

PRELIMINARY GEOTECHNICAL INVESTIGATION
TENTATIVE TRACT 20793
67 LOT RESIDENTIAL DEVELOPMENT
(LINDEN AVENUE PROJECT), CITY OF RIALTO
SAN BERNARDINO COUNTY, CALIFORNIA

FOR

LYTLE DEVELOPMENT COMPANY
285 W. RIALTO AVENUE, SUITE B
RIALTO, CALIFORNIA 92376

W.O. 8972-A-IE SEPTEMBER 12, 2025
REVISED SEPTEMBER 15, 2025



Geotechnical • Geologic • Coastal • Environmental
18451 Collier Avenue, Suite A, Lake Elsinore, California 92530
TEL: (951) 471-0700 - FAX: (951) 471-0702
www.geosoils.com

September 12, 2025
Revised September 15, 2025

W.O. 8972-A-IE

Lytle Development Company
285 W. Rialto Avenue, Suite B
Rialto, California 92376

Attention: Mr. Kevin Lynch

Subject: Preliminary Geotechnical Investigation, Tentative Tract 20793, 67 Lot Residential Development (Linden Avenue Project), City of Rialto, San Bernardino County, California

Dear Mr. Lynch:

In accordance with your request and authorization, GeoSoils, Inc. (GSI) is presenting the results of our preliminary geotechnical investigation of Tentative Tract 20793 (Linden project) in the City of Rialto, San Bernardino County, California. The purpose of this study was to evaluate the onsite soils and geologic conditions and their effects on the proposed residential development, from a geotechnical viewpoint, and to provide code compliant conclusions and recommendations for future residential development of the property. A secondary function of this study was to provide feasibility level infiltration testing within the proposed area of the stormwater basin onsite, for use in Best Management Practice (BMP) design by the civil engineer of record for the project.

EXECUTIVE SUMMARY

Based on our review of the Tentative Tract Map by Encompass Associates, Inc., (EAI, 2025), the previous geotechnical studies by GSI on other nearby properties (see Appendix A), our recent field exploration, laboratory testing, geologic, and geotechnical engineering analysis, the residential development within the proposed project site appears feasible from a geotechnical viewpoint, provided the recommendations presented in the text of this report are appropriately implemented. The primary developmental considerations are summarized below:

- Our review indicates no known active faults are crossing the site, and the site is not located within an Alquist-Priolo Earthquake Fault Zone (California Department of Conservation, California Geological Survey [CGS], 2018). In addition, our review of the Geologic Hazard Overlay maps in the San Bernardino County Land Use Plan (County of San Bernardino, 2010), the site is not located within an area designated as having a potential for liquefaction. Our evaluation and general liquefaction screening process (pursuant to Special Publication 117 [SP117A, CGS, 2008]) indicates that the potential for liquefaction and associated adverse effects within the site is considered low.

- The earth materials encountered during our subsurface investigation included undocumented artificial fill, placed over a majority of the site, colluvium/topsoil, and Holocene-age (Quaternary) young alluvial-fan deposits (Morton and Matti, 2001). Removal of all undocumented artificial fill, colluvium/topsoil and near surface weathered young alluvial-fan deposits will be necessary prior to fill placement, in areas proposed for settlement-sensitive improvements. Approximate depths of removals are outlined in the conclusions and recommendations section of this report. For preliminary planning purposes, these depths are estimated to be on the order of 7 to 11 feet across the majority of the site, or potentially more where infiltration-pond grading was previously performed on the northern portion of the site. Variations from these thicknesses should be anticipated. Actual depths of removals will be evaluated in the field during grading by the geotechnical consultant.
- Due to the nature of the earth materials encountered onsite, localized oversized rock materials (i.e., greater than 12 inches in diameter) are anticipated be generated during rough grading, which will require specialized placement techniques. Per the San Bernardino County guidelines, oversized materials 12 inches or greater, should not be placed within 10 feet of finish pad grades. This may necessitate the need for scraper or dozer pits or rockfill blankets if the oversized materials (≥ 12 inches) cannot be reduced in size during grading. Oversized rock materials should be placed in accordance with the Rock Placement Guidelines section of this report. Proper disclosure to all interested and affected parties, and any homeowners association, will be required regarding the potential for difficult excavation due to oversize materials and their hold-down distances from finish grade, etc. Local utility entities may also require a more stringent backfill size restriction (i.e., less than 6 inches or so in maximum size) which may require some screening during underground utility backfill placement.
- Percolation testing was performed to evaluate the feasibility of the site in regards to the design and installation of stormwater BMP systems. Further discussions of the test procedures used, the onsite USDA soil groups, general infiltration basin siting requirements and limitations, along with the converted infiltration rates obtained, are presented in following sections of this report.
- The recommendations presented in this report should be incorporated into the design and construction considerations of the project.

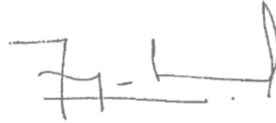
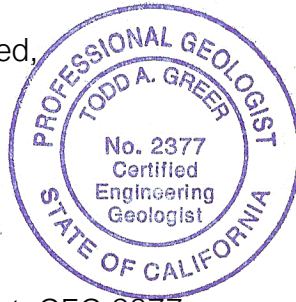
The opportunity to be of service is sincerely appreciated. If you should have any questions, please do not hesitate to contact our office.

Respectfully submitted,

GeoSoils, Inc.



Todd A. Greer
Engineering Geologist, CEG 2377



Jorge F. Meneses, PhD, PE, GE, BC. GE, F.ASCE
Geotechnical Engineer, GE 3041



MAM/TAG/JFM/li

Distribution: (1) Addressee (via pdf)

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ATTACHMENTS:

Appendix A - References	Rear of Text
Appendix B - Test Pit Logs	Rear of Text
Appendix C - Field Percolation Logs.....	Rear of Text

Appendix E - Laboratory Test Results.....	Rear of Text
Appendix D – Seismic Ground Motion Values.....	Rear of Text
Appendix F - General Earthwork and Grading Guidelines	Rear of Text
Plate 1 - Geotechnical Map.....	Rear of Text

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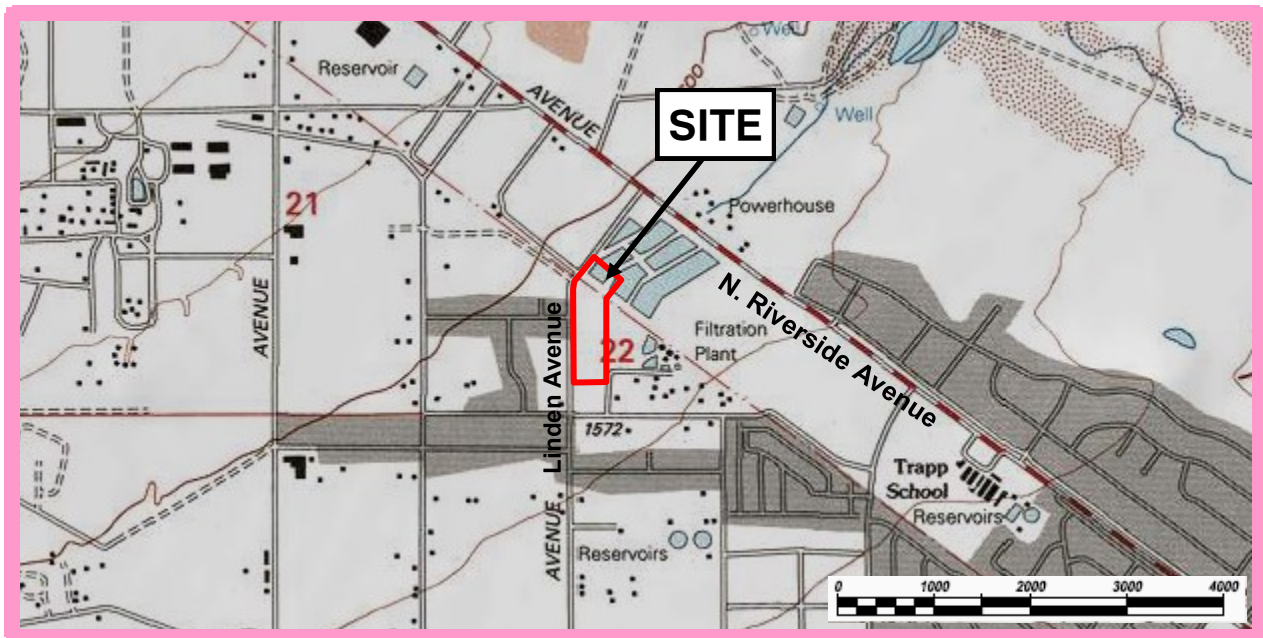
SCOPE OF SERVICES

The scope of our services has included the following:

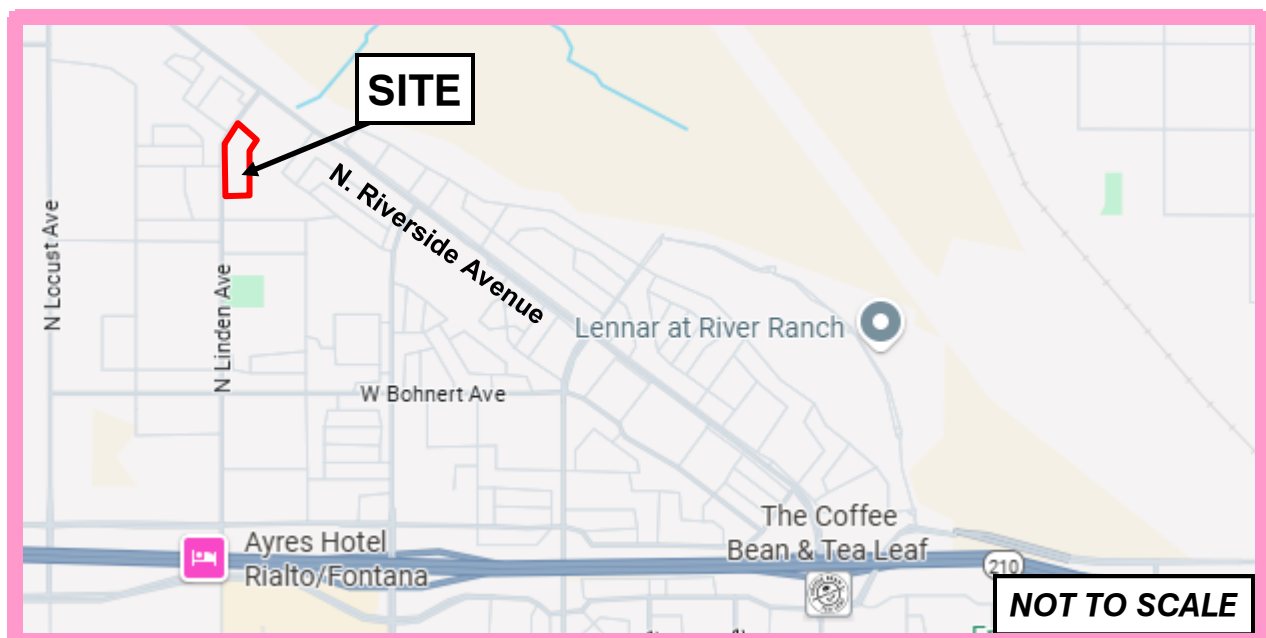
1. Review of available soils and geologic data for the site area, including previous geotechnical reports by GSI in the vicinity of the subject site (Appendix A).
2. Geologic site reconnaissance and geologic mapping of surficial deposits.
3. Our field investigation was conducted on June 6, 2025, using a rubber-tired backhoe. Subsurface exploration consisted of the advancement of 10 exploratory test pits throughout the property for geotechnical logging, and sampling (Appendix B). Two (2) of the test pits advanced were used for feasibility level infiltration testing. The field infiltration data sheets are presenting in Appendix C.
4. Seismic ground motion values (Appendix D).
5. Laboratory testing of representative soil samples collected during our subsurface exploration program. Testing included maximum density testing, expansion index, sieve analysis, resistance value (R-value), and sulfate and corrosion testing of the materials encountered during our onsite field investigation. Results of our laboratory testing are provided in Appendix E.
6. Appropriate engineering and geologic analyses of data collected, and preparation of this geotechnical report, including test pit logs, infiltration test results, laboratory test results, regional seismic data and ground motion values, general earthwork factors, recommendations for site grading, and preliminary foundation design recommendations.

SITE LOCATION AND CONDITIONS

The project area consists of a slightly curved and loosely rectangular shaped 6.93-acre parcel in the City of Rialto, San Bernardino County, California (see Figure 1, Site Location Map). In general, the site may be described as a relatively flat to gently sloping remnant of the young alluvial fans that make up the higher topographic benches (i.e., the Rialto Bench), of the outwash area of the Lytle Creek Wash. Topographically, drainage is generally directed to the south- southeast via sheetflow and constructed drainage courses. According to the Tentative Tract Map by EAI (2025), elevations across the site range from a high of approximately 1,595 feet Mean Sea Level (MSL) on the north to approximately 1,576 feet MSL at the southwest corner of the project site. Therefore, overall relief across the site is on



Base Map: TOPO! Copyright 2003 National Geographic, USGS Devore Quadrangle, 7.5 Minute, Riverside Co., California -- dated 1954.



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SITE LOCATION MAP

Figure 1

the order of approximately 19 feet. Historically, the site was used during construction of the adjacent West Valley Water District (WVWD) water filtration facility to the east and south. During construction in 2004, additional storage basins were constructed at the WVWD filtration facility, and the excavation spoils brought to, and used to cap the subject site with the undocumented artificial fill. The adjacent areas to the west- northwest have been residentially developed with single-family tract homes.

PROPOSED DEVELOPMENT

Based on our review of the 30-scale tentative tract map by EAI (2025), it is our understanding that the proposed development of the project would consist of the construction of an alley-loaded 67 lot residential development, a dog park, tot-lot and other parklets are proposed along with the installation of underground utility and interior and exterior roadway infrastructure improvements. In addition, a water quality BMP basin is proposed in the southern portion of the site, along with perimeter block wall improvements. We further understand that the buildings are proposed as one- or two-story structures with slab-on-grade foundation systems. Building loads are assumed to be typical for these types of relatively light residential structures. Typical cut and fill grading techniques would be required to bring the site to design elevations. The need for import soils is currently unknown. Sewage disposal is anticipated to be accommodated by connection to the regional disposal system. Due to the flat-laying nature of the site, no significant cut or fill slopes are currently proposed (EAI, 2025).

FIELD STUDIES

As indicated above, field studies conducted during our evaluation of the property consisted of geologic reconnaissance mapping, and the excavation and logging of 10 exploratory test pits throughout the property for evaluation of near-surface soil and geologic conditions. Two (2) of the exploratory test pits were used for infiltration feasibility testing. The field exploration was performed on June 6, 2025. The test pits were logged by a staff geologist from our firm who also collected representative bulk samples for appropriate laboratory testing. The logs of the test pits are presented in Appendix B. The field data sheets for the infiltration testing conducted are presented in Appendix C. The approximate locations of the test pits advanced onsite and infiltration testing locations are presented on Plate 1 (Geotechnical Map).

REGIONAL AND SITE GEOLOGY

Regional Geologic Setting

The subject property is located on the northeastern fringe of a prominent natural geomorphic province in southwestern California known as the Peninsular Ranges. The Peninsular

Ranges are characterized by steep, elongated ranges and valleys that trend northwestward, whereas, the Transverse Ranges trend east-west. The site area is situated within the Rialto-Colton Basin of the Perris block, an eroded mass of Cretaceous-age and older rocks. The Perris Block is defined by the San Jacinto and Elsinore-Chino faults, to the east and west, respectively, and by the Cucamonga fault, to the north. It is located on coalescing alluvial fans derived locally from alluvium and ancient debris flows originating from the San Gabriel Mountains (basement rock uplifted by bounding faults) to the north, and the Jurupa Mountains (an isolated knob of basement rock) to the south.

Local Geology

The site may be characterized as being primarily underlain by Holocene-age young alluvial-fan deposits (Morton and Matti, 2001), which historically emanated from Lytle Creek Wash. The alluvial-fan deposits onsite are mantled by a layer of undocumented artificial fill, on the order of approximately 2 to 6 feet in thickness, relatively thin layers of colluvium/topsoil underlie the artificial fill. As described by Miller and Matti (2001), the young alluvial-fan deposits (Unit 5, Qyf₅) are generally coarse-grained alluvial-fans having slightly dissected surfaces, typically have high coarse to fine clast ratios, and can also be distinguished by the higher terrace level (i.e., the Rialto Bench). At depth, the young alluvial-fan deposits encountered during this study were generally observed to be varying hues of light gray, and dark to light brown silty sands, and sands that were friable (uncemented) and contained abundant sub-rounded gravel- to boulder-sized clasts. Due to the friable nature of surficial soils, localized caving was encountered in a majority of the excavations (see Appendix B).

Site Geologic Units

The site geologic units encountered during our subsurface investigation and site geologic mapping included undocumented artificial fill mantling the site, colluvium/topsoil, and Quaternary-age young alluvial-fan deposits (Unit 5, Qyf₅). The distribution of mappable units is shown on Plate 1 (Geotechnical Map). Supplemental descriptions for the geologic units are shown in the Test Pit Logs (Appendix B). The geologic units are generally described below (from youngest to oldest):

Artificial Fill - Undocumented (Map Symbol - Afu)

Undocumented artificial fill was identified during our subsurface investigation mantling the colluvium and topsoil materials. These surficial fill materials are associated with the past grading and construction in 2004 of several water storage basins in the neighboring WWD filtration facility, and then in 2009 where the majority of the site appears to have been used as a construction yard. These surficial soils were generally observed to range from two (2) feet (TP-6) to as much as six (6) feet in thickness (TP-7). As encountered onsite, the artificial fill materials were typically light gray, friable, fine- to coarse-grained gravelly sands and sandy gravels with few to some boulder-sized clasts ranging from 12 to 24 inches in diameter. The undocumented fill materials were generally dry and loose. Due to the potentially compressible nature of these surficial soils, and the volume of boulder-sized clasts observed

that would require special handling techniques, they are considered unsuitable for support of structures or improvements in their existing state and therefore, will be need to be removed and recompacted.

Colluvium and Topsoil (Unmapped)

Colluvium and topsoil materials were observed in our subsurface investigation beneath the artificial fill, and mantling the young alluvial-fan deposits onsite. These materials appear to be associated with in-place weathering, and previous tilling for weed abatement purposes. These subsurface soils were generally observed to be about 1 to 2 feet in thickness. As encountered onsite, the colluvium and topsoil is typically dark brown, fine- to coarse-grained silty sands, with sub-rounded gravels and cobbles, with a few roots and rootlets, generally marking the boundary of the colluvium and topsoil. The colluvium and topsoil materials were generally dry to damp, loose, and friable. Due to the potentially compressible nature of these surficial soils, they are considered unsuitable for support of structures or improvements in their existing state and therefore, will be need to be removed and recompacted.

Quaternary Young Alluvial-Fan Deposits - Unit 5 (Map Symbol - Qyf₅)

Regional geologic mapping by Morton and Matti (2001) indicates that Quaternary-age young alluvial-fan deposits (Unit 5) are present across the entirety of the site. As encountered onsite, these deposits generally consisted of light gray to dark brown to light brown gravelly sands and sands with gravels. Locally abundant sub-rounded gravel and cobbles, to a few boulder-sized clasts, up to 30 inches in diameter, were encountered during our subsurface investigation. These deposits were generally dry to locally damp and loose to medium dense with depth. On a preliminary basis, these deposits generally have a very low expansion potential. Where unweathered, these deposits appear suitable for the support of additional engineered fill and settlement-sensitive improvements. During site grading the weathered portions (upper 3 to 4 feet) should be removed to competent/dense young alluvial-fan deposits.

FAULTING, REGIONAL SEISMICITY, AND GROUND MOTION VALUES

Local Faulting

The site is situated in an area of active as well as potentially-active faults. The nearby San Bernardino segment of the San Jacinto fault zone is considered active and is included within an Alquist-Priolo Earthquake Fault Zone. Our review of published data indicates that there are no known active faults crossing the site, and the site is not within an Alquist-Priolo Earthquake Fault Zone (CGS, 2018). During our review of aerial photographs (USDA, 1980), we did not observe any photolineaments transecting the site. However, a splay of the San Bernardino segment of the San Jacinto fault is located approximately 1.9 miles (3.0 kilometers) to the southeast. This fault should have the greatest effect on the site in the form of strong ground shaking.

Seismic Ground Motion Values

Using a latitude of 34.1527 degrees and a longitude of -117.4004, risk category of II and based on the site conditions, the following table summarizes the design criteria obtained from the 2022 CBC, Chapter 16 Structural Design, Section 1613, Earthquake Loads. The ASCE Hazard Tool (ASCE Hazard Tool) has been used to aid in design. Since the site class is D, a site-specific procedure would be required, but it is not performed as we follow the Exception of Section 11.4.8.1, ASCE 7-16, Supplement 3.

