction, Vibro-Piers, Dynamic Deep Compaction and Compaction Grouting. However; these methods require mobilization of special equipments and will probably not to be economical for a project of this size. Detailed design criteria for alternative mitigation measures are beyond the scope of this investigation. Pending completion of the evaluation of the alternatives by the design team and the owner, this report will provide foundation design criteria which based on our judgement will provide the most reasonable balance between cost and mitigation.

Based on our analysis we judge that the combination of remedial grading and “Rigid Conventional Shallow Foundation system” for support of the proposed structures will provide the most reasonable balance between the cost and liquefaction mitigation within the designated liqefiable area. A conventional shallow foundation system in conjunction with a concrete slab-on-grade floor appears to be suitable for support of the improvements within the remaining nonliquefiable portion of the site. Accordingly, recommended design criteria for these foundation schemes are presented in the following sections.

8.2 Rigid Shallow Foundations

Foundation support for the proposed structures within the designated liqefiable area could be derived by utilizing a continuous interconnected grade beam foundations embedded within the newly placed compacted fill.

Allowable design parameters for foundations are as follows:

- Minimum depth for interior and exterior footing…………………………………………………2 feet (measured from lowest adjacent soil grade)
- Minimum footing width………………………………………………………………………………….1.5 feet
- Footings should be capable of spanning an unsupported distance of minimum 10 feet
- No isolated footing is allowed
- Allowable bearing capacity (pounds per square foot), (FS ≥ 3)
    a. Sustained loads ………………………………………..……………………………..……..1,500 psf
    b. Transient loads (1/3 allowable increase for wind and seismic)……….2,000 psf
- Resistance to lateral loads
    a. Passive soil resistance (pounds per cubic foot) ……………………………..200 pcf
    b. Coefficient of sliding friction………………………………………………..……………0.35

Footings can be designed to resist lateral loads by using a combination of sliding friction and passive resistance. The coefficient of friction should be applied to dead load forces only. Passive resistance should be reduced by one-third and the upper one-foot of passive resistance should be neglected where the soil is not confined by the slabs or pavement.

For the properly constructed foundations in accordance with the foregoing criteria, total static post-construction settlement from the anticipated structural loads is estimated to be on the order of one-inch. Differential settlement on the order of ½ of total settlement should be anticipated over a distance of 40 feet.

8.3 Shallow Conventional Foundations

Foundation support for the proposed structures within the remaining non-liqufiable portions of the site could be derived by utilizing a conventional, shallow foundation system embedded within the properly compacted fill soils in accordance with the following criteria:

- Minimum depth measured from lowest adjacent grade .... 1.5 feet
- Minimum footing width .... 1.5 feet
- Allowable bearing capacity (pounds per square foot), (FS $\geq$ 3)
    a. Sustained loads...1,500 psf
    b. Total loads (1/3 allowable increase for wind and seismic) ....2,000 psf
- Resistance to lateral loads
    a. Passive soils resistance (pounds per cubic foot) ....200 psf
    b. Coefficient of sliding friction 0.35

Footings can be designed to resist lateral loads by using a combination of sliding friction and passive resistance. The coefficient of friction should be applied to dead load forces only; and Passive resistance should be reduced by one third. For foundations with no sliding friction at the base (foundations resisting uplift loads), 100% of passive resistance could be utilizes. The upper one foot of passive resistance should be neglected where the soil is not confined by the slabs or pavement.

For the properly constructed foundations in accordance with the foregoing criteria, total static post-construction settlement from the anticipated structural loads is estimated to be on the order of 1 inch. Differential settlement on the order of ½ of total settlement should be anticipated over a distance of 40 feet.

8.4 Footing Setbacks

Footings for structures adjacent to retaining walls should be deepened so as to extend below a 1:1 projection from the heel of the wall. Alternatively, walls may be designed to accommodate structural loads from buildings or appurtenances.

Footings should maintain a minimum horizontal setback of H/3 (H=slope height) from the base of the footing to the descending slope face and no less than 10 feet, nor need to be greater than 40 feet.

Footings adjacent to unlined drainage swales or underground utilities (if any) should be deepened to a minimum of 6-inches below the invert of the adjacent unlined swale or utilities. This distance is measured from the footing face at the bearing elevation

8.5 Interior Slabs-on-Grade

The project structural engineer should design the interior concrete slab-on-grade floor. However; as a minimum, it is recommended that a minimum of 5-inch thick slab, reinforced with No. 4 bars located at 12 inches on center, both ways, be constructed for the structures located in the liqefiable area. For the structures located in the nonliqefaible areas of the site, a minimum of 4-inch thick slab, reinforced with No. 3 bars located at 12 inches on center, both ways, could be constructed.

A layer of free draining, clean (washed) ¾ -inch crushed rock, at least 4 inches thick layer should be placed below the slab. Subgrade materials should not be allowed to desiccate between grading and the construction of the concrete slabs. The floor slab subgrade should be thoroughly and uniformly moistened prior to placing concrete. A moisture vapor retarder/barrier should be placed beneath slabs where moisture sensitive floor coverings will be installed. The vapor barrier should comply with the requirements of ASTM E1745 (Class "A"), and should be installed in accordance with ASTM E1643. The vapor barrier should be at least 10-mil thick and should be sealed at all splices, around the plumbing, and at the perimeter of slab areas, Every effort should be made to provide a continuous barrier and care should be taken not to puncture the membrane.

Current construction practice typically includes placement of a 2-inch thick sand cushion between the bottom of the concrete slab and the moisture vapor retarder/barrier. This cushion can provide some protection to the vapor retarder/barrier during construction and may assist in reducing the potential for edge curling in the slab during curing. However, the sand layer also provides a source of moisture vapor

To the underside of the slab that can increase the time required to reduce moisture vapor emissions to limits acceptable for the type of floor covering placed on top of the slab. The slab can be placed directly on the vapor retarder/barrier. The floor covering manufacturer should be contacted to determine the volume of moisture vapor allowable and any treatment needed to reduce moisture vapor emissions to acceptable limits for the particular type of floor covering installed. The project team should determine the appropriate treatment for the specific application.

8.6 Exterior Slabs-On-Grade

It is recommended that a minimum 4-inch thick slab reinforced with No.3 bars located at 18-inches on center, both ways, be constructed.

Slabs should be provided with weakened plane joints. Joints should be placed in accordance with the American Concrete Institute (ACI) guidelines. Proper control joints should be provided to reduce the potential for damage resulting from shrinkage. Subgrade materials should not be allowed to desiccate between grading and the construction of the concrete slabs. The floor slab subgrade should be thoroughly and uniformly moistened prior to placing concrete.

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

8.7 Conventional Retaining Walls

8.7.1 Foundations

The recommendations provided in the foundation sections of this report are also applicable to conventional retaining walls.

8.7.2 Lateral Earth Pressure

The following parameters are based on the use of low-expansion potential backfill materials within a 1:1 (H: V) line projected from the heel of the retaining wall.

The active earth pressure for the design of unrestrained earth retaining structures with level backfills can be taken as equivalent to the pressure of a fluid weighing 40 pcf. The at-rest earth pressure for the design of restrained earth retaining structures with level backfills can be taken as equivalent to the pressure of a fluid weighing 60 pcf. An additional 20 pcf should be added to these values for walls with a 2:1(H: V) sloping backfill. The above values assume a granular and drained backfill condition. Higher lateral earth pressures would apply if walls retain expansive clay soils.

An increase in earth pressure equivalent to an additional 2 feet of retained soil can be used to account for surcharge loads from light traffic. Surcharge due to other loading within an approximate 1½:1 (H: V) projection from the back of the wall will increase the lateral pressures provided above and should be incorporated into the wall design.

Where required, seismic earth pressures can be taken as equivalent to the pressure of a fluid weighing 30 pounds per cubic foot (pcf). The resultant force will be acting at 1/3 H feet from top of the wall. This value is for level backfill conditions and do not include a factor of safety. The seismic pressure is in addition to the static lateral earth pressures.

Retaining walls should be designed to resist hydrostatic pressures or be provided with a back-drain to reduce the accumulation of hydrostatic pressures. Back-drains may consist of a two-foot wide zone of ¾-inch crushed rock. The back-drain should be separated from the adjacent soils using a non-woven filter fabric, such as Mirafi 140N or equivalent. A perforated pipe (Schedule 40 PVC) should be installed at the base of the back-drain and sloped to discharge to a suitable storm drain facility. As an alternative, a geo-composite drainage system such as Miradrain 6000 or equivalent placed behind the wall and connected to a suitable storm drain facility can be used. The project architect should provide waterproofing specifications and details.

8.8 Pool Design

The proposed pool should be designed according to the following criteria:

- Design shell as free standing.
- Utilize 65 pcf equivalent fluid pressure for static active lateral soil loading.
- Provide for hydrostatic pressure relief.
- In the case of a spa being planned structurally continuous with the pool shell, the spa should either be designed to be entirely supported by the pool shell (i.e., cantilevered) or the spa support should be derived at a depth comparable to that of the pool (i.e., deep).

8.9 Corrosivity

One sample of the onsite soils was tested to provide a preliminary indication of the corrosion potential of the onsite soils. The test results are presented in **Appendix B**. A brief discussion of the corrosion test results is provided in the following section.

- The sample tested had a soluble sulfate concentration of 0.004 percent, which indicates the sample has a low sulfate corrosion potential relative to concrete. However; we recommend that type II cement with maximum 0.50 water/cement ratio in accordance with California Building Code (CBC) standard 1904 (Durability Requirements) be utilized. Concrete mix design, materials, placement, curing, and finishing should be in conformance with the Standard Specifications for Public Works Construction "Green book", and American concrete Institute (ACI) specifications.
- The sample tested had a chloride concentration of 0.001 percent, which indicates the sample has a low chloride corrosion potential relative to metal.
- The sample tested had a minimum resistivity of 3100 ohm-cm, which indicates the sample is highly corrosive to ferrous metals.
- The sample tested had a pH of 7.0, which indicates the sample is neutral in nature.

Additional testing should be performed after grading to evaluate the as-graded corrosion potential of the onsite soils. We are not corrosion engineers. A corrosion consultant should be retained to provide corrosion control recommendations if deemed necessary.

9.0 PAVEMENT DESIGN RECOMMENDATIONS

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

The upper 24-inches of pavement subgrade soils should be scarified; moisture conditioned to at least 2 to 4 percent above optimum moisture content and compacted to at least 95 percent of the laboratory standard (ASTM D1557). If loose or yielding materials are encountered during subgrade preparation, evaluation should be performed by EEI.

Aggregate base materials should be properly prepared (i.e., processed and moisture conditioned) and compacted to at least 95 percent of the maximum dry density as determined by ASTM D1557. All pavement section changes should be properly transitioned. Although not anticipated, if adverse conditions are encountered during the preparation of subgrade materials, special construction methods may need to be employed. A representative of the project geotechnical engineer should be present for the preparation of subgrade and aggregate base. For preliminary design purposes, we have assumed an R-Value of 15 for the materials likely to be exposed at subgrade. For design purposes we have assumed a Traffic Index (TI) of 5.0 for the parking stalls and a Traffic Index (TI) of 6.0 for drive areas. This assumed TI should be verified as necessary by the Civil Engineer or Traffic Engineer.

TABLE 3
Preliminary Pavement Design Recommendations

Traffic Index (TI) / Intended Use	Pavement Surface	Aggregate Base Material (1)
5	3.0-inches Asphalt Concrete	8.0-inches
6	3.0-inches Asphalt Concrete	11.0-inches
Concrete Pavement Section	6.0-inches Portland Cement Concrete	6.0-inches

(1) R-Value of 78 for Caltrans Class 2 aggregate base

The recommended pavement sections provided in **Table 3** are intended as a minimum guideline. If thinner or highly variable pavement sections are constructed, increased maintenance and repair could be expected. If the actual traffic index (TI) increases beyond our assumed values, increased maintenance and repair could be required for the pavement section. Final pavement design should be verified by testing of soils exposed at subgrade after grading has been completed. Thicker pavement sections could result if R-Value testing indicates lower value.

10.0 DEVELOPMENT RECOMMENDATIONS

10.1 Landscape Maintenance and Planting

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

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

10.2 Site Drainage

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

Final surface grades around structures should be designed to collect and direct surface water away from structures and toward appropriate drainage facilities. The ground around the structure should be graded so that surface water flows rapidly away from the structure without ponding. In general, we recommend that the ground adjacent to the structure slope away at a gradient of at least 2 percent. Densely vegetated areas where runoff can be impaired should have a minimum gradient of at least 5 percent within the first 5 feet from the structure. Roof gutters with downspouts that discharge directly into a closed drainage system are recommended on structures. Drainage patterns established at the time of fine grading should be maintained throughout the life of the proposed structures.

10.3 Structure Setback from Retention Devices

We recommend that retention/disposal devices be situated at least three times their depth, or a minimum of 15 feet (whichever is greater), from the outside bottom edge of structural foundations. Structural foundations include (but are not limited to) buildings, loading docks, retaining walls, and screen walls. All storm water disposal systems should be checked and maintained on regular intervals. Stormwater devices including bioswales that are located closer than 10 feet from any foundations/footings should be lined with an impermeable membrane to reduce the potential for saturation of foundation soils. Foundations may also need to be deepened.

10.4 Utility Trench Backfill

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

Select material should be used when called for on the drawings, or when recommended by the geotechnical engineer. Care should be taken during backfill and compaction operations to maintain alignment and prevent damage to the joints. The backfill should be kept free from oversized material, chunks of highly plastic clay, or other unsuitable or deleterious material. Backfill soils should be non-expansive, non-corrosive, and compatible with native earth materials. Backfill materials and testing should be in accordance with the requirements of the local governing jurisdiction.

