South Shore Testing & Environmental

23811 Washington Ave, Suite C110, #112, Murrieta, CA 92562 E-mail: ss.testing@aol.com
Phone: (951) 239-3008 FAX: (951) 239-3122

March 8, 2018

Mr. Kelly D. Olauson
C/O Naggar & Associates
445 South D Street
Perris, California 92570

SUBJECT: **GEOTECHNICAL UPDATE INVESTIGATION**
Proposed "Palomar Crossings" ±66.92-Acre Mixed Commercial/Retail and Residential Development
Northeast Corner of State Highway 74 and Palomar Road
City of Menifee, Riverside County, California
Work Order No. 3801801.00U

Dear Mr. Olauson:

Pursuant to your authorization, a Geotechnical Update Investigation was conducted on the subject site in accordance with the 2016 California Building Code, Section 1803.5.11. Attached as **Plate 1**, the Geotechnical Map is a not-to-scale image of the Menifee North Specific Plan indicating the boundaries of the subject site, and the approximate location of the previous exploration borings (T.H.E., 2010 & 2011), and pertinent geotechnical information. The purpose of our investigation was to update the engineering parameters of the onsite soils to current proposed site development and the 2016 California Building Code (CBC).

Introduction

This investigation has been conducted resulting from a 2016 California Building Code Chapter 18 requirement for preliminary geotechnical investigation being conducted for all projects in Seismic Category D. This investigation will address geotechnical conditions existing on the site as they may pertain to the proposed high density residential, commercial and retail development with associated streets, parking and landscape areas that are to be constructed on the site. It is our understanding that the structures will be a typical one and/or two-story type V structures. Contained herein also are preliminary recommendations for foundation design for the proposed construction.

Proposed Development

It is our understanding that the proposed site use consists of the construction of high density residential, general retail, and commercial development with associated infrastructure. Typical cut/fill grading techniques, overexcavation and recompaction, will be required to achieve design grade and that the proposed structures will utilize continuous and column footings with slab-on-grade construction.

Site Description

The subject site consists of an irregular-shaped ±66.92-acre parcel of land (including SCE and flood control easements) located in the northerly portion of the City of Menifee, Riverside County, California. The site is bordered on the north by an unimproved dirt access road (Stone Lane), vacant agricultural land and existing residential development (northeast corner); on the east by Menifee Road and Man-of War Lane (dirt road); on the south by State Highway 74 and commercial/industrial development; and, on the west by Palomar Road, commercial/retail development and vacant land.

The subject site is currently vacant and undeveloped. Topographically, the subject site is in an area of relatively flat terrain with natural gradients less than 2% to the south-southwest. Drainage on the subject site is accomplished by sheetflow toward the south-southwest toward State Highway 74. At the time of our current investigation, vegetation on the subject site generally consisted of a dried growth of annual weeds and several large trees on the southeast corner of the site, site of a former single-family residence. Total relief of the site is approximately 20-ft. with a high elevation of approximately 1485-feet above mean sea level (msl) towards the northeast corner of the subject parcels and a low elevation of approximately 1465-feet (msl) towards the southwest corner of the subject site.

Subsurface Conditions

The USGS Geologic Map of the Romoland 7.5' Quadrangle (Morton, 2003) indicates the formational earth materials underlying the site are late to middle Pleistocene old alluvial fan deposits (map symbol Qoaf), which extended to the maximum depth explored of 51.5-ft bgs (THE, 2010 & 2011). This unit generally consisted of inter-lensing silty Sand (Unified Soil Classification-SM) and sandy Silts (ML). Minor amounts < 3-ft) of undocumented fill may be encountered on the southeast corner of the subject site, site of a former single-family residence. A brief description of the geologic units underlying the site that are considered pertinent to proposed development follows:

LEGEND

UNITS

Qoaf - OLD ALLUVIAL FAN DEPOSITS

SYMBOLS

B-4A - APPROXIMATE LOCATION OF EXPLORATORY BORINGS (THE, 2011)

B-10 - APPROXIMATE LOCATION OF EXPLORATORY BORINGS (THE, 2010)

SOUTH SHORE TESTING & ENVIRONMENTAL

GEOTECHNICAL MAP
PROPOSED "PALOMAR CROSSINGS" ±66.92-ACRE MIXED
COMMERCIAL/RETAIL & RESIDENTIAL DEVELOPMENT
NORTHEAST CORNER OF STATE HIGHWAY 74 AND PALOMAR ROAD
CITY OF MENIFEE, RIVERSIDE COUNTY, CALIFORNIA

WORK ORDER: 3801801.00U DATE MARCH 2018 PLATE: 1 OF

Old Alluvial Fan Deposits (Map Symbol – Qof)

The old alluvial fan deposits were encountered at the ground surface throughout the subject parcels and extended to the maximum depth explored of 51.5-ft (B-1A & B-1 [THE, 2011 & 2010]). This unit generally consisted of silty Sand (Unified Soil Classification-SM) and sandy Silts (ML). The silty Sands can be described as yellow brown to orange brown, fine to medium grained, minor coarse and gravel size, medium dense to dense, moderately graded and micaceous in part. The silts can generally be described as dark brown to orange brown, sandy in part, slightly moist to moist, micaceous and medium dense to dense.

Groundwater

Groundwater was not encountered within the previous exploratory borings, which were advanced to a maximum depth of 51.5-ft bgs (B-1A & B-1 [THE, 2011 & 2010]) on the southerly portion of the site adjacent to State Highway 74 or lower elevations of the subject parcels. The project site is located within the Perris Valley Hydrologic subarea of the Perris Basin. Regional groundwater is at least 100-ft bgs (Department of Water Resources, 1978). Minor fluctuations can and will likely occur in moisture or free water content of the soil owing to rainfall and irrigation over time. In addition, the depth to groundwater can fluctuate seasonally because of planned groundwater management.

Excavation Characteristics

We anticipate that the old alluvial fan deposits can be excavated to proposed depths with moderate ease to moderate difficulty utilizing conventional grading equipment (i.e. Caterpillar D-9 bulldozer or equivalent) in proper working condition.

Seismicity

The site is located in a region of generally high seismicity, as is all of southern California. During its design life, the site is expected to experience strong ground motions from earthquakes on regional and/or local causative faults. The subject site is not located within either a State of California (Hart, 2000) or County of Riverside (County of Riverside GIS Website, 2018) Fault-Rupture Hazard Zone for active faulting. No active faults are known to traverse the site (Morton, 2003 & Dibblee, 2003).

Active fault zones regional to the site include the San Jacinto fault (San Jacinto Valley segment), the Elsinore fault (Glen Ivy segment) and the San Andreas fault (Southern segment), which are located 15.7-kilometers northeast, 20.0-km southwest, and 37.2-km northeast, respectively. The following table lists the known faults that would have the most significant impact on the site:

FAULT	MAXIMUM PROBABLE EARTHQUAKE (MOMENT MAGNITUDE)	SLIP RATE	FAULT TYPE
San Jacinto (San Jacinto Valley segment) (15.7-km NE)	6.9	12 mm/year	A
Elsinore (Glen Ivy Segment) (20-km SW)	6.8	5 mm/year	A
San Andreas (Southern segment) (37.2-km NE)	7.2	25 mm/year	A

2016 California Building Code (CBC) -Seismic Parameters:

Based on the geologic setting and soil conditions encountered, the soils underlying the site are classified as "Site Class D, Stiff Soil Profile", according to the CBC. The seismic parameters according to the CBC are summarized in the USGS Design Maps Summary Report presented in **Appendix B**. The corresponding value for peak ground acceleration from the design response spectrum based on the 2016 CBC seismic parameters is **0.500g**.

SEISMIC EFFECTS

Ground Accelerations

The most significant earthquake to affect the property is a 6.9 Richter magnitude earthquake on the San Jacinto fault zone (San Jacinto Valley segment). Based on Section 1803.5.12 of the 2016 California Building Code, peak ground accelerations modified for site class effects (PGA_M) of approximately **0.500g** are possible for the design earthquake.

Ground Cracks

The risk of surface rupture as a result of active faulting is considered negligible based on the absence of known active faulting on the site (Morton, 2003 & Dibblee, 2003). Ground cracks can and do appear on sites for a variety of reasons including, but not limited to, strong seismic shaking, imperfections in subsurface strata (either man-made or natural), and the expansive nature of some soils near the ground surface. Therefore, the possibility of minor cracks at the ground surface for the life of the project cannot be fully eliminated.

Landslides

No geomorphic expression of landsliding or slope instability was noted during our site mapping or aerial photograph review. The subject parcels are located in an area of relatively flat, gently sloping terrain and a far distance away from any steep natural slopes. No landslides have been mapped in the area (Morton, 2003 and Riverside County GIS, 2018). The risk of seismically induced landsliding to affect the proposed development is low.

Liquefaction

The site is not within either a State of California or County of Riverside designated or mapped liquefaction hazard zone. Therefore, coupled with the absence of shallow groundwater (+100-ft) and the underlying medium dense to dense old alluvial fan deposits; it is our opinion that liquefaction is not anticipated, and further analysis appears to be unwarranted.

Seismically Induced Soil Settlement

The proposed footings are anticipated to be founded medium dense to dense engineered fill materials. The settlement potential, under seismic loading conditions for these onsite materials, in our opinion, is low.

Seiches and Tsunami

Considering the location of the site in relation to large bodies of water, seiches and tsunamis are not considered potential hazards of the site.

Rockfall Potential

The subject site is located in an area of relatively flat terrain that is free of boulder outcroppings. The potential for rockfall is anticipated to be negligible.

CONCLUSIONS AND RECOMMENDATIONS

Conclusions

General

The development of the site as proposed is both feasible and safe from a geotechnical standpoint if the recommendations contained herein are implemented during design and construction.

1. Per the not-to-scale "Site Planning Map", the proposed development will encompass the entire site with access to the site from Palomar Road, State Highway 74, and Menifee Road.

2. The previous geotechnical reports (THE, 2010 & 2011) and excavations indicate that suitable material for support of fill and/or structure is near the surface on the site. Earth materials on the site are also suitable for use as compacted structural fill.

3. Observation, classification, and testing indicate that the near surface soils have a very low expansion potential (EI's <20) consisting of a low plastic silty Sand and sandy Silt (THE, 2010 & 2011).

4. Based on the previous exploratory borings (THE, 2010 & 2011), older alluvial fan sedimentary units were encountered at the ground surface throughout the subject site and extended to the total depth explored of 51.5-ft bgs.

RECOMMENDATIONS

Site Grading

General

Based on the site plan, we anticipate that the proposed structures will be founded in fill and minor cut/fill grading (overexcavation and recompaction) will needed to achieve design grade. The subject site appears to be close to a balanced job, no import or export is anticipated to achieve final grades. It is important to note that all imported soils must be observed and approved by the soil engineer prior to use as fill to verify compliance with project specifications and consistency with onsite soils with respect to expansion potential and structural contact pressure.

Site Specific Grading

A representative of this firm shall be present to observe the bottoms of all excavations. A representative of this firm shall be present during all fill placement operations to monitor and test as the earth materials are being placed. This observation and testing is intended to assure compliance with the recommendations of this report as well as project specifications as they relate to earthwork construction, County and State ordinances and Table 1705.6 of the 2016 California Building Code.

The upper 1 to 2-ft of old alluvial fan deposits are considered loose and potentially compressible in their existing state, and will require complete removal and recompaction or offsite disposal. Removals should extend until medium dense alluvial soils are encountered with a minimum in-place density of 85 percent of the dry density as determined by ASTM D1557 test method. The exposed alluvial soils should be free of pinpoint pores and roots. Depths of removals within the pad areas should extend a minimum of 3-ft bgs or 2-ft below the bottom of the deepest footing, whichever is deeper. Removals should extend a minimum of 5-ft outside the building footprint or to a depth equal to the removal depth, whichever is greater. Depths of removals within the proposed driveways and parking areas should extend a minimum of 2-ft bgs. The exposed surfaces shall be suitably processed by moisture conditioning to near optimum moisture content, then compacted in the upper 12-inches to the minimum compaction requirement prior to placing fill. No structural fill shall be placed on any ground without first being observed by a representative of the company providing this report and then providing written certification that the ground is competent and prepared to receive fill.

Onsite soils derived from excavations will be suitable for use as structural fill provided they are free of large rock (8" or larger) and organic debris or construction waste. Approved fill material should be placed in 6 to 8-inch loose lifts, brought to optimum moisture content, and compacted to a minimum of 90% of the maximum laboratory dry density, as determined by the ASTM D 1557-12 test method. No rocks larger than 8-inches in diameter should be used as fill material as they inhibit the compaction process. Rocks larger than 8 inches may be removed or crushed and used as fill material. Broken concrete slab shall also be reduced in size to be less than 8-inches in the major direction. Rocks larger than 8-inches that cannot be crushed, organic materials, asphaltic concrete or oil bearing surface aggregate should be removed from the graded area and in the case of oil bearing materials, removed and taken to an appropriate dump site that is designed to handle such.

All earthwork should be done in accordance with the specifications contained in **Appendix C**. Additionally, it will be the responsibility of the owner and or the grading contractor to provide this firm with schedule information for grading activities that require observation and testing. It is preferred that we have a minimum of 48 hours of notice for such.

It will also be recommended that at the completion of rough grading, additional testing of engineering characteristics such as expansion potential and ancillary testing should take place to determine final design requirements for foundations, slabs and concrete used.

Expansion Index Testing

Previous Expansion Index testing was performed on a representative onsite soil samples collected during subsurface investigations of the subject site (THE, 2010 & 2011). The Expansion Index results were 2 & 5 or **Non Expansive** (Expansion Index ≤20 – 2016 CBC, Section 1803.5.3). Expansion Index testing should be conducted on representative samples exposed within the upper 3-ft of the building pads at the completion of rough grading. Final footing recommendations should be made at that time. Expansion testing should also be performed on imported soils prior to their approval as structural fill material.