2022 CBC SEISMIC GROUND MOTIONS		
PARAMETER	VALUE	2022 CBC OR REFERENCE
Risk Category	I, II, or III	Table 1604.5
Site Class	D	Chapter 20, ASCE 7-16
MCE _R spectral acceleration - (0.2 sec), S_s	2.198 g	Section 1613.2.1
MCE _R spectral acceleration - (1.0 sec), S_1	0.851 g	Section 1613.2.1
Site Coefficient, F_a	1.0	ASCE 7-16, Supplement 3
Site Coefficient, F_v	1.7	ASCE 7-16, Supplement 3
MCE _R spectral acceleration (0.2 sec), S_{MS}	2.198 g	Section 1613.2.3
MCE _R spectral acceleration (1.0 sec), S_{M1}	2.170 g	ASCE 7-16, Supplement 3
Design spectral acceleration (0.2 sec), S_{DS}	1.465 g	Section 1613.2.4
Design spectral acceleration (1.0 sec), S_{D1}	1.447 g	ASCE 7-16, Supplement 3
PGA _M	1.0 g	ASCE 7-16, Section 11.8.3
Seismic Design Category	E	Section 1613.2.5

Conformance to the criteria above for seismic design does not constitute any kind of guarantee or assurance that significant structural damage or ground failure will not occur in the event of a large earthquake. The primary goal of seismic design is to protect life, not to eliminate all damage, since such design may be economically prohibitive. Cumulative effects of seismic events are not addressed in the 2022 CBC (CBSC, 2022) and regular maintenance and repair following locally significant seismic events will likely be necessary, as is the case in all southern California.

GROUNDWATER

Regional groundwater was not encountered within any of the subsurface excavations advanced during our field work. Likewise, seeps, springs, or other indications of a high groundwater level were not noted on the subject property during the time of our field investigation. Based on our review of the California Department of Water Resources (CDWR) Water Data Library (2025), the depth to groundwater in nearby wells ranges from

approximately 344.5 feet below the ground surface (11/15/2010) in the water treatment facility directly east of the site (Well Station No. 341539N1173987W002), to 421.5 feet below the ground surface (11/23/2010) further south on Linden Avenue, just north of Birdsall Park (Well Station No. 341483N1173993W001). Based on the above, regional groundwater will not be encountered during grading; however, the possibility of transient and localized perched groundwater conditions, cannot be precluded within existing drainage areas, and impermeable zones. These observations reflect site conditions at the time of our field work and do not prevent future changes in local groundwater conditions from excessive irrigation, precipitation, or that were not evident, at the time of our investigation.

LIQUEFACTION POTENTIAL

Seismically induced liquefaction is a phenomenon in which cyclic stresses, produced by earthquake-induced ground motion, create excess pore pressures in soils. The soils may thereby acquire a high degree of mobility and lead to lateral movement, sliding, sand boils, consolidation and settlement of loose sediments, and other damaging deformation. This phenomenon occurs only below the water table; but after liquefaction has developed, it can propagate upward into overlying, non-saturated soil as excess pore water dissipates. Typically, liquefaction has a relatively low potential at depths greater than 50 feet.

Liquefaction has two principal effects. One is the consolidation of loose sediments with resultant settlement of the ground surface. The other is lateral sliding. Significant permanent lateral movement generally occurs only when there is considerable differential loading on susceptible soils, such as fill or natural ground slopes on alluvium.

Based on our review of the Geologic Hazard Overlay map in the San Bernardino County Land Use Plan (County of San Bernardino, 2010), the site does not lie within a zone of generalized liquefaction susceptibility, and is designated as non-susceptible to liquefaction.

Based on previous site specific data from nearby historic investigations by GSI and others, we found there is a potential for seismic activity; however, based on the generally dense granular and coarse onsite soils, current hydrologic regime and the depth to regional groundwater, the potential for liquefaction to affect the site is considered very low.

SUBSIDENCE

Our experience in the site vicinity and review of readily available data indicate that the site area may be subsiding owing to down-faulting along bordering fault zones, or regional groundwater withdrawal. The scope of this potential for affecting the subject site is beyond the scope of this current study. However, based on our review of Fife et al. (1976) aerial subsidence within the generalized area may be on the order of 0.4 feet. The main subsidence zone is located near San Bernardino, which is approximately 7 miles southeast of the site. It should be noted that areal subsidence generally occurs at the transition condition between materials of substantially different engineering properties (i.e., bedrock and alluvium), or

active faults. At the site, no potential for this transition condition appears to exist; however, faults transect the site. Our review of available stereoscopic aerial photographs (USDA, 1980) also showed no features generally associated with areal subsidence (e.g., radially-directed drainages flowing into a depression(s), linear depressions associated with mountain fronts).

The Rialto-Colton Basin is an adjudicated water basin. Past overdraft had resulted in subsidence as early as 1971, and resulted in some damage to existing infrastructure. Pumping induced drawdown and subsequent aquifer-system compaction were identified as the most likely cause of the land subsidence in the affected area. The main areas of known subsidence lie several miles to the southeast of the subject site (Fife et al., 1976). Since the groundwater basin is now adjudicated, it is unlikely that the Watermaster would allow groundwater extraction to increase to the point that significant subsidence would occur onsite.

OTHER GEOLOGIC HAZARDS

The following list includes other geologic and seismic related hazards that have been considered during our evaluation of the site. The hazards listed are considered negligible or mitigated as a result of site location, soil characteristics, and typical site development procedures:

- Liquefaction
- Lateral Spreading
- Dynamic Settlement
- Surface Fault Rupture
- Ground Lurching or Shallow Ground Rupture
- Tsunami
- Seiche

Based on our review of Geologic Hazard Overlay maps in the San Bernardino County Land Use Plan (County of San Bernardino, 2010), the site does not lie within a generalized landslide susceptibility zone. However, due to the non-cohesive (friable) materials that exist onsite, caving and sloughing should be anticipated in all subsurface excavations and trenching. Appropriate safety considerations for potential caving and sloughing, such as shoring or sloping layback cuts, should be incorporated into the construction design details.

LABORATORY TESTING

Classification

Soils were classified visually according to the Unified Soils Classification System. The soil classifications are shown on the Test Pit Logs, Appendix B.

Moisture Density

The field moisture contents and dry unit weights were measured for the onsite soils encountered in the exploratory test pits. Field moisture and density tests were performed using the nuclear (densometer) methods per ASTM D 6938 (Procedure A). The results of the field moisture and density testing are presented on the Test Pit Logs (Appendix B).

Laboratory Standard

The maximum density and optimum moisture content was determined for the major soil types encountered within the exploratory test pits. A rock correction was determined and the maximum density values listed below reflect the "rock corrected" test results. The laboratory standard used was ASTM D 1557. The moisture-density relationship obtained for the site soils are shown below:

SOIL TYPE	LOCATION AND DEPTH (ft.)	MAXIMUM DRY DENSITY (pcf)	OPTIMUM MOISTURE CONTENT (%)
Well Graded SAND with SILT and GRAVEL, Dark Yellowish Brown	P-1, TP-4, TP-6 @ 6'-9' (Composite)	141.3	5.4
Well Graded SAND with SILT and GRAVEL, Dark Yellowish Brown	TP-6 @ 2'	140.5	5.0

Expansion Potential

Expansion Index (E.I.) testing and expansion potential classification were performed in general accordance with ASTM Standard D 4829 on a representative sample of the onsite soils collected from the field exploration. Based on the preliminary test results obtained, the soils exhibit a very low expansion potential (E.I. 0-20). Additional E.I. testing should be conducted at the conclusion of site grading to further evaluate the preliminary test results obtained. The results of the expansion index testing are presented in the following table.

SAMPLE LOCATION AND DEPTH (FT)	EXPANSION INDEX	EXPANSION POTENTIAL
TP-6 @ 2'	0	Very Low

Particle-Size Analysis

A particle-size evaluation was performed on representative soil samples (P-1/TP-4/TP-6 (composite) @ 6-9 feet and TP-6 @ 2 feet) in accordance with ASTM D 6913. The testing

was used to evaluate the soil classification in accordance with the Unified Soil Classification System (USCS). The results of the particle-size evaluation indicate that the tested samples are a well-graded sand with silt and gravel, to a well-graded gravel with sand, to a silty Gravel with sand. The testing results indicate a range between 48.4 to 45.9 percent gravel, 44.5 to 48.7 percent sand, and 7.1 to 5.4 percent fines [USCS Symbol - SW-SM and GW-GM]). The particle-size test results are presented in Appendix E.

Corrosion

A typical sample of the site soil materials were analyzed for soluble sulfates, chloride, pH, and resistivity. The soluble sulfate and corrosion potential results are shown as follows:

LOCATION AND DEPTH (FT.)	SOLUBLE SULFATES PERCENTAGE BY WEIGHT	CHLORIDE (PPM)	PH	RESISTIVITY (OHMS-CM)
TP-6 @ 2'	< 0.003	11	7.4	24,000

For preliminary planning purposes, and based upon the soluble sulfate test results and the latest edition of the 2022 CBC (CBSC, 2022), the soluble sulfate content is classified as “S0” per the ACI 318-14 (0.00 to 0.10 soil percentage by weight is considered Exposure Class “S0”). As such, sulfate-resistant concrete is currently not anticipated. Based on the results of the resistivity and pH testing, the onsite soils are generally considered slightly alkaline (a pH of 7.4 to 7.8 is considered slightly alkaline) and are considered mildly corrosive to ferrous metals in a saturated state (over 10,000 ohm-cm is considered mildly corrosive). Chloride levels are generally low.

Although the site soils are categorized as being mildly corrosive toward ferrous metals, it is our understanding that ferrous metals embedded in properly poured and formed concrete with the proper mix should be adequately protected from these conditions. GSI does not consult in the field of corrosion engineering. Therefore, additional comments and recommendations should be obtained from a qualified corrosion engineer based on the level of corrosion protection required for the project, as recommended by the project architect or structural engineer, assuming Exposure Classes “S0,” “W0,” and “C1” conditions, per ACI 318-14, as the foundations would likely be exposed to moisture.

R-Value Test

An evaluation of the resistance value (R-value) for a representative sample of the site soils was performed in accordance with the latest revisions to the Department of Transportation, State of California, Material & Research Test Method No. 301. Test results obtained indicated an R-value of 78. Based on the test results obtained, preliminary pavement designs for the project have been developed, and are provided in the Preliminary Pavement Design Recommendations section of this report. The R-value test results obtained are presented in Appendix E.

STORMWATER INFILTRATION FEASIBILITY

As mentioned previously, a secondary purpose of this study was to perform percolation and infiltration feasibility testing within the proposed stormwater quality BMP system. In accordance with the guidelines outlined in Appendix D of the San Bernardino County (SBC, 2013) Technical Guidance Document for Water Quality Management Plans, two (2) percolation tests were conducted onsite within the approximate area of the currently proposed water quality basin, at a depth of approximately 10 feet below existing site grades. The percolation testing was performed to further evaluate site conditions with respect to the proposed water quality basin that will retain and filter onsite storm water. The percolation testing was performed in general conformance with the SBC (2013) and CASQA (2003) design handbooks for such testing. The field percolation testing and geologic logging were performed by a trained field technician and staff geologist from our firm. Logs of the test pits advanced for this study can be found in Appendix B. For convenience, the field percolation data sheets are presented in Appendix C. Procedures for testing are outlined briefly below:

Percolation Test Procedures

Test Borings:

1. Backhoe excavated to a depth of 9 feet. Then, the percolation cup further excavated with hand tools another 1 foot in depth.
2. Diameter - 8 inches.
3. After the removal of loose materials, 2 inches of gravel was placed on the bottom of each test boring.
4. A perforated pipe was then installed within each percolation cup to facilitate accurate field measurements and prevent caving during the pre-soak period and testing intervals.

Pre-Soaking: After the installation of the perforated pipes, the boring was filled with clear water to a depth of approximately 15 inches. The testing period for the percolation tests commenced immediately, as all the water seeped away while the tester was present and sandy soils were known to exist onsite.

Sandy Soil Test: During the sandy soil criteria test period, two (2) consecutive measurements were conducted at each test location at intervals of approximately 25 minutes. More than 6 inches of water seeped away during each of the two (2) test measurements within the percolation test locations; therefore, the testing began using the criteria for sandy soils testing at all locations.

Testing: After required pre-soak assessment and sandy soil criteria test periods, percolation testing measurements were made. A column of clear water was re-established within each of the test locations. The drop in water level was measured from a fixed reference point, refilling after each test measurement. For each test, a series of test measurements were taken for an additional hour, at time intervals of 10 minutes.

Precision: All test measurements were read to the nearest 1/4-inch.

Test Results: Calculations from our field testing indicate a percolation rate between 0.67 and 3.08 minutes/inch. Per the SBC (2013) guidelines, the percolation rates obtained were then converted to infiltration rates using the "Porchet Method," to be used by the design engineer for appropriate sizing of the basins. The converted infiltration rates obtained varied between 2.54 and 18.95 inches/hour, yielding an overall average of 10.74 inches/hour. Typically, the lowest infiltration rate (2.54 inches/hour) obtained is applied to the design. The converted infiltration rates, along with the formulas used are presented in Figure 2 (Percolation Rate to Infiltration Rate Conversion).

USDA Site Soil Groups, Soil Units, Ksat Values

Our review of the United States Department of Agriculture (USDA, 2025) Web Soil Survey, indicates only one (1) major soil unit underlies the site. The Tujunga gravelly loamy sand, 0 to 9 percent slopes (Map Unit Symbol: TvC) is distributed throughout the entirety of the site. Based on our review, the soils onsite belong to Hydrologic Soil Group "A," with the capacity of the most limiting soil layer to transmit water classified as "high to very high." The "Ksat" value (i.e., hydraulic conductivity or infiltration rate) for the soil type onsite was evaluated to be within the range of 5.95 to 19.98 inches per hour.

Infiltration Basin Siting Requirements

Our review of the general infiltration basin siting requirements and limitations (CASQA, 2003), indicates sites characterized as belonging to Hydrologic Soil Groups "A," "B," and "C" may be suitable for infiltration, requiring a minimum soil infiltration rate of 0.5 inches/hour (CASQA, 2003). Based on our review of historic regional groundwater levels, and provided a relatively shallow basin is constructed, a minimum 10-foot vertical separation from the bottom of the basin to the top of historic high groundwater levels appears to be maintained. The design engineer will need to review basin siting requirements by CASQA (2003) and the converted infiltration rates obtained during this study with respect to the proposed water quality basin.

Onsite Storm Water Quality Best Management Practice (BMP) Systems

Based on our review, onsite infiltration-runoff retention systems (OIRRS) are currently planned for Best Management Practices (BMP's) or Low Impact Development (LID) principles for the project (EAI, 2025). As such, certain guidelines must be followed in the planning, design, and construction of such systems. Such systems, if improperly designed or implemented without consideration of the geotechnical aspects of site conditions, can contribute to flooding, saturation of bearing materials beneath site improvements, slope instability, and possible concentration and contribution of pollutants into the groundwater or storm drain or utility trench systems.

Percolation Rate to Infiltration Rate Conversion

$$* \text{ Infiltration Rate } (I_t) = \frac{\Delta H \pi r^2 60}{t(\pi r^2 + 2\pi r H_{av})} = \frac{\Delta H 60 r}{\Delta t(r + 2H_{av})}$$

Where:

- I_t = tested infiltration rate, inches/hour
- ΔH = change in head over the time interval, inches
- Δt = time interval, minutes
- r = effective radius of test hole
- H_{av} = average head over the time interval, inches

		Δt	Init Level	Fnl Level	ΔH	H_{av}	I_t	Low = 2.54 Average = 10.74 **DIR = 1.27
Infiltration Test Numbers	TP-9/P-1 @ 9 ft.	10	15	11 3/4	3 1/4	13 3/8	2.54	
	TP-10/P-2 @ 9 ft.	10	15	0	15	7 1/2	18.95	

* Conversion per the "Porchet Method" (RCFWCD, 2011)

$$FOS = 2.0 = 2.54/2 = 1.27 = DIR^{**}$$

**DIR = Design Infiltration Rate (Inches/Hour)

A key factor in these systems is the infiltration rate (often referred to as the percolation rate) which can be ascribed to, or determined for, the earth materials within which these systems are installed. Additionally, the infiltration rate of the designed system (which may include gravel, sand, mulch or topsoil, or other amendments) will need to be considered. The project infiltration testing is very site specific, any changes to the location of the proposed OIRRS or estimated size of the OIRRS, may require additional infiltration testing. Locally, relatively impermeable formations include: clayey surficial soils, igneous and metamorphic bedrock, as well as future fine grained fill soils.

Some of the methods which are used for onsite infiltration include percolation basins, dry wells, bio-swale and bio-retention, permeable pavers and pavement, infiltration trenches, filter boxes and subsurface infiltration galleries and chambers. Some of these systems are constructed using native and import soils, perforated piping, and filter fabrics while others employ structural components such as stormwater infiltration chambers and filters and separators. Every site will have characteristics which should lend themselves to one or more of these methods, but not every site is suitable for OIRRS. In practice, OIRRS are usually initially designed by the project design civil engineer. Selection of methods should include (but should not be limited to) review by licensed professionals including the geotechnical engineer, hydrogeologist, engineering geologist, project civil engineer, landscape architect, environmental professional, and industrial hygienist. Applicable governing agency requirements should be reviewed and included in design considerations.

The following geotechnical guidelines should be considered when designing onsite infiltration-runoff retention systems:

- It is not good engineering practice to allow water to saturate soils, especially near slopes or improvements; however, the controlling agency or authority may now require the use of OIRRS.
- Areas adjacent to, or within, the OIRRS that are subject to inundation should be properly protected against scouring, undermining, and erosion, in accordance with the recommendations of the design engineer.
- Where infiltration systems are located near slopes or improvements, impermeable liners and subdrains should be used along the bottom or sides of bioretention swales and basins located within the influence of such slopes and structures. Impermeable liners used in conjunction with bioretention basins should consist of a 30-mil polyvinyl chloride (PVC) membrane that is covered by a minimum of 12 inches of clean soil, free from rocks and debris, with a maximum 4:1 (h:v) slope inclination, or flatter, and meet the following minimum specifications:

Solid Soils Specific Gravity (ASTM D792): 1.2 (g/cc, min.); Tensile (ASTM D882): 73 (lb/in-width, min); Elongation at Break (ASTM D882): 380 (% min); Modulus (ASTM D882): 32 (lb/in-width, min.); and Tear Strength (ASTM D1004): 8 (lb/in, min); Seam Shear Strength (ASTM D882) 58.4 (lb/in, min); Seam Peel Strength (ASTM D882) 15 (lb/in, min).

- Subdrains should consist of at least 4-inch diameter Schedule 40 or SDR 35 drain pipe with perforations oriented down. The drain pipe should be sleeved with a filter sock.
- Storm drain, standpipes, and utilities that cross BMPs should be slurried with a 1½-sack slurry mix, to 5 feet outside the structure.
- Infiltrations systems should not be installed within approximately 8 feet of building foundations utility trenches, and walls, or a 1:1 (horizontal to vertical [h:v]) slope (down and away) from the bottom elements of these improvements. Alternatively, deepened foundation (pile or pier) supported improvements may be used.
- Infiltrations systems should not be installed adjacent to pavement or hardscape improvements. Alternatively, deepened and thickened edges and curbs or impermeable liners may be used in areas adjoining the OIRRS.
- As with any OIRRS, localized ponding and groundwater seepage should be anticipated. The potential for seepage or perched groundwater to occur after site development should be disclosed to all interested and affected parties.
- Infiltration systems should not be installed where the vertical separation of the groundwater level is less than approximately 10 feet from the base of the system.
- Infiltration systems should be designed using a suitable factor of safety (FOS) to account for uncertainties in the known infiltration rates (as generally required by the controlling authorities), and reduction in performance over time.
- As with any OIRRS, proper care will need to be provided. Best management practices should be followed at all times, especially during inclement weather. Provisions for the management of any siltation, debris within the OIRRS, or overgrown vegetation (including root systems) should be considered. An appropriate inspection schedule will need to be adopted and provided to all interested and affected parties.
- Any designed system will require regular and periodic maintenance, which may include rehabilitation or complete replacement of the filter media (e.g., sand, gravel, filter fabrics, topsoil, mulch) or other components used in construction, so that the design life exceeds 15 years. Due to the potential for piping and adverse seepage conditions, a burrowing rodent control program should also be implemented onsite.
- Newly established vegetation and landscaping (including phreatophytes) may have root systems that will influence the performance of the OIRRS or nearby LID systems.
- The potential for surface flooding, in the case of system blockage, should be evaluated by the design engineer.

- Any proposed utility backfill materials (i.e., inlet/outlet piping or other subsurface utilities) located within or near the proposed area of the OIRRS may become saturated. This is due to the potential for piping, water migration, and seepage along the utility trench line backfill. If utility trenches cross or are proposed near the OIRRS, cut-off walls or other water barriers will need to be installed to mitigate the potential for piping and excess water entering the utility backfill materials. Planned or existing utilities may also be subject to piping of fines into open-graded gravel backfill layers unless separated from overlying or adjoining OIRRS by geotextiles or slurry backfill.
- The use of OIRRS above existing utilities that might degrade or corrode with the introduction of water and seepage should be avoided.

PRELIMINARY EARTHWORK FACTORS

Preliminary earthwork factors (shrinkage and bulking) for the subject property have been estimated based upon our field and laboratory testing, visual site observations, and experience in the site area. It is apparent that shrinkage would vary with depth and with areal extent over the site based on previous site use. Variables include vegetation, weed control, discing, and previous filling or exploring. However, all these factors are difficult to define in a three-dimensional fashion.