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

11.0 PLAN REVIEW

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

12.0 LIMITATIONS

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

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

13.0 REFERENCES

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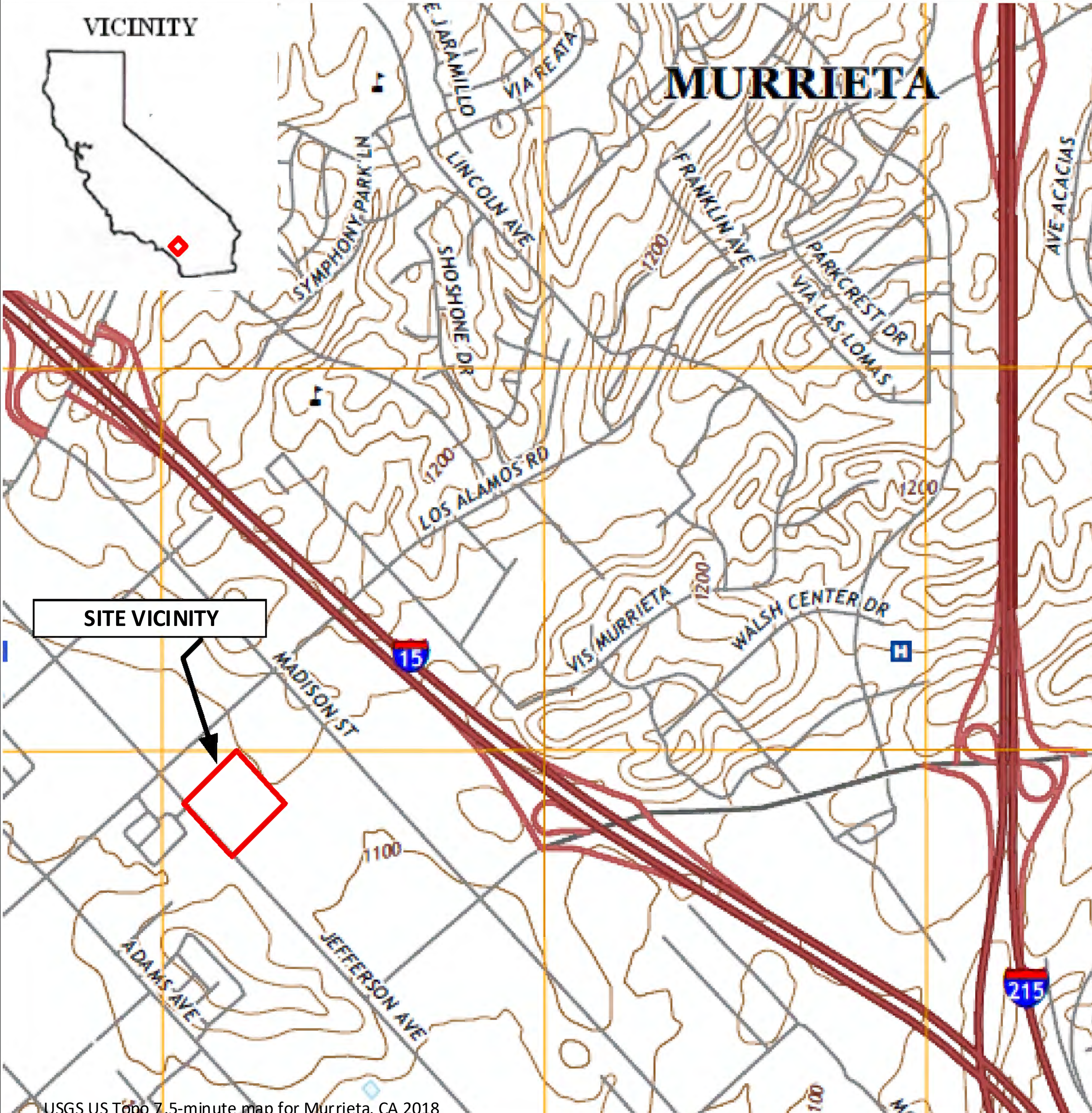
FIGURES



LEGEND

N

Scale: 1" = 1250 feet

0 625 ft 1250 ft 2500 ft

Note: All Locations Are Approximate

SITE VICINITY MAP

Pacific West Development, LP

9-acre Jefferson Street Multi-family Development

Lot 91 MB008/359 SD TR T L WC

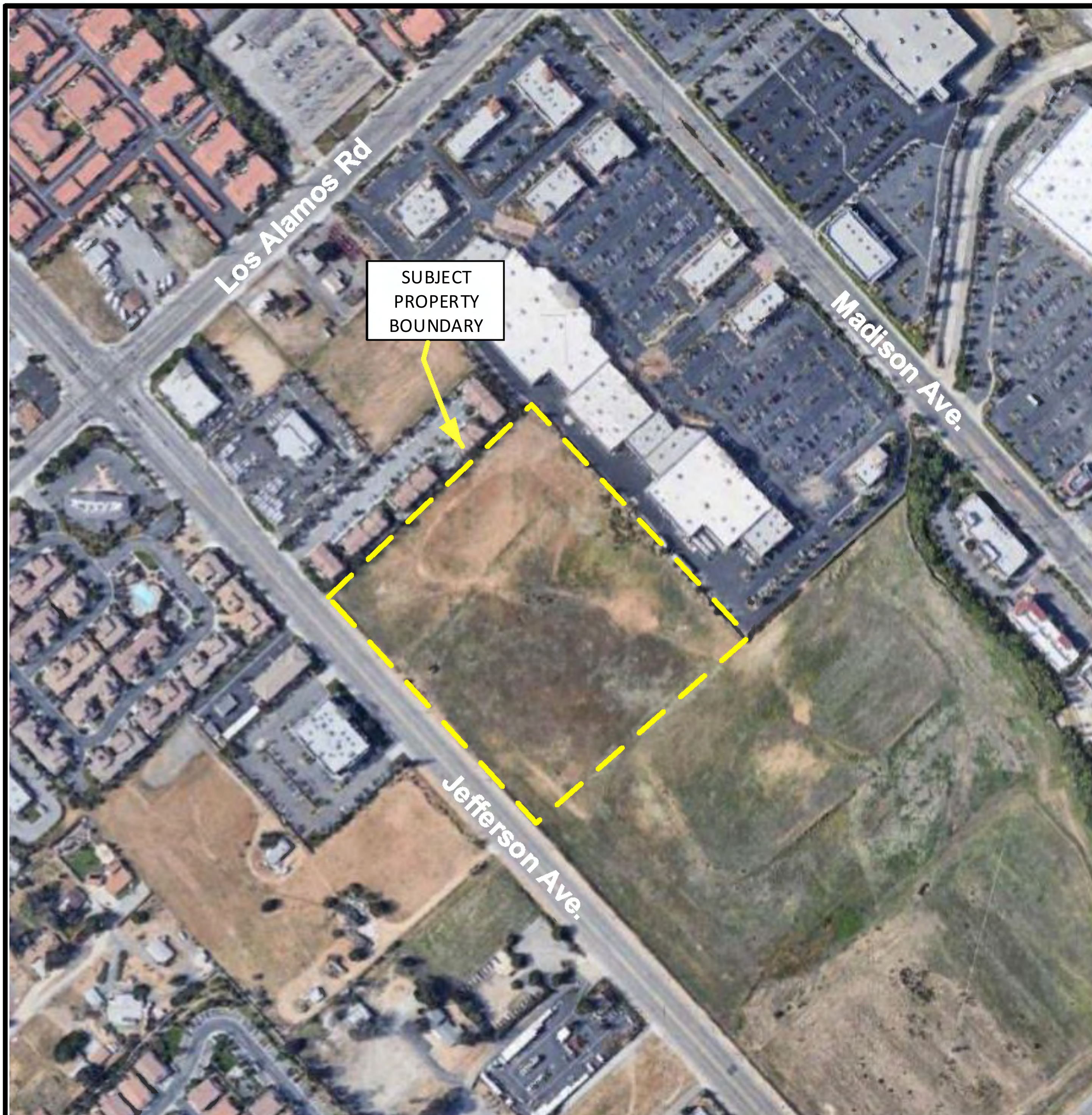
APN 949-220-048

Murrieta, Riverside County, CA

EEI Project PWD-72978.4B

EEI Engineering Solutions

FIGURE 1



Source: Google Earth, 2019

N

Scale: 1" = 500'

0 250 ft 500 ft 1000 ft

Note: All Locations Are Approximate

AERIAL SITE MAP

Pacific West Development, LP

9-acre Jefferson Street Multi-family Development

Lot 91 MB008/359 SD TR T L WC

APN 949-220-048

Murrieta, Riverside County, CA

EEI Project PWD-72978.4B

EEI Engineering Solutions

FIGURE 2

Source: Google Earth, 2019

- **CPT-9-5** *Approximate Location of CPT tests*
- **B-9-2** *Approximate Locations of exploratory borings*
- **B-8** *Approximate Locations of exploratory borings from previous investigation 12/12/00*

N

Scale: 1" = 500'

0 | 250 ft | 500 ft | 1000 ft

Note: All Locations Are Approximate

GEOTECHNICAL MAP

Pacific West Development, LP

9-acre Jefferson Street Multi-family Development

Lot 91 MB008/359 SD TR T L WC

APN 949-220-048

Murrieta, Riverside County, CA

EEI Project PWD-72978.4B

EEI Engineering Solutions

FIGURE 3

APPENDIX A
SOIL CLASSIFICATION CHART AND BORING LOGS

UNIFIED SOIL CLASSIFICATION (ASTM D-2487-98)

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

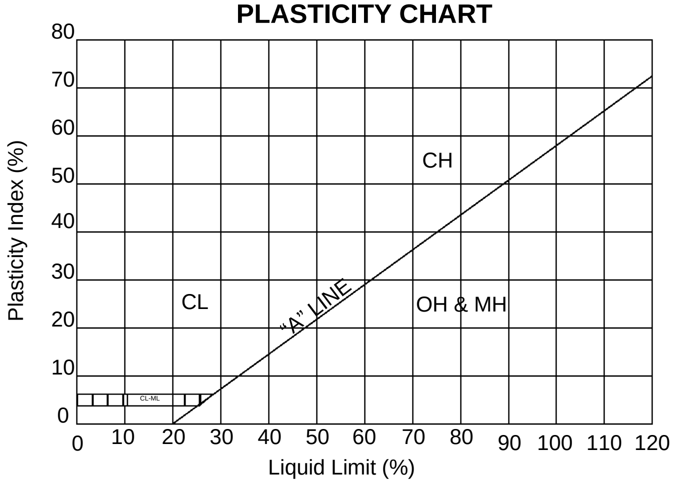
SAMPLER TYPES

- SPT
- Modified California (2.5" I.D.)
- Bulk
- Rock Core
- Shelby Tube
- Water Level

OTHER TESTS

ATT – Atterberg Limit (Plasticity Index)
CD – Consolidated Drained Triaxial
CON – Consolidation
COR – Corrosivity)
DS – Direct Shear
EI – Expansion Index
MAX – Maximum Density
RV – R-Value
SA – Sieve Analysis
-#200 - Percent Passing #200 Sieve
TV – Torvane Shear
UU – Unconsolidated Undrained Triaxial

PLASTICITY CHART



PENETRATION RESISTANCE (Recorded As Blows/Foot)				
SAND & GRAVEL		SILT & CLAY		
Relative Density	Blows/Foot* N_{60}	Consistency	Blows/Foot* N_{60}	Strength**(KSF)
Very Loose	0-4	Very Soft	0 - 2	0 – 0.5
Loose	4-10	Soft	2 - 4	0.5 – 1.0
Medium Dense	10-30	Medium Stiff	4 - 8	1.0 – 2.0
Dense	30-50	Stiff	8 - 15	2.0 – 4.0
Very Dense	Over 50	Very Stiff	15 - 30	4.0 – 8.0
		Hard	Over 30	Over 8.0

* Number of blows of 140LB hammer falling 30 inches to drive a 2 inch O.D. (1-3/8 inch I.D.) split barrel sampler the last 12 inches of an 18-inch drive (ASTM-1586 Standard Penetration Test)

** Undrained shear strength in kips/sq. ft. As determined by laboratory testing or approximated by the standard penetration test, pocket penetrometer, torvane, or visual observation

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LEGEND TO SOIL DESCRIPTIONS

APPENDIX A

BOREHOLE LOG

Number: B-9-1	**Client:** Pacific West Development	**Sheet:** 1 of 1

Location: Proposed 9-Acre Jefferson Street Multi-family Development
Lot 91 MB008/359 SD TR T L WC, APN 949-220-048, Murrieta, Riverside County, California

Date Started:	Date Finished:	EEI Rep:	Project No.:	Drill Rig/Sampling Method	Borehole Diameter:
4/18/2020	4/18/2020	BS	PWD-72978.4B	Ingersoll-Rand A300 / 8" Hollow Stem Auger 140 lbs Manual Hammer	8"

SAMPLE LOG / **BOREHOLE LOG**

Bulk	Sample Type	Blows Per 6"	Dry Unit Wt. (pcf)	Moisture (%)	Depth In Feet	USCS Symbol	Geologic Description (SoilType, Color, Grain, Minor Soil Component, Moisture, Density, Odor, Etc.)
Bulk (0–5)					0–1	ML	**ALLUVIUM** Silty CLAY, dark brown to reddish brown, very moist, very stiff, fine to medium grained. Low plasticity. Trace caliche.
	MC	6, 10, 13	126	14	2.5–4		
	SPT	12, 26, 39			5–6.5		
	SPT	9, 20, 23			7.5–9		
	SPT	17, 15, 26			12.5–14		
	MC	17, 27, 42	116	20	17.5–19		
					21		@21' Groundwater encountered
					22	ML	
					23	CL-ML	**PAUBA FORMATION (Qpfs)** Silty CLAYSTONE, yellow brown to orangish brown, very moist to wet, very soft, fine to medium grained with minor coarse sand, poorly indurated, highly weathered. Oxidation staining. Trace caliche.
	SPT	11, 21, 28			25–26.5		

Total Depth: 26.5'
No Groundwater Encountered
Backfilled with Native Soil

BOREHOLE LOG

Number:	B-9-2
Client:	Pacific West Development
Sheet:	1 of 1
Location:	Proposed 9-Acre Jefferson Street Multi-family Development Lot 91 MB008/359 SD TR T L WC, APN 949-220-048, Murrieta, Riverside County, California
Date Started:	4/18/2020
Date Finished:	4/18/2020
EEI Rep:	BS
Project No.:	PWD-72978.4B
Drill Rig/Sampling Method	Ingersoll-Rand A300 / 8" Hollow Stem Auger 140 lbs Manual Hammer
Borehole Diameter:	8"