Bearing Value and Footing Geometry

A safe allowable bearing value of 2,300 psf for foundations embedded into observed competent engineered fill soils may be used for design of continuous footings that maintain a minimum width of 12-inches and a minimum depth of at least 18-inches below the lowest adjacent grade. The bearing value may be increased by 10% for each additional foot of depth and/or width to a maximum of 3,200 psf. The bearing value may be increased by one-third for seismic or other temporary loads. Continuous footings, for single-story or equivalent structures, should have a minimum width of 12-inches and depth of 18-inches and conform to the minimum criteria of the 2016 CBC for non-expansive soils (EI = 2 & 5). Continuous footings, for two-story or equivalent structures, should have a minimum width of 15-inches and depth of 24-inches and conform to the minimum criteria of the 2016 CBC for non-expansive soils (EI = 2 & 5). The use of isolated column footings is not discouraged, however, where utilized, should have a minimum embedment of 18-inches below lowest soil grade. The minimum distance of the bottom outside edge of all footings and any slope face shall be 5 feet. All footings should be embedded a minimum of 12 inches into observed competent native materials or properly compacted fill, regardless of depth below the adjacent ground surface.

Settlement

The bearing value recommended above reflects a total settlement of 0.5" and a differential settlement of 0.5" within a horizontal distance of 20 feet (L/480). Most of this settlement is expected to occur during construction and as the loads are being applied.

Concrete Slabs

All concrete slabs on grade should be 4 inches thick, minimum. They should be underlain by 2-inches of sand or approved non-expansive onsite materials. Imported or approved onsite materials may be utilized for this purpose. Contractors should be advised that when pouring during hot or windy weather conditions, they should provide large slabs with sufficiently deep weakened plane joints to inhibit the development of irregular or unsightly cracks. Also, 4-inch thick slabs should be jointed in panels not exceeding 8-ft in both directions to augment proper crack direction and development.

Moisture Barrier

When the intrusion of moisture through concrete slabs is objectionable, particularly with interior slabs where flooring is moisture sensitive, a vapor barrier should be installed onto the subgrade prior to the pouring of concrete. It should consist of a minimum 10-mil visqueen, protected from puncture with 2-inches of sand above. This is considered a minimum recommendation as there are other devices that provide as good as or better moisture protection. The project architect and or structural engineer may recommend alternative devices for moisture protection.

Reinforcement

From a Geotechnical standpoint, continuous footings should be reinforced with a minimum of two number 4 steel bar placed at the top and bottom. In no case, should the content of steel in concrete footings be less than the recommended minimums of the appropriate sections of the A.C.I. standards. Slabs should be reinforced with a minimum of number 3 steel bars placed at the center of thickness at 18-inch centers both ways (CBC 2016). These are considered minimums and additional requirements may be imposed by other structural engineering design requirements. In addition, at the completion of grading, testing of the near surface soils may indicate that different or more stringent reinforcing schedule minimums may be appropriate. Careful consideration should be given to the recommendations that will be contained in the final report of compaction test results and foundation design requirements.

Concrete

Based on previous corrosivity suite testing (THE, 2010 & 2011), Type II Portland cement concrete can be utilized for the subject site. Previous laboratory analysis results indicated that percentage by weight for soluble sulfates in onsite soils varied from ND (no detection), 0.08 to 0.007, which equates to a **Negligible** sulfate exposure (0.0 to 0.10 percentage by weight) per American Concrete Institute (ACI), 318, Table 4.3.1 (2005). Soluble sulfate content testing should be conducted at completion of rough grading to confirm concentration of sulfite ions within the onsite earth materials.

Corrosion Suite Testing

Previous corrosivity test results testing (THE, 2010 & 2011), indicated that the onsite near surface soils are moderately corrosive (NACE International, 1984). South Shore Testing & Environmental does not practice corrosion engineering. If specific information or evaluation relating to the corrosivity of the onsite or any import soil is required, we recommend that a competent corrosion engineer be retained to interpret or provide additional corrosion analysis and mitigation. Corrosion suite testing should be conducted at the completion of rough grading.

Lateral Loads

The bearing value of the soil may be increased by one third for short duration loading (wind, seismic). Lateral loads may be resisted by passive forces developed along the sides of concrete footings or by friction along the bottom of concrete footings. The value of the passive resistance for level ground may be computed using an equivalent fluid density of 370 pcf for level ground. The total force should not exceed 3,000 psf. A coefficient of friction of .37 may be used for the horizontal soil/concrete interface for resistance of lateral forces. If friction and passive forces are combined, then the passive values should be reduced by one third.

Preliminary Structural Section

We recommend the following preliminary structural section for proposed parking and driveway areas for the subject site. The preliminary design of the pavement sections for the proposed parking and driveways, interior streets, street improvements are based on an assumed R-value of 25. R-value testing should be conducted at the completion of rough grading to verify soils exposed at subgrade, and a final structural section should be recommended at that time. The Traffic Indexes (TI) for Palomar Road and State Highway 74 are assumed values of 8.0 (Industrial Collector Street – Riverside County Standard No. 114) and 11.0 (Expressway – Standard 114), The TI's for Palomar Road , State Highway 74 and any interior streets should be verified by the project civil engineer. The recommended preliminary pavement sections are:

AREA	TI	PAVEMENT SECTION
Parking Lot	5.5	0.25' (3.0") AC over 0.67' (8.0") ABII
Driveway (light traffic)	5.5	0.25' (3.0") AC over 0.67' (8.0") ABII
Driveway (heavy traffic)	8.0	0.38' (4.6") AC over 1.05' (12.6") ABII
Interior Streets	5.5	0.25' (3.0") AC over 0.67' (8.0") ABII
Palomar Road	8.0	0.38' (4.6") AC over 1.05' (12.6") ABII
State Highway 74	11.0	0.57' (6.8") AC over 1.51' (18.1") ABII

It is recommended that the subgrade materials be compacted to a depth of 1 foot below subgrade elevation and that both the subgrade materials and the ABII be compacted to 95% relative to the maximum density of the respective materials, as determined by ASTM D1557 laboratory tests. R-Value testing should be conducted on imported soils prior to their approval as structural fill material

Oversize Rock

No oversize rock is anticipated as the subject site is underlain by old alluvial fan deposits that are free of large rock. If any oversize material is to be generated during grading operations, it should be disposed of off-site, utilized in landscaping, or placed in an approved rock fill in accordance with **Appendix C** of this report.

Earthwork Factors

Earthwork Factors

Shrinkage results when a volume of material removed at one density is compacted to a higher density. A shrinkage factor of 15 percent for the upper 2-ft of old alluvial soils should be anticipated when excavating and compacting the onsite old alluvial soils to an average relative compaction of 92 percent. An increase in relative compaction, or deeper removals, could correspond to an increase in shrinkage values. Subsidence, as a result of ground preparation, may also be anticipated on the order of 0.15 feet, occurring mostly during site construction.

Slope Stability & Construction

We anticipate that cut and fill slopes constructed at a 2:1 (horizontal:vertical) slope ratio to a maximum vertical height of 10-ft will be surficially and grossly stable if constructed in accordance with the recommendations presented in this report and in **Appendix C** of this report. Owing to the relatively flat nature of the subject site, no fill and/or cut slopes over 3-ft in vertical height are anticipated for the subject parcels.

Utility Trench Backfill

All trench excavations should be conducted in accordance with Cal-OSHA standards as a minimum. The soils encountered within our exploratory trenches are generally classified as Type "C" soil in accordance with the CAL/OSHA (2013) excavation standards. Based upon a soil classification of Type "C", the temporary excavations should not be inclined steeper than 1.5:1 (h:v) for a maximum depth of 20-ft. For temporary excavations, deeper than 20-ft or for conditions that differ from those described for Type "C" in the CAL/OSHA excavation standards, the project geotechnical engineer should be contacted.

Utility trench backfill should be compacted to a minimum of 90 percent of the maximum dry density determined in laboratory testing by the ASTM D 1557-12 test method. It is our opinion that utility trench backfills consisting of onsite or approved sandy soils can best be placed by mechanical compaction to a minimum of 90 percent of the maximum dry density. The upper 1-ft of utility trench excavations located within pavement areas should be compacted to a minimum of 95 percent of the maximum dry density.

Fine Grading and Site Drainage

Fine grading of areas outside of proposed structures should be accomplished such that positive drainage exists away from all footings in accordance with 2016 CBC and local governing agency requirements. Run-off should be conducted in a non-erosive manner toward approved drainage devices per approved plans. No run-off should be allowed to concentrate and flow over the tops of slopes.

Construction

South Shore Testing & Environmental, or a duly designated representative, should be present during all earthwork construction in accordance with the standard specifications contained at the back of this report, to test and or confirm the conditions encountered during this study. In addition, post earthwork construction monitoring should be conducted at the following stages:

- At the completion of rough grading of the building pad so that a finished surface compaction test may be obtained. Moisture content near optimum will necessarily need to be maintained, both to maintain proper compaction and to prevent wind erosion of the pad.

- At the completion of foundation excavations, but prior to the placement of steel and or other construction materials in them. As a requirement of this report, the undersigned must, in writing, certify that the foundations meet the minimum requirements of this report and the building plans for depth and width along with the earth materials being the appropriate moisture content and compaction. Backfilling of over deepened footings with earth materials will not be allowed and must be poured with concrete. Consequential changes and differences may exist throughout the earth materials on the site. It may be possible that certain excavations may have to be deepened slightly if earth materials are found to be loose or weak during these observations.

- Any other pertinent post construction activity where soils are excavated or manipulated or relied upon in any way for the performance of buildings or hardscape features.

Supplemental Recommendations

If at any time during grading or construction on this site, conditions are found to be different than those indicated in this report, it is essential that the soil engineer be notified. The soil engineer reserves the right to modify in any appropriate way the recommendations of this report if site conditions are found to be different than those indicated in this report.

- The soil at the surface is observed to be compact silty coarse grained soil. It is minimally to non-erosive. It is dense at shallow depths, on the order of 2-ft and water does not percolate well into the onsite soils.

- Cuts to 5-ft, or slightly more will stand vertical for normal time periods associated with construction of backcuts for fill slopes or retaining walls. Time periods for unsupported cuts 5-ft or greater vertical should be limited to 60 days in the non-rainy season and 30 days in the rainy season.

Grading and Foundation Plan Reviews

Once grading and foundation plans are finalized, Grading Plan and Foundation Plan Reviews should be performed to review plans and confirm that the plans are in general conformance with recommendations presented in this report.

Construction Monitoring

Observation and testing by South Shore Testing & Environmental is necessary to verify compliance with recommendations contained in this report and to confirm that the geotechnical conditions encountered are consistent with those encountered. South Shore Testing & Environmental should conduct construction monitoring during any fill placement and subgrade preparation prior to placement of fill or construction materials.

LIMITATIONS

Our investigation was performed using the degree of care and skill ordinarily exercised, under similar circumstances, by reputable Geotechnical Engineers and Geologists practicing in this or similar localities. No other warranty, expressed or implied, is made as to the conclusions and professional advice included in this report.

The report is issued with the understanding that it is used only by the owner and it is the sole responsibility of the owner or their representative to ensure that the information and recommendations contained herein are brought to the attention of the architect, engineer, and appropriate jurisdictional agency for the project and incorporated into the plans; and the necessary steps are taken to see that the contractor and subcontractors carry out such recommendations contained herein during construction and in the field.

The samples taken and used for testing and the observations made are believed representative; however, soil and geologic conditions can vary significantly between test locations. The evaluation or identification of the potential presence of hazardous or corrosive materials was not part of the scope of services provided by **South Shore Testing & Environmental**, or its assigns.

The findings of this report are valid as of the present date. However, changes in the condition of a property can occur with the passage of time, whether due to natural processes or the works of man on this or adjacent properties. In addition, changes in applicable or appropriate standards may occur, whether they result from legislation or the broadening of knowledge. Accordingly, the findings of this report may be invalidated wholly or partially by changes outside our control. Therefore, this report is subject to review and revision as changed conditions are identified.

The firm that performed the geotechnical investigation for this project should be retained to provide testing observation services during construction to maintain continuity of geotechnical interpretation and to check that the recommendations presented herein are implemented during site grading, excavation of foundations and construction of improvements.

If another geotechnical firm is selected to perform the testing and observation services during construction operations, that firm should prepare a letter indicating their intent to assume the responsibilities of project geotechnical engineer of record. Selection of another firm to perform any of the recommended activities or failure to retain the undersigned to perform the recommended activities wholly absolves **South Shore Testing & Environmental**, the undersigned, and its assigns from all liability arising directly or indirectly from any aspects of this project.

This opportunity to be of service is sincerely appreciated. If you have any questions, please call.

Respectfully submitted,

South Shore Testing & Environmental

John P. Frey
Project Geologist

William C. Hobbs, RCE 42265
Civil Engineer

ATTACHMENTS

Plate 1 – Geotechnical Map (not-to-scale)
Appendix A - References
Appendix B - USGS Design Maps Summary Report
Appendix C - Standards of Grading

APPENDIX A

References

REFERENCES

California Building Standards Commission, 2016, "2016 California Building Code", California Code of Regulations, Title 24, Part 2, Volumes 1 and 2 of 2.

California Division of Mines and Geology, 2000, "Digital Images of Official Maps of Alquist-Priolo Earthquake Fault Zones of California", Southern Region, DMG CD 2000-003.

California Division of Mines and Geology, 1996, "Probabilistic Seismic Hazard Assessment for the State of California", CDMG Open File Report 96-08,

California Division of Mines and Geology, 1996, "Guidelines for Evaluating the Hazard of Surface Fault Rupture," Adopted by the State Mining and Geology Board on May 9, 1996.

California Division of Mines and Geology, revised 1985, "Fault-Rupture Hazard Zones in California, Alquist-Priolo Special Studies Zones Act of 1972 with Index to Special Studies Zones Maps", Special Publication 42.

County of Riverside GIS Website, 2018.

Department of Water Resources, Southern District, April 1978, "Water Resources Evaluation of the San Jacinto Area", District Report.

Dibblee, Thomas W., 2003, "Geologic Map of the Perris Quadrangle, Riverside County, California", Dibblee Geology Center Map #DF-112, Scale: 1" = 2,000'.

Dibblee, Thomas W., 2003, "Geologic Map of the Sunnymead/South ½ of Redlands Quadrangles, San Bernardino and Riverside Counties, California", Dibblee Geology Center Map #DF-110, Scale: 1" = 2,000'.

Hart, E.W., 2000, "Fault-Rupture Hazard Zones in California", California Division of Mines and Geology Special Publication 42, CD-003 (CD-ROM Version).

Morton, D.M., 2004, "Preliminary Digital Geologic Map of the Santa Ana 30' x 60' Quadrangle, Southern California (Version 2.0)", U.S. Geological Survey in Cooperation with the California Geologic Survey, Open-File Report 99-172, Scale: 1"=100,000'.