Therefore, the information presented below represents average shrinkage values (excluding the removal of oversize materials):

Undocumented Artificial Fill (Afu)	15% to 20% shrinkage
Colluvium and Topsoil	12% to 18% shrinkage
Young Alluvial Fan Deposits (Qyf ₅)	10% to 18% shrinkage

The above factors indicate that earthwork balance for the site would be difficult to define and flexibility in design is essential to achieve a balanced end product. Subsidence due to equipment loadings (dynamic compaction) may be on the order of up to 0.10 feet, but will depend on haul routes, and other factors.

PRELIMINARY CONCLUSIONS AND RECOMMENDATIONS

Based on our current field exploration, laboratory testing, and our engineering and geologic analyses, it is our opinion that the project site appears suited for the proposed residential development from a geotechnical engineering and geologic viewpoint, provided that the recommendations presented in the following sections are incorporated into the design and construction phases of site development. The recommendations presented below should be incorporated in the design, grading, and construction considerations.

General

1. Geotechnical engineering and compaction testing services should be provided during grading operations to assist the contractor in removing unsuitable soils and in his effort to compact the fill.
2. Geologic observations should be performed during grading to verify and further evaluate geologic conditions. Although unlikely, if adverse geologic structures are encountered, supplemental recommendations and earthwork may be warranted.
3. In general, and based upon the available data to date, groundwater is not expected to be a factor in development of the site. Due to the sandy nature of the site materials and terrain, minor seepage may be encountered throughout the site along with seasonal perched water within the drainage areas. Given the high permeability of onsite soils and lack of significant canyon areas, no subdrain systems are currently anticipated. However, subdrainage systems for the control of localized groundwater seepage, cannot be precluded and should be anticipated.
4. Based upon our field explorations throughout the property, the site materials should be rippable to the depths anticipated for development.
5. Due to the non-cohesive nature of the onsite materials, caving and sloughing may be anticipated to be a factor in subsurface excavations and trenching. Therefore, current local and state and federal safety ordinances for subsurface trenching should be enforced.
6. General Earthwork, Grading Guidelines are provided at the end of this report as Appendix F. Specific recommendations are provided below.

Demolition/Grubbing

1. Any existing surficial or subsurface structures (i.e., foundations, septic systems, etc.), major vegetation, and any miscellaneous debris should be removed from the areas of proposed grading.
2. The project geotechnical consultant should be notified of any previous foundation, irrigation lines, septic tanks or leach fields, or other subsurface structures that are uncovered during the recommended removals, so that appropriate remedial recommendations can be provided.
3. Cavities or loose soils (including all previous exploratory test pits) remaining after demolition and site clearance should be cleaned out, observed by the geotechnical consultant, processed, and replaced with fill that has been moisture conditioned to at least optimum moisture content and compacted to at least 90 percent of the laboratory standard, if not removed by proposed remedial grading.

Treatment of Existing Ground

1. Removal of all undocumented artificial fill, subsurface colluvium and topsoil, and near surface weathered young alluvial fan deposits will be necessary prior to fill placement, in areas proposed for settlement-sensitive improvements. The near surface deposits should be removed to competent materials (i.e., greater than or equal to 85 percent saturation, or greater than or equal to 105 pcf for in-place native materials), if not removed by proposed excavation within areas proposed for settlement-sensitive improvements. For preliminary planning purposes, these depths are estimated to be on the order of 7 to 11 feet across the majority of the site, or potentially more where infiltration-pond grading was previously performed on the northern portion of the site. Variations from these thicknesses should be anticipated. A minimum of 2 feet of compacted fill should underlie the proposed footings. Actual depths of removals will be evaluated in the field during grading by the geotechnical consultant.
2. Subsequent to the above removals, the fill should be cleansed of any oversized rock, and the upper 6 to 8 inches of the exposed subsoils should be scarified, brought to at least optimum moisture content, and recompact to a minimum relative compaction of 90 percent of the laboratory standard.
3. The existing onsite soils may be reused as compacted fill provided that any significant concentrations of vegetation and miscellaneous trash and debris, and oversized rock (>12 inches) are removed prior to or during fill placement. In general, any oversized rock constituents greater than 12 inches in any one dimension should not be placed within 10 feet of finish pad grade.
4. Localized deeper removal may be necessary due to buried drainage channel meanders or dry porous materials. The project geotechnical consultant/geologist should observe all removal areas during the grading.

Fill Placement

1. Fill materials should be cleansed of major vegetation, debris, and oversized rock (greater than 12 inches) prior to placement.
2. Fill materials should be brought to at least optimum moisture, placed in thin 6- to 8-inch lifts and mechanically compacted to obtain a minimum relative compaction of 90 percent of the laboratory standard (ASTM D 1557).
3. Any oversized rock materials greater than 12 inches in diameter should not be placed within 10 feet of finish grade. The maximum particle size for utilities in this area may be as stringent as 4 to 6 inches, as directed by the controlling authority. The use of particles greater than 4 to 6 inches in utility backfill may require screening prior to re-use. Procedures for oversized rock are outlined in the "Rock Placement Guidelines" section of this report. Additional guidelines for rock placement are provided in Appendix F.

4. Any import materials should be observed and evaluated suitable by the geotechnical consultant prior to placement on the site. Foundation designs may be altered if import materials have a greater sulfate or expansion values than the onsite materials encountered during our preliminary investigation.

ROCK PLACEMENT GUIDELINES

Rock Disposal

We anticipate that soils to be used as fill material for the subject project will contain some rock. Appropriately, the need for rock disposal will be necessary during grading operations on the site. Generally, for the purpose of this report, the materials may be described as either 8 inches or less, greater than 8 and less than 36 inches, and greater than 36 inches. These three categories set the basic dimensions for where and how the materials are to be placed.

Materials 8 Inches in Diameter or Less

Since rock fragments along with the overburden materials are anticipated to be a part of the materials used in the grading of the site, criteria are needed to facilitate the placement of these materials within guidelines which would be workable during the rough grading, post-grading improvements, and serve as acceptable compacted fill.

1. Fines and rock fragments 8 inches or less in diameter may be placed as compacted fill cap materials within the building pads, slopes, and street areas as described below. The rock fragments and fines should be brought to at least optimum moisture content and compacted to a minimum relative compaction of 90 percent of the laboratory standard.
2. The purpose for the 8-inch diameter limits is to allow reasonable sized rock fragments into the fill under selected conditions (optimum moisture or above) surrounded with compacted fines. The 8-inch diameter size also allows a greater volume of the rock fragments to be handled during grading, while staying in reasonable limits for later onsite excavation equipment (backhoes and trenchers) to excavate footings and utility lines.
3. Fill materials 8 inches or less in diameter should be placed, but not limited to, within the upper 10 feet of proposed fill pads, the entire street right-of-way width, including the proposed overexcavated areas and replacement fill areas, or to a depth of 10 feet below finish grade.

Materials Greater Than 8 inches and Less Than 36 Inches in Diameter

1. During the process of excavation, a significant amount of rock fragments or constituents larger than 8 inches in diameter will be generated. These significant

amounts of oversized materials between greater than 8 and less than 36 inches in diameter may be incorporated into the fills utilizing a series of rock blankets.

2. Each rock blanket should consist of rock fragments of approximately greater than 8 and less than 36 inches in diameter along with fines generated from the proposed cuts and overburden materials from removal areas. The blankets should be limited to 24 to 36 inches in thickness and should be placed with granular fines which are flooded into and around the rock fragments.
3. Rock blankets should be restricted to areas which are at least 1 foot below the lowest utility invert within the street right-of-way, and 10 feet below finish grade on the proposed fill lots.
4. Compaction may be achieved by utilizing wheel rolling methods with scrapers and water trucks, track-walking by bulldozers, and sheepsfoot tampers.
5. Each rock blanket should be completed with its surface compacted prior to placement of any subsequent rock blanket or rock windrow.

Materials Greater Than 36 Inches in Diameter

1. If encountered, oversize rock greater than 36 inches in diameter should be placed in single rock windrows. The windrows should be at least 15 feet or an equipment width apart, whichever is greatest, and at least 10 feet below final grade.
2. The void spaces between rocks in windrows should be filled with the more granular soils by flooding them into place.
3. A minimum vertical distance of 3 feet between soil fill and rock lift should be maintained. Also, the windrows should be staggered from lift to lift. Rock windrows should not be placed closer than 20 feet from the face of fill slopes.
4. Larger rocks too difficult to be placed into windrows may be individually placed into a dozer trench. Each trench should be excavated into the compacted fill or dense natural ground a minimum of 1 foot deeper than the size of the rock to be buried. After the rocks are placed in the trench (not immediately adjacent to each other), granular fill material should be flooded into the trench to fill the voids.
5. The oversize rock trenches should be no closer together than 15 feet at a particular elevation and at least 20 feet from any slope face. Trenches at higher elevations should be staggered and there should be 4 feet of compacted fill between the top of one trench and the bottom of the next higher trench. Placement of rock into these trenches should be under the full-time inspection of the soils engineer.
6. Consideration should be given to using oversize materials in open space "green belt" areas that would be designated as non-structural fills.

PRELIMINARY RECOMMENDATIONS - FOUNDATIONS

General

Preliminary recommendations for foundation design and construction are provided in the following sections. These preliminary recommendations have been developed from our understanding of the currently planned site development, site observations, subsurface exploration, laboratory testing, and engineering analyses. Foundation design should be re-evaluated at the conclusion of site grading and remedial earthwork for the as-graded soil conditions. Although not anticipated, revisions to these recommendations may be necessary.

The information and recommendations presented in this section are not meant to supersede design by the project structural engineer or civil engineer specializing in structural design. Upon request, GSI could provide additional input and consultation regarding soil parameters, as related to foundation design.

General recommendations for foundations are provided in the following sections, and are not intended to preclude the transmission of water or water vapor through the foundations or slabs. Further discussion and recommendations are provided within the soil moisture transmission considerations section of this report.

Expansive and Corrosive Soils

The laboratory testing conducted for this study indicates that the onsite soils do not meet the criteria of detrimentally expansive soils as defined in Section 1803.5.2 of the 2022 CBC. With recommended site grading, the overall expansive character of site soils is anticipated to be very low expansive.

Preliminary testing indicates that site soils present a negligible sulfate exposure to concrete (Exposure Class "S0" [per Table 19.3.2.1 of ACI 318-14]). However, reinforced concrete mix design for foundations, slab-on-grade floors, and pavements should also conform to Exposure Classes "S0," "W0," and "C1" in Table 19.3.1.1 of the ACI (2014), as concrete would likely be exposed to moisture.

General Foundation Design

The following foundation design recommendations are presented as minimum criteria from a soils engineering viewpoint.

1. The foundation systems should be designed and constructed in accordance with guidelines presented in the 2022 CBC.
2. An allowable bearing value of 2,000 pounds per square foot (psf) may be used for the design of footings that are founded entirely into properly compacted engineered fill or

suitable formation. This value may be increased by 10 percent for each additional 12 inches in footing depth, to a maximum value of 2,500 psf. These values may be increased by one-third when considering short duration seismic or wind loads. Foundations should not simultaneously bear on formation and engineered fill.

3. For foundations deriving passive resistance from engineered fill, a passive earth pressure may be computed as an equivalent fluid having a density of 150 pcf, with a maximum earth pressure of 2,250 psf.
4. The upper 6 inches of passive pressure should be neglected if not confined by slabs or pavement.
5. For lateral sliding resistance, a 0.25 coefficient of friction may be used for a concrete to soil contact when multiplied by the dead load.
6. When combining passive pressure and frictional resistance, the passive pressure component should be reduced by one third.
7. All footing setbacks from slopes should comply with Figure 1808.7.1 of the 2022 CBC. GSI recommends a minimum horizontal setback distance of 7 feet as measured from the bottom, outboard edge of the footing to the slope face.
8. 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.
9. All footing excavations should be observed by a GSI representative subsequent to trenching and prior to concrete form and reinforcement placement. The purpose of the observations is to evaluate that the excavations have been made into the recommended bearing material and to the minimum widths and depths recommended for construction.
10. If loose or compressible materials are exposed within the footing excavation, a deeper footing or removal and recompaction of the subgrade materials would be recommended at that time.
11. Footing trench spoil and any excess soils generated from trench excavations should be compacted to a minimum relative compaction of 90 percent, if not removed from the site.

Provided that the earthwork and foundation recommendations in this reported are adhered, foundations bearing on engineered fill should be designed to accommodate a total settlement of 1 inch, and a differential settlement of ½ inch over a 30-foot horizontal span. Based on the soil conditions, a seismic settlement of ½ inch and seismic differential settlement of ¼ inch over a 30-foot horizontal span are estimated.

Slab Subgrade Pre-Soaking

Pre-moistening of the slab subgrade soil is recommended. The moisture content of the subgrade soils should be equal to or greater than optimum moisture to a depth equivalent to the perimeter grade beam or cut-off wall depth in the slab areas (typically 12 inches for very low expansive soil conditions).

Pre-moistening or pre-soaking should be evaluated by the soils engineer 72 hours prior to vapor retarder placement. In summary:

EXPANSION POTENTIAL	PAD SOIL MOISTURE	CONSTRUCTION METHOD	SOIL MOISTURE RETENTION
Very Low (E.I. = 0-20)	Upper 12 inches of pad at or above soil optimum moisture	Wetting or reprocessing	Periodically wet or cover with plastic after trenching. Evaluation 72 hours prior to placement of concrete.

Perimeter Cut-Off Walls

Perimeter cut-off walls should be at least 12 inches deep (below the lowest adjacent grade) for very low expansive soil conditions. The cut-off walls may be integrated into the slab design or independent of the slab. The cut-off walls should be a minimum of 6 inches thick (wide).

SOIL MOISTURE TRANSMISSION CONSIDERATIONS

GSI has evaluated the potential for moisture or water vapor transmission through the concrete floor slab, in light of typical floor coverings and improvements. Slab moisture emission rates range from about 2 to 27 lbs/24 hours/1,000 square feet for a typical slab (Kanare, 2005), while floor covering manufacturers generally recommend about 3 lbs/24 hours as an upper limit. The recommendations in this section are not intended to preclude the transmission of moisture or water vapor through the foundation or slabs. Foundation systems and slabs should not allow moisture or water vapor to enter into structures so as to cause damage to another building component or to limit the installation of the type of flooring materials typically used for the particular application (State of California, 2025). These recommendations may be exceeded or supplemented by a “waterproofing” consultant, the project architect, or the structural consultant. Thus, the client will need to evaluate the following in light of a cost vs. benefit analysis (owner expectations and repairs/replacement), along with disclosure to all interested/affected parties.

Moisture or water vapor transmission will occur in a new concrete slab-on-grade floor as a result of chemical reactions taking place within the curing concrete. Moisture or water vapor transmission through concrete floor slabs as a result of concrete curing has the potential to adversely affect sensitive floor coverings depending on the thickness of the concrete floor slab and the duration of time between the placement of concrete, and the floor covering installation. It is possible that a slab moisture sealant may be needed prior to the placement of sensitive floor coverings if a thick slab-on-grade floor is used and the time frame between concrete and floor covering placement is relatively short.

Considering the E.I. test results presented herein, the known soil conditions in the region, the anticipated typical moisture or water vapor transmission rates, floor coverings, and improvements (to be chosen by the client and project architect) that can tolerate moisture or water vapor transmission rates without significant distress, the following alternatives are provided:

- Construct a thicker concrete slab-on-grade floor.
- Concrete slab-on-grade floor underlayment should consist of a 15-mil vapor retarder, or equivalent, with all laps sealed per the 2022 CBC and the manufacturer's recommendations. The vapor retarder should comply with ASTM E 1745 - Class A criteria (i.e., Stego Wrap, Husky Yellow Guard, or approved equivalent), and be installed in accordance with ACI 302.1R-15 and ASTM E 1643.
- The 15-mil vapor retarder (ASTM E 1745 - Class A) should be installed per the recommendations of the manufacturer, including all penetrations (i.e., pipe, ducting, rebar, etc.).
- The concrete slab-on-grade floor should be immediately underlain by a sand cushion consisting of 2 inches of clean sand ($SE \geq 30$), placed atop a 15-mil vapor retarder (ASTM E-1745 -Class A, per Engineering Bulletin 119 [Kanare, 2005]) that is installed per the recommendations of the manufacturer, including all penetrations (i.e., pipe, ducting, rebar, etc.). The manufacturer shall provide instructions for lap sealing, including minimum width of lap, method of sealing, and either supply or specify suitable products for lap sealing (ASTM E 1745), and per Code.

ACI 302.1R-15 (ACI, 2015) states, "Experience has shown, however, that the greatest level of protection for floor coverings, coatings, or building environments is provided when the vapor retarder/barrier is placed in direct contact with the slab. Placing concrete in direct contact with the vapor retarder/barrier eliminates the potential for water from sources such as rain, saw-cutting, curing, cleaning, or compaction to become trapped within the fill course. Wet or saturated fill above the vapor retarder/barrier can significantly lengthen the time required for a slab to dry to a level acceptable to the manufacturers of floor coverings, adhesives, and coatings. A fill layer sandwiched between the vapor retarder/barrier and the concrete slab-on-grade floor also serves as an avenue for moisture to enter and travel freely beneath

the slab, which can lead to an increase in moisture within the slab once it is covered. Moisture can enter the fill layer through voids, tears, or punctures in the vapor retarder/barrier.” Therefore, additional observation and testing will be necessary for the cushion or sand layer for moisture content, and relatively uniform thicknesses, prior to the placement of concrete.

- Additional concrete mix design recommendations should be provided by the structural consultant or waterproofing specialist. Concrete finishing and workability should be addressed by the structural consultant and a waterproofing specialist.
- Where concrete admixtures are used, the structural consultant should also make changes to the concrete in the grade beams and footings in kind, so that the concrete used in the foundation and floor slab is designed or treated for more uniform moisture protection.
- The owner should be specifically advised which building areas are suitable for tile flooring, vinyl flooring, or other types of moisture and vapor-sensitive flooring and which building areas are not suitable for these types of flooring applications. In all planned floor areas, flooring shall be installed per the manufacturer’s recommendations.
- Additional recommendations regarding moisture or water vapor transmission should be consistent with the specified floor coverings indicated by the architect.

Regardless of the mitigation, some limited moisture and water vapor transmission through the foundation and concrete slab-on-grade floor cannot be entirely precluded and should be anticipated. Construction crews may require special training for installation of certain product(s), as well as concrete finishing techniques. The use of specialized product(s) should be approved by the slab designer, project architect, and the waterproofing consultant.

PRELIMINARY RETAINING WALL DESIGN PARAMETERS

General

Based on our review, the majority of onsite earth materials are derived from weathering of plutonic rocks which exhibit a very low expansion potential (E.I. 0-20). These materials appear to predominantly consist of sands and silty sands with gravel and minor amounts of clay content. Recommendations for the design and construction of conventional masonry retaining walls are provided herein. Recommendations for specialty walls (e.g., crib, earthstone, geogrid) can be provided upon request, and would be based on site specific conditions. If walls allow water to accumulate in the backfill or at their toe via a water quality basin, the water should be conveyed via a non-erosive device to an appropriate inlet, per the recommendations of the design civil engineer.

Conventional Retaining Walls

The design parameters provided below assume native onsite materials with an expansion index up to an E.I. of 20 are used to backfill any retaining wall. The type of backfill, should be specified by the wall designer, and clearly shown on the plans. Building walls, below grade, should be water-proofed. Waterproofing should also be provided for site retaining walls in order to reduce the potential for efflorescence staining.

Preliminary Retaining Wall Foundation Design

The preliminary foundation design for conventional retaining walls should incorporate the following recommendations:

Minimum Footing Embedment - 18 inches below the lowest adjacent grade (excluding landscape layer [upper 6 inches]).

Minimum Footing Width - 24 inches.

Allowable Bearing Pressure - An allowable vertical bearing pressure of 2,500 pcf may be used in the preliminary design of retaining wall foundations provided that the footing maintains a minimum width of 24 inches and extends at least 18 inches into approved engineered fill overlying dense formational materials. This pressure may be increased by one-third for short-term wind and seismic loads.

Passive Earth Pressure - A passive earth pressure of 200 pcf with a maximum earth pressure of 2,500 psf may be used in the preliminary design of retaining wall foundations provided the foundation is embedded into properly compacted silty to clayey sand fill.

Lateral Sliding Resistance - A 0.25 coefficient of friction may be used for a concrete to soil contact when multiplied by the dead load. When combining passive pressure and frictional resistance, the passive pressure component should be reduced by one-third.

Backfill Soil Density - A soil density of 120 to 130 pcf may be used in the design of retaining wall foundations. This assumes an average engineered fill compaction of at least 90 percent of the laboratory standard (ASTM D 1557).

Footing Setbacks - All retaining wall footing setbacks from slopes should comply with Figure 1808.7.1 of the 2022 CBC. GSI recommends a minimum horizontal setback distance of 7 feet, as measured from the bottom, outboard edge of the footing to the face to the 2:1 (h:v) slope face.

Settlement - Provided that the earthwork and foundation recommendations in this report are adhered to, foundations bearing on approved non-detrimentally expansive,

engineered fill should be designed to accommodate a total static settlement of 1 inch and a differential static settlement of ½ inch over a 30-foot horizontal span.

Any retaining wall footings near the perimeter of the site, or not within areas of placed compacted fills will likely need to be deepened into unweathered dense formational materials for adequate vertical and lateral bearing support.

Restrained Walls

Any retaining wall that will be restrained prior to placing and compacting backfill material or retaining walls that have re-entrant or male corners, should be designed for an at-rest equivalent fluid pressure (EFP) of 59 pcf. The design should include any applicable surcharge loading. For areas of male or re-entrant corners, the restrained wall design should extend a minimum distance of twice the height of the wall (2H) laterally from the corner.