SAMPLE LOG / BOREHOLE LOG

Bulk	Sample Type	Blows Per 6"	Dry Unit Wt. (pcf)	Moisture (%)	Depth In Feet	USCS Symbol	Graphic Log	Geologic Description (SoilType, Color, Grain, Minor Soil Component, Moisture, Density, Odor, Etc.)
					0–1	ML		**Alluvium** Sandy SILT, dark brown, very moist, soft to slightly stiff, fine to medium grained. Low plasticity.
	MC	10 50/5.5"	125	13	1–6	SM		**PAUBA FORMATION (Qpfs)** Silty SANDSTONE, reddish brown to orange brown, very moist, moderately soft, fine to medium grained, moderately indurated to cemented, moderately weathered.
	SPT	25 50/5.5"						

Depth scale: 1–34 feet

Total Depth: 6'
No Groundwater Encountered
Backfilled with Native Soil

APPENDIX B
LABORATORY TEST DATA

GRAIN SIZE DISTIBUTION - PASSING #200 SIEVE

ASTM METHOD D422

Boring No.	**B-9-1**						
Depth	0-5 ft.						ft
Total Sample Weight	196.0						gm
Retained on #200 Sieve	104.9						gm
Passing #200 Sieve	91.1						gm
Fines Content	**46.5**						%

Boring No.							
Depth							ft
Total Sample Weight							gm
Retained on #200 Sieve							gm
Passing #200 Sieve							gm
Fines Content							%

Boring No.							
Depth							ft
Total Sample Weight							gm
Retained on #200 Sieve							gm
Passing #200 Sieve							gm
Fines Content							%

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3146 Tiger Run Ct. Suite # 118 Carlsbad, Ca. 760-431-3747

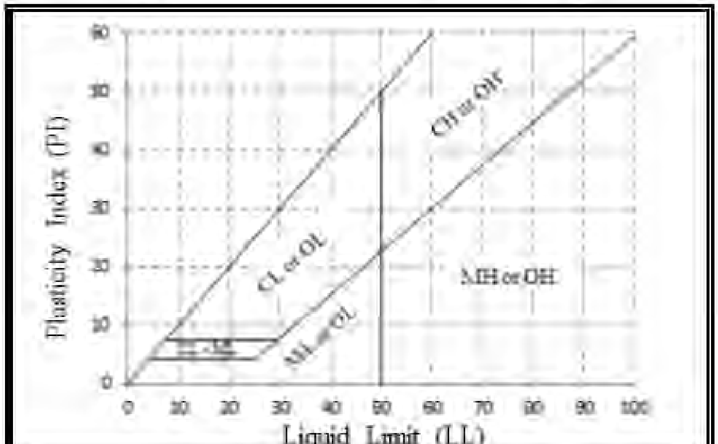
Client:	Pacific West Dev.
Project Name:	Jefferson Ave.
Project Number:	PWD-72978.4b
Date:	4/23/20
Tested by:	B D
Reviewed by:	

LIQUID LIMIT, PLASTIC LIMIT, AND PLASTICITY INDEX

ASTM METHOD D 4318

Sample	B-9-1 @ 0-5 ft.					
	Liquid Limit			Plastic Limit		
Test Number	**1**	**2**	**3**	**1**	**2**	**2**
Container Number	9	14	21	28	30	33
Weight of Container (g)	14.27	14.06	14.46	14.23	14.18	14.22
Wet Weight of Soil and Container (g)	25.97	24.97	25.09	0.00	0.00	0.00
Dry Weight of Soil and Container (g)	0.00	0.00	0.00	0.00	0.00	0.00
Number of Blows	0	0	0			
Moisture Content (%)	-182.0	-177.6	-173.5	0.0	0.0	0.0

Liquid Limit =	**0.0**	L. L. could not be determined
Plastic Limit =	**0.0**	
Plasticity Index =	**0.0**	Soil is non-plastic.
Classification:		

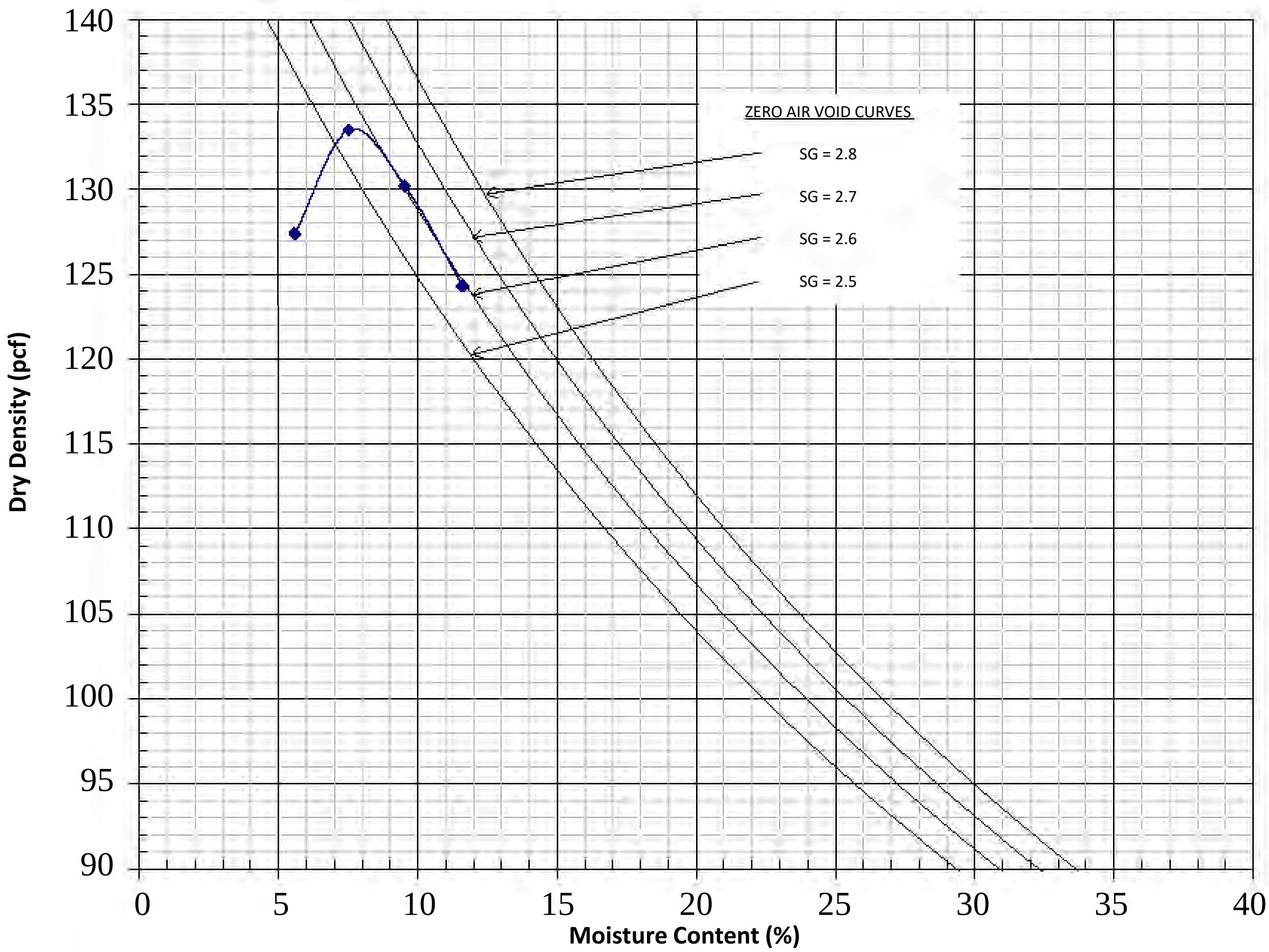


3146 Tiger Run Ct. Suite # 118 Carlsbad, Ca. 92010 760-431-3747

Client:	Pacific West Dev.
Project Name:	Jefferson Ave.
Job Number:	PWD-72978.4b
Date:	4/27/20
Boring Number:	B-9-1
Depth:	0-5 ft.
Soil Description:	Brown Fine Silty Sand ML
Tested by:	B D

LABORATORY COMPACTION ASTM D 1557

Sample	1	2	3	4
Mold and Wet Soil (lbs.)	8.74	9.04	9.01	8.88
Small Mold (lbs.)	4.26	4.26	4.26	4.26
Wet Soil (lbs.)	4.48	4.78	4.75	4.62
Wet Density (pcf)	134.5	143.5	142.6	138.7
Tare and Wet Soil (gm.)	100.00	100.00	100.00	100.00
Tare and Dry Soil (gm.)	94.70	93.00	91.30	89.60
Moisture (%)	**5.6**	**7.5**	**9.5**	**11.6**
Dry Density (pcf)	**127.4**	**133.5**	**130.2**	**124.3**



Maximum Density 134.0 pcf @ 8.0 % Moisture

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3146 Tiger Run Ct. Suite # 118 Carlsbad, Ca. 92010 760-431-3747

Client:	Pacific West Dev.
Project Name:	Jefferson Ave.
Project Number:	PWD-72978.4b
Date:	4/28/2020
Procedure:	D-1557-A
Boring/Sample No.:	B-9-1
Depth/Location:	0-5 ft.
Soil Description:	Brown Sandy Silt SM
Tested By:	B D

EXPANSION INDEX TEST

ASTM METHOD D4829

B-9-1 @ 0-5 ft.

		% Saturation of Re-molded Sample		Moisture Content of Final Sample	
Tare No.-	33	Wt. of Soil and Ring (g) -	624.5	Wt. of Soil and Ring (g) -	646
Wet Weight and Tare (g)-	135.5	Ring Weight (g) -	198.6	Ring Weight (g) -	198.6
Dry Weight and Tare(g)-	128.8	Wet Weight of Soil (g) -	425.9	Wet Weight of Soil (g) -	447.4
Tare Weight (g)-	39.3	Dry Weight of Soil (g) -	396.2	Dry Weight of Soil (g) -	396.2
Water Loss (g) -	6.7	Volume of Ring (ft^3) -	0.0073	Weight of Water (g) -	51.2
Dry Weight (g) -	89.5	Dry Density (pcf) -	119.7	Final Moisture (%)	12.9
Initial Moisture (%) -	7.5	Initital Saturation (%) -	49.5	Final Saturation (%) -	85.5

Expansion Test - UBC (144 PSF)				
	Date	Time	Reading	
Add Weight	4/24/20	8:35	0.000	
10 Minutes		8:45	0.000	Initial Reading
Add Water		10:17	0.005	
		12:41	0.008	
	4/27/20	6:48	0.011	Final Reading

EImeasured	=	11
EI_{50}	=	11

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

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3146 Tiger Run Ct. Suite #118 Carlsbad Ca. 92010 760-431-3747

Client:	Pacific West Dev.
Project Name:	Jefferson Ave.
Project No.:	PWD-72978.4b
Date:	4/24/2020
Boring/Sample No.:	B-9-1
Depth/Location:	0-5 ft.
Soil Description:	Red-Brn. Silty Sand ML
Tested By:	B D

DIRECT SHEAR TEST (ASTM D3080)

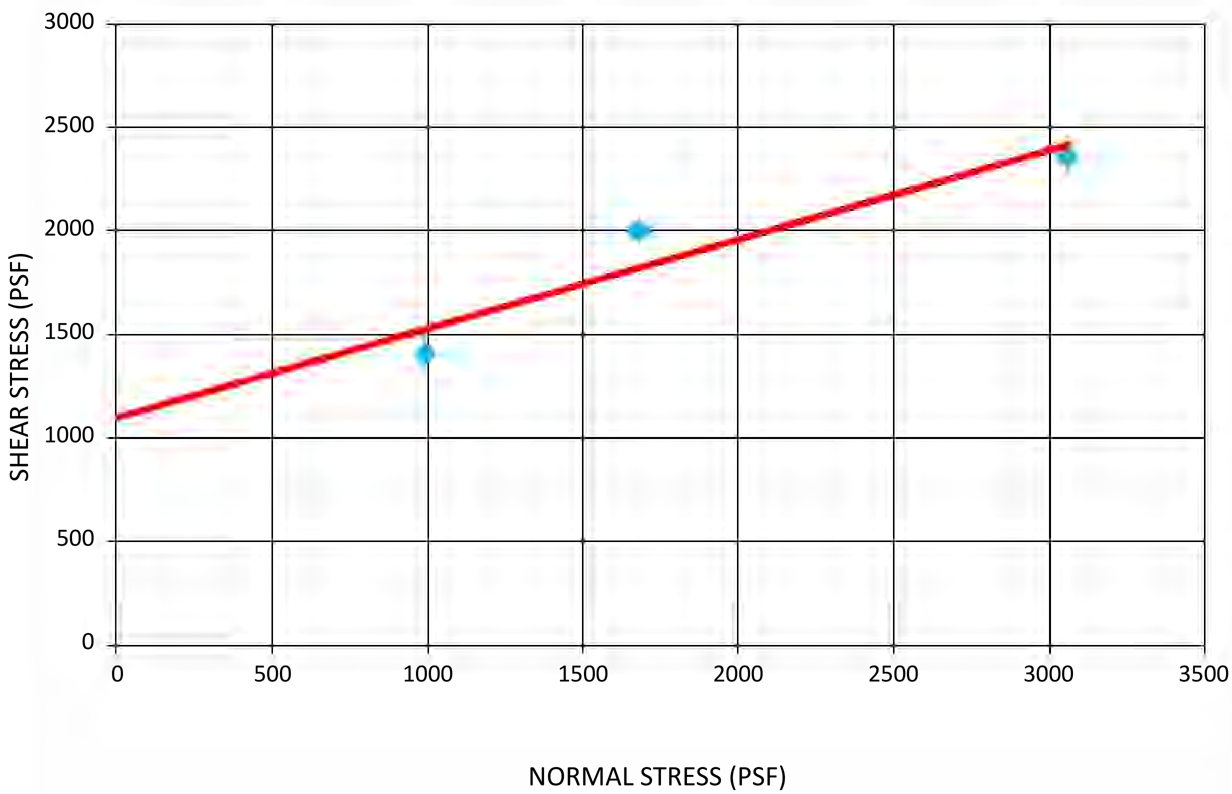
B-9-1 @ 2.5 ft.