Morton, D.M., 2003, "Geologic Map of the Romoland 7.5' Quadrangle Riverside County, California", U.S. Geological Survey, Open-File Report 03-102, Scale: 1"=2,000'.

Morton, D.M., 1991, "Geologic Map of the Romoland 7.5' Quadrangle Riverside County, California", U.S. Geological Survey, Open-File Report 90-701, Scale: 1"=2,000'.

REFERENCES (Continued)

T.H.E. Soils Company, Inc., 2011, "Supplement to Preliminary Geotechnical Investigation, Proposed 58.48-Acre Mixed Commercial/Retail and Residential Development, APN Nos.: 329-100-025, -026, -027, -030, -031 & -032 and 329-090-069, -070, -071, & -072, Northeast Corner of State Highway 74 and Palomar Road, Romoland Area, Riverside County, California", Work Order No. 14431001.00A, Dated November 10, 2011.

T.H.E. Soils Company, Inc., 2010, "Preliminary Geotechnical Investigation, Proposed 38.82-Acre Commercial/Retail Development, APN Nos.: 329-100-025, -026, -027, -030, -031 & -032, Northeast Corner of State Highway 74 and Palomar Road, Romoland Area, Riverside County, California", Work Order No. 14431001.00, Dated June 16, 2010.

Woodford, A. O., Shelton, J. S., Doehring, D. O., and Morton, R. K., 1971, "Pliocene-Pleistocene History of the Perris Block, Southern California", GSA Bulletin, Volume 82, Number 12.

AERIAL PHOTOGRAPHS UTILIZED

YEAR/SCALE	FLIGHT #/FRAME #	AGENCY
1948/1"=2,625'	12RS 643 12AF/89, 90	Riv Co Flood Control
1962/1"=2,000'	Co. Flight/1 16 – 1 18	Riv Co Flood Control
1974/1"=2,000'	Co. Flight/520	Riv Co Flood Control
1980/1"=2,000'	Co. Flight/552, 553	Riv Co Flood Control
1984/1"=1,600'	Co. Flight/961, 962	Riv Co Flood Control
1990/1"=1,600'	Co. Flight/11-21, 11-22	Riv Co Flood Control
1995/1"=1,600'	Co. Flight/11-24, 11-25	Riv Co Flood Control

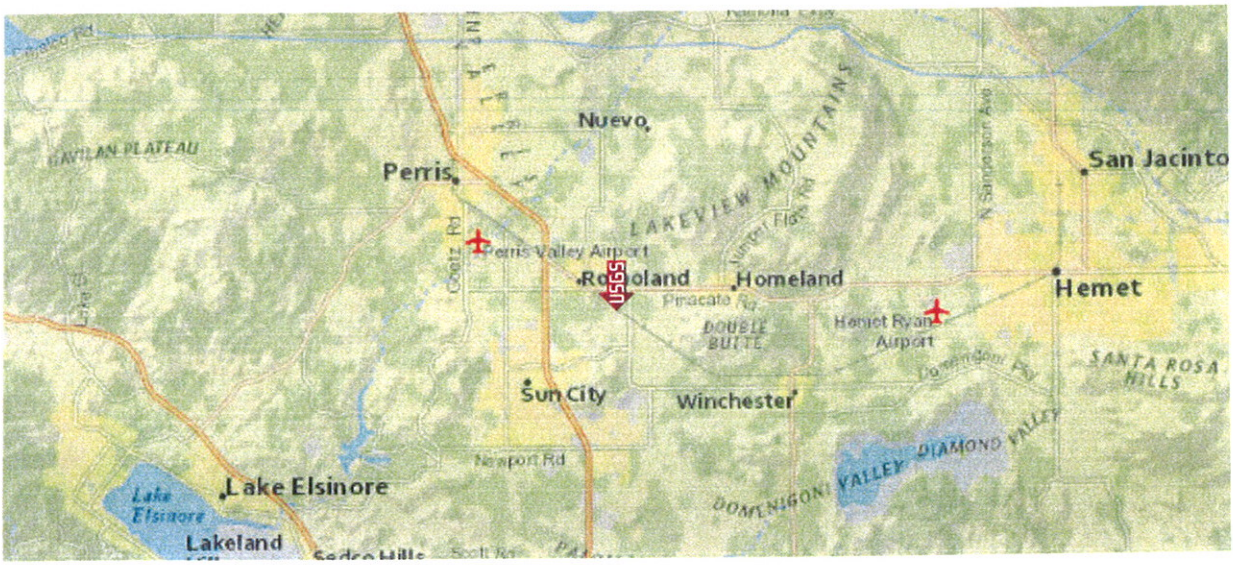
APPENDIX B

USGS Design Maps Summary Report

USGS Design Maps Summary Report

User-Specified Input

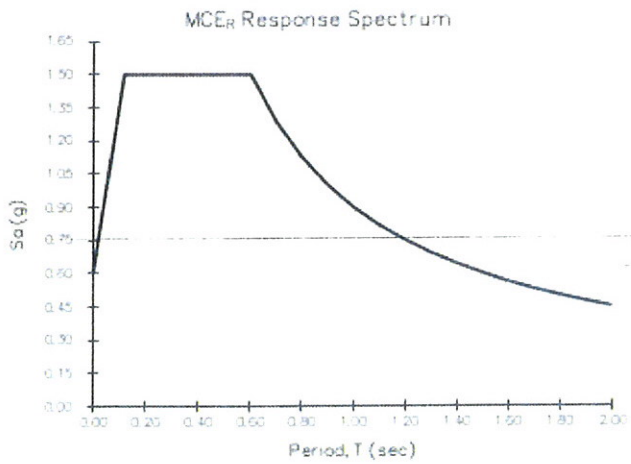
Report Title	Palomar Crossings Wed March 7, 2018 18:31:03 UTC
Building Code Reference Document	ASCE 7-10 Standard (which utilizes USGS hazard data available in 2008)
Site Coordinates	33.7446°N, 117.1595°W
Site Soil Classification	Site Class D – "Stiff Soil"
Risk Category	I/II/III

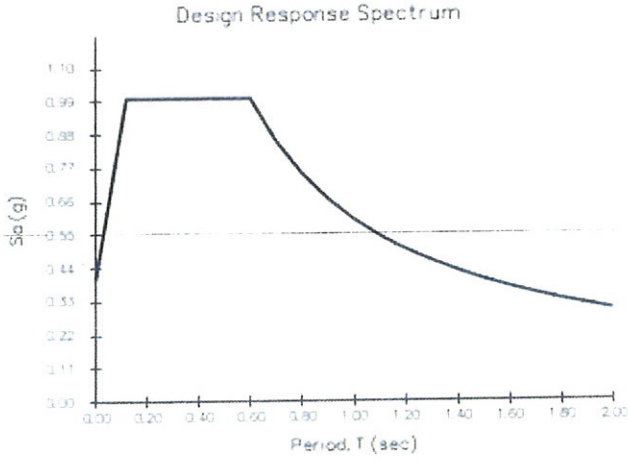


USGS-Provided Output

S_S =	1.500 g	S_{MS} =	1.500 g	S_{DS} =	1.000 g
S_1 =	0.600 g	S_{M1} =	0.900 g	S_{D1} =	0.600 g

For information on how the SS and S1 values above have been calculated from probabilistic (risk-targeted) and deterministic ground motions in the direction of maximum horizontal response, please return to the application and select the "2009 NEHRP" building code reference document.





For PGA_M, T_L, C_{RS}, and C_{R1} values, please view the detailed report.

USGS Design Maps Detailed Report

ASCE 7-10 Standard (33.7446°N, 117.1595°W)

Site Class D – "Stiff Soil", Risk Category I/II/III

Section 11.4.1 — Mapped Acceleration Parameters

Note: Ground motion values provided below are for the direction of maximum horizontal spectral response acceleration. They have been converted from corresponding geometric mean ground motions computed by the USGS by applying factors of 1.1 (to obtain S_S) and 1.3 (to obtain S_1). Maps in the 2010 ASCE-7 Standard are provided for Site Class B. Adjustments for other Site Classes are made, as needed, in Section 11.4.3.

From Figure 22-1 [1] $S_S = 1.500$ g

From Figure 22-2 [2] $S_1 = 0.600$ g

Section 11.4.2 — Site Class

The authority having jurisdiction (not the USGS), site-specific geotechnical data, and/or the default has classified the site as Site Class D, based on the site soil properties in accordance with Chapter 20.

Table 20.3-1 Site Classification

Site Class	$\bar{v}_s$	$\bar{N}$ or $\bar{N}_{ch}$	$\bar{s}_u$
A. Hard Rock	>5,000 ft/s	N/A	N/A
B. Rock	2,500 to 5,000 ft/s	N/A	N/A
C. Very dense soil and soft rock	1,200 to 2,500 ft/s	>50	>2,000 psf
D. Stiff Soil	600 to 1,200 ft/s	15 to 50	1,000 to 2,000 psf
E. Soft clay soil	<600 ft/s	<15	<1,000 psf
	Any profile with more than 10 ft of soil having the characteristics: • Plasticity index PI > 20, • Moisture content $w \geq 40\%$, and • Undrained shear strength $\bar{s}_u$ < 500 psf		
F. Soils requiring site response analysis in accordance with Section 21.1	See Section 20.3.1		

For SI: 1ft/s = 0.3048 m/s 1lb/ft² = 0.0479 kN/m²

Section 11.4.3 — Site Coefficients and Risk-Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameters

Table 11.4-1: Site Coefficient F_a

Site Class	Mapped MCE_R Spectral Response Acceleration Parameter at Short Period				
	$S_S \leq 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S \geq 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight-line interpolation for intermediate values of S_S

For Site Class = D and S_S = 1.500 g, F_a = 1.000

Table 11.4-2: Site Coefficient F_v

Site Class	Mapped MCE_R Spectral Response Acceleration Parameter at 1-s Period				
	$S_1 \leq 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 \geq 0.50$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	See Section 11.4.7 of ASCE 7				

Note: Use straight-line interpolation for intermediate values of S_1

For Site Class = D and S_1 = 0.600 g, F_v = 1.500

Equation (11.4–1): $S_{MS} = F_a S_S = 1.000 \times 1.500 = 1.500$ g

Equation (11.4–2): $S_{M1} = F_v S_1 = 1.500 \times 0.600 = 0.900$ g

Section 11.4.4 — Design Spectral Acceleration Parameters

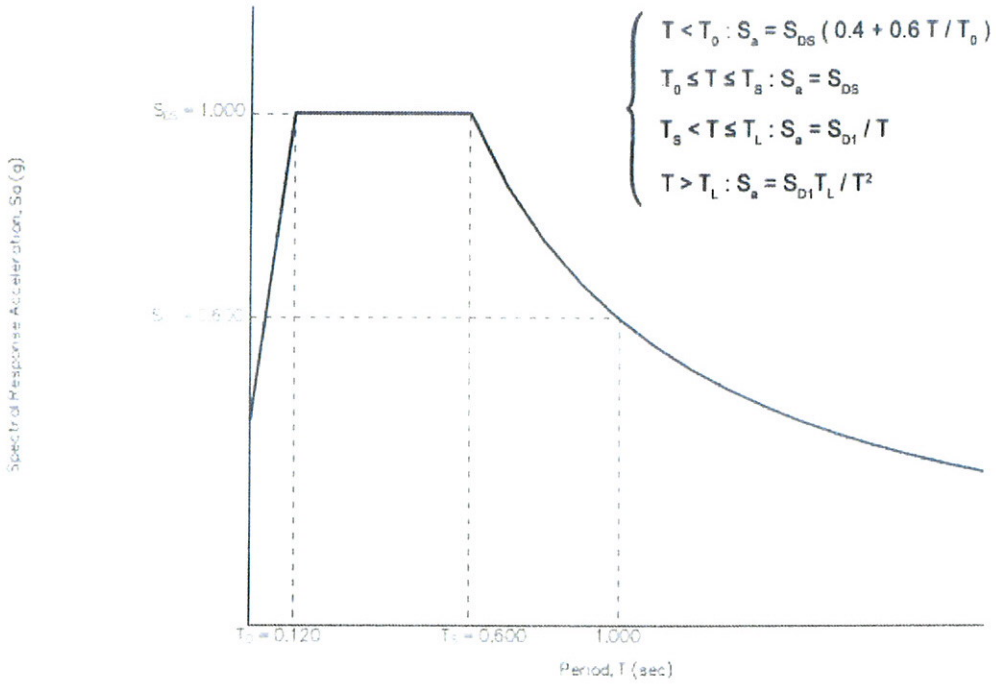
Equation (11.4–3): $S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3} \times 1.500 = 1.000$ g

Equation (11.4–4): $S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3} \times 0.900 = 0.600$ g

Section 11.4.5 — Design Response Spectrum

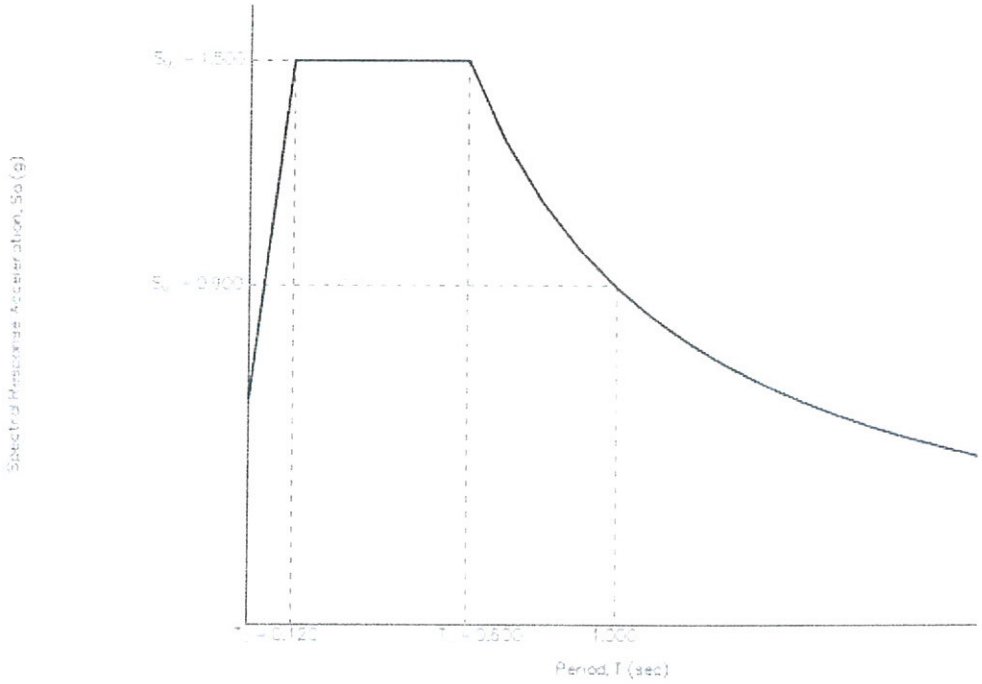
From Figure 22-12 [3] $T_L = 8$ seconds

Figure 11.4–1: Design Response Spectrum



Section 11.4.6 — Risk-Targeted Maximum Considered Earthquake (MCE_R) Response Spectrum

The MCE_R Response Spectrum is determined by multiplying the design response spectrum above by 1.5.