Cantilevered Walls

The recommendations presented below are for cantilevered retaining walls with retained soil heights up to 10 feet. Design parameters for retaining walls less than 3 feet in height may be superseded by the San Bernardino County regional standard design. Active earth pressure may be used for retaining wall design, provided the top of the wall is not restrained from minor deflections. An equivalent fluid pressure approach may be used to compute the horizontal pressure against the wall. Appropriate fluid unit weights are given below for specific slope gradients of the retained material. These do not include other superimposed loading conditions due to traffic, structures, seismic events, or adverse geologic conditions. When wall configurations are finalized, the appropriate loading conditions for superimposed loads can be provided upon request.

For preliminary planning purposes, the structural consultant or wall designer should incorporate the surcharge of traffic on the back of retaining walls, if traffic will occur within a horizontal distance of “H” from the backside of the retaining walls, where “H” equals the retained soil height. The traffic surcharge may be taken as 100 psf/ft in the upper 5 feet of the wall for light passenger vehicle traffic (i.e., cars, vans, pick-up trucks, etc.). This does not include the surcharge of parked vehicles which should be evaluated at a higher surcharge to account for the effects of seismic loading. Equivalent fluid pressures for the design of cantilevered retaining walls are provided in the following table:

SURFACE SLOPE OF RETAINED MATERIAL (HORIZONTAL:VERTICAL)	EQUIVALENT FLUID WEIGHT P.C.F. ⁽²⁾
Level ⁽¹⁾	38
2:1 (h:v)	58

Seismic Surcharge

The Mononobe-Okabe method is commonly utilized for calculating seismically induced active and passive lateral earth pressures and is based on the limit equilibrium Coulomb theory for static stress conditions. This method entails three fundamental assumptions (e.g., Seed and Whitman, 1970): Wall movement is sufficient to ensure either active or passive conditions, the driving soil wedge inducing the lateral earth pressures is formed by a planar failure surface starting at the heel of the wall and extending to the free surface of the backfill, and the driving soil wedge and the retaining structure act as rigid bodies, and therefore, experiences uniform accelerations throughout the respective bodies (U.S. Army Corps of Engineers, 2003, Engineering and Design - Stability Analysis of Concrete Structures).

- Seismic Lateral Earth Pressure = 35 pcf (equivalent fluid weight).

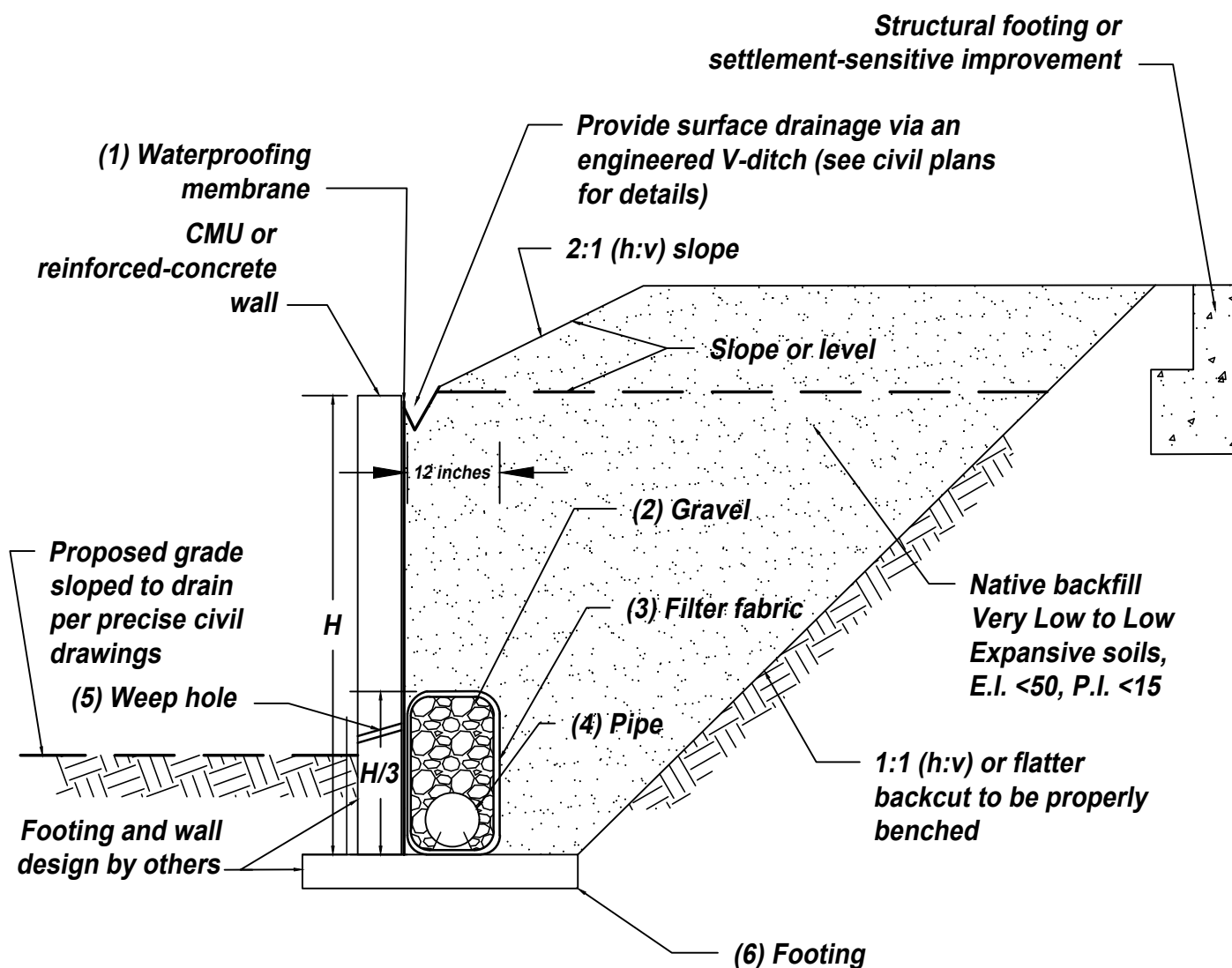
The seismic lateral earth pressure given above is a triangle increasing with depth, and the resultant of this pressure is an increment of force which should be applied to the back of the wall at 1/3 of the wall height from the wall base. The seismic increment of earth pressure should be added to the static active earth pressure. Even for the at-rest (K_0) condition, the seismic increment of earth pressure should be added to the static active earth pressure, not to the at-rest static earth pressure (SEAOC Seismology Committee, 2019).

Per 2019 CBC Section 1803.5.12 dynamic seismic lateral earth pressures shall be applied to foundation walls and retaining walls supporting more than 6 feet of backfill. Dynamic seismic lateral earth pressures may also be applied to shorter walls at the discretion of the structural engineer.

Retaining Wall Backfill and Drainage

Positive drainage must be provided behind all retaining walls in the form of gravel wrapped in geofabric and outlets. A backdrain system is considered necessary for retaining walls that are 2 feet or greater in height. Details 1, 2, and 3, present the backdrainage options discussed below. Backdrains should consist of a 4-inch diameter perforated PVC or ABS pipe encased in either Class 2 permeable filter material or $\frac{3}{4}$ -inch to $1\frac{1}{2}$ -inch gravel wrapped in approved filter fabric (Mirafi 140 or equivalent). The filter material should extend a minimum of 1 horizontal foot behind the base of the walls and upward at least 1 foot. . For limited access and confined areas, (panel) drainage behind the wall may be constructed in accordance with Detail 2 (Retaining Wall Backfill and Subdrain Detail Geotextile Drain).

Materials with an expansion index (E.I.) potential of greater than 50 should not be used as backfill for retaining walls. Retaining wall backfill materials should be moisture conditioned and mixed to achieve the soil's optimum moisture content, placed in relatively thin lifts (6 to 8 inches) with relatively light equipment, and compacted to at least 90 percent relative compaction.



(1) Waterproofing membrane.

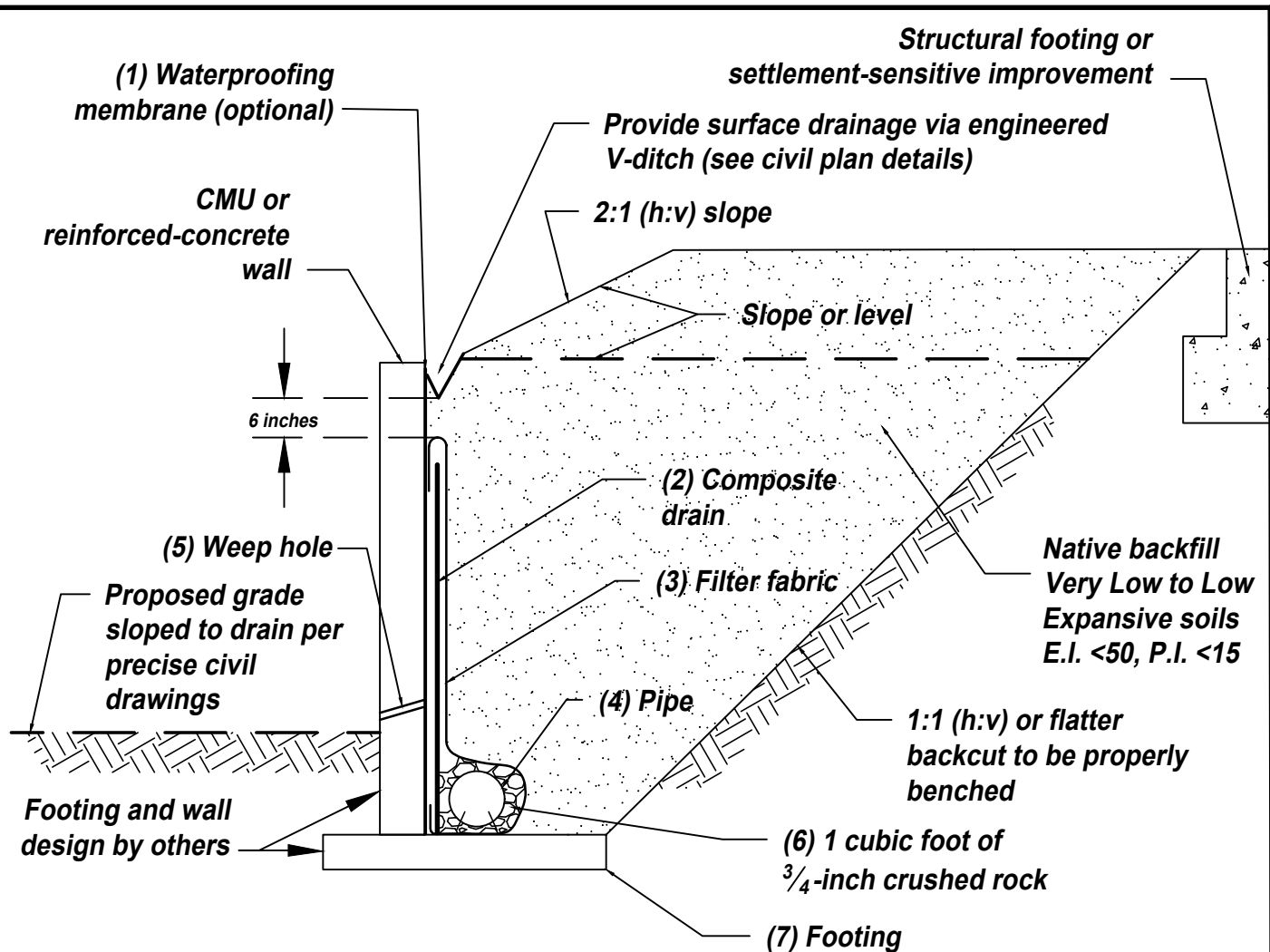
(2) Gravel: Clean, crushed, $\frac{3}{4}$ to $1\frac{1}{2}$ inch.

(3) Filter fabric: Mirafi 140N or approved equivalent.

(4) Pipe: 4-inch-diameter perforated PVC, Schedule 40, or approved alternative with minimum of 1 percent gradient sloped to suitable, approved outlet point (perforations down).

(5) Weep holes: For CMU walls, Omit grout every other block, at or slightly above finished surface. For reinforced concrete walls, minimum 2-inch diameter weep holes spaced at 20 foot centers along the wall and placed 3 inches above finished surface. Design civil engineer to provide drainage at toe of wall. No weep holes for below-grade walls.

(6) Footing: If bench is created behind the footing greater than the footing width using level fill or cut natural earth materials, an additional "heel" drain will likely be required by geotechnical consultant.



(1) Waterproofing membrane (optional): Liquid boot or approved mastic equivalent.

(2) Drain: Miradrain 6000 or J-drain 200 or equivalent for non-waterproofed walls; Miradrain 6200 or J-drain 200 or equivalent for waterproofed walls (all perforations down).

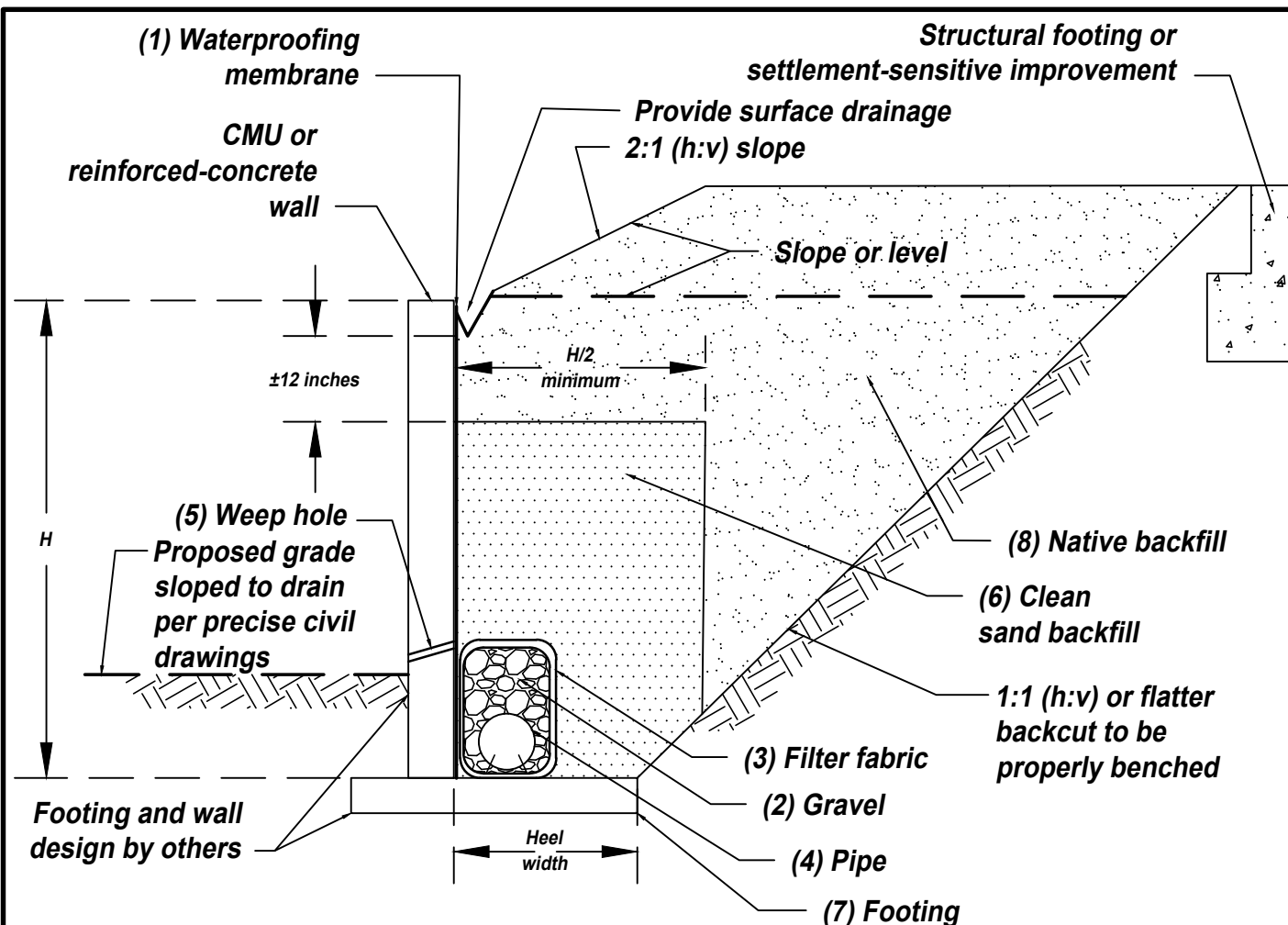
(3) Filter fabric: Mirafi 140N or approved equivalent; place fabric flap behind core.

(4) Pipe: 4-inch-diameter perforated PVC, Schedule 40, or approved alternative with minimum of 1 percent gradient to proper outlet point (perforations down).

(5) Weep holes: For CMU walls, Omit grout every other block, at or slightly above finished surface. For reinforced concrete walls, minimum 2-inch diameter weep holes spaced at 20 foot centers along the wall and placed 3 inches above finished surface. Design civil engineer to provide drainage at toe of wall. No weep holes for below-grade walls.

(6) Gravel: Clean, crushed, 3/4 to 1 1/2 inch.

(7) Footing: If bench is created behind the footing greater than the footing width using level fill or cut natural earth materials, an additional "heel" drain will likely be required by geotechnical consultant.



(1) Waterproofing membrane: Liquid boot or approved mastic equivalent.

(2) Gravel: Clean, crushed, $\frac{3}{4}$ to $1\frac{1}{2}$ inch.

(3) Filter fabric: Mirafi 140N or approved equivalent.

(4) Pipe: 4-inch-diameter perforated PVC, Schedule 40, or approved alternative with minimum of 1 percent gradient to proper outlet point (perforations down).

(5) Weep hole: For CMU walls, Omit grout every other block, at or slightly above finished surface. For reinforced concrete walls, minimum 2-inch diameter weep hole spaced at 20 foot centers along the wall and placed 3 inches above finished surface. Design civil engineer to provide drainage at toe of wall. No weep holes for below-grade walls.

(6) Clean sand backfill: Must have sand equivalent value (S.E.) of 35 or greater; can be densified by water jetting upon approval by geotechnical engineer.

(7) Footing: If bench is created behind the footing greater than the footing width using level fill or cut natural earth materials, an additional "heel" drain will likely be required by geotechnical consultant.

(8) Native backfill: If E.I. < 21 and S.E. > 35 then all sand requirements also may not be required and will be reviewed by the geotechnical consultant.

Outlets should consist of a 4-inch diameter solid PVC or ABS pipe spaced no greater than approximately 100 feet apart, with a minimum of two outlets, one on each end. The use of weep holes, only, in walls higher than 2 feet, is not recommended. The surface of the backfill should be sealed by pavement or the top 18 inches compacted with native soil ($E.I. \leq 20$). Proper surface drainage should also be provided. For additional mitigation, consideration should be given to applying a water-proof membrane to the back of all retaining structures. The use of a waterstop should be considered for all concrete and masonry joints.

Wall and Retaining Wall Footing Transitions

Site walls are anticipated to be founded on footings designed in accordance with the recommendations in this report. Should wall footings transition from cut to fill, the structural consultant or wall designer may specify either:

- a) A minimum of a 2-foot overexcavation and recompaction of cut materials for a distance of $2H$, from the point of transition, where “H” equals the wall height.
- b) Increase the amount of reinforcing steel and wall detailing (i.e., expansion joints or crack control joints) such that an angular distortion of $1/360$ for a distance of $2H$ on either side of the transition may be accommodated. Expansion joints should be placed no greater than 20 feet on-center, in accordance with the structural engineer’s or wall designer’s recommendations, regardless of whether or not a transition condition exists. Expansion joints should be sealed with a flexible, non-shrink grout.
- c) Embed the footings entirely into suitable native formational material (i.e., deepened footings).

If transitions from cut to fill transect the wall footing alignment at an angle of less than 45 degrees (plan view), then the designer should follow recommendation “a” (above) and until such transition is between 45 and 90 degrees to the wall alignment.

Slope Setback Considerations for Footings

Footings should maintain a horizontal distance, X , between any adjacent descending slope face and the bottom outer edge of the footing, and minimally comply with the guidelines depicted on Figure 1808.7.1 of the 2022 CBC. The horizontal distance, X , may be calculated by using $X = h/3$, where h is the height of the slope. X should not be less than 7 feet, nor need not be greater than 40 feet. X may be maintained by deepening the footings.

PRELIMINARY PAVEMENT DESIGN RECOMMENDATIONS

General

The governing agency may retain the authority to approve the final structural design sections after subgrade elevations and actual resistance values (R-values) have been obtained at the conclusion of earthwork. Based on an R-value of 78, general review of pavement designs for other nearby projects, and for estimation and bidding purposes, the pavement sections provided herein should be considered for preliminary design. Typically, actual pavement sections will likely vary, therefore final pavement sections should be based on actual R-value testing performed during, or shortly after, roadway grading for any proposed street improvements.

Asphaltic Concrete (AC) Pavements

The preliminary design for Asphaltic Concrete (AC) was evaluated based on a tested R-value of R=78 and the use of concrete shoulders (curb or gutter) at the edge of pavement. GSI does not recommend the use of an ADTT value of less than 25 for any pavement section, unless the ADTT significantly less than 25 is certified by a civil engineer specializing in traffic engineering.