Sample Data	
Remolded:	Natural
Remarks:	Sample inundated prior to testing
Soil Description:	Brown Fine Sandy Silt ML

Test Results		
Average Initial Moisture =	14.1	%
Average Dry Density =	119.9	pcf
Average Final Moisture =	15.4	%

Peak Strength	ϕ = 23 deg.	c = 1098 psf

SHEAR TEST DIAGRAM



3146 Tiger Run Ct. Suite # 118 Carlsbad, Ca. 92010 760-431-3747

Client:	Pacific West Development
Project Name:	Jefferson Ave.
Project No.:	PWD-72978.4b
Date:	4/22/20
Boring/Sample No:	B-9-1
Depth/Location:	2.5 ft.
Soil Description:	Brown Fine Sandy Silt ML
Tested by:	B D

LABORATORY REPORT

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

CLARKSON LABORATORY AND SUPPLY INC.
350 Trousdale Dr. Chula Vista, Ca. 91910 www.clarksonlab.com
ANALYTICAL AND CONSULTING CHEMISTS

Date: April 29, 2020
Purchase Order Number: PWD-72978.4B
Sales Order Number: 47832
Account Number: EEI
To:

EEI Environmental Equalizers Inc
3146 Tiger Run Court, Suite 118
Carlsbad, CA 92010
Attention: Mohammad Joolazedah

Laboratory Number: SO7773-1 Customers Phone: 760-431-3747

Sample Designation:

One soil sample received on 04/24/20 at 4:10pm,
from Jefferson Ave Projec# PWD-72978.4B marked
as B-9-1@0'-5' ft

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

pH 7.0

Water Added (ml)	Resistivity (ohm-cm)
10	9200
5	4800
5	3400
5	3300
5	3100
5	3400
5	3600

29 years to perforation for a 16 gauge metal culvert.
38 years to perforation for a 14 gauge metal culvert.
53 years to perforation for a 12 gauge metal culvert.
67 years to perforation for a 10 gauge metal culvert.
82 years to perforation for a 8 gauge metal culvert.

Water Soluble Sulfate	Calif. Test 417	0.004%
Water Soluble Chloride	Calif. Test 422	0.001%

Rosa Bernal
RMB/dbb

APPENDIX C
EARTHWORK and GRADING GUIDELINES

EARTHWORK AND GRADING GUIDELINES

GENERAL

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

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

EARTHWORK OBSERVATIONS AND TESTING

Geotechnical Consultant

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

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

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

Laboratory and Field Tests

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

Contractor's Responsibility

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

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

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

SITE PREPARATION

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

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

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

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

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

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

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

COMPACTED FILLS

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

EXCAVATIONS

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

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

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

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

SUBDRAIN INSTALLATION

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

COMPLETION

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

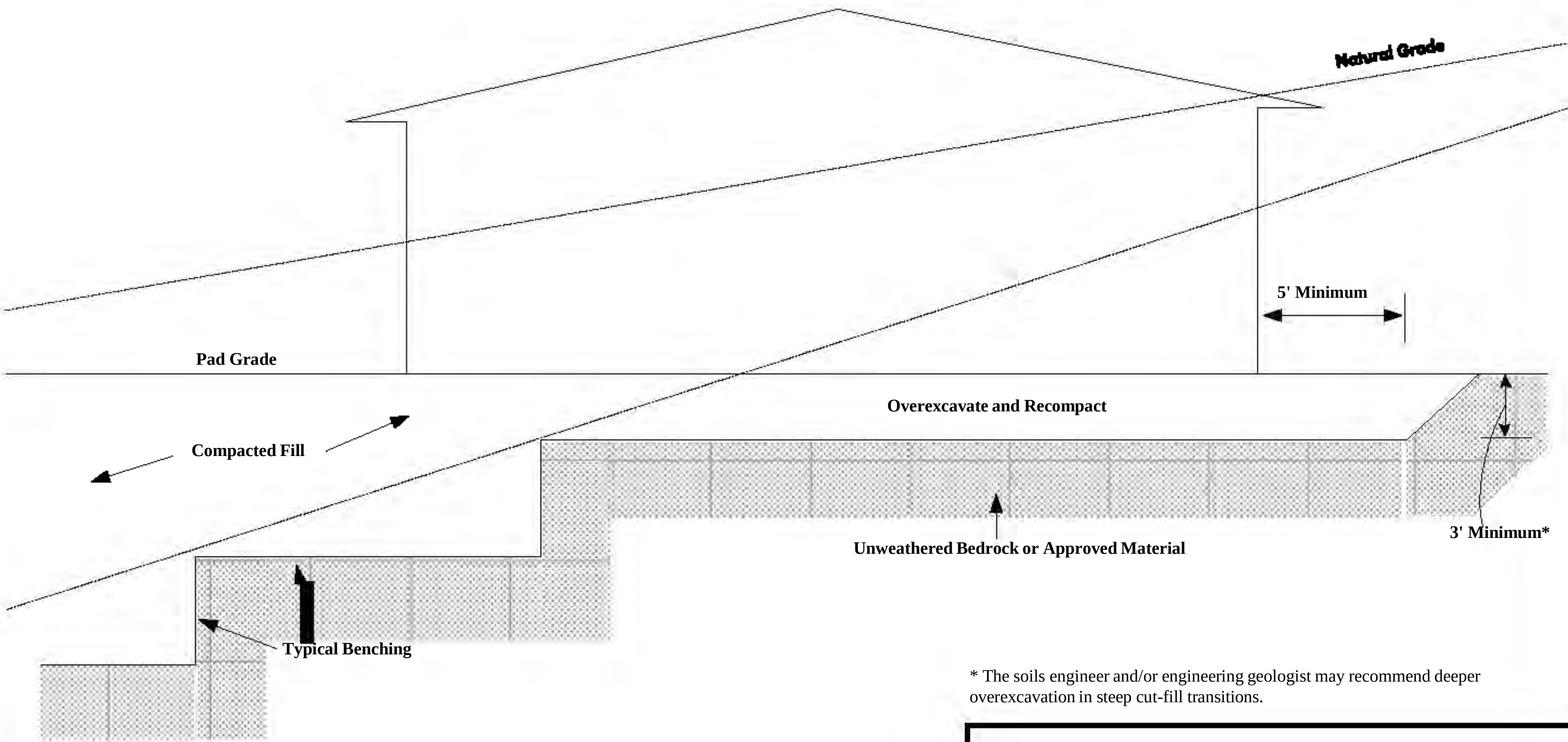
After completion of grading and after the soil engineer and engineering geologist have finished their observations, final reports should be submitted subject to review by the controlling governmental agencies. No additional grading should be undertaken without prior notification of the soil engineer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

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

ATTACHMENTS

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

TRANSITION LOT DETAIL
CUT LOT – MATERIAL TYPE
TRANSITION



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

Note: Figure not to scale

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

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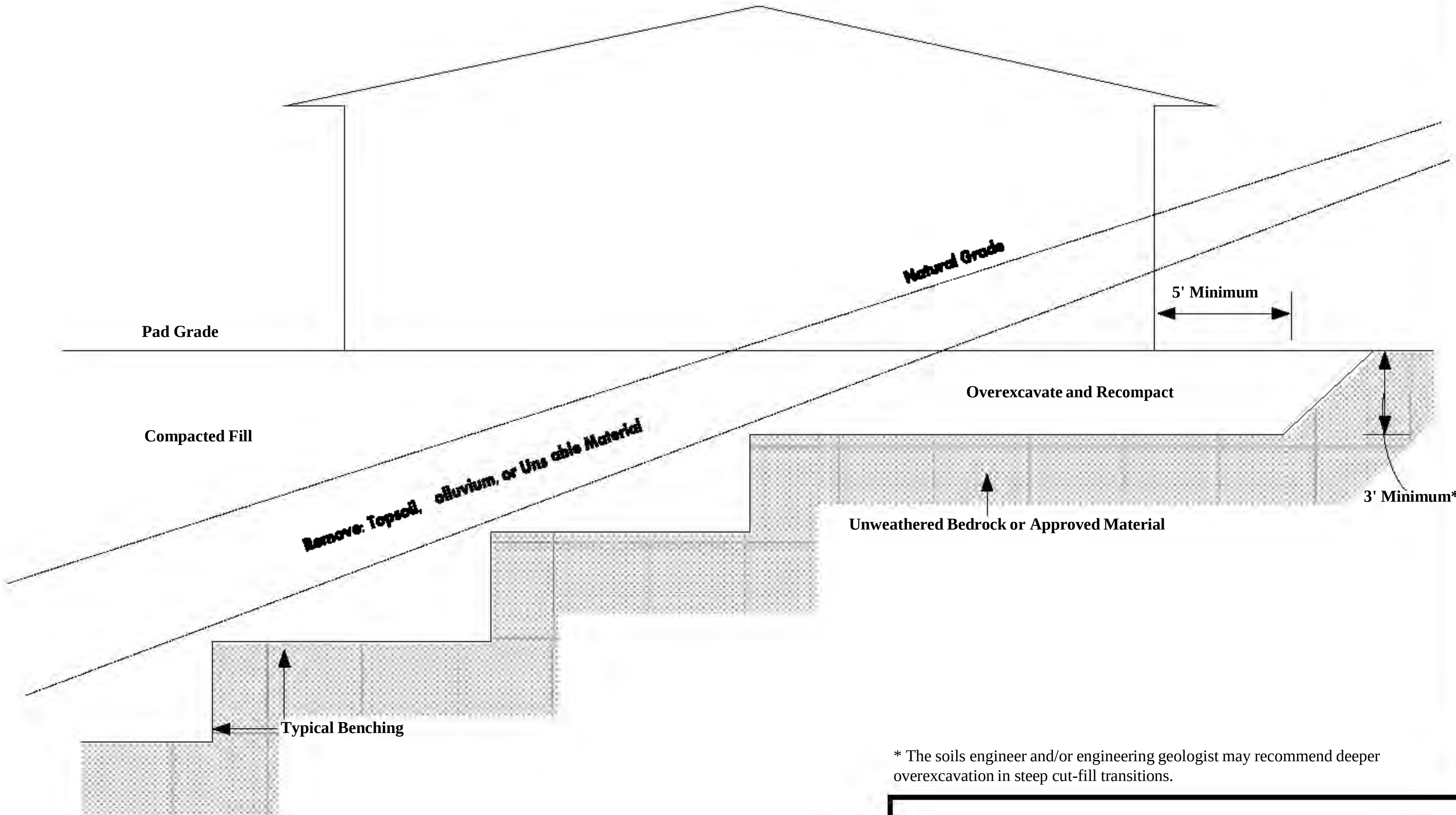
FIGURE A

TRANSITION LOT DETAIL
CUT – FILL – DAYLIGHT TRANSITION



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

Note: Figure not to scale

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

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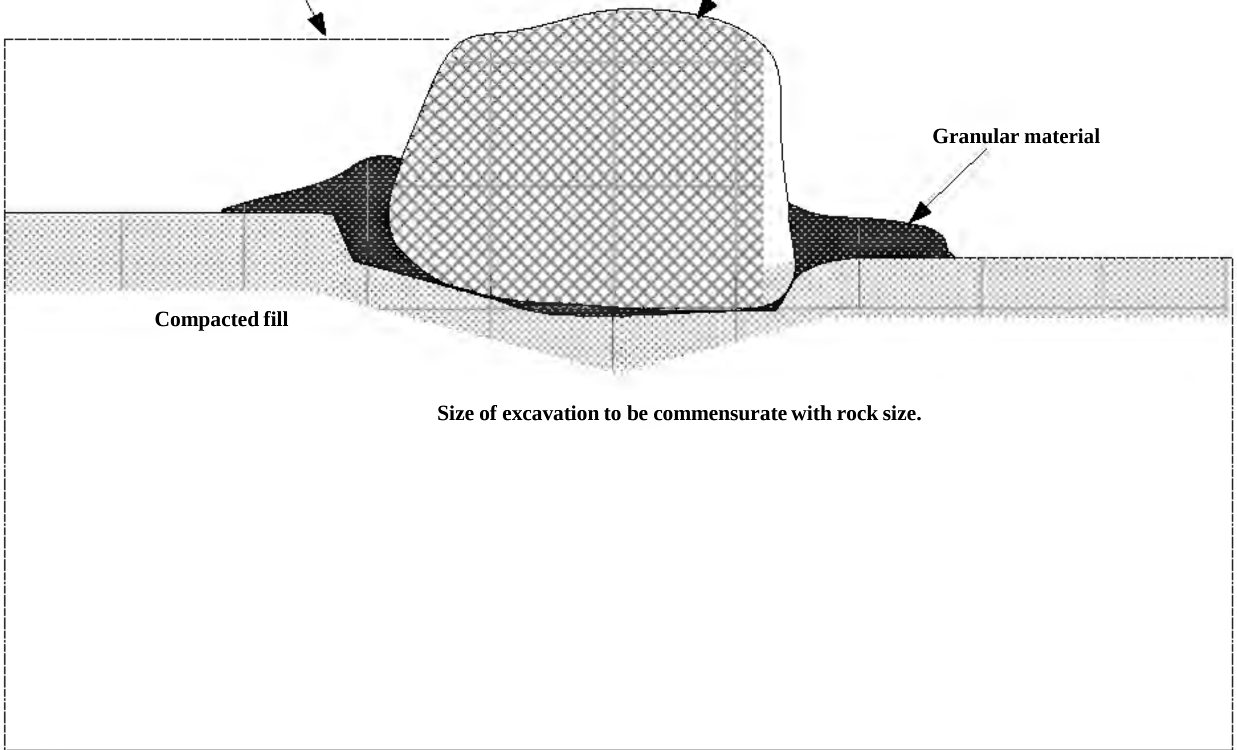
FIGURE B