Section 11.8.3 — Additional Geotechnical Investigation Report Requirements for Seismic Design Categories D through F

From Figure 22-7 [4] PGA = 0.500

Equation (11.8-1): $PGA_M = F_{PGA}PGA = 1.000 \times 0.500 = 0.5$ g

Table 11.8-1: Site Coefficient F_{PGA}

Site Class	Mapped MCE Geometric Mean Peak Ground Acceleration, PGA				
	PGA ≤ 0.10	PGA = 0.20	PGA = 0.30	PGA = 0.40	PGA ≥ 0.50
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight-line interpolation for intermediate values of PGA

For Site Class = D and PGA = 0.500 g, F_{PGA} = 1.000

Section 21.2.1.1 — Method 1 (from Chapter 21 - Site-Specific Ground Motion Procedures for Seismic Design)

From Figure 22-17 [5] C_{RS} = 1.057

From Figure 22-18 [6] C_{R1} = 1.032

Section 11.6 — Seismic Design Category

Table 11.6-1 Seismic Design Category Based on Short Period Response Acceleration Parameter

VALUE OF S_{DS}	RISK CATEGORY		
	I or II	III	IV
$S_{DS} < 0.167g$	A	A	A
$0.167g \le S_{DS} < 0.33g$	B	B	C
$0.33g \le S_{DS} < 0.50g$	C	C	D
$0.50g \le S_{DS}$	D	D	D

For Risk Category = I and S_{DS} = 1.000 g, Seismic Design Category = D

Table 11.6-2 Seismic Design Category Based on 1-S Period Response Acceleration Parameter

VALUE OF S_{D1}	RISK CATEGORY		
	I or II	III	IV
$S_{D1} < 0.067g$	A	A	A
$0.067g \le S_{D1} < 0.133g$	B	B	C
$0.133g \le S_{D1} < 0.20g$	C	C	D
$0.20g \le S_{D1}$	D	D	D

For Risk Category = I and S_{D1} = 0.600 g, Seismic Design Category = D

Note: When S_1 is greater than or equal to 0.75g, the Seismic Design Category is **E** for buildings in Risk Categories I, II, and III, and **F** for those in Risk Category IV, irrespective of the above.

Seismic Design Category ≡ "the more severe design category in accordance with Table 11.6-1 or 11.6-2" = D

Note: See Section 11.6 for alternative approaches to calculating Seismic Design Category.

References

1. *Figure 22-1*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-1.pdf
2. *Figure 22-2*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-2.pdf
3. *Figure 22-12*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-12.pdf
4. *Figure 22-7*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-7.pdf
5. *Figure 22-17*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-17.pdf
6. *Figure 22-18*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-18.pdf

APPENDIX C

Standards of Grading

STANDARD GRADING AND EARTHWORK SPECIFICATIONS

These specifications present **South Shore Testing & Environmental**, standard recommendations for grading and earthwork.

No deviation from these specifications should be permitted unless specifically superseded in the geotechnical report of the project or by written communication signed by the Soils Consultant. Evaluations performed by the Soils Consultant during the course of grading may result in subsequent recommendations which could supersede these specifications or the recommendations of the geotechnical report.

1.0 GENERAL

1.1 The Soils Consultant is the Owner's or Developer's representative on the project. For the purpose of these specifications, observations by the Soils Consultant include observations by the Soils Engineer, Soils Engineer, Engineering Geologist, and others employed by and responsible to the Soils Consultant.

1.2 All clearing, site preparation, or earthwork performed on the project shall be conducted and directed by the Contractor under the allowance or the supervision of the Soils Consultant.

1.3 The Contractor should be responsible for the safety of the project and satisfactory completion of all grading. During grading, the Contractor shall remain accessible.

1.4 Prior to the commencement of grading, the Soils Consultant shall be employed for the purpose of providing field, laboratory, and office services for conformance with the recommendations of the geotechnical report and these specifications. It will be necessary that the Soils Consultant provide adequate testing and observations so that he may provide an opinion as to determine that the work was accomplished as specified. It shall be the responsibility of the Contractor to assist the Soils Consultant and keep him apprised of work schedules and changes so that he may schedule his personnel accordingly.

1.5 It shall be the sole responsibility of the Contractor to provide adequate equipment and methods to accomplish the work in accordance with applicable grading codes, agency ordinances, these specifications, and the approved grading plans. If, in the opinion of the Soils Consultant, unsatisfactory conditions, such as questionable soil, poor moisture condition, inadequate compaction, adverse weather, etc, are resulting in a quality of work less then required in these specifications, the Soils Consultant will be empowered to reject the work and recommend that construction be stopped until the conditions are rectified.

1.6 It is the Contractor's responsibility to provides safe access to the Soils Consultant for testing and/or grading observation purposes. This may require the excavation of the test pits and/or the relocation of grading equipment.

1.7 A final report shall be issued by the Soils Consultant attesting to the Contractor's conformance with these specifications.

2.0 SITE PREPARTION

2.1 All vegetation and deleterious material shall be disposed of off-site. This removal shall be observed by the Soils Consultant and concluded prior to fill placement.

2.2 Soil, Alluvium or bedrock materials determined by the Soils Consultant as being unsuitable for placement in compacted fills shall be removed from the site or used in open areas as determined by the Soils Consultant. Any material incorporated as a part of a compacted fill must be approved by the Soils Consultant prior to fill placement.

2.3 After the ground surface to receive fill has been cleared, it shall be scarified, disced and/or bladed by the Contractor until it is uniform and free from ruts, hollows, hummocks, or other uneven features which may prevent uniform compaction.

The scarified ground surface shall then be brought to optimum moisture, mixed as required, and compacted as specified. If the scarified zone is greater than twelve inches in depth, the excess shall be removed and placed in lifts not to exceed six inches or less.

Prior to placing fill, the ground surface to receive fill shall be observed, tested, and approved by the soils consultant.

2.4 Any underground structures or cavities such as cesspools, cisterns, mining shafts, tunnels, septic tanks, wells, pipe lines, or others are to be removed or treated in a manner prescribed by the Soils Consultant.

2.5 In cut-fill transitions lots and where cut lots are partially in soil, colluvium or unweathered bedrock materials, in order to provide uniform bearing conditions, the bedrock portion of the lot extending a minimum of 5 feet outside of building lines shall be over excavated a minimum of 3 feet and replaced with compacted fill. Greater over excavation could be required as determined by Soils Consultant. Typical details are attached.

3.0 COMPACTED FILLS

3.1 Material to be placed as fill shall be free of organic matter and other deleterious substances, and shall be approved by the Soils Consultant. Soils of poor gradation, expansion, or strength characteristics shall be placed in areas designated by Soils Consultant or shall be mixed with other soils to serve as satisfactory fill material, as directed by the Soils Consultant.

3.2 Rock fragments less than six inches in diameter may be utilized in the fill, provided

- They are not placed or nested in concentrated pockets
- There is sufficient amount of approved soil to surround the rocks
- The distribution of rocks is supervised by the Soils Consultant

3.3 Rocks greater than twelve inches in diameter shall be taken off-site, or placed in accordance with the recommendations of the Soils Consultant, areas designated as suitable for rock disposal (A typical detail for Rock Disposal is attached.)

3.4 Material that is spongy, subject to decay, or otherwise considered unsuitable shall not be used in the compacted fil.

3.5 Representative samples of materials to be utilized as compacted fill shall be analyzed by the laboratory of the Soils Consultant to determine the physical properties. If any material other than that previously tested is encountered during grading, the appropriate analysis of this material shall be conducted by the Soils Consultant before being approved as fill material.

3.6 Material used in the compacting process shall be evenly spread, watered, processed, and compacted in thin lifts not to exceed six inches in thickness to obtain a uniformly dense layer. The fill shall be placed and compacted on a horizontal plane, unless otherwise approved by the Soils Consultant.

3.7 If the moisture content or relative compaction varies from that required by the Soils Consultant, the Contractor shall rework the fill until it has been approved by the Soils Consultant.

3.8 Each layer shall be compacted to at least 90 percent of the maximum density in compliance with the testing method specified by the controlling government agency or ASTM 1557-70, whichever applies.

If compaction to a lesser percentage is authorized by the controlling governmental agency because of a specific land use or expansive soil conditions the area to receive fill compacted to less than 90 percent shall either be delineated on the grading plan and/or appropriate reference made to the area in the geotechnical report.

3.9 All fills shall be keyed and benched through all topsoil, colluvium, alluvium, or creep material, into sound bedrock, or firm material where the slope receiving fill exceeds a ratio of five horizontal to one vertical or in accordance with the recommendations of the Soils Consultant.

3.10 The key for side hill fills shall be a minimum width of 15 feet within bedrock or firm materials, unless otherwise specified in the geotechnical report, (see detail attached.)

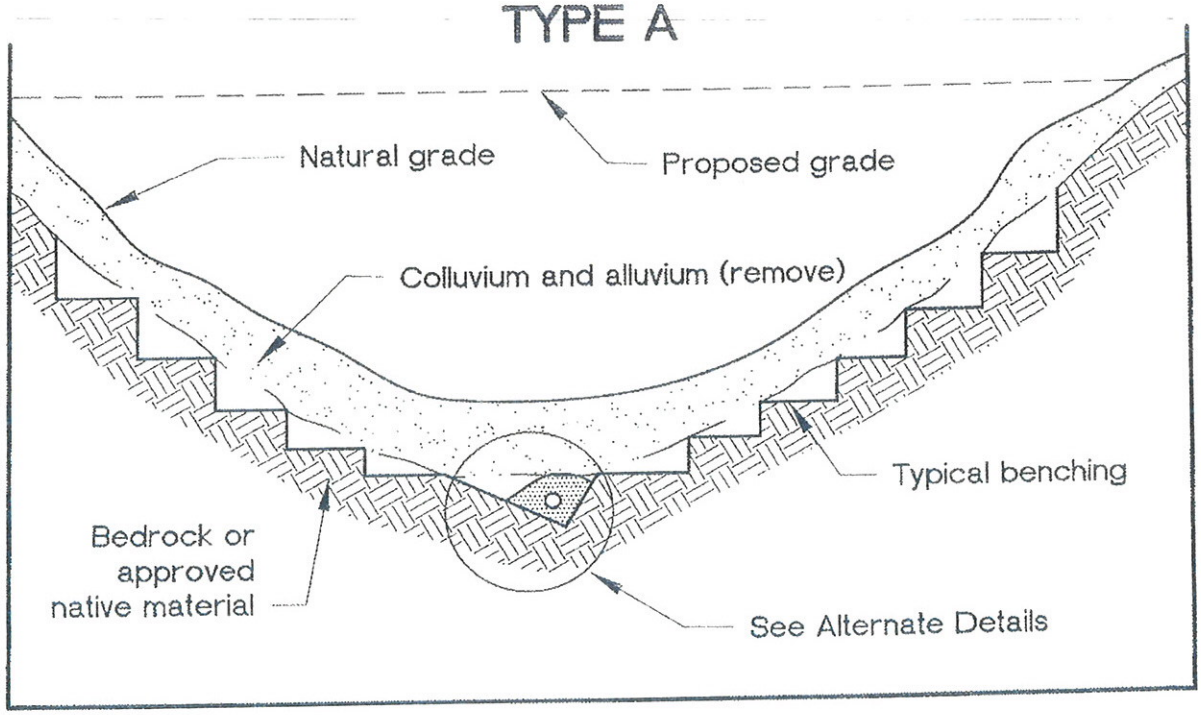
3.11 Sub drainage devices shall be constructed in compliance with the ordinances of the controlling governmental agency, or with the recommendations of the Soils Consultant. (Typical Canyon Subdrain details are attached.)

3.12 The contractor will be required to obtain a minimum relative compaction of at least 90 percent out to the finish slope face of fill slopes, buttresses, and stabilization fills. This may be achieved by either over building the slope and cutting back to the compacted core, or by direct compaction of the slope face with suitable equipment, or by any other procedure, which produces the required compaction approved by the Soils Consultant.

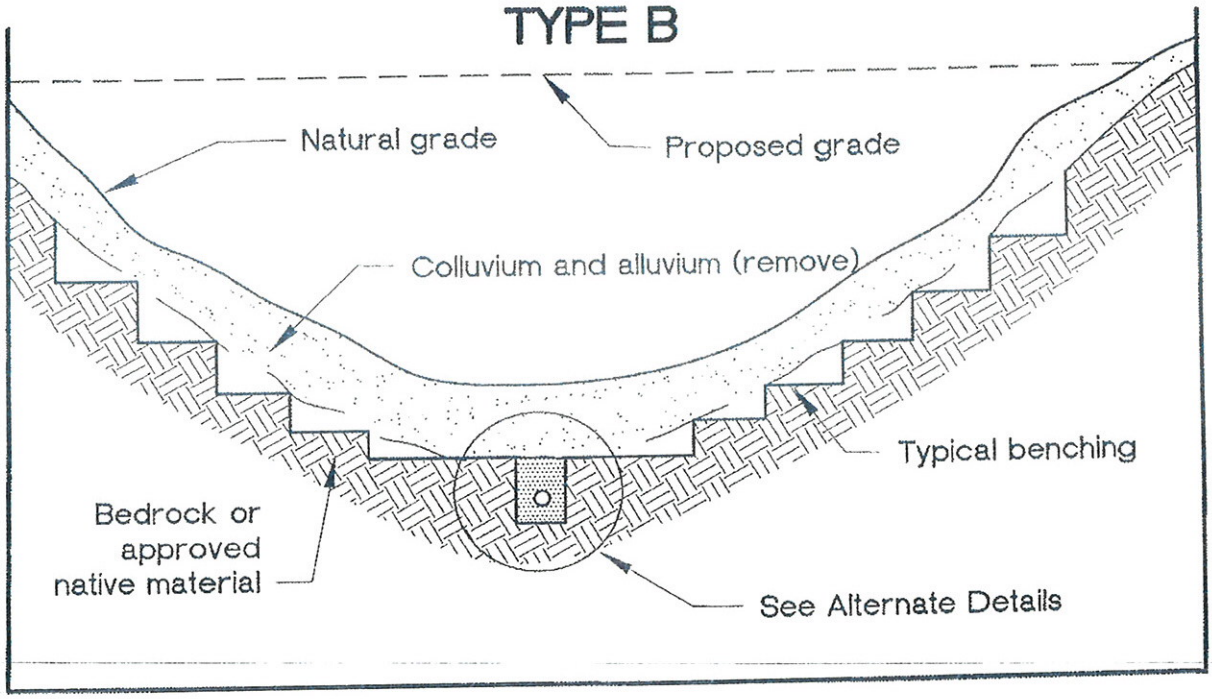
3.13 All fill slopes should be planted or protected from erosion by other methods specified in the Soils report.