The preliminary pavement sections presented in the following tables are based on R-value testing, traffic indices likely for the project (to be assigned by the project civil engineer), and the guidelines presented in the latest revision to the California Department of Transportation "Highway Design Manual," seventh edition. It is our understanding that the minimum asphalt section required by the City of Rialto - Public Works Department (COR, 2011) for both a local street and a collector street is 4 inches of asphaltic concrete (AC), on 6 inches of Class 2 aggregate base. Based on the R-value testing conducted, the following preliminary pavement design sections are presented.

STREET CLASSIFICATION	ASSUMED TRAFFIC INDEX (T.I.) ¹	STANDARD PAVEMENT DESIGNS		
		R-VALUE	AC INCHES*	CLASS 2 BASE ROCK ² INCHES*
Interior/Local Streets	5.0	78	4.0*	6.0*
Collector Street	6.0	78	4.0*	6.0*
Secondary Arterial Street	8.0	78	5.0*	6.0*
¹ T.I.s are selected by the civil consultant's evaluation of traffic patterns and type. ² Assumed R-values for base rock R=78 - Greenbook or Cal-Trans standard Class 2 base rock. * City/County minimum asphaltic concrete or crushed aggregate base requirements may vary.				

The preliminary pavement sections provided above are intended as a minimum guideline. If thinner or highly variable pavement sections are constructed, increased maintenance and repair could be expected. If the ADT (average daily traffic) or ADTT (average daily truck traffic) increases beyond that intended, as reflected by the T.I. used for design, increased maintenance and repair could be required for the pavement section. Consideration should be given to the increased potential for distress from overuse of paved street areas by heavy equipment or construction related heavy traffic (e.g., concrete trucks, loaded supply trucks), particularly when the final section is not in place (i.e., topcoat). Best management construction practices should be followed at all times, especially during inclement weather.

Portland Concrete Cement (PCC) Pavement

The preliminary design for Portland Cement Concrete Pavement (PCCP) was evaluated using a subgrade R-value of 78, and a modulus of rupture (MR) of 400 and 450 psi. GSI does not recommend the use of an ADTT value of less than 25 for any pavement section, unless the ADTT significantly less than 25 is certified by a civil engineer specializing in traffic engineering. The preliminary PCCP sections are provided in the following table.

PORTLAND CONCRETE CEMENT PAVEMENTS (PCCP)*					
TRAFFIC AREAS	CONCRETE TYPE	PCP THICKNESS (INCHES)	TRAFFIC AREAS	CONCRETE TYPE	PCCP THICKNESS (INCHES)
Light Vehicles	520-C-2500	6.0	Dumpster Aprons (Trash Service)	520-C-2500	7.0
	560-C-3250	5.0		560-C-3250	6.0
NOTE: All PCCP is designed as un-reinforced and bearing directly on compacted subgrade. However, a 4-inch thick leveling course of compacted aggregate base, or crushed rock may be considered to improve performance. All PCCP should be properly detailed (jointing, etc.) per the industry standard. Pavements may be additionally reinforced with #4 reinforcing bars, placed 18 inches on center, each way, for improved performance.					
* To be re-evaluated based on exposed field conditions and R-values obtained following rough grading.					

The transition of the pavement from parking to traffic lanes should be made over a distance of 24 inches, with crack control joints (weaken plane) or contact joints at the end of the transition. A minimum 4-inch layer of base rock, in traffic and bus stop areas, should be considered to improve traffic lane performance. Base rock may consist of either ¾-inch crushed rock or Caltrans Class 2 aggregate base. Crushed rock may be compacted by vibratory methods. Aggregate base should be compacted to a minimum relative compaction of 95 percent.

Weakened Plane Joints

Transverse and longitudinal weakened plane joints may be constructed per Caltrans standard specifications. Transverse weakened plane joints should be spaced no farther than 15 feet apart, and no closer than 5 feet. Longitudinal weakened plane joints should be spaced no farther than 20 feet apart, and no less than 5 feet.

Expansion Joints and Contact Joints

Transverse expansion joints should be constructed at 120-foot spacings, or in accordance with City standards. Transverse and longitudinal contact joints should be constructed in accordance with the recommendations of the design engineer. Within large slab areas, joint spacings should be no greater than 20 feet.

Slab Reinforcement

The preliminary PCC Pavements for this project are designed as unreinforced and should perform adequately, assuming proper construction.

The upper 12 inches of subgrade should be compacted to a minimum relative compaction of 95 percent of the maximum dry density (ASTM D 1557). Aggregate base compaction should be 95 percent of the maximum dry density (ASTM D 1557). If adverse conditions (i.e., saturated ground, etc.) are encountered during preparation of subgrade, special construction methods may need to be employed. These recommendations should be considered preliminary. R-value testing and pavement design analysis should be performed upon completion of grading for the project.

PAVEMENT GRADING RECOMMENDATIONS

General

All section changes should be properly transitioned. If adverse conditions are encountered during the preparation of subgrade materials, special construction methods may need to be employed. A GSI representative should be present for the preparation of subgrade, base rock, and asphalt concrete.

Subgrade

Within street and parking areas, all surficial deposits of loose soil material should be removed and recompact as recommended. After the loose soils are removed, the bottom is to be scarified to a depth of at least 6 inches, moisture conditioned as necessary and compacted to 95 percent of the maximum laboratory density or the County of San Bernardino minimum, as specified by ASTM D 1557.

Deleterious material, excessively wet or dry pockets, concentrated zones of oversized rock fragments, and any other unsuitable materials encountered during grading should be removed. The compacted fill material should then be brought to the elevation of the proposed subgrade for the pavement. The subgrade should be proof-rolled in order to promote a uniform firm and unyielding surface. All grading and fill placement should be observed by the project geotechnical consultant.

Crushed Aggregate Base Rock

Compaction tests are required for the recommended base section. Minimum relative compaction required will be 95 percent of the laboratory maximum density as determined by ASTM test method D 1557 or Caltrans Test Method Number California 216. Base aggregate should be in accordance to the Caltrans Class 2 base rock (minimum R-value=78).

Paving

Prime coat may be omitted if all of the following conditions are met:

1. The asphalt pavement layer is placed within two weeks of completion of base or subbase course.
2. Traffic is not routed over completed base before paving
3. Construction is completed during the dry season of May through October.
4. The base is kept free of debris prior to placement of asphaltic concrete.

If construction is performed during the wet season of November through April, prime coat may be omitted if no rain occurs between completion of base course and paving and the time between completion of base and paving is reduced to three days, provided the base is free of loose soil or debris. Where prime coat has been omitted and rain occurs, traffic is routed over base course, or paving is delayed, measures shall be taken to restore base course, and subgrade to conditions that will meet specifications as directed by the geotechnical consultant.

Drainage

Positive drainage should be provided for all surface water to drain towards the area swale, curb and gutter, or to an approved drainage channel. Positive site drainage should be maintained at all times. Water should not be allowed to pond or seep into the ground, such as from behind unprotected curbs, both during and after grading. If planters or landscaping are adjacent to paved areas, measures should be taken to minimize the potential for water to enter the pavement section, such as thickened edges, and enclosed planters. Also, best management construction practices should be strictly adhered to at all times to minimize the potential for distress during construction and roadway improvements.

Additional Considerations

To mitigate perched groundwater, consideration should be given to installation of subgrade separators (cut-offs) between pavement subgrade and landscape areas, although this is not a requirement from a geotechnical standpoint. Cut-offs, if used, should be 6 inches wide and at least 12 inches below the pavement subgrade contact or 12 inches below the crushed aggregate base rock, if used.

DRIVEWAY, CONCRETE APRONS, FLATWORK, AND OTHER IMPROVEMENTS

The soil materials on site may be expansive. The effects of expansive soils are cumulative, and typically occur over the lifetime of any improvements. On relatively level areas, when the soils are allowed to dry, the desiccation and swelling process tends to cause heaving and distress to flatwork and other improvements. The resulting potential for distress to improvements may be reduced, but not totally eliminated. To that end, it is recommended that the developer should notify any owners and interested/affected parties of this long-term potential for distress. To reduce the likelihood of distress, the following recommendations are presented for all exterior flatwork:

1. The subgrade area for concrete slabs should be compacted to achieve a minimum 90 percent relative compaction, and then be presoaked to 2 to 3 percentage points above (or 125 percent of) the soils' optimum moisture content, to a depth of 18 inches below subgrade elevation. If very low expansive soils are present, only optimum moisture content, or greater, is required and specific presoaking is not warranted. The moisture content of the subgrade should be proof tested within 72 hours prior to concrete placement.
2. Exterior concrete slabs should be cast over a non-yielding surface, consisting of a 4-inch layer of Class 2 base, crushed rock, gravel, or clean sand (or City/County minimum, whichever is greater), that should be compacted and level prior to placement of concrete. If very low expansive soils are present, the base, rock, gravel, or sand may be deleted. The layer or subgrade should be wet-down completely prior to placement of concrete, to minimize loss of concrete moisture to the surrounding earth materials.
3. Exterior slabs should be a minimum of 4 inches thick. Driveway slabs and approaches should additionally have a thickened edge (12 inches) adjacent to all landscape areas, to help impede infiltration of landscape water under the slab. Trash disposal (dumpster) area aprons should be a minimum of 5 inches thick and meet minimum City standards, as necessary.
4. The use of transverse and longitudinal control joints is recommended to help control slab cracking due to concrete shrinkage or expansion.

For sidewalks or narrow slabs, control joints should be provided at intervals of every 6 feet. The slabs should be separated from the foundations and sidewalks with expansion joint filler material.
5. No traffic should be allowed upon the newly placed concrete slabs until they have been properly cured to within 75 percent of design strength. Concrete compression strength should be a minimum of 2,500 psi.

6. Driveways, sidewalks, and patio slabs adjacent to the structure should be separated from the structure with thick expansion joint filler material. In areas directly adjacent to a continuous source of moisture (e.g., irrigation, planters), all joints should be additionally sealed with flexible mastic.
7. Planters and walls (sound walls or retaining walls) should not be tied to the structure.
8. Overhang structures should be supported on the slabs, or structurally designed with continuous footings tied in at least two directions.
9. Any masonry landscape or sound walls that are to be constructed throughout the property should be grouted and articulated in segments no more than 20 feet long. These segments should be keyed or doweled together.
10. If settlement concerns or expansive soils are present, utilities may be enclosed within a closed utilidor (vault) or designed with flexible connections to accommodate differential settlement and expansive soil conditions.
11. Positive site drainage should be maintained at all times. Finish grade on the building pad should provide a minimum of 1 to 2 percent fall to the street, as indicated herein. Drainage reversals could occur, including post-construction settlement, if relatively flat drainage gradients are not periodically maintained by the owner and interested/affected parties.
12. Air conditioning (A/C) units should be supported by slabs that are incorporated into the building foundation or constructed on a rigid slab with flexible couplings for plumbing and electrical lines. A/C waste water lines should be drained to a suitable non-erosive outlet.
13. Shrinkage cracks could become excessive if proper finishing and curing practices are not followed. Finishing and curing practices should be performed per the Portland Cement Association (PCA) guidelines. Mix design should incorporate rate of curing for climate and time of year, sulfate content of soils, corrosion potential of soils, and fertilizers used on site.

DEVELOPMENT CRITERIA

Slope Maintenance and Planting

Water has been shown to weaken the inherent strength of all earth materials. Slope stability is significantly reduced by overly wet conditions. Positive surface drainage away from slopes should be maintained and only the amount of irrigation necessary to sustain plant life should be provided for planted slopes. Over-watering should be avoided as it adversely affects site improvements, and causes perched groundwater conditions. Graded slopes constructed

utilizing onsite materials would be erosive. Eroded debris may be minimized and surficial slope stability enhanced by establishing and maintaining a suitable vegetation cover soon after construction. Compaction to the face of fill slopes would tend to minimize short-term erosion until vegetation is established. Plants selected for landscaping should be light weight, deep rooted types that require little water and are capable of surviving the prevailing climate. Jute-type matting or other fibrous covers may aid in allowing the establishment of a sparse plant cover. Utilizing plants other than those recommended above will increase the potential for perched water, staining, mold, etc., to develop. A rodent control program to prevent burrowing should be implemented. Irrigation of natural (ungraded) slope areas is generally not recommended. These recommendations regarding plant type, irrigation practices, and rodent control should be provided to each homeowner. Over-steepening of slopes should be avoided during building construction activities and landscaping.

Drainage

Adequate lot surface drainage is a very important factor in reducing the likelihood of adverse performance of foundations, hardscape, and slopes. Surface drainage should be sufficient to prevent ponding of water anywhere on a lot, and especially near structures and tops of slopes. Lot surface drainage should be carefully taken into consideration during fine grading, landscaping, and building construction. Therefore, care should be taken that future landscaping or construction activities do not create adverse drainage conditions. Positive site drainage within lots and common areas should be provided and maintained at all times. Drainage should not flow uncontrolled down any descending slope. Water should be directed away from foundations and not allowed to pond or seep into the ground. In general, the area within 5 feet around a structure should slope away from the structure. We recommend that unpaved lawn and landscape areas have a minimum gradient of 1 percent sloping away from structures, and whenever possible, should be above adjacent paved areas. Consideration should be given to avoiding construction of planters adjacent to structures (buildings, pools, spas, etc.). Pad drainage should be directed toward the street or other approved area(s). Although not a geotechnical requirement, roof gutters, downspouts, or other appropriate means may be used to control roof drainage. Downspouts, or drainage devices, should outlet a minimum of 5 feet from structures or into a subsurface drainage system. Areas of seepage may develop due to irrigation or heavy rainfall, and should be anticipated. Minimizing irrigation will lessen this potential. If areas of seepage develop, recommendations for minimizing this effect could be provided upon request.

Erosion Control

Cut and fill slopes will be subject to surficial erosion during and after grading. Onsite earth materials have a moderate to high erosion potential. Consideration should be given to providing hay bales and silt fences for the temporary control of surface water, from a geotechnical viewpoint.

Landscape Maintenance

Only the amount of irrigation necessary to sustain plant life should be provided. Over-watering the landscape areas will adversely affect proposed site improvements. We would recommend that any proposed open-bottom planters adjacent to proposed structures be eliminated for a minimum distance of 10 feet. As an alternative, closed-bottom type planters could be utilized. An outlet placed in the bottom of the planter, could be installed to direct drainage away from structures or any exterior concrete flatwork. If planters are constructed adjacent to structures, the sides and bottom of the planter should be provided with a moisture barrier to prevent penetration of irrigation water into the subgrade. Provisions should be made to drain the excess irrigation water from the planters without saturating the subgrade below or adjacent to the planters. Graded slope areas should be planted with drought resistant vegetation. Consideration should be given to the type of vegetation chosen and their potential effect upon surface improvements (i.e., some trees will have an effect on concrete flatwork with their extensive root systems). From a geotechnical standpoint leaching is not recommended for establishing landscaping. If the surface soils are processed for the purpose of adding amendments, they should be recompact to 90 percent minimum relative compaction.

Gutters and Downspouts

As previously discussed in the drainage section, the installation of gutters and downspouts should be considered to collect roof water that may otherwise infiltrate the soils adjacent to the structures. If utilized, the downspouts should be drained into PVC collector pipes or other non-erosive devices (e.g., paved swales or ditches; below grade, solid tight-lined PVC pipes), that will carry the water away from the house, to an appropriate outlet, in accordance with the recommendations of the design civil engineer. Downspouts and gutters are not a requirement; however, from a geotechnical viewpoint, provided that positive drainage is incorporated into project design (as discussed previously).

Subsurface and Surface Water

Subsurface and surface water are not anticipated to affect site development, provided that the recommendations contained in this report are incorporated into final design and construction and that prudent surface and subsurface drainage practices are incorporated into the construction plans. Perched groundwater conditions along zones of contrasting permeabilities may not be precluded from occurring in the future due to site irrigation, poor drainage conditions, or damaged utilities, and should be anticipated. Should perched groundwater conditions develop, this office could assess the affected area(s) and provide the appropriate recommendations to mitigate the observed groundwater conditions. Groundwater conditions may change with the introduction of irrigation, rainfall, or other factors.

Site Improvements

If in the future, any additional improvements (e.g., pools, spas) are planned for the site, recommendations concerning the geological or geotechnical aspects of design and construction of said improvements could be provided upon request. Pools or spas should not be constructed without specific design and construction recommendations from GSI, and this construction recommendation should be provided to the homeowners, any homeowners association, or other interested parties. This office should be notified in advance of any fill placement, grading of the site, or trench backfilling after rough grading has been completed (e.g., any grading, utility trench and retaining wall backfills, flatwork)

Tile Flooring

Tile flooring can crack, reflecting cracks in the concrete slab below the tile, although small cracks in a conventional slab may not be significant. Therefore, the designer should consider additional steel reinforcement for concrete slabs-on-grade where tile will be placed. The tile installer should consider installation methods that reduce possible cracking of the tile such as slipsheets. Slipsheets or a vinyl crack isolation membrane (approved by the Tile Council of North America or Ceramic Tile Institute of America) are recommended between tile and concrete slabs on grade.

Additional Grading

This office should be notified in advance of any fill placement, supplemental regrading of the site, or trench backfilling after rough grading has been completed. This includes completion of grading in the street, driveway approaches, driveways, parking areas, and utility trench and retaining wall backfills.

Trenching and Temporary Construction Backcuts

Considering the nature of the onsite earth materials, it should be anticipated that caving or sloughing could be a factor in subsurface excavations and trenching. Shoring or excavating the trench walls or backcuts at the angle of repose (typically 25 to 45 degrees [except as specifically superseded within the text of this report]), should be anticipated. All excavations should be observed by an engineering geologist or geotechnical engineer from GSI, prior to workers entering the excavation or trench, and minimally conform to CAL-OSHA, state, and local safety codes. Should adverse conditions exist, appropriate recommendations would be offered at that time. The above recommendations should be provided to any contractors or subcontractors, or homeowners, etc., that may perform such work.

Underground Utility Trench Backfill

1. All underground utility trench backfill, including under-slab utility trench backfill, should be uniformly moisture conditioned to at least optimum moisture content and then be compacted to obtain a minimum relative density of 90 percent of the

laboratory standard (per ASTM D 1557). Observation, selective probing, and field density testing should be provided to evaluate the desired results.

2. Exterior trenches adjacent to, and within areas extending below a 1:1 plane projected from the outside bottom edge of the footing, and all trenches beneath hardscape features and in slopes, should be compacted to at least 90 percent of the laboratory standard. Sand backfill, unless excavated from the trench, should not be used in these backfill areas. Compaction testing and observations, along with probing, should be accomplished to evaluate the desired results.
3. All trench excavations should conform to CAL-OSHA, state, and local safety codes.
4. Prior to underground utility installation, the bottom of underground utility trenches, exposing unsuitable soils should be overexcavated to expose suitable, unweathered young alluvial fan deposits, cleared of oversized (greater than 8 inches) rock, and then be backfilled to the planned cut elevation with the excavated soils in accordance with the "Fill Placement" section of this report.

SUMMARY OF RECOMMENDATIONS REGARDING GEOTECHNICAL OBSERVATION AND TESTING

We recommend that observation and testing be performed by GSI at each of the following construction stages:

- During grading and pad recertification.
- During excavation greater than 4 feet.
- After excavation of building footings, retaining wall footings, and free standing walls footings, prior to the placement of reinforcing steel or concrete.
- Prior to pouring of any concrete slabs-on-grade or flatwork, after presoaking/presaturation of building pads and other flatwork subgrade, before the placement of concrete, reinforcing steel, capillary break (e.g., sand, pea-gravel), or vapor barriers (e.g., Stego Wrap, Husky Guard).
- During placement of backfill for area drain, interior (under-slab) plumbing, and underground utility line trenches.
- During retaining wall subdrain construction and backfill.
- During vehicular and pedestrian pavement construction.
- During slope construction or repair.

- When any unusual soil conditions are encountered during any construction operations, subsequent to the issuance of this report.
- When any homeowner improvements (e.g., flatwork, spas, pools, walls), are constructed, prior to construction.
- A report of geotechnical observation and testing should be provided at the conclusion of each of the above stages, in order to provide concise and clear documentation of site work, and to comply with code requirements.

OTHER DESIGN PROFESSIONALS AND CONSULTANTS

The design civil engineer, structural engineer, post-tension designer, architect, landscape architect, wall designer, and other design professionals, should review the recommendations provided herein, incorporate those recommendations into all their respective plans, and by explicit reference, make this report part of their project plans. This report presents minimum design criteria for the design of slabs, foundations and other elements possibly applicable to the project. These criteria should not be considered as substitutes for actual designs by the structural engineer or designer. Please note that the recommendations contained herein are not intended to preclude the transmission of water or vapor through the slab or foundation. The structural engineer or foundation or slab designer should provide recommendations to not allow water or vapor to enter into the structure so as to cause damage to another building component, or so as to limit the installation of the type of flooring materials typically used for the particular application.

The structural engineer or designer should analyze actual soil-structure interaction and consider, as needed, bearing, expansive soil influence, and strength, stiffness and deflections in the various slab, foundation, and other elements in order to develop appropriate, design-specific details. As conditions dictate, it is possible that other influences will also have to be considered. The structural engineer or designer should consider all applicable codes and authoritative sources where needed. If analyses by the structural engineer/designer result in less critical details than are provided herein as minimums, the minimums presented herein should be adopted. It is considered likely that some, more restrictive details will be required.