ROCK DISPOSAL PITS



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

Note: Figure not to scale

EARTHWORK AND GRADING GUIDELINES
ROCK DISPOSAL PITS

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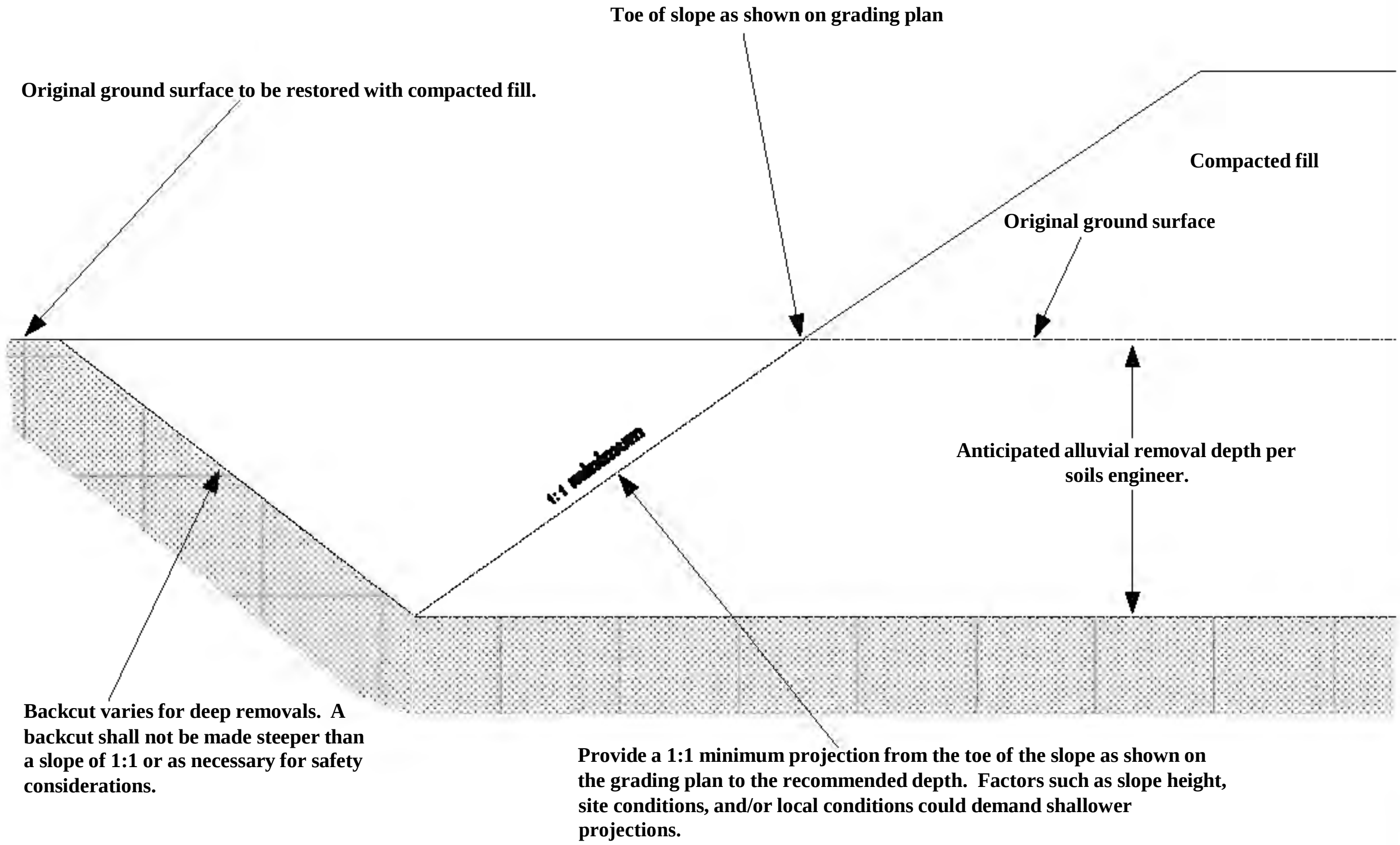
FIGURE C

DETAIL FOR FILL SLOPE TOEING OUT ON FLAT ALLUVIATED CANYON



Note: Figure not to scale

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

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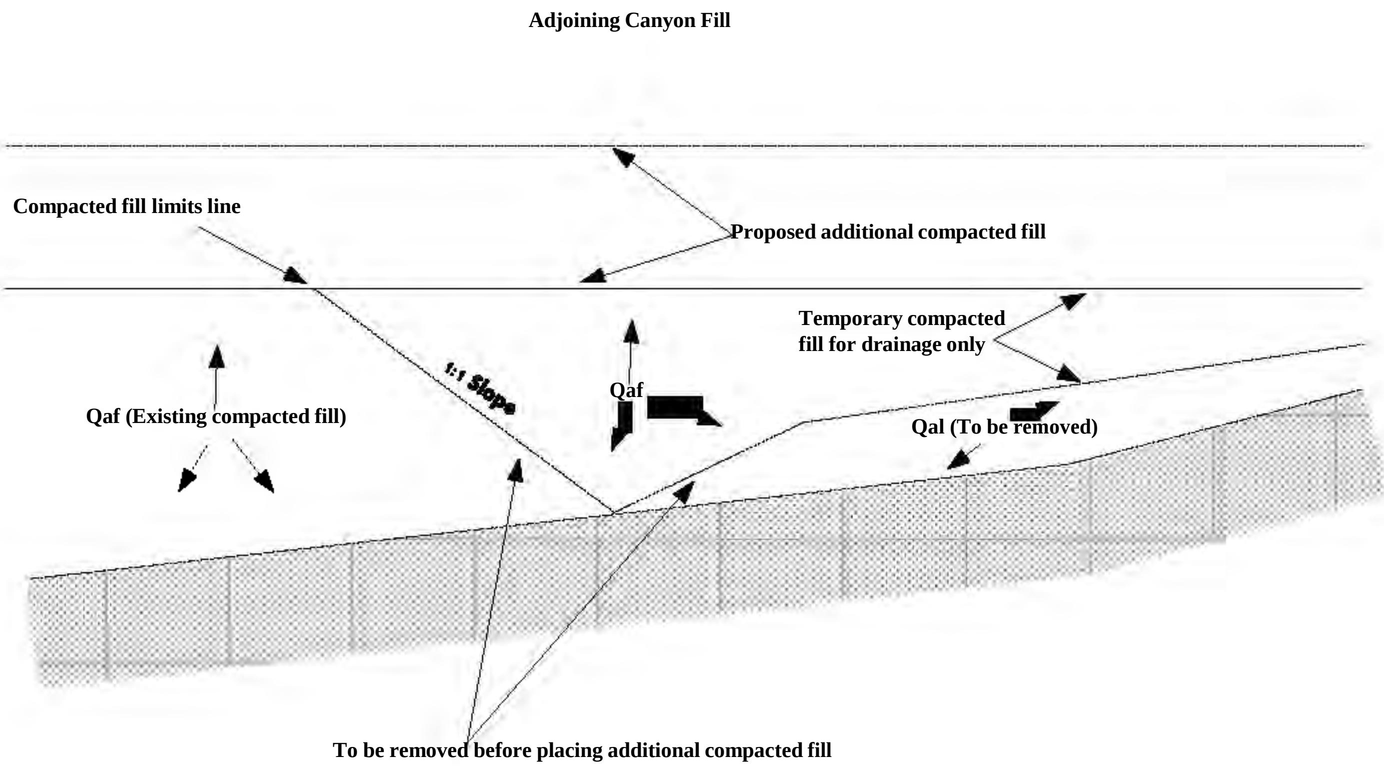
FIGURE D

REMOVAL ADJACENT TO EXISTING FILL



Legend
Qaf - Artificial Fill
Qal - Alluvium

Note: Figure not to scale

EARTHWORK AND GRADING GUIDELINES
REMOVAL ADJACENT TO EXISTING FILL

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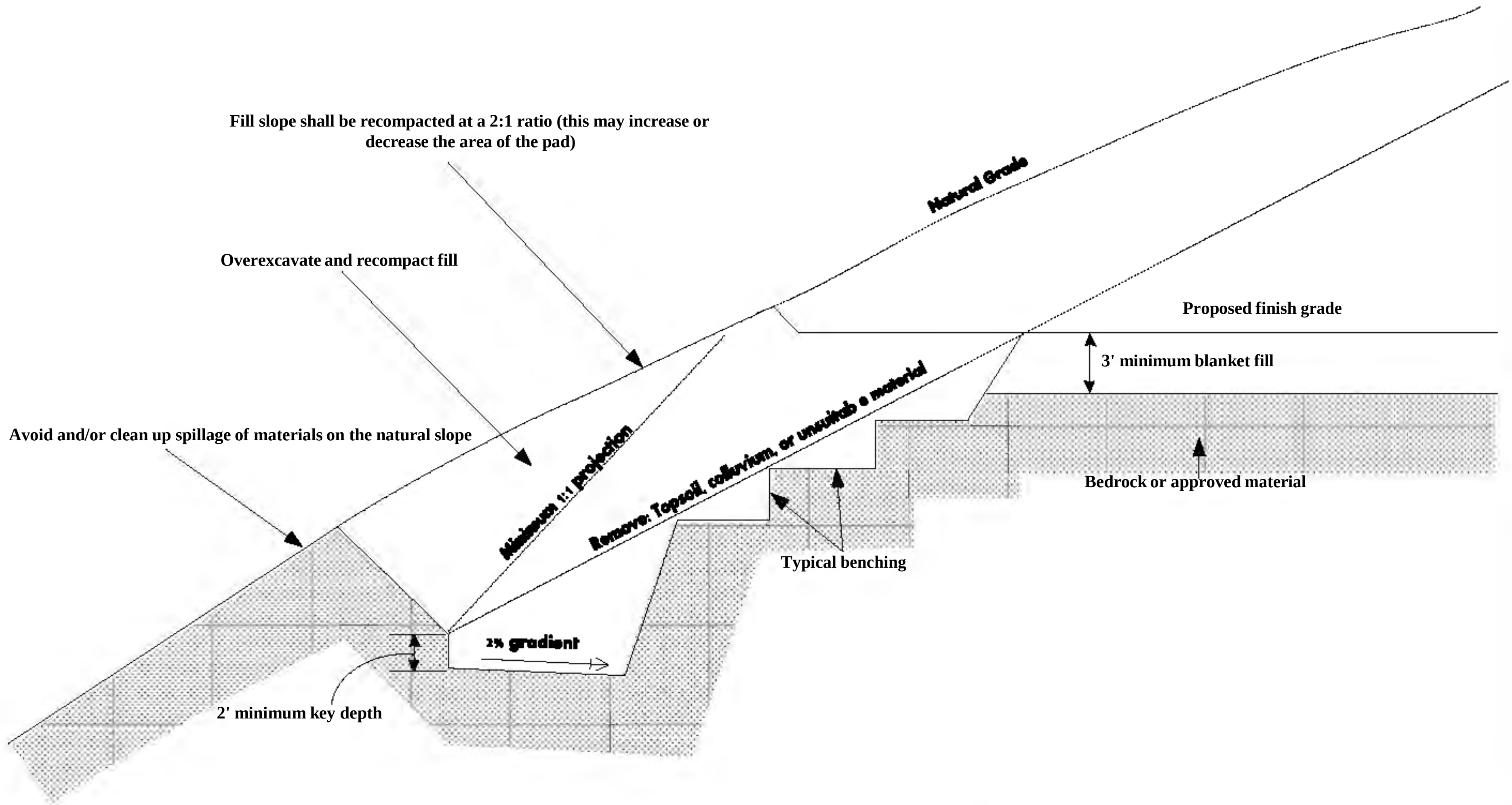
FIGURE E

DAYLIGHT CUT LOT DETAIL



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

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

Note: Figure not to scale

EARTHWORK AND GRADING GUIDELINES
DAYLIGHT CUT LOT DETAIL

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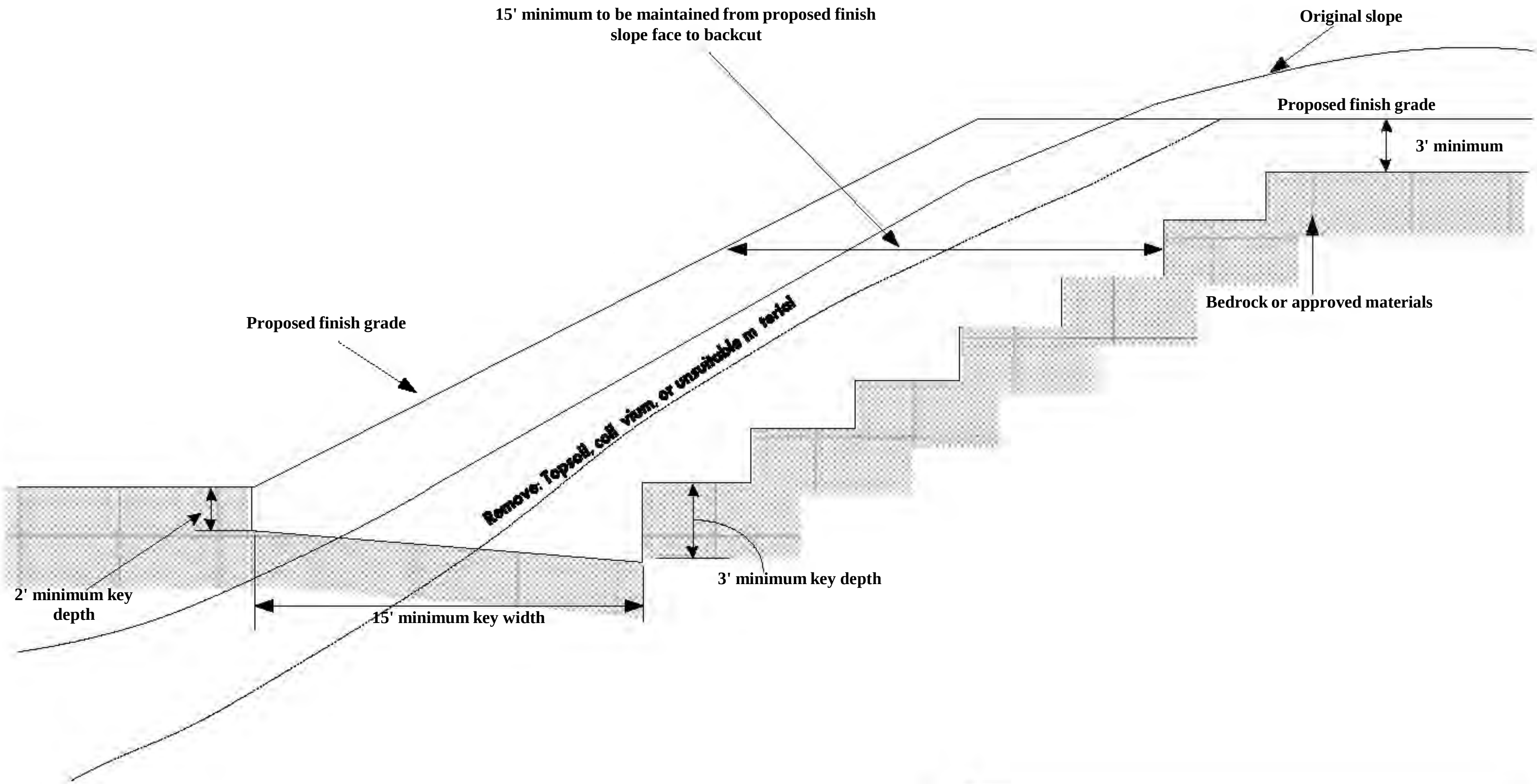
FIGURE F