3.14 Fill-over-cut slopes shall be properly keyed through topsoil, colluvium or creep material into rock or firm materials and the transition shall be stripped of all soils prior to placing fill (see attached detail.)

TYPE A

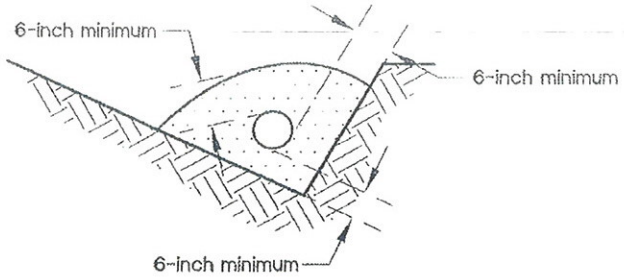


TYPE B

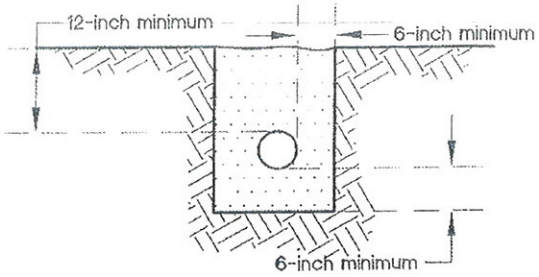


Selection of alternate subdrain details, location, and extent of subdrains should be evaluated by the geotechnical consultant during grading.

SOUTH SHORE TESTING	CANYON SUBDRAIN DETAIL	Plate 1



A-1



B-1

Filter material: Minimum volume of 9 cubic feet per lineal foot of pipe.

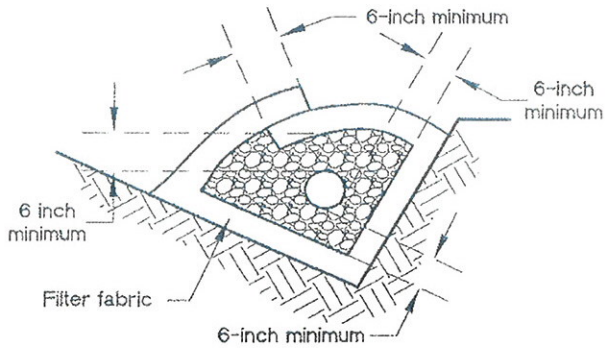
Perforated pipe: 6-inch-diameter ABS or PVC pipe or approved substitute with minimum 8 perforations (¼-inch diameter) per lineal foot in bottom half of pipe (ASTM D-2751, SDR-35, or ASTM D-1527, Schd. 40).

For continuous run in excess of 500 feet, use 8-inch-diameter pipe (ASTM D-3034, SDR-35, or ASTM D-1785, Schd. 40).

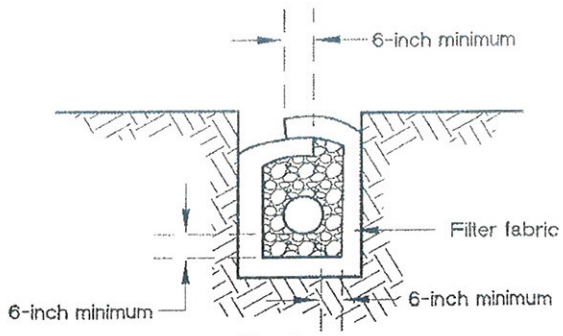
FILTER MATERIAL

Sieve Size	Percent Passing
1 inch	100
¾ inch	90-100
⅜ inch	40-100
No. 4	25-40
No. 8	18-33
No. 30	5-15
No. 50	0-7
No. 200	0-3

ALTERNATE 1: PERFORATED PIPE AND FILTER MATERIAL



A-2

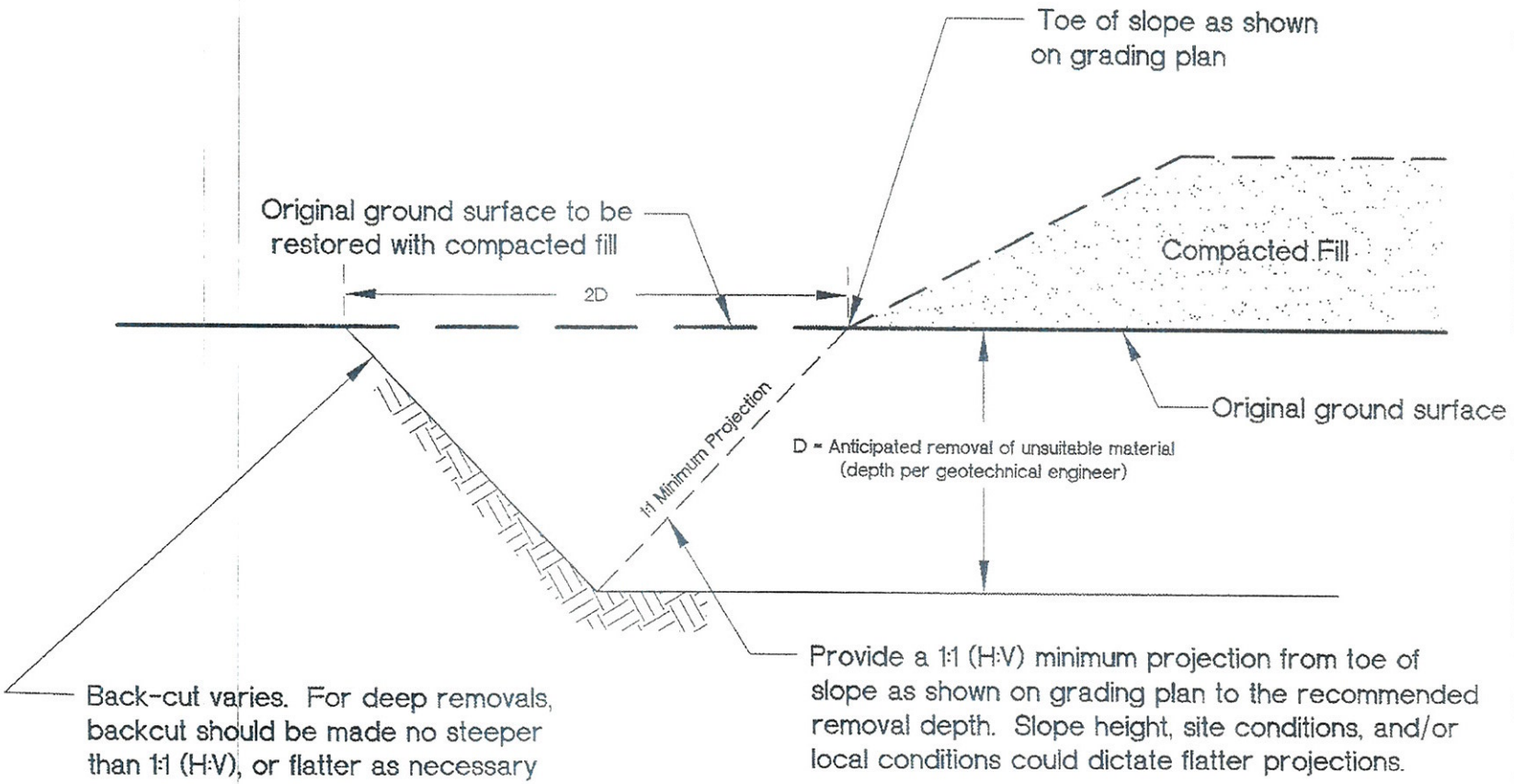


B-2

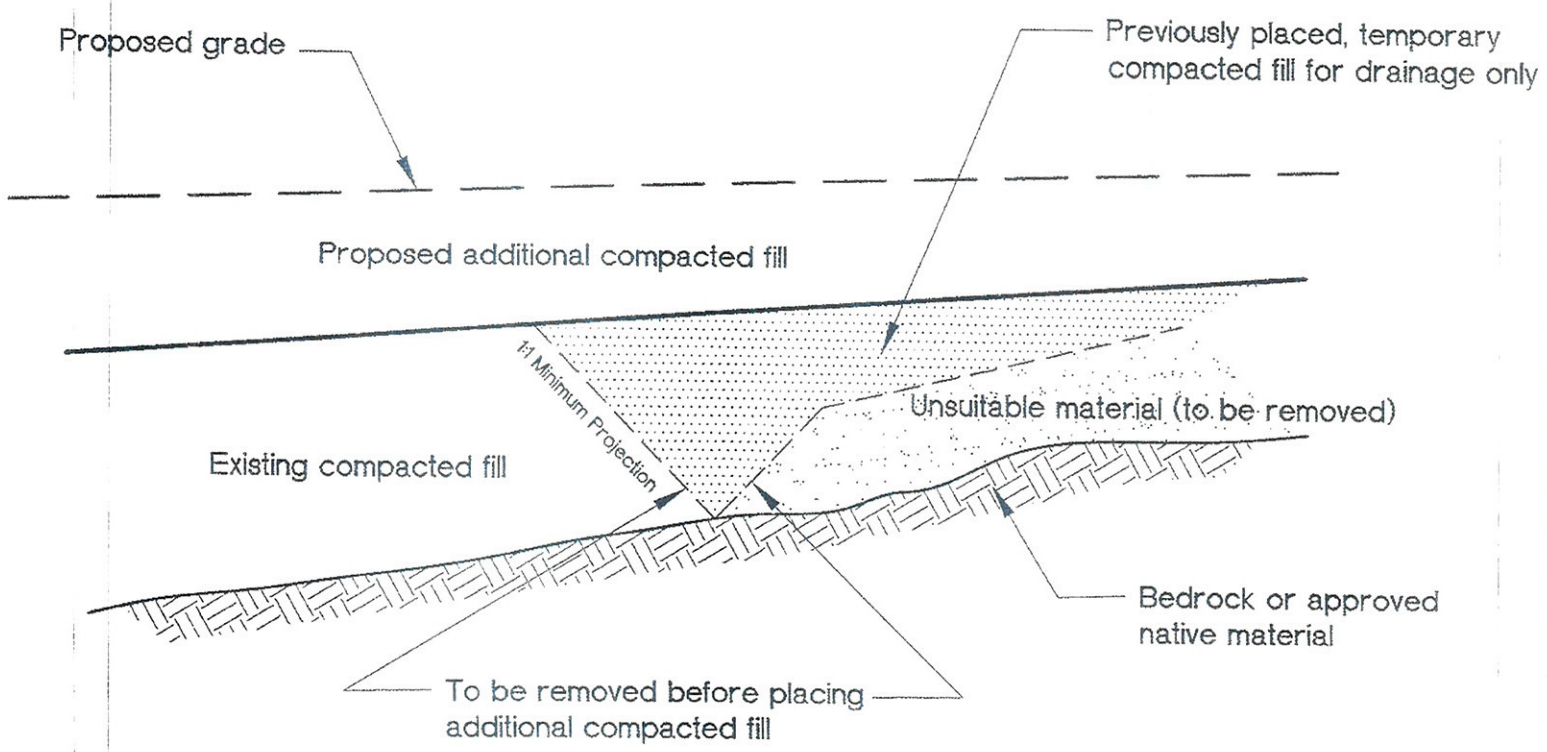
Gravel Material: 9 cubic feet per lineal foot.
Perforated Pipe: See Alternate 1
Gravel: Clean ¾-inch rock or approved substitute.
Filter Fabric: Mirafi 140 or approved substitute.