If the structural engineer or designer has any questions or requires further assistance, they should not hesitate to call or otherwise transmit their requests to GSI. In order to mitigate potential distress, the foundation or improvement's designer should confirm to GSI and the governing agency, in writing, that the proposed foundations and improvements can tolerate the amount of differential settlement or expansion characteristics and other design criteria specified herein.

PLAN REVIEW

Final project plans (e.g., grading, precise grading, foundation, retaining wall, landscaping), should be reviewed by this office prior to construction, so that construction is in accordance with the conclusions and recommendations of this report. Based on our review, supplemental recommendations or further geotechnical studies may be warranted.

LIMITATIONS

The materials encountered on the project site and utilized for our analysis are believed representative of the area; however, soil and bedrock materials vary in character between excavations and natural outcrops or conditions exposed during mass grading. Site conditions may vary due to seasonal changes or other factors.

Inasmuch as our study is based upon our review and engineering analyses and laboratory data, the conclusions and recommendations are professional opinions. These opinions have been derived in accordance with current standards of practice, and no warranty, either express or implied, is given. Standards of practice are subject to change with time. GSI assumes no responsibility or liability for work or testing performed by others, or their inaction; or work performed when GSI is not requested to be onsite, to evaluate if our recommendations have been properly implemented. Use of this report constitutes an agreement and consent by the user to all the limitations outlined above, notwithstanding any other agreements that may be in place. In addition, this report may be subject to review by the controlling authorities. Thus, this report brings to completion our scope of services for this portion of the project. All samples will be disposed of after 30 days, unless specifically requested by the client, in writing.

APPENDIX A

REFERENCES

APPENDIX A

REFERENCES

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APPENDIX B

TEST PIT LOGS

UNIFIED SOIL CLASSIFICATION SYSTEM					CONSISTENCY OR RELATIVE DENSITY																	
Major Divisions			Group Symbols	Typical Names	CRITERIA																	
Coarse-Grained Soils More than 50% retained on No. 200 sieve	Gravels 50% or more of coarse fraction retained on No. 4 sieve	Clean Gravels	GW	Well-graded gravels and gravel-sand mixtures, little or no fines	<u>Standard Penetration Test</u> Penetration Resistance N (blows/ft) Relative Density <table><tr><td>0 - 4</td><td>Very loose</td></tr><tr><td>4 - 10</td><td>Loose</td></tr><tr><td>10 - 30</td><td>Medium</td></tr><tr><td>30 - 50</td><td>Dense</td></tr><tr><td>> 50</td><td>Very dense</td></tr></table>			0 - 4	Very loose	4 - 10	Loose	10 - 30	Medium	30 - 50	Dense	> 50	Very dense					
			0 - 4	Very loose																		
		4 - 10	Loose																			
		10 - 30	Medium																			
	30 - 50	Dense																				
	> 50	Very dense																				
	GP	Poorly graded gravels and gravel-sand mixtures, little or no fines																				
	Gravel with	GM	Silty gravels gravel-sand-silt mixtures																			
		GC	Clayey gravels, gravel-sand-clay mixtures																			
	Sands more than 50% of coarse fraction passes No. 4 sieve	Clean Sands	SW	Well-graded sands and gravelly sands, little or no fines																		
SP			Poorly graded sands and gravelly sands, little or no fines																			
Sands with Fines		SM	Silty sands, sand-silt mixtures																			
		SC	Clayey sands, sand-clay mixtures																			
		<u>Standard Penetration Test</u> Penetration Resistance N (blows/ft) Consistency Unconfined Compressive Strength (tons/ft²) <table><tr><td><2</td><td>Very Soft</td><td><0.25</td></tr><tr><td>2 - 4</td><td>Soft</td><td>0.25 - .050</td></tr><tr><td>4 - 8</td><td>Medium</td><td>0.50 - 1.00</td></tr><tr><td>8 - 15</td><td>Stiff</td><td>1.00 - 2.00</td></tr><tr><td>15 - 30</td><td>Very Stiff</td><td>2.00 - 4.00</td></tr><tr><td>>30</td><td>Hard</td><td>>4.00</td></tr></table>			<2	Very Soft	<0.25	2 - 4	Soft	0.25 - .050	4 - 8	Medium	0.50 - 1.00	8 - 15	Stiff	1.00 - 2.00	15 - 30	Very Stiff	2.00 - 4.00	>30	Hard	>4.00
					<2	Very Soft	<0.25															
2 - 4	Soft				0.25 - .050																	
4 - 8	Medium				0.50 - 1.00																	
8 - 15	Stiff				1.00 - 2.00																	
15 - 30	Very Stiff				2.00 - 4.00																	
>30	Hard				>4.00																	
Fine-Grained Soils 50% or more passes No. 200 sieve	Silts and Clays Liquid limit 50% or less				ML	Inorganic silts, very fine sands, rock flour, silty or clayey fine sands																
					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 sands or silts, elastic silts																			
		CH	Inorganic clays of high plasticity, fat clays																			
		OH	Organic clays of medium to high plasticity																			
Highly Organic Soils		PT	Peat, mucic, and other highly organic soils																			

3"		3/4"		#4	#10	#40	#200 U.S. Standard Sieve
Unified Soil Classification	Cobbles	Gravel		Sand			Silt or Clay
		coarse	fine	coarse	medium	fine	

MOISTURE CONDITIONS

Dry

Absence of moisture: dusty, dry to the touch

trace

0 - 5 %

C

Core Sample

Slightly Moist

Below optimum moisture content for compaction

few

5 - 10 %

S

SPT Sample

Moist

Near optimum moisture content

little

10 - 25 %

B

Bulk Sample

Very Moist

Above optimum moisture content

some

25 - 45 %

—

Groundwater

Wet

Visible free water; below water table

Qp

Pocket Penetrometer

BASIC LOG FORMAT:

Group name, Group symbol, (grain size), color, moisture, consistency or relative density. Additional comments: odor, presence of roots, mica, gypsum, coarse grained particles, etc.

EXAMPLE:

Sand (SP), fine to medium grained, brown, moist, loose, trace silt, little fine gravel, few cobbles up to 4" in size, some hair roots and rootlets.



W.O. 8972-A-IE
 Lytle Development Company
 Linden Avenue, Rialto
 Logged By: JM
 Date June 6, 2025

EXPLORATORY TEST PIT LOGS

TEST PIT NO.	ELEV. (ft.)	DEPTH (ft.)	GROUP SYMBOL	SAMPLE DEPTH (ft.)	MOISTURE (%)	FIELD DRY DENSITY (pcf)	DESCRIPTION
TP-1	1,586	0-4	GM/SP				ARTIFICAIL FILL - UNDOCUMENTED (Afu): Gravely SAND to sand with GRAVEL, light gray, dry, loose, friable; abundant sub-rounded cobbles and gravel, few boulder size clasts up to 20 inches in diameter, some caving.
		4-5	SM				COLLUVIUM/TOPSOIL (Not Mapped): (left in place): Silty SAND, dark brown, damp, loose; few roots and rootlets, friable, abundant sub-rounded cobbles and gravel sized clasts.
		5-10	GM/SP	Nuke @ 6	2.3	116.3	QUATERNARY YOUNG ALLUVIAL-FAN DEPOSITS, UNIT 5 (Qyf₅): Gravely SAND to sand with GRAVEL, light gray to dark brown with depth, dry to damp, loose to medium dense with depth; friable, abundant sub-rounded cobbles and gravel, few boulder size clasts up to 24 inches in diameter, some caving.
							Total Depth = 10'. No Groundwater/Seepage encountered. Some caving encountered. Backfilled on 6-6-25.
TP-2	1,587	0-5	GM/SP				ARTIFICAIL FILL - UNDOCUMENTED (Afu): Gravely SAND to sand with GRAVEL, light gray, dry, loose; friable, abundant sub-rounded cobbles and gravel, few boulder size clasts up to 13 inches in diameter, some caving.
		5-7	SM				COLLUVIUM/TOPSOIL (Not Mapped): (left in place): SILTY SAND, dark brown, moist, loose; abundant sub-rounded cobbles and gravel, few roots and rootlets, friable.
		7-11	GM/SP	Nuke @ 7	1.3	117.9	QUATERNARY YOUNG ALLUVIAL-FAN DEPOSITS, UNIT 5 (Qyf₅): Gravely SAND to sand with GRAVEL, light gray to dark brown with depth, dry, loose to medium dense with depth; friable, abundant sub-rounded cobbles and gravel, few boulder size clasts up to 18 inches in diameter, some caving.
							Total Depth = 11'. No Groundwater/Seepage encountered. Some caving encountered. Backfilled on 6-6-25.



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TEST PIT NO.	ELEV. (ft.)	DEPTH (ft.)	GROUP SYMBOL	SAMPLE DEPTH (ft.)	MOISTURE (%)	FIELD DRY DENSITY (pcf)	DESCRIPTION
TP-3	1,592	0-3	GM/SP				ARTIFICAIL FILL - UNDOCUMENTED (Afu): Gravely SAND to Sand with GRAVEL, light gray, dry, loose; abundant sub-rounded cobbles and gravel, friable, few boulder size clasts up to 18 inches in diameter, some caving.
		3-4	SM				COLLUVIUM/TOPSOIL (Not Mapped): (left in place): Silty SAND, dark brown, dry to damp, loose; abundant sub-rounded cobbles and gravel, few roots and rootlets, friable.
		4-10	GM/SP	Nuke @ 6	4.3	119.3	QUATERNARY YOUNG ALLUVIAL-FAN DEPOSITS, UNIT 5 (Qyf₅): Gravely SAND to sand with GRAVEL, light gray to dark brown with depth, dry to damp, loose to medium dense with depth; friable, abundant sub-rounded cobbles and gravel, few boulder size clasts up to 18 inches in diameter, some caving.
							Total Depth = 10'. No Groundwater/Seepage encountered. Some caving encountered. Backfilled on 6-6-25.
TP-4	1,594	0-5	GM/SP				ARTIFICAIL FILL - UNDOCUMENTED (Afu): Gravely SAND to sand with GRAVEL, light gray, dry, loose; friable, abundant sub-rounded cobbles and gravel, few boulder size clasts up to 18 inches in diameter, some caving.
		5-6	SM				COLLUVIUM/TOPSOIL (Not Mapped): (left in place): SILTY SAND, dark brown, damp, loose; abundant sub-rounded cobbles and gravel, few roots and rootlets, friable.
		6-11	GM/SP	Nuke @ 7	3.9	118.6	QUATERNARY YOUNG ALLUVIAL-FAN DEPOSITS, UNIT 5 (Qyf₅): Gravely SAND to sand with GRAVEL, light gray to dark brown with depth, dry, loose to med dense with depth; friable, abundant sub-rounded cobbles and gravel, few boulder size clasts up to 12 inches in diameter, some caving.
							Total Depth = 11'. No Groundwater/Seepage encountered. Some caving encountered. Backfilled on 6-6-25.



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EXPLORATORY TEST PIT LOGS

TEST PIT NO.	ELEV. (ft.)	DEPTH (ft.)	GROUP SYMBOL	SAMPLE DEPTH (ft.)	MOISTURE (%)	FIELD DRY DENSITY (pcf)	DESCRIPTION
TP-5	1,596	0-5	GM/SP				ARTIFICAIL FILL - UNDOCUMENTED (Afu): Gravely SAND to sand with GRAVEL, light gray, dry, loose; abundant sub-rounded cobbles and gravel, friable, few boulder size clasts up to 18 inches in diameter, some caving.
		5-7	SM				COLLUVIUM/TOPSOIL (Not Mapped): (left in place): Silty SAND, dark brown, moist, loose; few roots and rootlets, friable, abundant sub-rounded cobbles and gravel sized clasts.
		7-11	GM/SP	Nuke @ 7	3.9	118.6	QUATERNARY YOUNG ALLUVIAL-FAN DEPOSITS, UNIT 5 (Qyf₅): Gravely SAND to sand with GRAVEL, light gray to dark brown with depth, damp, loose to medium dense with depth; abundant sub-rounded cobbles and gravel, friable, few boulder size clasts up to 12 inches in diameter, some caving.
							Total Depth = 11'. No Groundwater/Seepage encountered. Some caving encountered. Backfilled on 6-6-25.
TP-6	1,590	0-2	GM/SP				ARTIFICAIL FILL - UNDOCUMENTED (Afu): Gravely SAND to sand with GRAVEL, light gray, dry, loos; friable, few boulder size clasts up to 24 inches in diameter, abundant sub-rounded cobbles and gravel, some caving.
		2-3	SM				COLLUVIUM/TOPSOIL (Not Mapped): (left in place): SILTY SAND, dark brown, dry, loose; few roots and rootlets, abundant sub-rounded cobbles and gravel, friable.
		3-7	GM/SP	Nuke @ 4	2.9	115.3	QUATERNARY YOUNG ALLUVIAL-FAN DEPOSITS, UNIT 5 (Qyf₅): Gravely SAND to sand with GRAVEL, light gray to dark brown with depth, dry, loose to medium dense with depth; friable, abundant sub-rounded cobbles and gravel, few boulder size clasts up to 18 inches in diameter, some caving.
							Total Depth = 7'. No Groundwater/Seepage encountered. Some caving encountered. Backfilled on 6-6-25.



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EXPLORATORY TEST PIT LOGS

TEST PIT NO.	ELEV. (ft.)	DEPTH (ft.)	GROUP SYMBOL	SAMPLE DEPTH (ft.)	MOISTURE (%)	FIELD DRY DENSITY (pcf)	DESCRIPTION
TP-7	1,589	0-6	GM/SP				ARTIFICAIL FILL - UNDOCUMENTED (Afu): Gravely SAND to sand with GRAVEL, light gray, dry, loose; friable, abundant sub-rounded cobbles and gravel, few boulder size clasts up to 18 inches in diameter, some caving.
		6-7	SM				COLLUVIUM/TOPSOIL (Not Mapped): (left in place): SILTY SAND, dark brown, moist, loose; abundant sub-rounded cobbles and gravel, few roots and rootlets, friable.
		7-11	GM/SP	Nuke @ 7	3.4	121.3	QUATERNARY YOUNG ALLUVIAL-FAN DEPOSITS, UNIT 5 (Qyf₅): Gravely SAND to sand with GRAVEL, light gray to dark brown with depth, moist, loose to medium dense with depth; friable, abundant sub-rounded cobbles and gravel, few boulder size clasts up to 18 inches in diameter, some caving.
							Total Depth = 11'. No Groundwater/Seepage encountered. Some caving encountered. Backfilled on 6-6-25.
TP-8	1,587	0-2	GM/SP				ARTIFICAIL FILL - UNDOCUMENTED (Afu): GRAVELY SAND to sand with Gravel, light gray, dry, loose, friable, abundant sub-rounded cobbles and gravel, few boulder size clasts up to 20 inches in diameter, some caving.
		2-3	SM				COLLUVIUM/TOPSOIL (Not Mapped): (left in place): SILTY SAND, Dark brown, moist, loose, abundant sub-rounded cobbles/gravel, few roots/rootlets, friable.
		3-9	GM/SP	Nuke @ 5	1.9	115.0	QUATERNARY YOUNG ALLUVIAL-FAN DEPOSITS, UNIT 5 (Qyf₅): Gravely SAND to sand with GRAVEL, light gray to dark brown with depth, moist, loose to med dense with depth; friable, abundant sub-rounded cobbles and gravel, few boulder size clasts up to 12 inches in diameter, some caving.
							Total Depth = 9' - Percolation Test at 10'. No Groundwater/Seepage encountered. Some caving encountered. Backfilled on 6-6-25.



W.O. 8972-A-IE
 Lytle Development Company
 Linden Avenue, Rialto
 Logged By: JM
 Date June 6, 2025

EXPLORATORY TEST PIT LOGS

TEST PIT NO.	ELEV. (ft.)	DEPTH (ft.)	GROUP SYMBOL	SAMPLE DEPTH (ft.)	MOISTURE (%)	FIELD DRY DENSITY (pcf)	DESCRIPTION
P-1	1,583	0-2					ARTIFICAIL FILL - UNDOCUMENTED (Afu): Gravely SAND to sand with GRAVEL, light gray, dry, loose; friable, abundant sub-rounded cobbles and gravel, few boulder size clasts up to 12 inches in diameter, some caving.
		2-3.5	SM				COLLUVIUM/TOPSOIL (Not Mapped): (left in place): Silty SAND, Dark brown, moist, loose, few roots/rootlets, friable, abundant sub-rounded cobbles/gravel.
		3.5-8	GM/SP	Nuke @ 7	2.9	111.0	QUATERNARY YOUNG ALLUVIAL-FAN DEPOSITS, UNIT 5 (Qyf₅): GRAVELY SAND to sand with Gravel, light gray to Dark brown with depth, moist, loose to med dense with depth; friable, abundant sub-rounded cobbles and gravel, few boulder size clasts up to 18 inches in diameter, some caving.
		8-9	SM				Silty SAND to Sand-SILT mixture, light brown, moist, medium dense, few to seldom large gravel clasts, moderately sorted.
		Total Depth = 9' - Percolation Test at 10'. No Groundwater/Seepage encountered. Some caving encountered. Backfilled on 6-6-25.					



W.O. 8972-A-IE
 Lytle Development Company
 Linden Avenue, Rialto
 Logged By: JM
 Date June 6, 2025

EXPLORATORY TEST PIT LOGS

TEST PIT NO.	ELEV. (ft.)	DEPTH (ft.)	GROUP SYMBOL	SAMPLE DEPTH (ft.)	MOISTURE (%)	FIELD DRY DENSITY (pcf)	DESCRIPTION
P-2	1,582	0-2					ARTIFICAIL FILL - UNDOCUMENTED (Afu): Gravely SAND to sand with GRAVEL, light gray, dry, loose; friable, abundant sub-rounded cobbles and gravel, few boulder size clasts up to 18 inches in diameter, some caving.
		2-3	SM				COLLUVIUM/TOPSOIL (Not Mapped): (left in place): SILTY SAND, Dark brown, moist, loose, few roots/rootlets, friable, abundant sub-rounded cobbles/gravel.
		3-8	GM/SP	Nuke @ 7	3.3	114.3	QUATERNARY YOUNG ALLUVIAL-FAN DEPOSITS, UNIT 5 (Qyf₅): GRAVELY SAND to sand with Gravel, light gray to Dark brown w/ depth, moist, loose to med dense w/ depth, friable, abundant sub-rounded cobbles/gravel, few boulder size clasts up to 30 inch in diameter, some caving.
		8-9	SM				Silty SAND to Sand-SILT mixture, light brown, dry to damp, medium dense, few to seldom large gravel clasts, moderately sorted.
							Total Depth = 9' No Groundwater/seepage Encountered Some caving encountered (34.15151, -117.40029)

APPENDIX C

FIELD PERCOLATION LOGS

Leach Line Percolation Data Sheet

Project: <u>Little Development - Linden</u>	W.O. Number: <u>8972-A-EE</u>
Test Hole No.: <u>R-1</u>	Date Excavated: <u>6/6/25</u>
Depth of Test Hole: <u>7</u>	Soil Classification: <u>SM</u>
Check for Sandy Soil Criteria Tested by: <u>MAH</u>	Date: <u>6/6/25</u> Presoak: <u>9:02 AM</u>
Actual Percolation Tested by: <u>MAH</u>	Date: <u>6/6/25</u>

Sandy Soil Criteria Test

Note: 5 gal water used in about 1 hour

Trial No.	Time	Time Interval (Min.)	Initial Water Level (Inches)	Final Water Level (Inches)	Δ in Water Level (Inches)
1	9:55	25	15	3	12
	10:20				
2	10:21	25	15	8.25	6.75
	10:46				

Use: Normal Sandy (Circle One) Soil Criteria

	Time	Time Interval (min)	Total Elapsed Time (Min.)	Initial Water Level (Inches)	Final Water Level (Inches)	Δ in Water Level (Inches)	Percolation Rate (min/inch)
First Hour	10:49	10	10	15	11	4	2.5
	10:59						
	11:00	10	20	15	11.25	3.75	2.67
	11:10						
Second Hour	11:12	10	30	15	11.25	3.75	2.67
	11:22						
	11:23	10	40	15	11.25	3.75	2.67
	11:33						
Third Hour	11:35	10	50	15	11.25	3.75	2.67
	11:45						
	11:45	10	60	15	11.75	3.25	3.08
	11:55						
Fourth Hour							
Fifth Hour							
Sixth Hour							

Leach Line Percolation Data Sheet

Project: <u>Lyth Development - Linden</u>	W.O. Number: <u>8972-A-IE</u>
Test Hole No.: <u>P-2</u>	Date Excavated: <u>6/6/2023</u>
Depth of Test Hole: <u>8 ft</u>	Soil Classification: <u>SP-Sd</u>
Check for Sandy Soil Criteria Tested by: <u>MMH</u>	Date: <u>6/6/2023</u> Presoak: <u>10:27 AM</u>
Actual Percolation Tested by: <u>MMH</u>	Date: <u>6/6/2023</u>

Sandy Soil Criteria Test

Note: 5 gal. water used in less than 20 min

Trial No.	Time	Time Interval (Min.)	Initial Water Level (Inches)	Final Water Level (Inches)	Δ in Water Level (Inches)
1	10:51	25	15"	0	15"
	11:16				
2	11:19	25	15	0	15
	11:44				

Use: Normal Sandy (Circle One) Soil Criteria

	Time	Time Interval (min)	Total Elapsed Time (Min.)	Initial Water Level (Inches)	Final Water Level (Inches)	Δ in Water Level (Inches)	Percolation Rate (min/inch)
First Hour	11:49	10	10	15	0	15	0.67
	11:59						
	11:59	10	20	15	0.5	14.5	0.69
	12:09						
Second Hour	12:10	10	30	15	0	15	0.67
	12:20						
	12:20	10	40	15	0	15	0.67
	12:30						
Third Hour	12:32	10	50	15	0	15	0.67
	12:42						
	12:45	10	60	15	0	15	0.67
	12:55						
Fourth Hour							
Fifth Hour							
Sixth Hour							

APPENDIX D

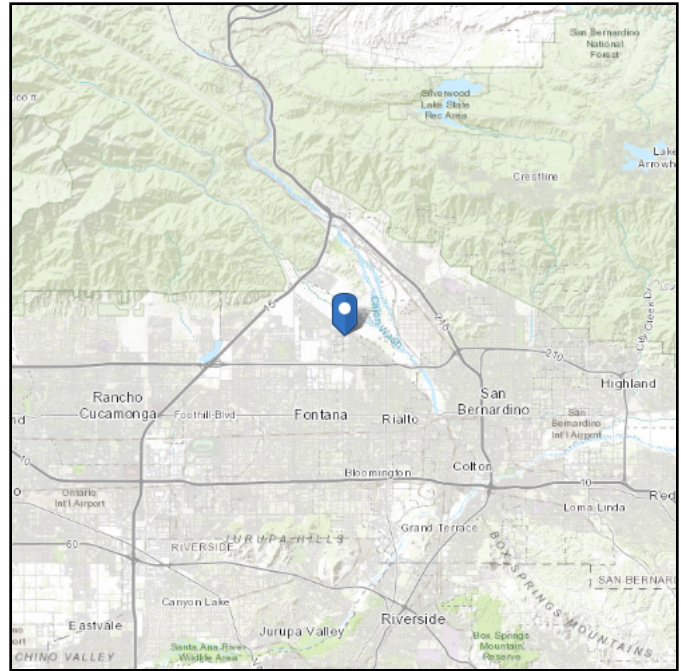
SEISMIC GROUND MOTION VALUES

ASCE Hazards Report

Address:
No Address at This Location

Standard: ASCE/SEI 7-16
Risk Category: II
Soil Class: D - Stiff Soil

Latitude: 34.1527
Longitude: -117.4004
Elevation: 1589.8059742957055 ft (NAVD 88)



Site Soil Class: D - Stiff Soil

Results:

S_S :	2.198	S_{D1} :	N/A
S_1 :	0.851	T_L :	12
F_a :	1	PGA :	0.909
F_v :	N/A	PGA_M :	1
S_{MS} :	2.198	F_{PGA} :	1.1
S_{M1} :	N/A	I_e :	1
S_{DS} :	1.465	C_v :	1.5

Ground motion hazard analysis may be required. See ASCE/SEI 7-16 Section 11.4.8.