SKIN FILL OF NATURAL GROUND



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

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

Note: Figure not to scale

EARTHWORK AND GRADING GUIDELINES
SKIN FILL OF NATURAL GROUND

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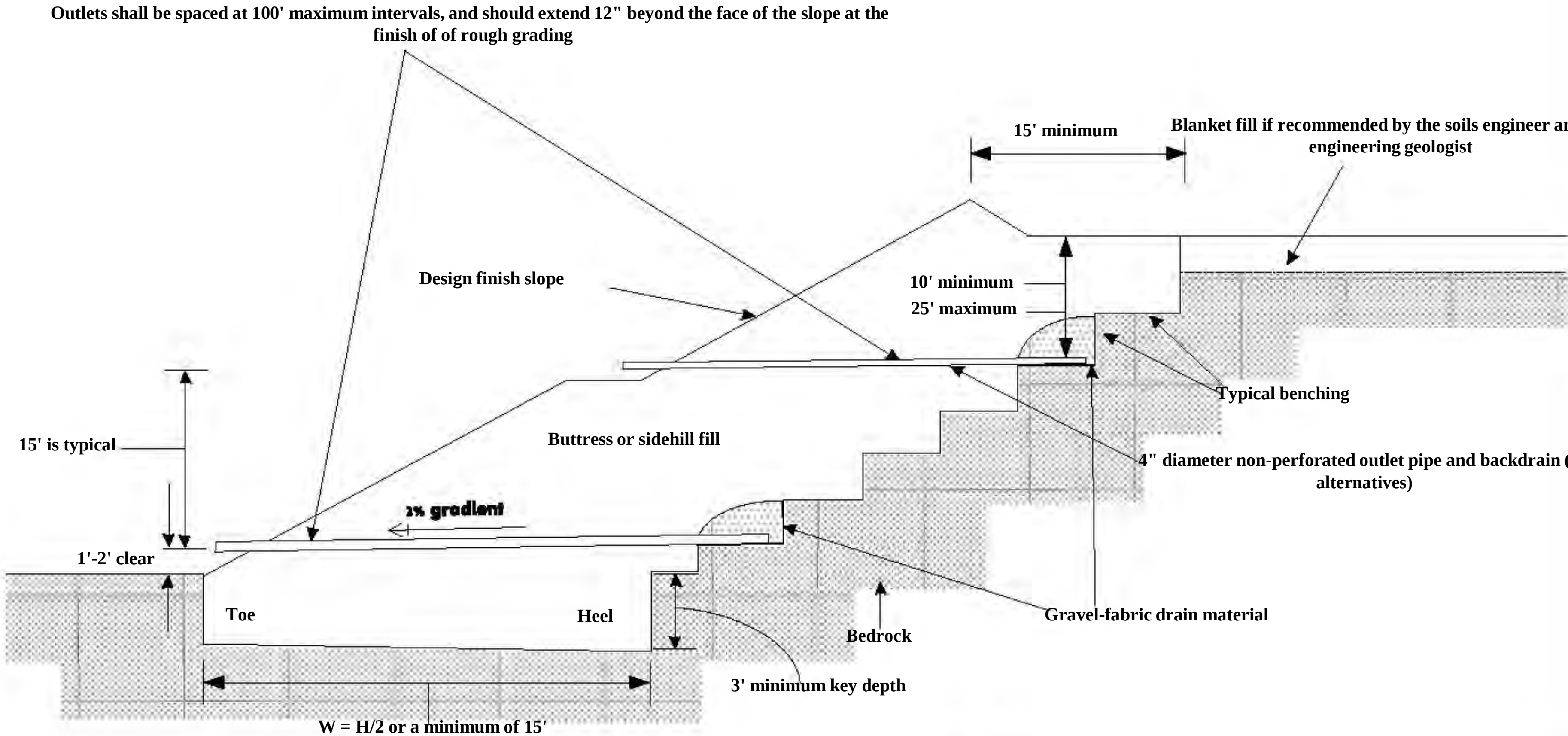
FIGURE G

TYPICAL STABILIZATION BUTTRESS FILL DESIGN



Note: Figure not to scale

EARTHWORK AND GRADING GUIDELINES

TYPICAL STABILIZATION BUTTRESS FILL DESIGN

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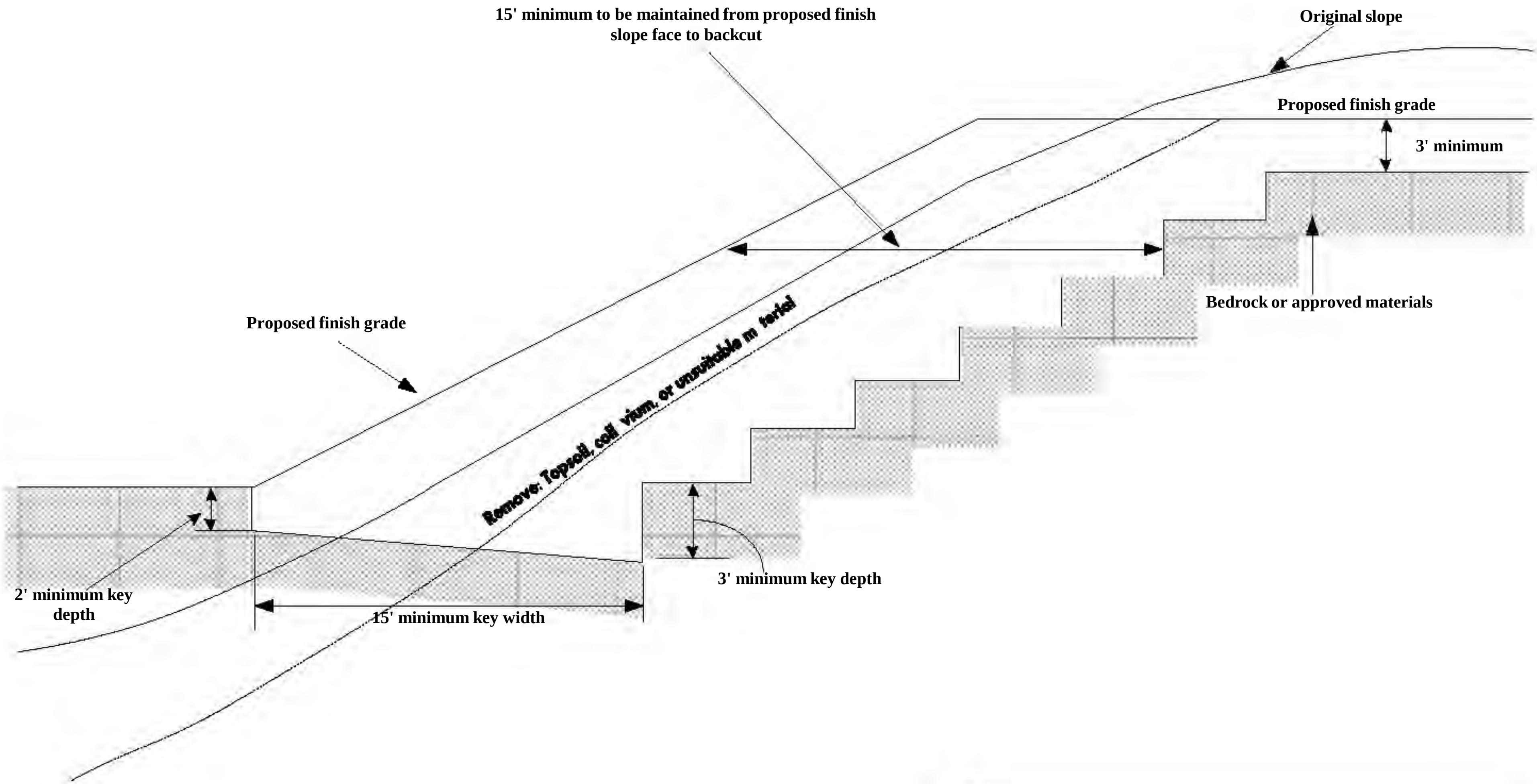
FIGURE H

SKIN FILL OF NATURAL GROUND



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

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

Note: Figure not to scale

EARTHWORK AND GRADING GUIDELINES
SKIN FILL OF NATURAL GROUND

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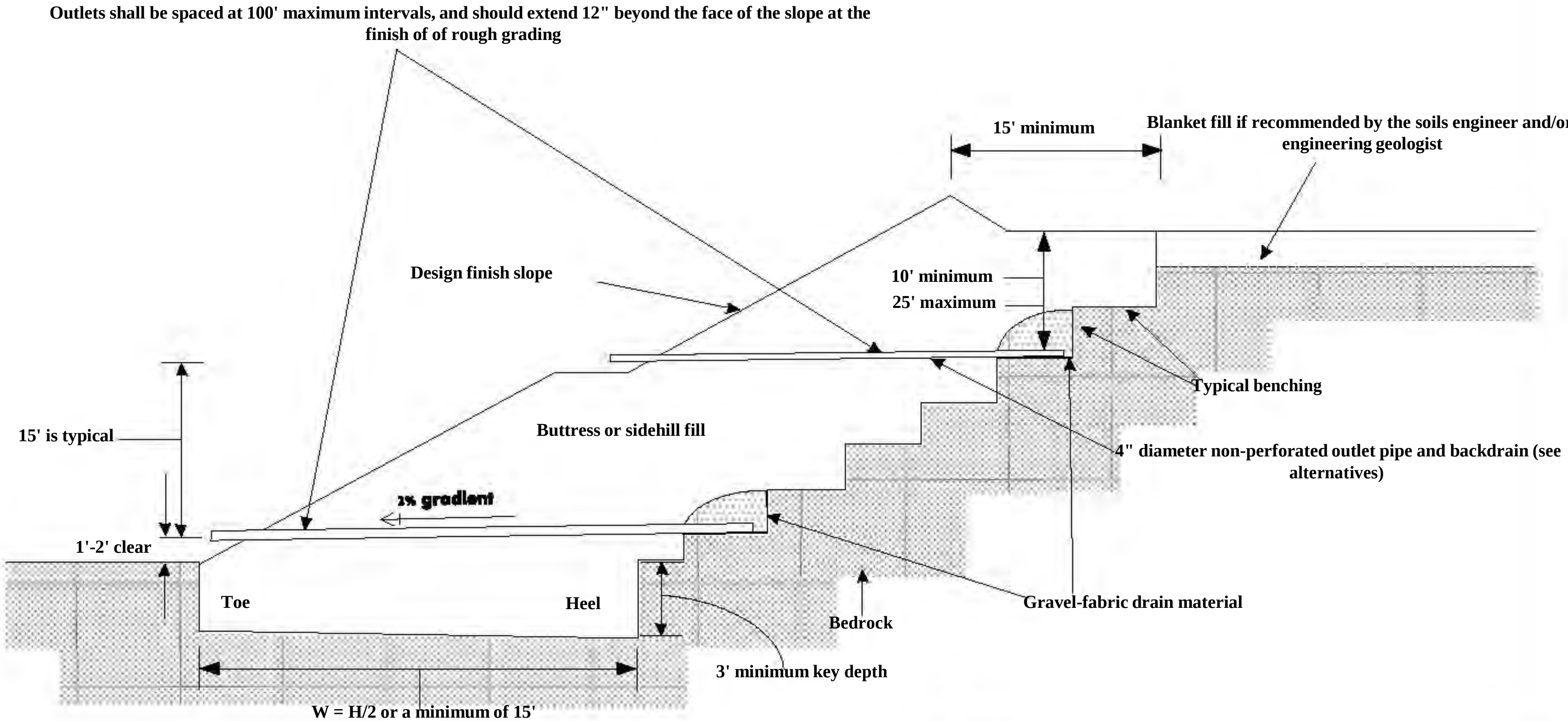
FIGURE G