ALTERNATE 2: PERFORATED PIPE, GRAVEL, AND FILTER FABRIC

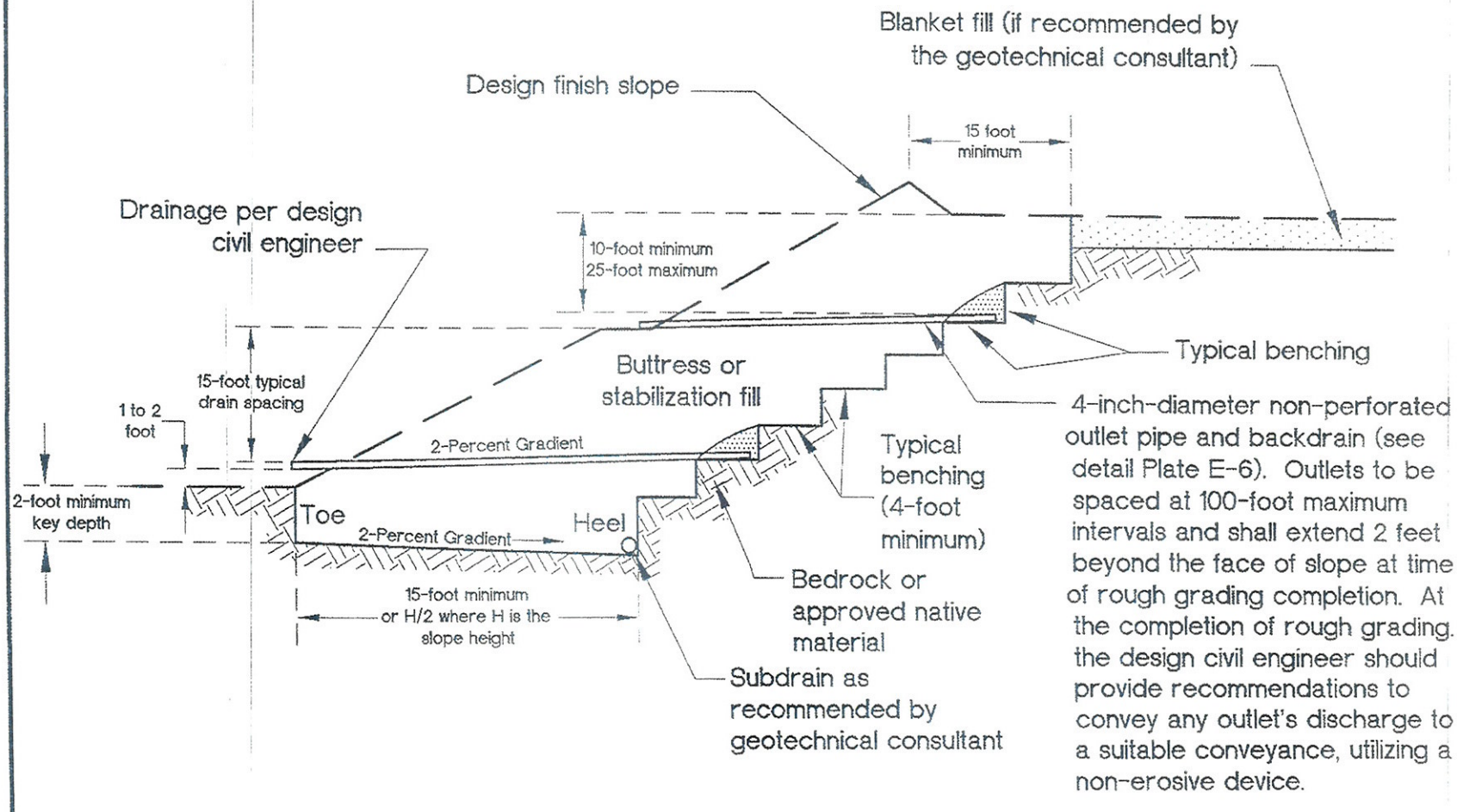
SOUTH SHORE TESTING	CANYON SUBDRAIN ALTERNATE DETAILS	Plate 2



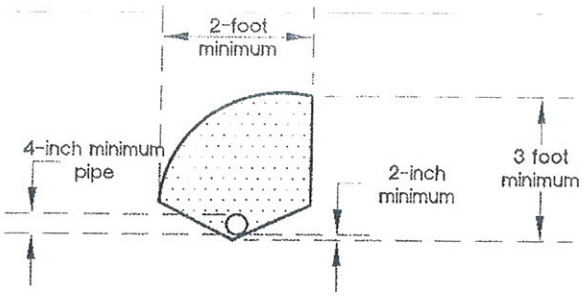
SOUTH SHORE TESTING	FILL SLOPE TOEING OUT ON FLAT ALLUVIATED CANYON DETAIL	PLATE 3

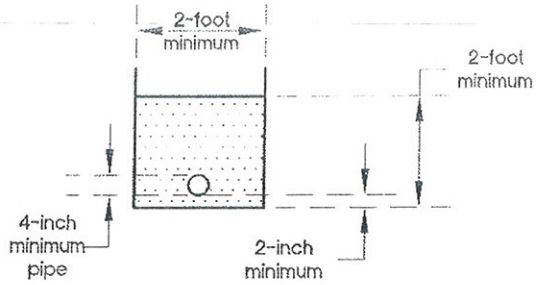


SOUTH SHORE TESTING	REMOVAL ADJACENT TO EXISTING FILL ADJOINING CANYON FILL DETAIL	Plate 4



SOUTH SHORE TESTING	TYPICAL STABILIZATION / BUTTRESS FILL DETAIL	Plate 5





Filter Material: Minimum of 5 cubic feet per lineal foot of pipe or 4 cubic feet per lineal feet 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 inches in all joints.

Minimum 4-Inch-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. Must be installed with perforations down at bottom of pipe. Provide cap at upstream end of pipe. Slope at 2 percent to outlet pipe. Outlet pipe to be connected to subdrain pipe with tee or elbow.

Notes:
1. Trench for outlet pipes to be backfilled and compacted with onsite soil.
2. Backdrains and lateral drains shall be located at elevation of every bench drain. First drain located at elevation just above lower lot grade. Additional drains may be required at the discretion of the geotechnical consultant.

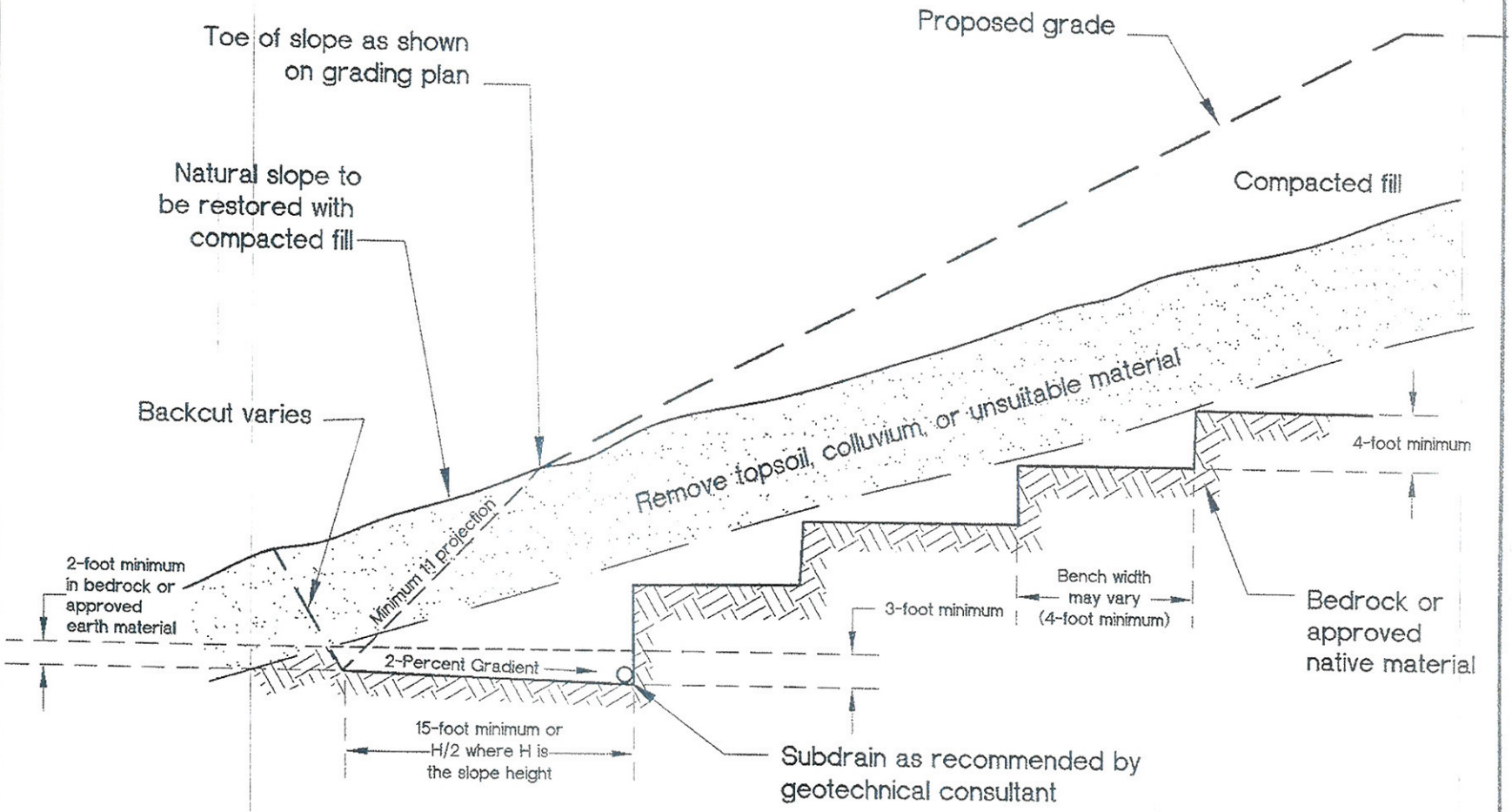
Filter Material shall be of the following specification or an approved equivalent.

Sieve Size	Percent Passing
1 inch	100
¾ inch	90-100
⅜ inch	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.

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

SOUTH SHORE TESTING	TYPICAL BUTTRESS SUBDRAIN DETAIL	Plate 6

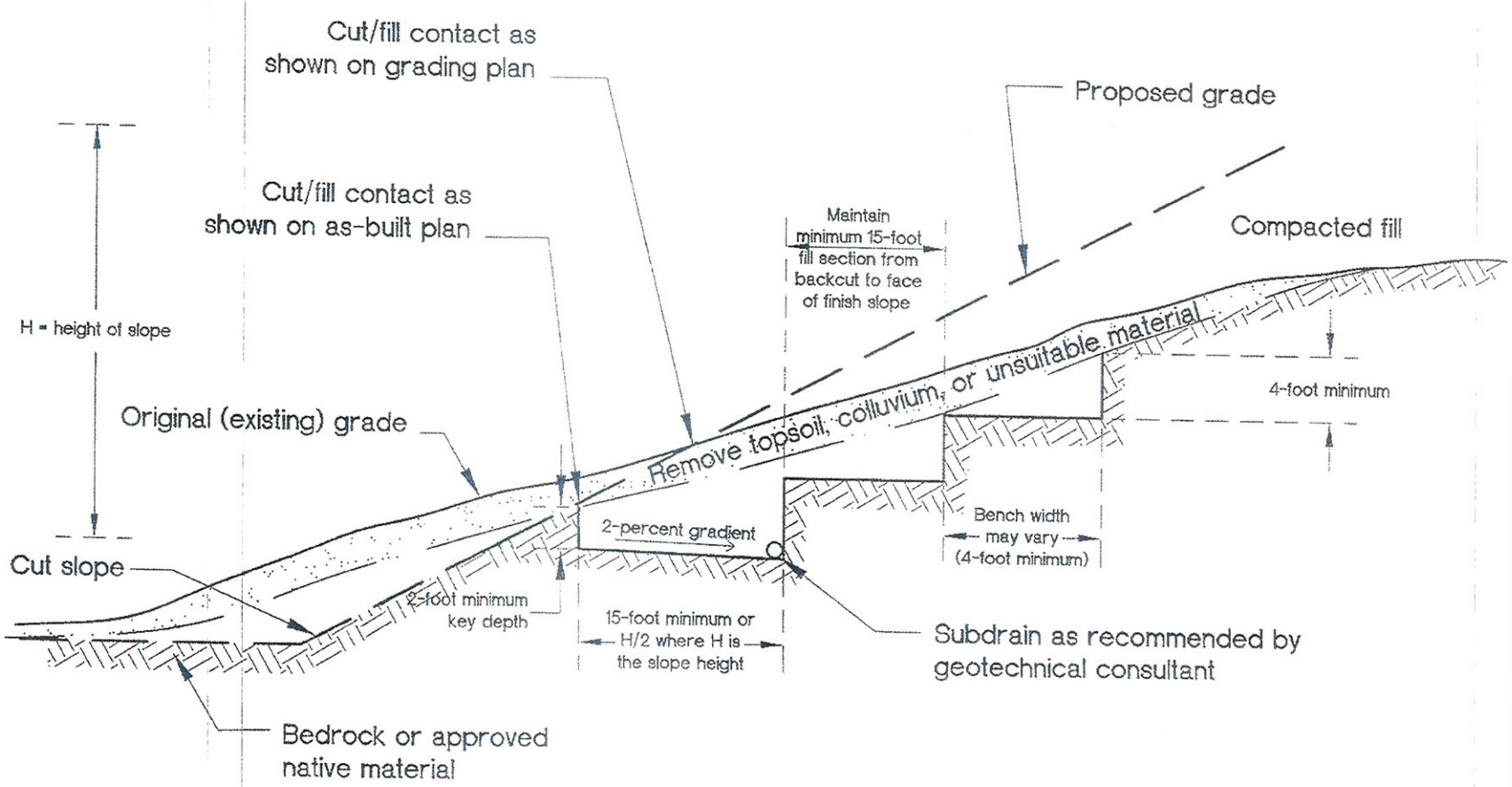


NOTES:

1. Where the natural slope approaches or exceeds the design slope ratio, special recommendations would be provided by the geotechnical consultant.