Data Accessed: Wed Jul 09 2025

Date Source: [USGS Seismic Design Maps](#)

The ASCE Hazard Tool is provided for your convenience, for informational purposes only, and is provided “as is” and without warranties of any kind. The location data included herein has been obtained from information developed, produced, and maintained by third party providers; or has been extrapolated from maps incorporated in the ASCE standard. While ASCE has made every effort to use data obtained from reliable sources or methodologies, ASCE does not make any representations or warranties as to the accuracy, completeness, reliability, currency, or quality of any data provided herein. Any third-party links provided by this Tool should not be construed as an endorsement, affiliation, relationship, or sponsorship of such third-party content by or from ASCE.

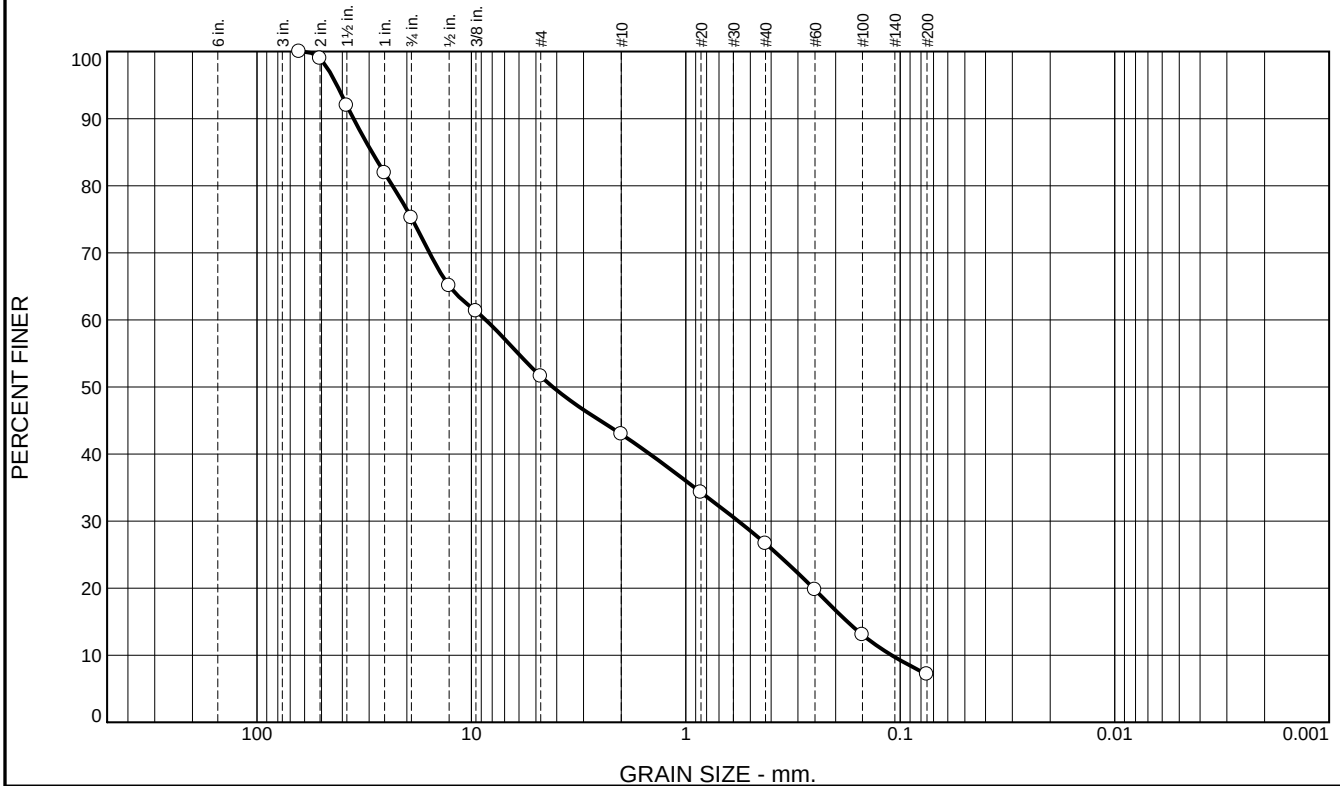
ASCE does not intend, nor should anyone interpret, the results provided by this Tool to replace the sound judgment of a competent professional, having knowledge and experience in the appropriate field(s) of practice, nor to substitute for the standard of care required of such professionals in interpreting and applying the contents of this Tool or the ASCE standard.

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APPENDIX E

LABORATORY TEST RESULTS

Particle Size Distribution Report - ASTM D6913



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	24.8	23.6	8.6	16.4	19.5	7.1	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
2.5	100.0		
2.0	99.0		
1.5	92.0		
1.0	81.9		
0.75	75.2		
0.5	65.1		
0.375	61.3		
#4	51.6		
#10	43.0		
#20	34.3		
#40	26.6		
#60	19.8		
#100	13.0		
#200	7.1		

* (no specification provided)

Material Description

Dark Yellowish Brown Well-Graded Sand With Silt & Gravel

Atterberg Limits

PL=

LL=

PI=

Coefficients

D₉₀= 35.4576

D₈₅= 29.0686

D₆₀= 8.5707

D₅₀= 4.1746

D₃₀= 0.5695

D₁₅= 0.1763

D₁₀= 0.1097

C_u= 78.13

C_c= 0.35

Classification

USCS= GW-GM

AASHTO=

Remarks

P-1, TP-4, TP-6 Composite

F.M.=4.72

Source of Sample: P-1 Depth: 6-9
Sample Number: P-1, TP-4, TP-6

Date: 7-14-25

GeoSoils, Inc.

Client: Lytle Development

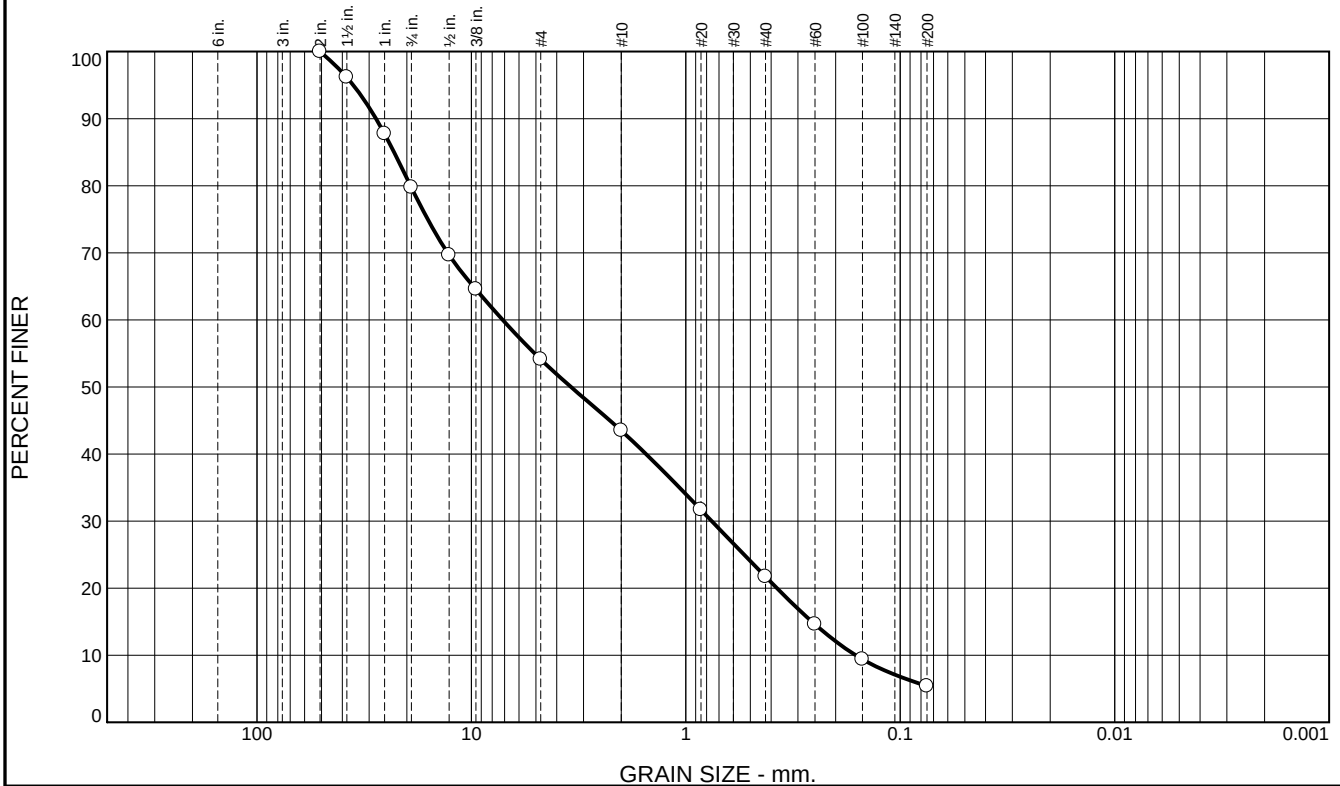
Project: Linden

Project No: 8972-A-IE

Plate E-1

Tested By: TR Checked By: TR

Particle Size Distribution Report - ASTM D6913



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	20.3	25.6	10.6	21.8	16.3	5.4	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
2.0	100.0		
1.5	96.2		
1.0	87.7		
0.75	79.7		
0.5	69.7		
0.375	64.6		
#4	54.1		
#10	43.5		
#20	31.7		
#40	21.7		
#60	14.6		
#100	9.4		
#200	5.4		

* (no specification provided)

Material Description

Dark Yellowish Brown Well-Graded Sand with Silt & Gravel

Atterberg Limits

PL=

LL=

PI=

Coefficients

D₉₀= 27.8445

D₈₅= 22.9484

D₆₀= 7.1440

D₅₀= 3.4331

D₃₀= 0.7570

D₁₅= 0.2581

D₁₀= 0.1613

C_u= 44.29

C_c= 0.50

Classification

USCS= SW-SM

AASHTO=

Remarks

F.M.=4.71

Source of Sample: TP-6
Sample Number: TP-6

Depth: 2.0

Date: 6-30-25

GeoSoils, Inc.

Client: Lytle Development
Project: Linden

Project No: 8972-A-IE

Plate E-2

Tested By: TR Checked By: TR

TEST SPECIMEN		A	B	C	D
Compactor air pressure	PSI	350	350	350	
Water added	%	4.4	4.8	5.3	
Moisture at compaction	%	8.8	9.2	9.7	
Height of sample	IN	2.54	2.53	2.58	
Dry density	PCF	129.6	128.6	127.7	
R-Value by exudation		82	78	76	
R-Value by exudation, corrected		82	78	77	
Exudation pressure	PSI	713	374	139	
Stability thickness	FT	0.23	0.28	0.31	
Expansion pressure thickness	FT	0.33	0.03	0.00	

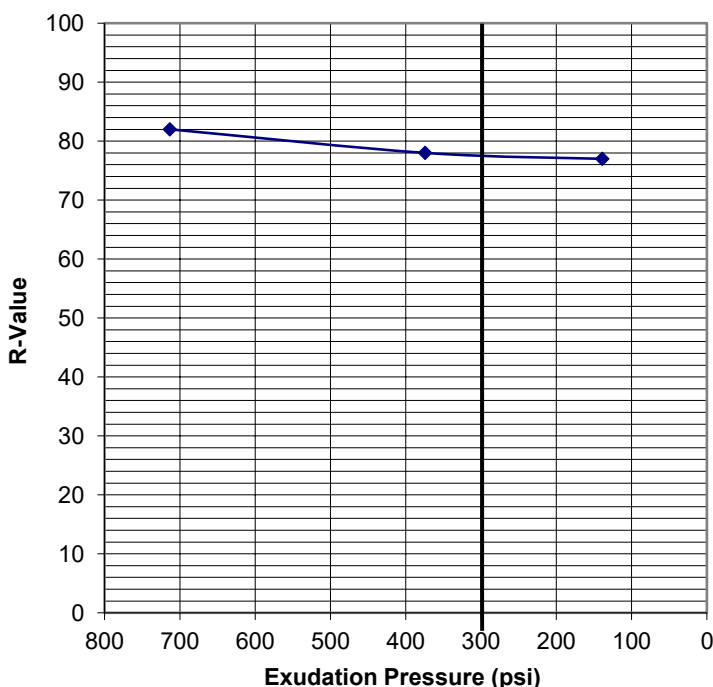
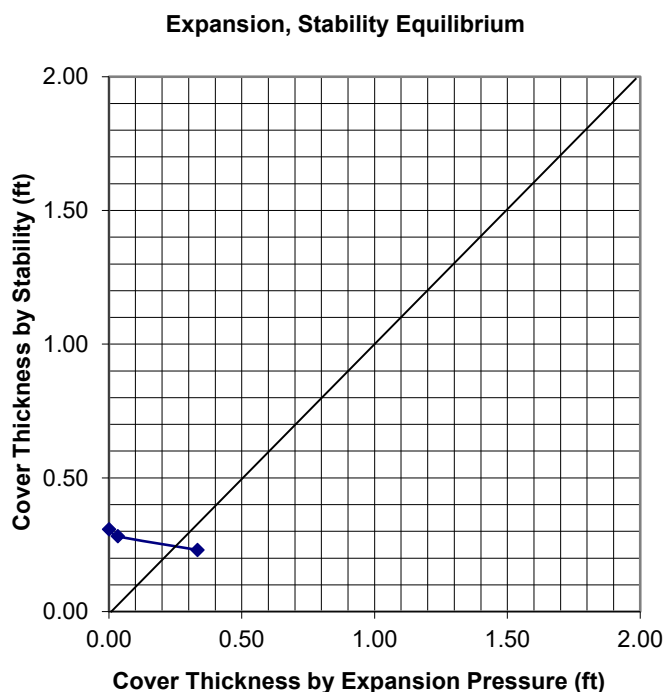
DESIGN CALCULATION DATA

Traffic index, assumed	5.0
Gravel equivalent factor, assumed	1.25
Expansion, stability equilibrium	0.25
R-Value by expansion	80
R-Value by exudation	78
R-Value at equilibrium	78

SAMPLE INFORMATION

Sample Location:	P-1, TP-4, TP-6
Sample Description:	Dark Yellowish Brown Well-Graded
Notes:	Sand with Silt & Gravel
	25% Retained on 3/4 inch sieve
Test Method:	Cal-Trans Test 301

R-Value By Exudation



GeoSoils, Inc.
1817 Aston Avenue, Suite 104
Carlsbad, CA 92008
Telephone: (760) 438-3155
Fax: (760) 931-0915

9/2/2010

R - VALUE TEST RESULTS

Project: Lytle Development

Number: 8972-A-IE

Date: July 2025

Plate D-3



1817 Aston Ave., Suite 104, Carlsbad CA 92008
Phone (760) 438-3155

CORROSION REPORT SUMMARY

Project No: 8972-A-IE
Project Name: Lytle Development
Report Date: July 8, 2025

SAMPLE ID	pH (H+)	Minimum Resistivity (ohm/cm)	Sulfate Content (wt%)	Chloride Content (mg/kg)
TP-6, 2.0ft	7.4	24000	<0.003	11

Sample testing in accordance with: pH - CTM 643, Resistivity - CTM 643
Sulfate - CTM 417, Chloride - CTM 422

Remarks: _____

APPENDIX F

GENERAL EARTHWORK AND GRADING GUIDELINES

GENERAL EARTHWORK, GRADING GUIDELINES, AND PRELIMINARY CRITERIA

General

These guidelines present general procedures and requirements for earthwork and grading as shown on the approved grading plans, including preparation of areas to be filled, placement of fill, installation of subdrains, excavations, and appurtenant structures or flatwork. The recommendations contained in the geotechnical report are part of these earthwork and grading guidelines and would supersede the provisions contained hereafter in the case of conflict. Evaluations performed by the consultant during the course of grading may result in new or revised recommendations which could supersede these guidelines or the recommendations contained in the geotechnical report. Generalized details follow this text.

The contractor is responsible for the satisfactory completion of all earthwork in accordance with provisions of the project plans and specifications and latest adopted Code. In the case of conflict, the most onerous provisions shall prevail. The project geotechnical engineer and engineering geologist (geotechnical consultant), or their representatives, should provide observation and testing services, and geotechnical consultation during the duration of the project.

EARTHWORK OBSERVATIONS AND TESTING

Geotechnical Consultant

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

The geotechnical consultant should provide testing and observation so that an evaluation may be made that the work is being accomplished as specified. It is the responsibility of the contractor to assist the consultants and keep them apprised of anticipated work schedules and changes, so that they may schedule their personnel accordingly.

All remedial removals, clean-outs, prepared ground to receive fill, key excavations, and subdrain installation should be observed and documented by the geotechnical consultant prior to placing any fill. It is the contractor's responsibility to notify the geotechnical consultant 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. Random or representative field compaction tests should be performed in accordance

with test methods ASTM designation D-1556, D-2937 or D-2922, and D-3017, at intervals of approximately ± 2 feet of fill height or approximately every 1,000 cubic yards 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 a geotechnical consultant, and staged approval by the governing agencies, as applicable. It is the contractor's responsibility to prepare the ground surface to receive the fill, to the satisfaction of the geotechnical consultant, and to place, spread, moisture condition, mix, and compact the fill in accordance with the recommendations of the geotechnical consultant. The contractor should also remove all non-earth material considered unsatisfactory by the geotechnical consultant.

Notwithstanding the services provided by the geotechnical consultant, it is the sole responsibility of the contractor to provide adequate equipment and methods to accomplish the earthwork in strict accordance with applicable grading guidelines, latest adopted Code or agency ordinances, geotechnical report(s), 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 or deleterious material, insufficient support equipment, etc., 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.

During construction, the contractor shall properly grade all surfaces to maintain good drainage and prevent ponding of water. The contractor shall take remedial measures to control surface water and to prevent erosion of graded areas until such time as permanent drainage and erosion control measures have been installed.

SITE PREPARATION

All major vegetation, including brush, trees, thick grasses, organic debris, and other deleterious material, should be removed and disposed of off-site. These removals must be concluded prior to placing fill. In-place existing fill, soil, alluvium, colluvium, or rock materials, as evaluated by the geotechnical consultant as being unsuitable, should be removed prior to any 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 geotechnical consultant.