TYPICAL STABILIZATION BUTTRESS FILL DESIGN



Note: Figure not to scale

EARTHWORK AND GRADING GUIDELINES

TYPICAL STABILIZATION BUTTRESS FILL DESIGN

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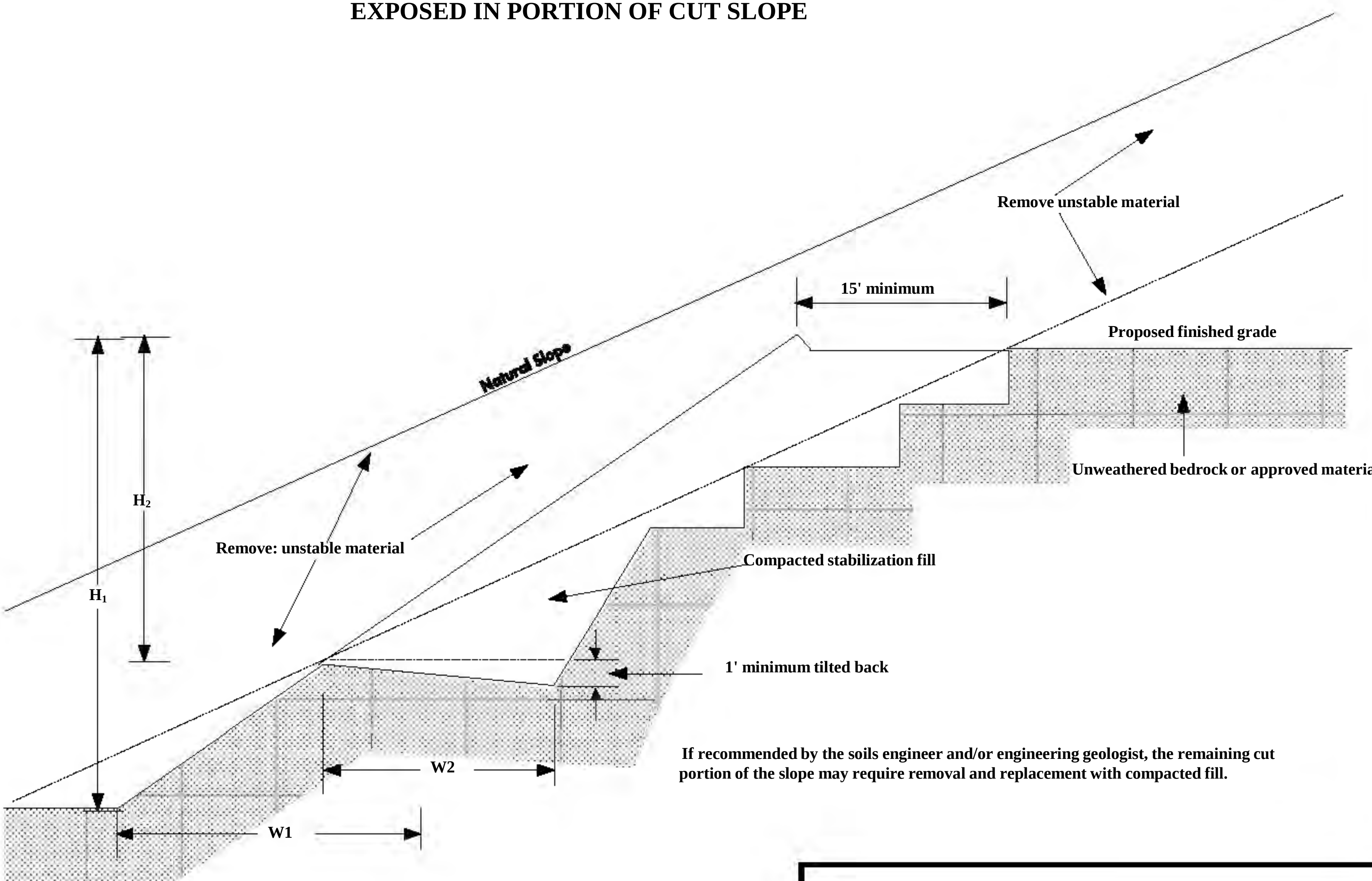
FIGURE H

STABILIZATION FILL FOR UNSTABLE MATERIAL EXPOSED IN PORTION OF CUT SLOPE



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

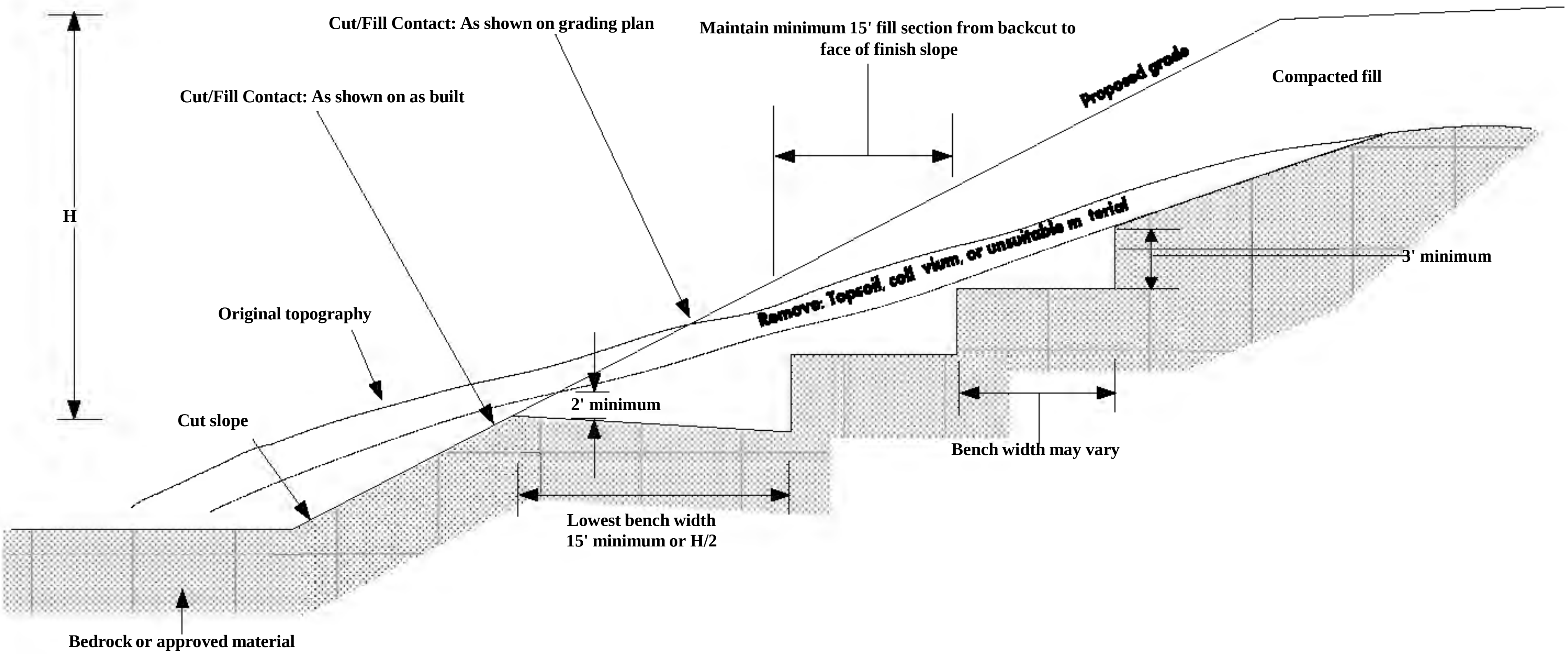
Note: Figure not to scale

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

EEI Engineering Solutions

FIGURE I

FILL OVER CUT DETAIL



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

Note: Figure not to scale

EARTHWORK AND GRADING GUIDELINES
FILL OVER CUT DETAIL

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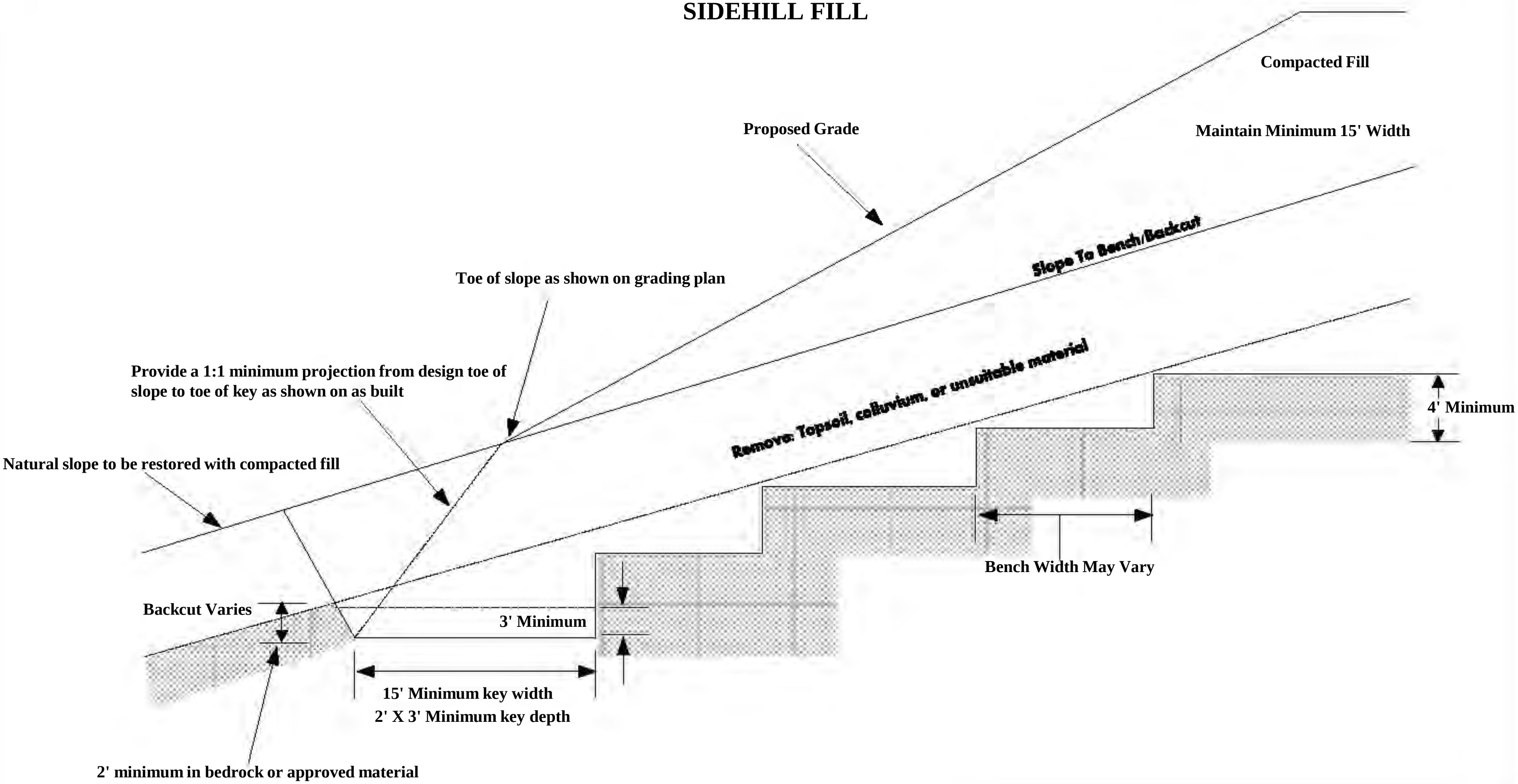
FIGURE J

FILL OVER NATURAL DETAIL
SIDEHILL FILL



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

Note: Figures not to scale

EARTHWORK AND GRADING GUIDELINES
FILL OVER NATURAL DETAIL SIDEHILL FILL

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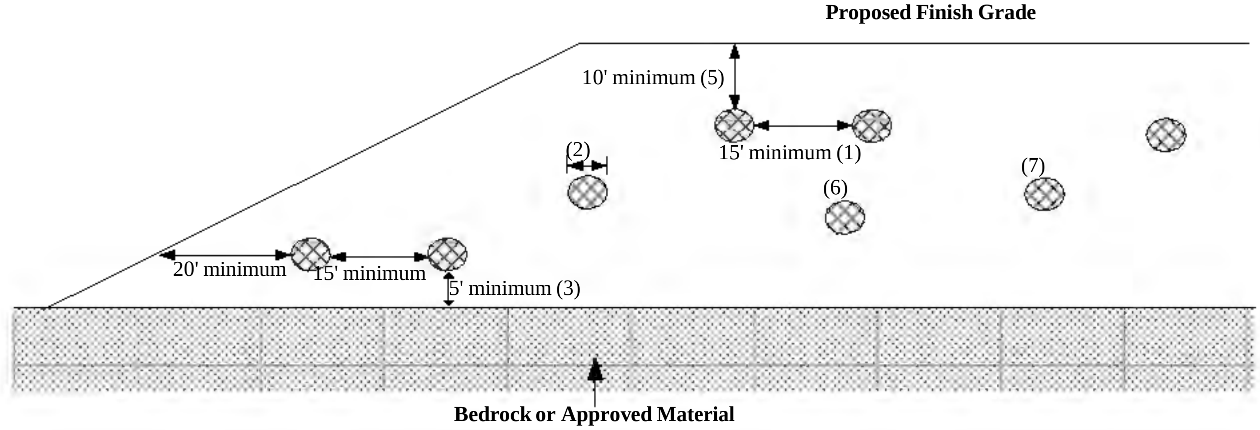
Engineering Solutions

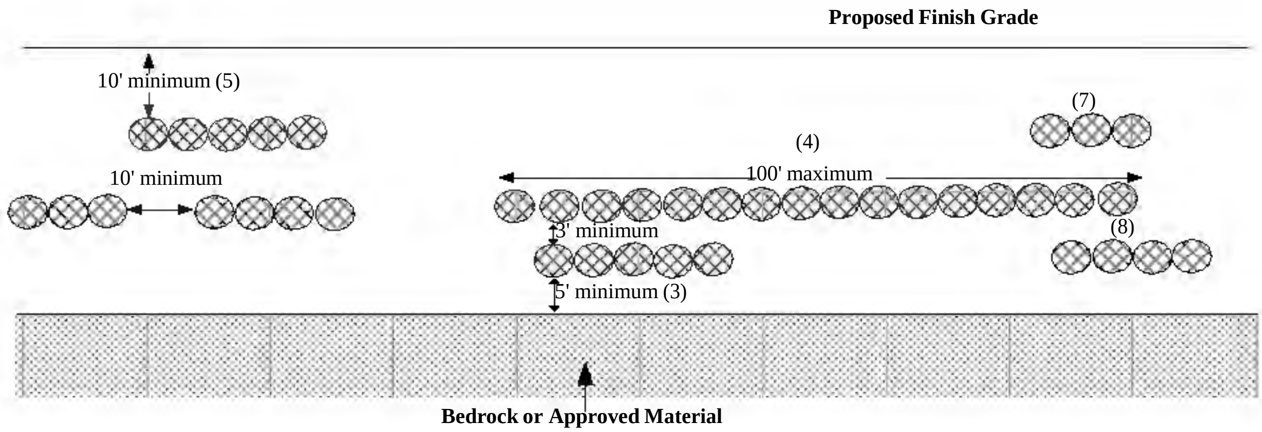
FIGURE K

OVERSIZE ROCK DISPOSAL

View Normal to Slope Face



View Parallel to Slope Face



Note:

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

Approximate Scale: 1" = 30'