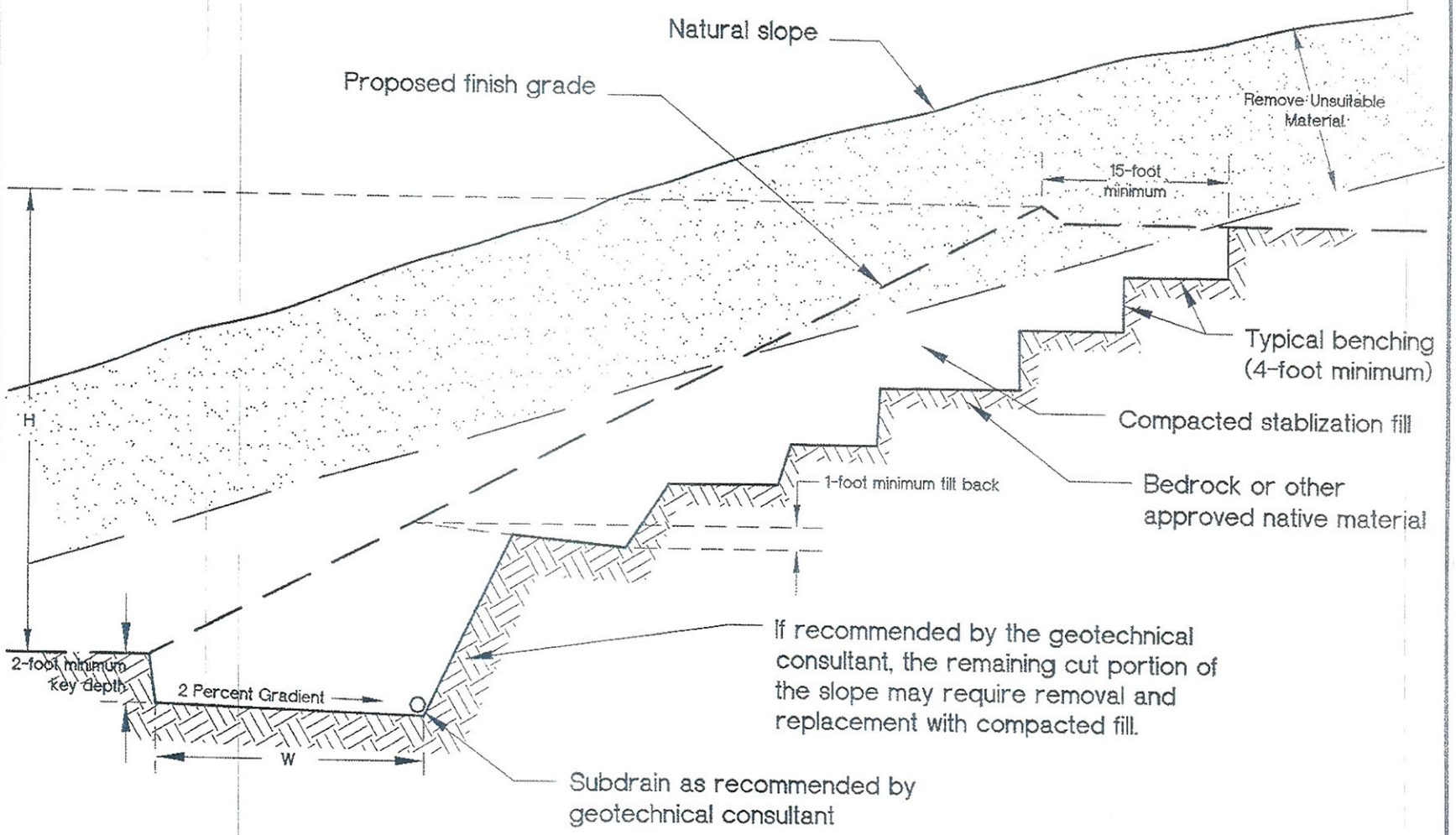
2. The need for and disposition of drains should be evaluated by the geotechnical consultant, based upon exposed conditions.

SOUTH SHORE TESTING	FILL OVER NATURAL (SIDEHILL FILL) DETAIL	Plate 7



NOTE: The cut portion of the slope should be excavated and evaluated by the geotechnical consultant prior to construction of the fill portion.

SOUTH SHORE TESTING	FILL OVER CUT DETAIL	Plate 8

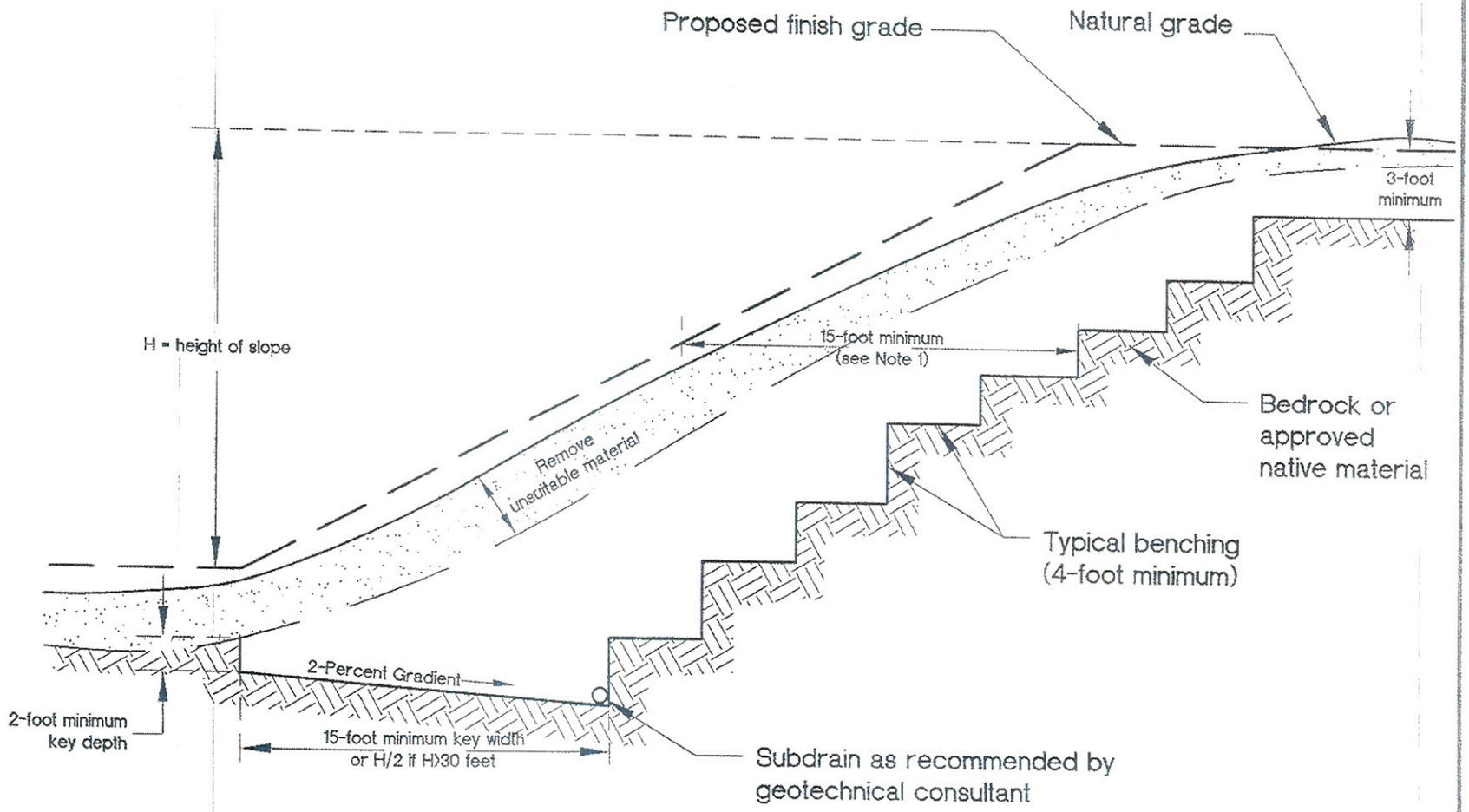


NOTES:

1. Subdrains may be required as specified by the geotechnical consultant.

2 W shall be equipment width (15 feet) for slope heights less than 25 feet. For slopes greater than 25 feet, W shall be evaluated by the geotechnical consultant. At no time, shall W be less than H/2, where H is the height of the slope.

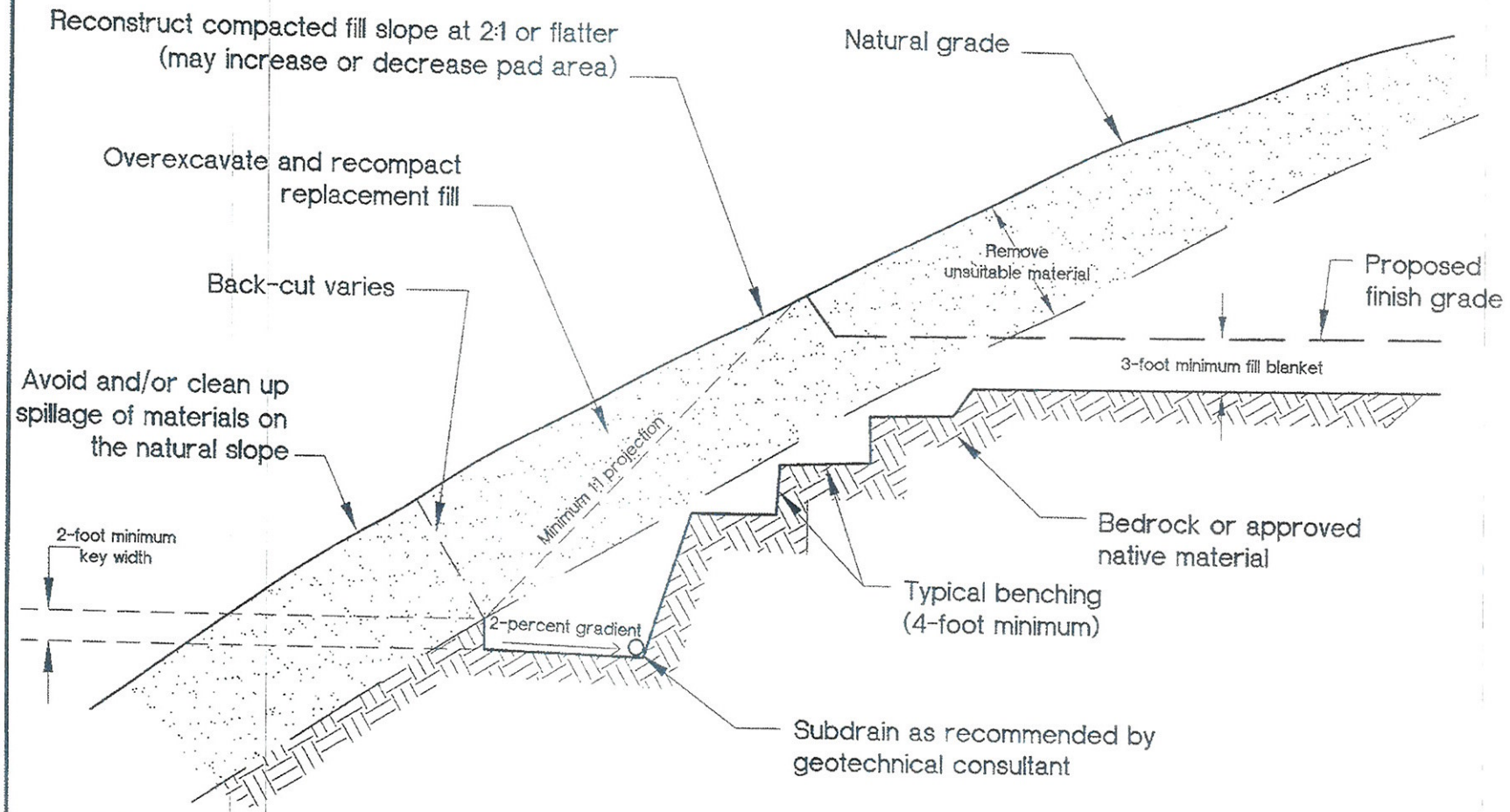
SOUTH SHORE TESTING	STABLIZATION FILL FOR UNSTABLE MATERIAL EXPOSED IN CUT SLOPE DETAIL	Plate 9



NOTES:

1. 15-foot minimum to be maintained from proposed finish slope face to backcut.
2. The need and disposition of drains will be evaluated by the geotechnical consultant based on field conditions.
3. Pad overexcavation and recompaction should be performed if evaluated to be necessary by the geotechnical consultant.

SOUTH SHORE TESTING	SKIN FILL OF NATURAL GROUND DETAIL	Plate 10

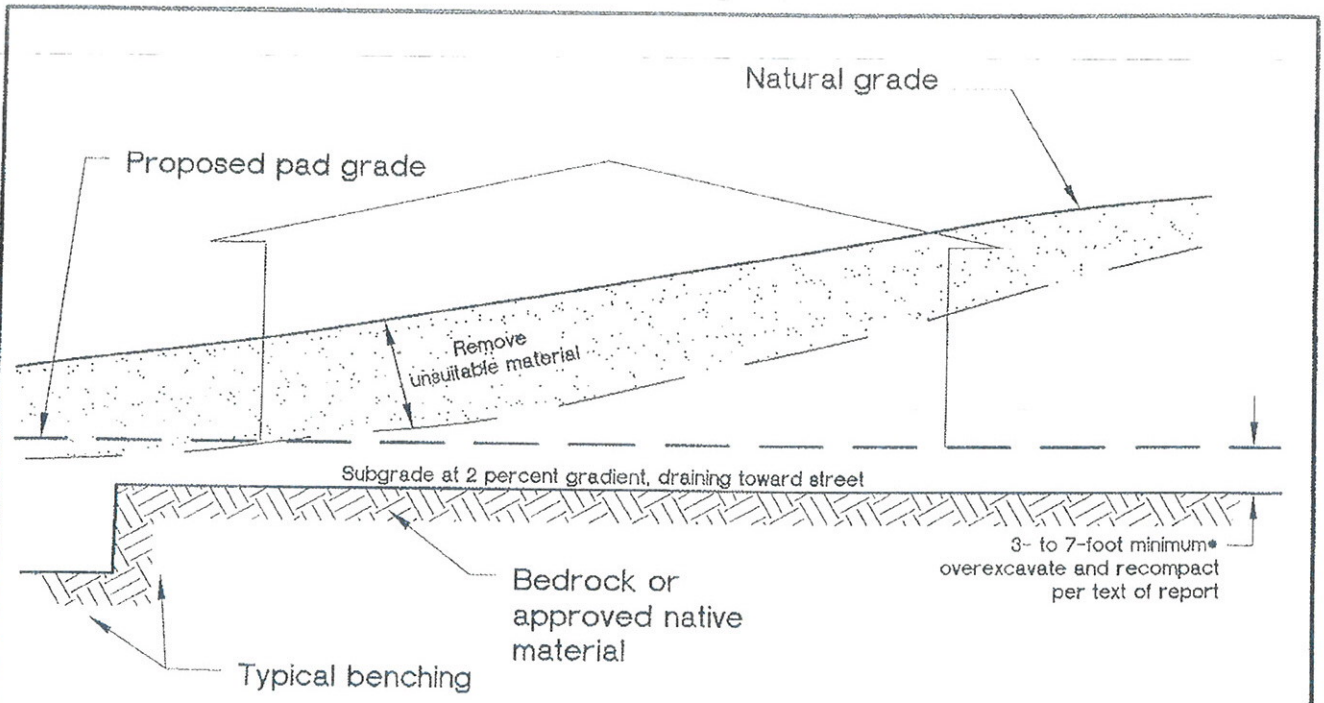


NOTES:

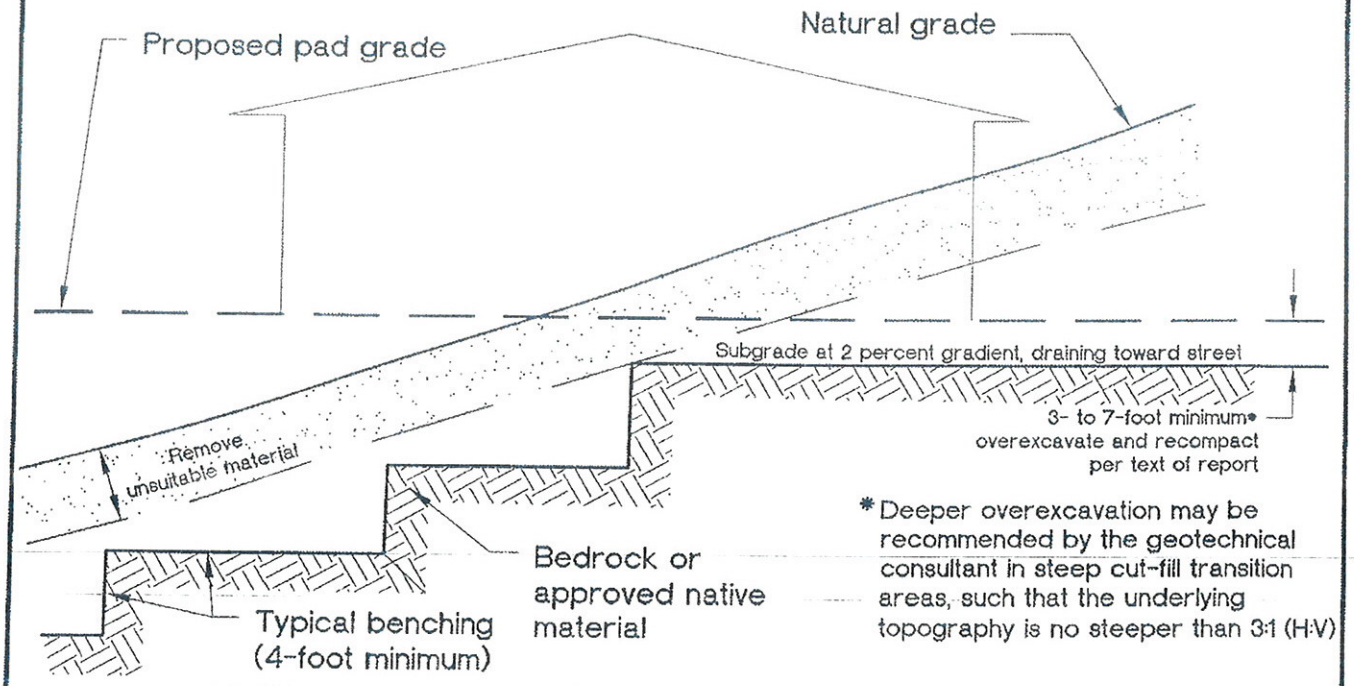
1. Subdrain and key width requirements will be evaluated based on exposed subsurface conditions and thickness of overburden.

2. Pad overexcavation and recompaction should be performed if evaluated necessary by the geotechnical consultant.

SOUTH SHORE TESTING	DAYLIGHT CUT LOT DETAIL	Plate 11



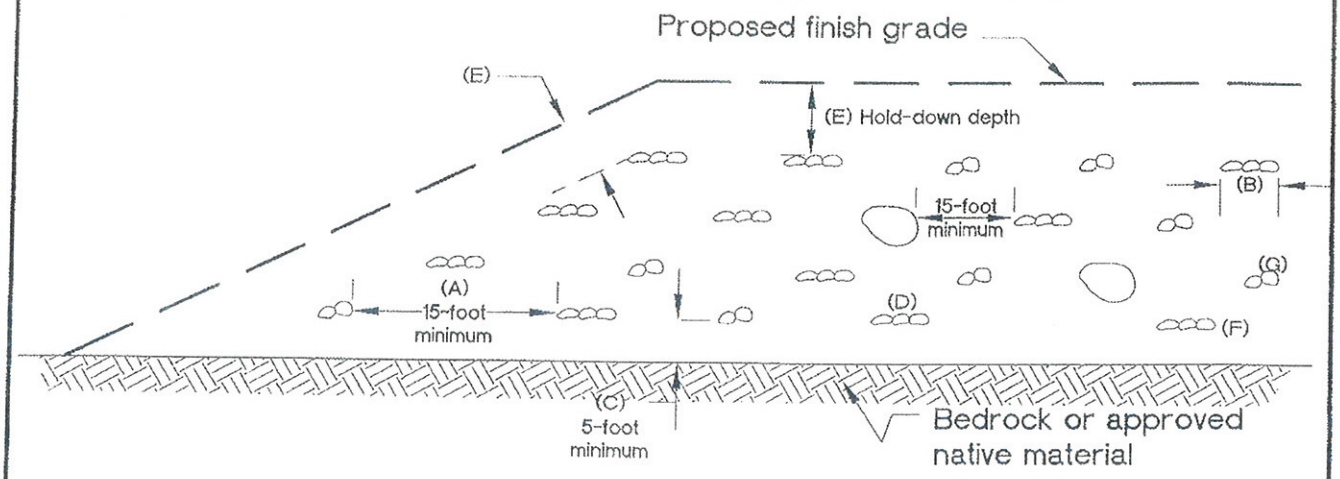
CUT LOT OR MATERIAL-TYPE TRANSITION



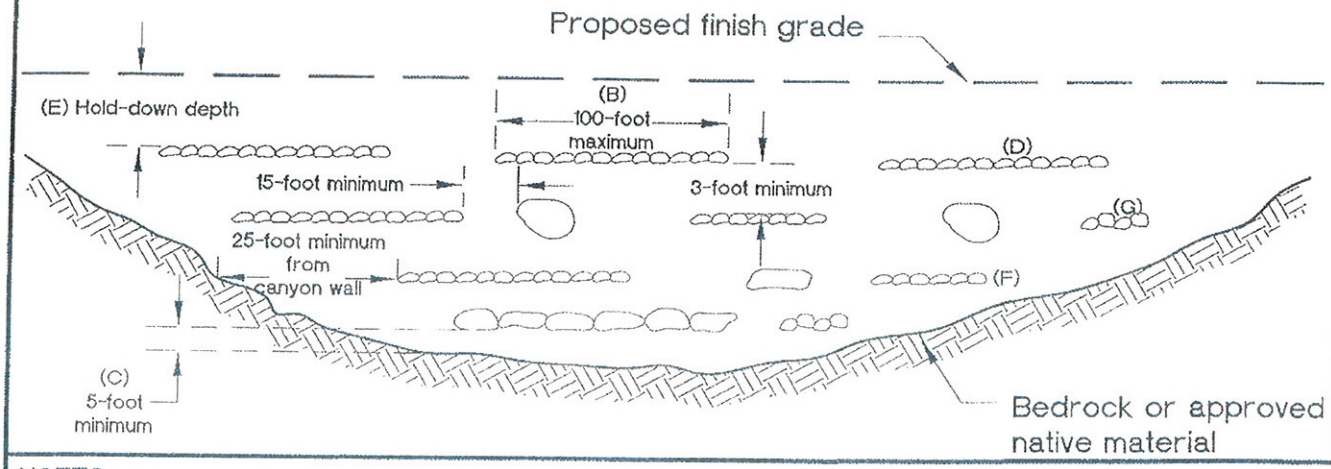
CUT-FILL LOT (DAYLIGHT TRANSITION)

SOUTH SHORE TESTING	TRANSITION LOT DETAILS	Plate 12

VIEW NORMAL TO SLOPE FACE



VIEW PARALLEL TO SLOPE FACE



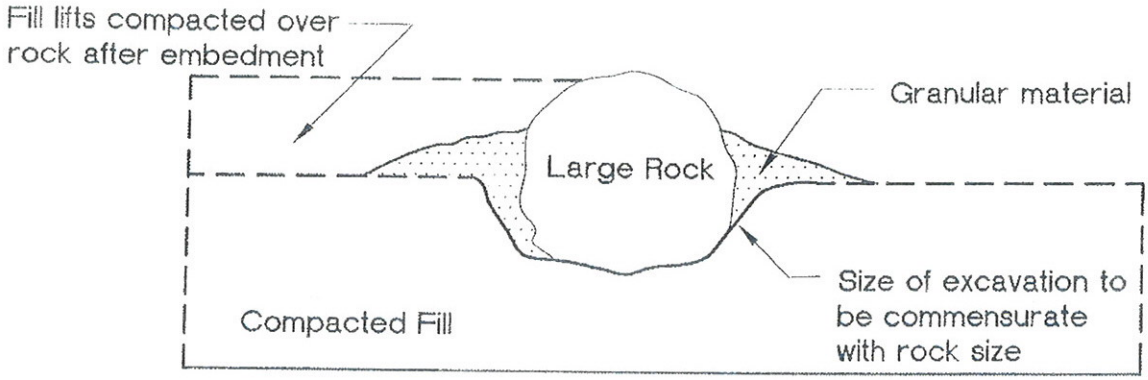
NOTES:

A. One equipment width or a minimum of 15 feet between rows (or windrows).
B. Height and width may vary depending on rock size and type of equipment. Length of windrow shall be no greater than 100 feet.
C. If approved by the geotechnical consultant, windrows may be placed directly on competent material or bedrock, provided adequate space is available for compaction.
D. Orientation of windrows may vary but should be as recommended by the geotechnical engineer and/or engineering geologist. Staggering of windrows is not necessary unless recommended.
E. Clear area for utility trenches, foundations, and swimming pools; Hold-down depth as specified in text of report, subject to governing agency approval.
F. All fill over and around rock windrow shall be compacted to at least 90 percent relative compaction or as recommended.
G. After fill between windrows is placed and compacted, with the lift of fill covering windrow, windrow should be proof rolled with a D-9 dozer or equivalent.

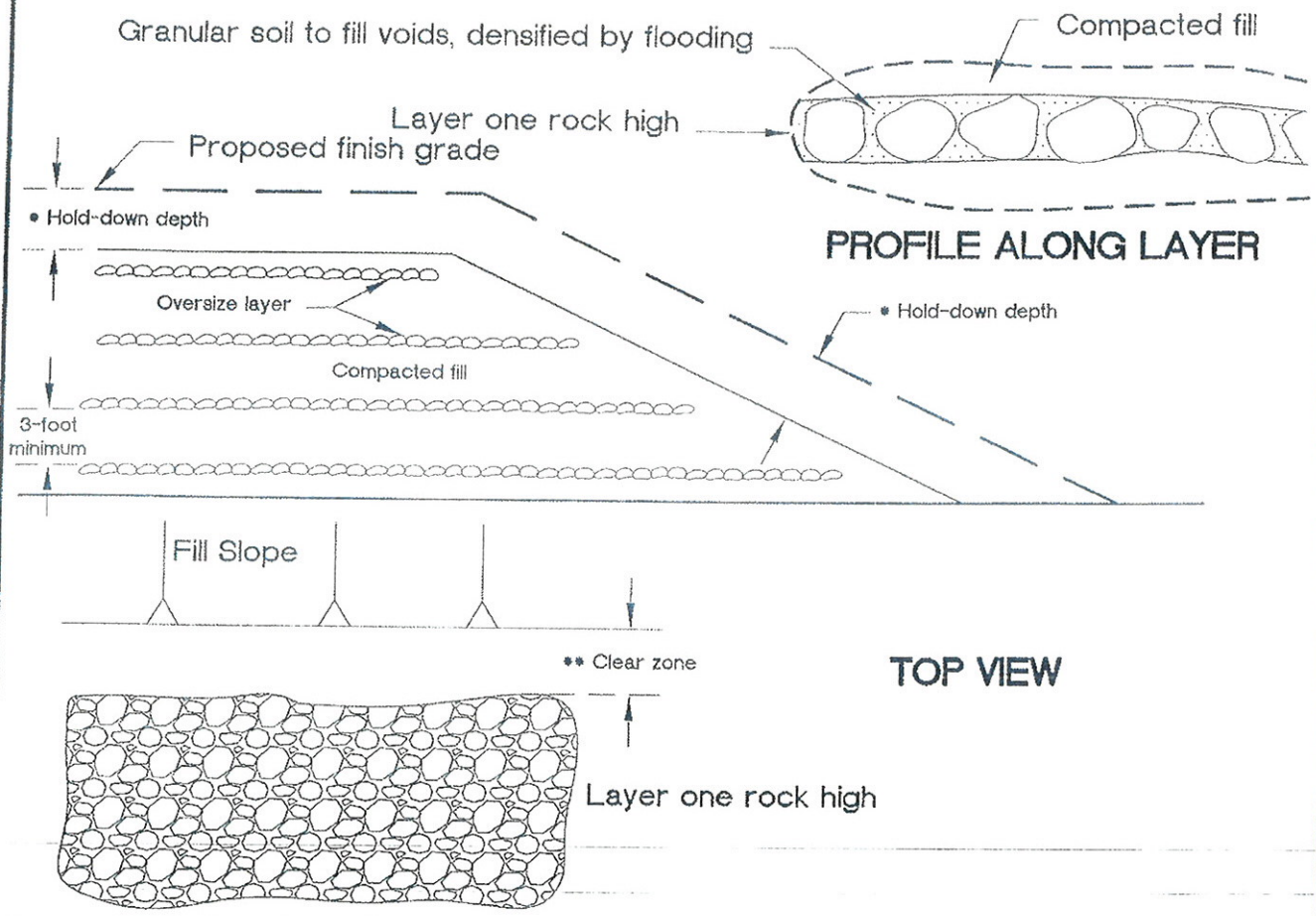
VIEWS ARE DIAGRAMMATIC ONLY AND MAY BE SUPERSEDED BY REPORT RECOMMENDATIONS OR CODE
ROCK SHOULD NOT TOUCH AND VOIDS SHOULD BE COMPLETELY FILLED

SOUTH SHORE TESTING	OVERSIZE ROCK DISPOSAL DETAIL	Plate 13

ROCK DISPOSAL PITS



ROCK DISPOSAL LAYERS



* Hold-down depth or below lowest utility as specified in text of report, subject to governing agency approval.

** Clear zone for utility trenches, foundations, and swimming pools, as specified in text of report.

VIEWS ARE DIAGRAMMATIC ONLY AND MAY BE SUPERSEDED BY REPORT RECOMMENDATIONS OR CODE

ROCK SHOULD NOT TOUCH AND VOIDS SHOULD BE COMPLETELY FILLED IN

SOUTH SHORE TESTING	ROCK DISPOSAL DETAIL	Plate 14