0 FT 18 FT 30 FT 60 FT

Note: All distances are approximate

EARTHWORK AND GRADING GUIDELINES
OVERSIZE ROCK DISPOSAL

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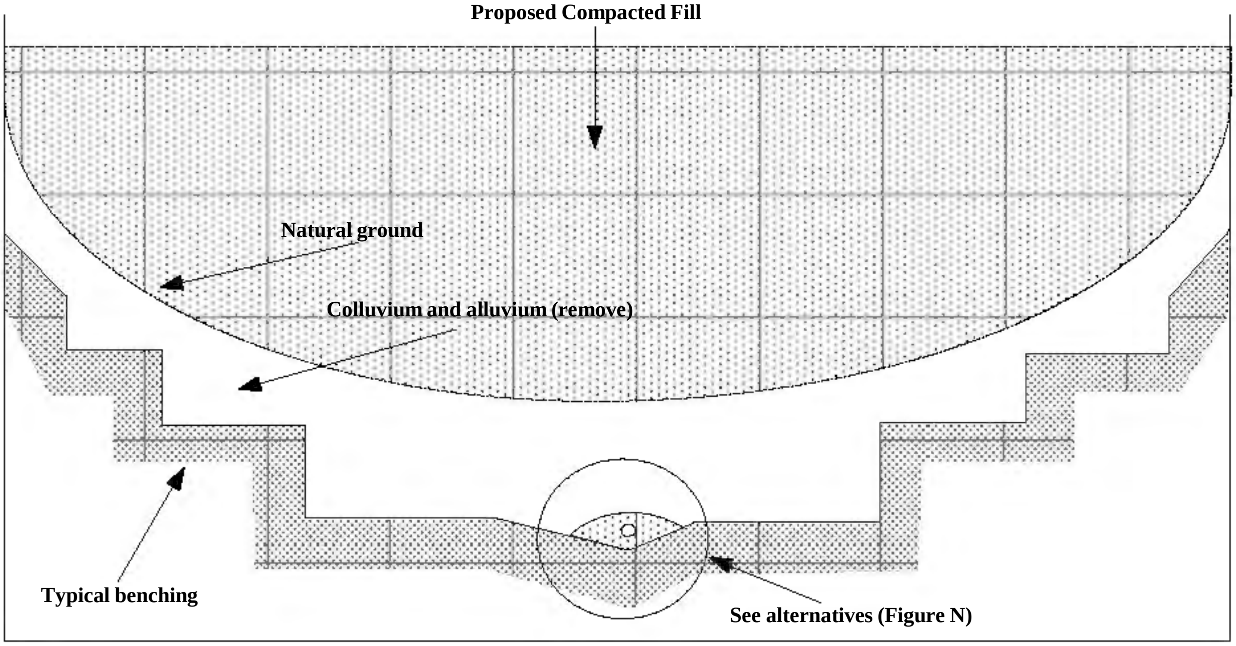
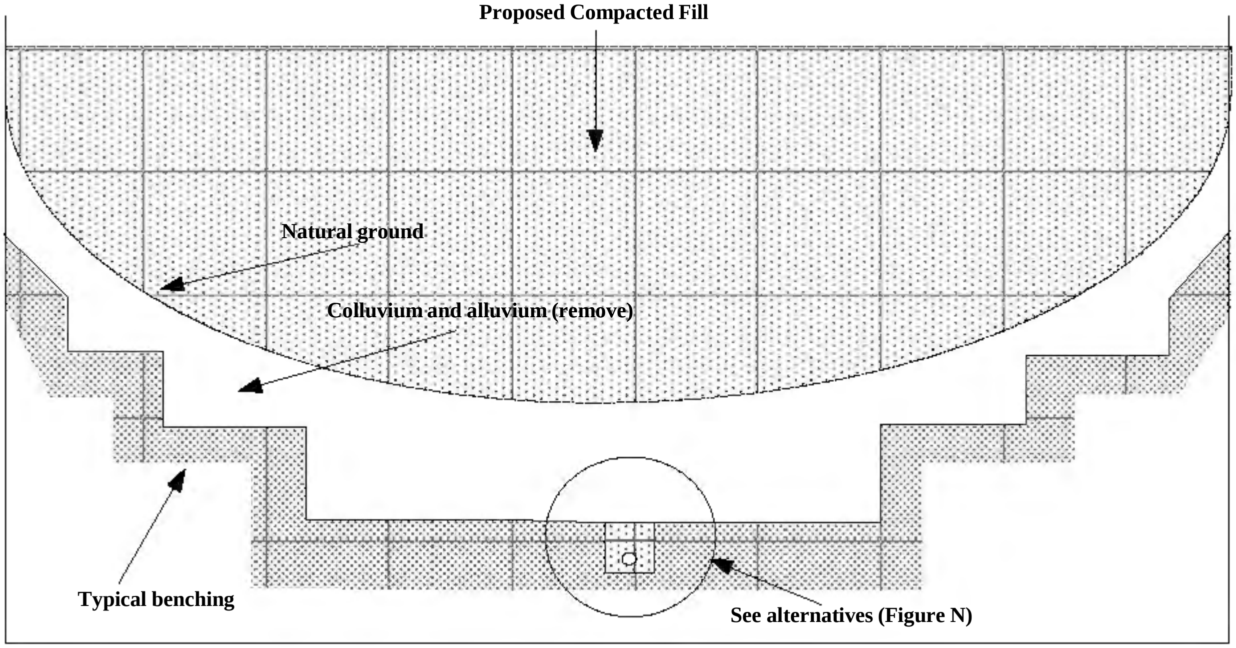
FIGURE L

CANYON SUBDRAIN DETAIL

Type A



Type B



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

Note: Figures not to scale

EARTHWORK AND GRADING GUIDELINES
CANYON SUBDRAIN DETAIL

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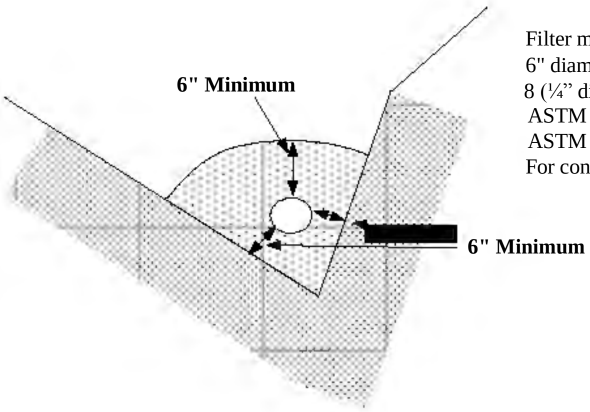
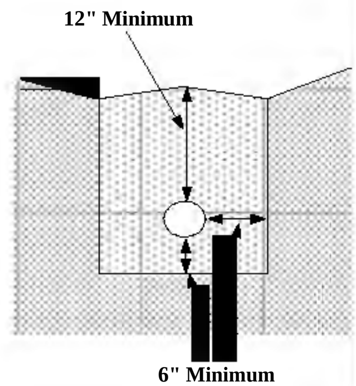
FIGURE M

CANYON SUBDRAIN ALTERNATE DETAILS

Alternate 1: Perforated Pipe and Filter Material



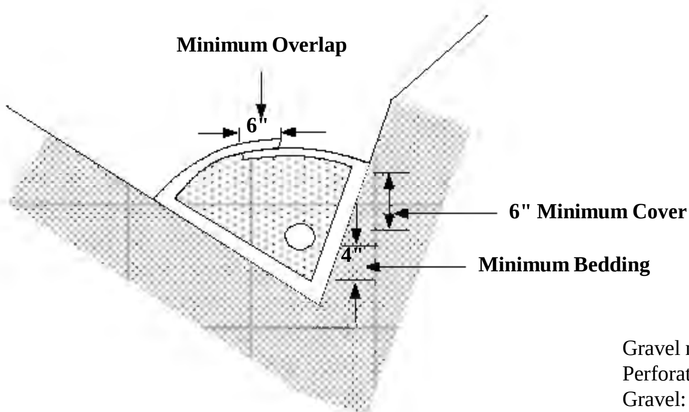
Filter material: Minimum volume of 9 $feet^3$/linear foot.
6" diameter ABS or PVC pipe or approved substitute with minimum 8 (¼" diameter) perforations per linear foot in bottom half of pipe.
ASTM D 2751, SDR 35 or ASTM D 1527, Schedule 40.
ASTM D 3034, SDR 35 or ASTM D 1785, Schedule 40.
For continuous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

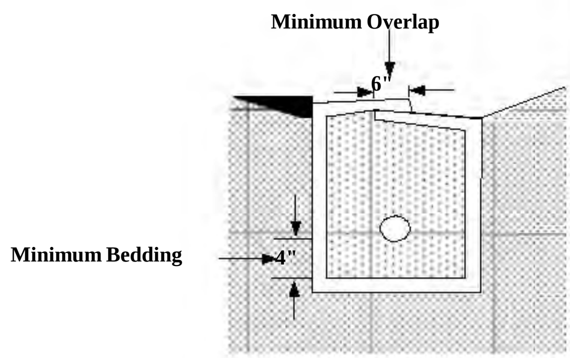


Filter Material

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

Alternate 2: Perforated Pipe, Gravel and Filter Fabric





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

EARTHWORK AND GRADING GUIDELINES
CANYON SUBDRAIN ALTERNATE DETAILS

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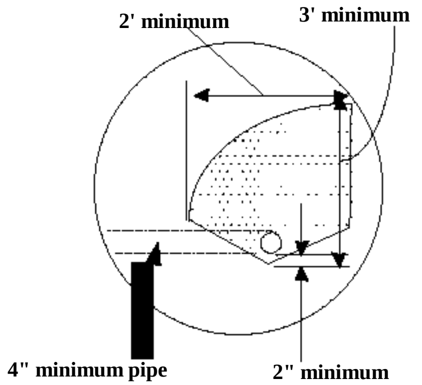
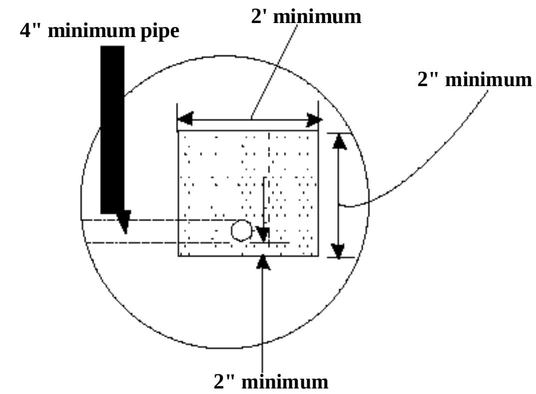
FIGURE N

Note: Figures not to scale

TYPICAL STABILIZATION BUTTRESS SUBDRAIN DETAIL





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

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

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

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

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

Filter Material

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

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

Filter Material

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

Sand equivalent: Minimum of 50

Note: Figures not to scale

EARTHWORK AND GRADING GUIDELINES
TYPICAL STABILIZATION BUTTRESS SUBDRAIN DETAIL

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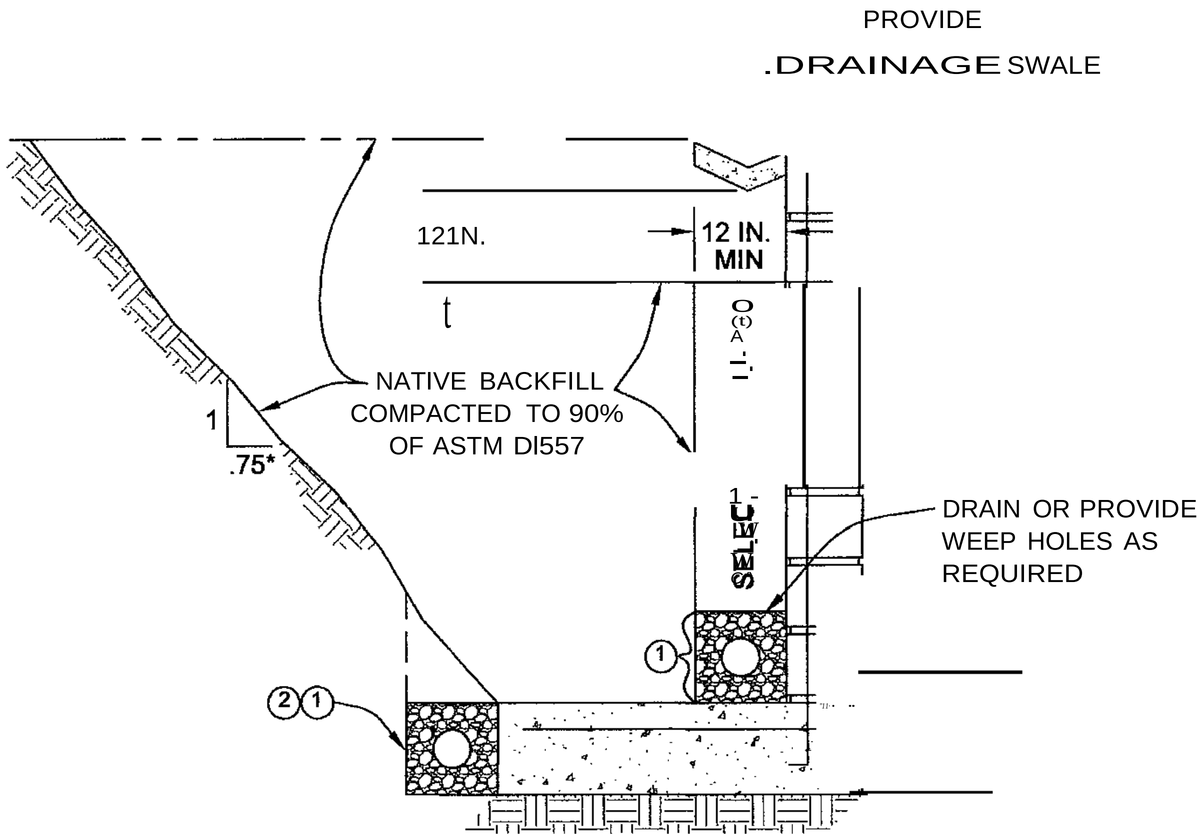
FIGURE O



* OR AS REQUIRED FOR SAFETY

NOTES

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

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

NOTE: FIGURE NOT TO SCALE

EARTHWORK & GRADING GUIDELINES
TYPICAL RETAINING WALL BACKFILL

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FIGURE P