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 geotechnical consultant. Soft, dry, spongy, highly

fractured, or otherwise unsuitable ground, extending to such a depth that surface processing cannot adequately improve the condition, should be overexcavated down to firm ground and approved by the geotechnical consultant before compaction and filling operations continue. Overexcavated and processed soils, which have been properly mixed and moisture conditioned, should be re-compacted 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 (ripped) to a minimum depth of 6 to 8 inches, or as directed by the geotechnical consultant. After the scarified ground is brought to optimum moisture content, or greater and mixed, the materials should be compacted as specified herein. If the scarified zone is greater than 6 to 8 inches in depth, it may be necessary to remove the excess and place the material in lifts restricted to about 6 to 8 inches in compacted thickness.

Existing ground which is not satisfactory to support compacted fill should be overexcavated as required in the geotechnical report, or by the on-site geotechnical consultant. Scarification, disc harrowing, or other acceptable forms of mixing should continue until the soils are broken down and free of large lumps or clods, until the working surface is reasonably uniform and free from ruts, hollows, hummocks, mounds, or other uneven features, which would inhibit compaction as described previously.

Where fills are to be placed on ground with slopes steeper than 5:1 (horizontal to vertical [h:v]), the ground should be stepped or benched. The lowest bench, which will act as a key, should be a minimum of 15 feet wide and should be at least 2 feet deep into firm material, and approved by the geotechnical consultant. In fill-over-cut slope conditions, the recommended minimum width of the lowest bench or key is also 15 feet, with the key founded on firm material, as designated by the geotechnical consultant. As a general rule, unless specifically recommended otherwise by the geotechnical consultant, the minimum width of fill keys should be equal to $\frac{1}{2}$ the height of the slope.

Standard benching is generally 4 feet (minimum) vertically, exposing firm, acceptable material. Benching may be used to remove unsuitable materials, although it is understood that the vertical height of the bench may exceed 4 feet. Pre-stripping may be considered for unsuitable materials in excess of 4 feet in thickness.

All areas to receive fill, including processed areas, removal areas, and the toes of fill benches, should be observed and approved by the geotechnical consultant prior to placement of fill. Fills may then be properly placed and compacted until design grades (elevations) are attained.

COMPACTED FILLS

Any earth materials imported or excavated on the property may be utilized in the fill provided that each material has been evaluated to be suitable by the geotechnical consultant. 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 geotechnical consultant. Soils of poor gradation, undesirable expansion potential, or substandard strength characteristics may be designated by the consultant as unsuitable and may require blending with other soils to serve as a satisfactory fill material.

Fill materials derived from benching operations should be dispersed throughout the fill area and blended with other approved material. 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 dimension greater than 12 inches, should not be buried or placed in fills unless the location of materials and disposal methods are specifically approved by the geotechnical consultant. Oversized material should be taken offsite, or placed in accordance with recommendations of the geotechnical consultant in areas designated as suitable for rock disposal. GSI anticipates that soils to be utilized as fill material for the subject project may contain some rock. Appropriately, the need for rock disposal may be necessary during grading operations on the site. From a geotechnical standpoint, the depth of any rocks, rock fills, or rock blankets, should be a sufficient distance from finish grade. This depth is generally the same as any overexcavation due to cut-fill transitions in hard rock areas, and generally facilitates the excavation of structural footings and substructures. Should deeper excavations be proposed (i.e., deepened footings, utility trenching, swimming pools, spas, etc.), the developer may consider increasing the hold-down depth of any rocky fills to be placed, as appropriate. In addition, some agencies/jurisdictions mandate a specific hold-down depth for oversize materials placed in fills. The hold-down depth, and potential to encounter oversize rock, both within fills, and occurring in cut or natural areas, would need to be disclosed to all interested/affected parties. Once approved by the governing agency, the hold-down depth for oversized rock (i.e., greater than 12 inches) in fills on this project is provided as 10 feet, unless specified differently in the text of this report. The governing agency may require that these materials need to be deeper, crushed, or reduced to less than 12 inches in maximum dimension, at their discretion.

To facilitate future trenching, rock (or oversized material), should not be placed within the hold-down depth feet from finish grade, the range of foundation excavations, future utilities, or underground construction unless specifically approved by the governing agency, the geotechnical consultant, or the developer's representative.

If import material is required for grading, representative samples of the materials to be utilized as compacted fill should be analyzed in the laboratory by the geotechnical consultant to evaluate its physical properties and suitability for use onsite. Such testing should be performed three (3) days prior to importation. If any material other than that previously tested is encountered during grading, an appropriate analysis of this material should be conducted by the geotechnical consultant as soon as possible.

Approved fill material should be placed in areas prepared to receive fill in near horizontal layers, that when compacted, should not exceed about 6 to 8 inches in thickness. The geotechnical consultant may approve thick lifts if testing indicates the grading procedures are such that adequate compaction is being achieved with lifts of greater thickness. Each layer should be spread evenly and blended to attain uniformity of material and moisture suitable for compaction.

Fill layers at a moisture content less than optimum should be watered and mixed, and wet fill layers should be aerated by scarification, or should be blended with drier material. Moisture conditioning, blending, and mixing of the fill layer should continue until the fill materials have a 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 the maximum density as evaluated by ASTM test designation D-1557, or as otherwise recommended by the geotechnical consultant. Compaction equipment should be adequately sized and should be specifically designed for soil compaction, or of proven reliability to efficiently achieve the specified 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 is in evidence, the particular layer or portion shall be re-worked until the required density and moisture content has been attained. No additional fill shall 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 geotechnical consultant.

In general, per the latest adopted Code, fill slopes should be designed and constructed at a gradient of 2:1 (h:v), or flatter. Compaction of slopes should be accomplished by over-building a minimum of 3 feet horizontally, and subsequently trimming back to the design slope configuration. Testing shall be performed as the fill is elevated 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 evaluation of fill slope compaction should be based on observation and testing of the finished slope face. Where compacted fill slopes are designed steeper than 2:1 (h:v), prior approval from the governing agency, specific material types, a higher minimum relative compaction, special reinforcement, and special grading procedures will be recommended.

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

1. An extra piece of 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 of the slope.

2. 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.
3. Field compaction tests will be made in the outer (horizontal) ± 2 to ± 8 feet of the slope at appropriate vertical intervals, subsequent to compaction operations.
4. After completion of the slope, the slope face should be shaped with a small tractor and then re-rolled with a sheepsfoot to achieve compaction to near the slope face. Subsequent to testing to evaluate compaction, the slopes should be grid-rolled to achieve compaction to the slope face. Final testing should be used to evaluate compaction after grid rolling.
5. Where testing indicates less than adequate compaction, the contractor will be responsible to rip, water, mix, and recompact the slope material as necessary to achieve compaction. Additional testing should be performed to evaluate compaction.

SUBDRAIN INSTALLATION

Subdrains should be installed in approved ground in accordance with the approximate alignment and details indicated by the geotechnical consultant. Subdrain locations or materials should not be changed or modified without approval of the geotechnical consultant. The geotechnical consultant may recommend and direct changes in subdrain line, grade, and drain material in the field, pending exposed conditions. The location of constructed subdrains, especially the outlets, should be recorded/surveyed by the project civil engineer. Drainage at the subdrain outlets should be provided by the project civil engineer.

EXCAVATIONS

Excavations and cut slopes should be examined during grading by the geotechnical consultant. If directed by the geotechnical consultant, further excavations or overexcavation and refilling of cut areas should be performed, or remedial grading of cut slopes should be performed. When fill-over-cut slopes are to be graded, unless otherwise approved, the cut portion of the slope should be observed by the geotechnical consultant prior to placement of materials for construction of the fill portion of the slope. The geotechnical consultant should observe all cut slopes, and should be notified by the contractor when excavation of cut slopes commences.

If, during the course of grading, unforeseen adverse or potentially adverse geologic conditions are encountered, the geotechnical consultant should investigate, evaluate, and make appropriate recommendations for mitigation of these conditions. The need for cut slope buttressing or stabilizing should be based on in-grading evaluation by the geotechnical consultant, whether anticipated or not.

Unless otherwise specified in geotechnical and geological report(s), 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, or in accordance with the recommendations of the geotechnical consultant.

COMPLETION

Observation, testing, and consultation by the geotechnical consultant should be conducted during the grading operations in order to state an opinion that all cut and fill areas are graded in accordance with the approved project specifications. After completion of grading, and after the geotechnical consultant has finished observations of the work, final reports should be submitted, and may be subject to review by the controlling governmental agencies. No further excavation or filling should be undertaken without prior notification of the geotechnical consultant or approved plans.

All finished cut and fill slopes should be protected from erosion and be planted in accordance with the project specifications or as recommended by a landscape architect. Such protection and planning should be undertaken as soon as practical after completion of grading.

PRELIMINARY OUTDOOR POOL/SPA DESIGN RECOMMENDATIONS

The following preliminary recommendations are provided for consideration in pool/spa design and planning. Actual recommendations should be provided by a qualified geotechnical consultant, based on site specific geotechnical conditions, including a subsurface investigation, differential settlement potential, expansive and corrosive soil potential, proximity of the proposed pool/spa to any slopes with regard to slope creep and lateral fill extension, as well as slope setbacks per Code, and geometry of the proposed improvements. Recommendations for pools/spas and deck flatwork underlain by expansive soils, or for areas with differential settlement greater than ¼-inch over 40 feet horizontally, will be more onerous than the preliminary recommendations presented below.

The 1:1 (h:v) influence zone of any nearby retaining wall site structures should be delineated on the project civil drawings with the pool/spa. This 1:1 (h:v) zone is defined as a plane up from the lower-most heel of the retaining structure, to the daylight grade of the nearby building pad or slope. If pools/spas or associated pool/spa improvements are constructed within this zone, they should be re-positioned (horizontally or vertically) so that they are supported by earth materials that are outside or below this 1:1 plane. If this is not possible given the area of the building pad, the owner should consider eliminating these improvements or allow for increased potential for lateral/vertical deformations and associated distress that

may render these improvements unusable in the future, unless they are periodically repaired and maintained. The conditions and recommendations presented herein should be disclosed to all homeowners and any interested/affected parties.

General

1. The equivalent fluid pressure to be used for the pool/spa design should be 60 pounds per cubic foot (pcf) for pool/spa walls with level backfill, and 75 pcf for a 2:1 sloped backfill condition. In addition, backdrains should be provided behind pool/spa walls adjacent to slopes.
2. Passive earth pressure may be computed as an equivalent fluid having a density of 150 pcf, to a maximum lateral earth pressure of 1,000 pounds per square foot (psf).
3. An allowable coefficient of friction between soil and concrete of 0.30 may be used with the dead load forces.
4. When combining passive pressure and frictional resistance, the passive pressure component should be reduced by one-third.
5. Where pools/spas are planned near structures, appropriate surcharge loads need to be incorporated into design and construction by the pool/spa designer. This includes, but is not limited to landscape berms, decorative walls, footings, built-in barbeques, utility poles, etc.
6. All pool/spa walls should be designed as “free standing” and be capable of supporting the water in the pool/spa without soil support. The shape of pool/spa in cross section and plan view may affect the performance of the pool, from a geotechnical standpoint. Pools and spas should also be designed in accordance with the latest adopted Code. Minimally, the bottoms of the pools/spas, should maintain a distance $H/3$, where H is the height of the slope (in feet), from the slope face. This distance should not be less than 7 feet, nor need not be greater than 40 feet.
7. The soil beneath the pool/spa bottom should be uniformly moist with the same stiffness throughout. If a fill/cut transition occurs beneath the pool/spa bottom, the cut portion should be overexcavated to a minimum depth of 48 inches, and replaced with compacted fill, such that there is a uniform blanket that is a minimum of 48 inches below the pool/spa shell. If very low expansive soil is used for fill, the fill should be placed at a minimum of 95 percent relative compaction, at optimum moisture conditions. This requirement should be 90 percent relative compaction at over optimum moisture if the pool/spa is constructed within or near expansive soils. The potential for grading or re-grading of the pool/spa bottom, and attendant potential for shoring or slot excavation, needs to be considered during all aspects of pool/spa planning, design, and construction.

8. If the pool/spa is founded entirely in compacted fill placed during rough grading, the deepest portion of the pool/spa should correspond with the thickest fill on the lot.
9. Hydrostatic pressure relief valves should be incorporated into the pool and spa designs. A pool/spa under-drain system is also recommended, with an appropriate outlet for discharge.
10. All fittings and pipe joints, particularly fittings in the side of the pool or spa, should be properly sealed to prevent water from leaking into the adjacent soils materials, and be fitted with slip or expandable joints between connections transecting varying soil conditions.
11. An elastic expansion joint (flexible waterproof sealant) should be installed to prevent water from seeping into the soil at all deck joints.
12. A reinforced grade beam should be placed around skimmer inlets to provide support and mitigate cracking around the skimmer face.
13. In order to reduce unsightly cracking, deck slabs should minimally be 4 inches thick, and reinforced with No. 3 reinforcing bars at 18 inches on-center. All slab reinforcement should be supported to ensure proper mid-slab positioning during the placement of concrete. Wire mesh reinforcing is specifically not recommended. Deck slabs should not be tied to the pool/spa structure. Pre-moistening or pre-soaking of the slab subgrade is recommended, to a depth of 12 inches (optimum moisture content), or 18 inches (120 percent of the soil's optimum moisture content, or 3 percent over optimum moisture content, whichever is greater), for very low to low, and medium expansive soils, respectively. This moisture content should be maintained in the subgrade soils during concrete placement to promote uniform curing of the concrete and minimize the development of unsightly shrinkage cracks. Slab underlayment should consist of a 1- to 2-inch leveling course of sand (S.E.>30) and a minimum of 4 to 6 inches of Class 2 base compacted to 90 percent. Deck slabs within the H/3 zone, where H is the height of the slope (in feet), will have an increased potential for distress relative to other areas outside of the H/3 zone. If distress is undesirable, improvements, deck slabs or flatwork should not be constructed closer than H/3 or 7 feet (whichever is greater) from the slope face, in order to reduce, but not eliminate, this potential.
14. Pool/spa bottom or deck slabs should be founded entirely on competent bedrock, or properly compacted fill. Fill should be compacted to achieve a minimum 90 percent relative compaction, as discussed above. Prior to pouring concrete, subgrade soils below the pool/spa decking should be thoroughly watered to achieve a moisture content that is at least 2 percent above optimum moisture content, to a depth of at least 18 inches below the bottom of slabs. This moisture content should be maintained in the subgrade soils during concrete placement to promote uniform curing of the concrete and minimize the development of unsightly shrinkage cracks.

15. In order to reduce unsightly cracking, the outer edges of pool/spa decking to be bordered by landscaping, and the edges immediately adjacent to the pool/spa, should be underlain by an 8-inch wide concrete cutoff shoulder (thickened edge) extending to a depth of at least 12 inches below the bottoms of the slabs to mitigate excessive infiltration of water under the pool/spa deck. These thickened edges should be reinforced with two No. 4 bars, one at the top and one at the bottom. Deck slabs may be minimally reinforced with No. 3 reinforcing bars placed at 18 inches on-center, in both directions. All slab reinforcement should be supported on chairs to ensure proper mid-slab positioning during the placement of concrete.
16. Surface and shrinkage cracking of the finish slab may be reduced if a low slump and water-cement ratio are maintained during concrete placement. Concrete utilized should have a minimum compressive strength of 4,000 psi. Excessive water added to concrete prior to placement is likely to cause shrinkage cracking, and should be avoided. Some concrete shrinkage cracking, however, is unavoidable.
17. Joint and sawcut locations for the pool/spa deck should be determined by the design engineer or contractor. However, spacings should not exceed 6 feet on center.
18. Considering the nature of the onsite earth materials, it should be anticipated that caving or sloughing could be a factor in subsurface excavations and trenching. Shoring or excavating the trench walls/backcuts at the angle of repose (typically 25 to 45 degrees), should be anticipated. All excavations should be observed by a representative of the geotechnical consultant, including the project geologist or geotechnical engineer, prior to workers entering the excavation or trench, and minimally conform to Cal/OSHA ("Type C" soils may be assumed), state, and local safety codes. Should adverse conditions exist, appropriate recommendations should be offered at that time by the geotechnical consultant. GSI does not consult in the area of safety engineering and the safety of the construction crew is the responsibility of the pool/spa builder.
19. It is imperative that adequate provisions for surface drainage are incorporated by the homeowners into their overall improvement scheme. Ponding water, ground saturation and flow over slope faces, are all situations which must be avoided to enhance long term performance of the pool/spa and associated improvements, and reduce the likelihood of distress.
20. Regardless of the methods employed, once the pool/spa is filled with water, should it be emptied, there exists some potential that if emptied, significant distress may occur. Accordingly, once filled, the pool/spa should not be emptied unless evaluated by the geotechnical consultant and the pool/spa builder.
21. For pools/spas built within (all or part) of the Code setback or geotechnical setback, as indicated in the site geotechnical documents, special foundations are recommended to mitigate the effects of creep, lateral fill extension, expansive soils and settlement on the proposed pool/spa. Most municipalities or County reviewers

do not consider these effects in pool/spa plan approvals. As such, where pools/spas are proposed on 20 feet or more of fill, medium or highly expansive soils, or rock fill with limited “cap soils” and built within Code setbacks, or within the influence of the creep zone, or lateral fill extension, the following should be considered during design and construction:

OPTION A: Shallow foundations with or without overexcavation of the pool/spa “shell,” such that the pool/spa is surrounded by 5 feet of very low to low expansive soils (without irreducible particles greater than 6 inches), and the pool/spa walls closer to the slope(s) are designed to be free standing. GSI recommends a pool/spa under-drain or blanket system (see attached Typical Pool/Spa Detail). The pool/spa builders and owner in this optional construction technique should be generally satisfied with pool/spa performance under this scenario; however, some settlement, tilting, cracking, and leakage of the pool/spa is likely over the life of the project.

OPTION B: Pier supported pool/spa foundations with or without overexcavation of the pool/spa shell such that the pool/spa is surrounded by 5 feet of very low to low expansive soils (without irreducible particles greater than 6 inches), and the pool/spa walls closer to the slope(s) are designed to be free standing. The need for a pool/spa under-drain system may be installed for leak detection purposes. Piers that support the pool/spa should be a minimum of 12 inches in diameter and at a spacing to provide vertical and lateral support of the pool/spa, in accordance with the pool/spa designers recommendations current applicable Codes. The pool/spa builder and owner in this second scenario construction technique should be more satisfied with pool/spa performance. This construction will reduce settlement and creep effects on the pool/spa; however, it will not eliminate these potentials, nor make the pool/spa “leak-free.”

22. The temperature of the water lines for spas and pools may affect the corrosion properties of site soils, thus, a corrosion specialist should be retained to review all spa and pool plans, and provide mitigative recommendations, as warranted. Concrete mix design should be reviewed by a qualified corrosion consultant and materials engineer.
23. All pool/spa utility trenches should be compacted to 90 percent of the laboratory standard, under the full-time observation and testing of a qualified geotechnical consultant. Utility trench bottoms should be sloped away from the primary structure on the property (typically the residence).
24. Pool and spa utility lines should not cross the primary structure’s utility lines (i.e., not stacked, or sharing of trenches, etc.).
25. The pool/spa or associated utilities should not intercept, interrupt, or otherwise adversely impact any area drain, roof drain, or other drainage conveyances. If it is

necessary to modify, move, or disrupt existing area drains, subdrains, or tightlines, then the design civil engineer should be consulted, and mitigative measures provided. Such measures should be further reviewed and approved by the geotechnical consultant, prior to proceeding with any further construction.

26. The geotechnical consultant should review and approve all aspects of pool/spa and flatwork design prior to construction. A design civil engineer should review all aspects of such design, including drainage and setback conditions. Prior to acceptance of the pool/spa construction, the project builder, geotechnical consultant and civil designer should evaluate the performance of the area drains and other site drainage pipes, following pool/spa construction.
27. All aspects of construction should be reviewed and approved by the geotechnical consultant, including during excavation, prior to the placement of any additional fill, prior to the placement of any reinforcement or pouring of any concrete.
28. Any changes in design or location of the pool/spa should be reviewed and approved by the geotechnical and design civil engineer prior to construction. Field adjustments should not be allowed until written approval of the proposed field changes are obtained from the geotechnical and design civil engineer.
29. Disclosure should be made to homeowners and builders, contractors, and any interested/affected parties, that pools/spas built within about 15 feet of the top of a slope, or $H/3$, where H is the height of the slope (in feet), will experience some movement or tilting. While the pool/spa shell or coping may not necessarily crack, the levelness of the pool/spa will likely tilt toward the slope, and may not be esthetically pleasing. The same is true with decking, flatwork and other improvements in this zone.
30. Failure to adhere to the above recommendations will significantly increase the potential for distress to the pool/spa, flatwork, etc.
31. Local seismicity and the design earthquake will cause some distress to the pool/spa and decking or flatwork, possibly including total functional and economic loss.
32. The information and recommendations discussed above should be provided to any contractors, subcontractors, or homeowners, interested/affected parties, etc., that may perform or may be affected by such work.

JOB SAFETY

General

At GSI, getting the job done safely is of primary concern. The following is the company's safety considerations for use by all employees on multi-employer construction sites. On-ground personnel are at highest risk of injury, and possible fatality, on grading and construction projects. GSI recognizes that construction activities will vary on each site, and

that site safety is the prime responsibility of the contractor; however, everyone must be safety conscious and responsible at all times. To achieve our goal of avoiding accidents, cooperation between the client, the contractor, and GSI personnel must be maintained.

In an effort to minimize risks associated with geotechnical testing and observation, the following precautions are to be implemented for the safety of field personnel on grading and construction projects:

Safety Meetings: GSI field personnel are directed to attend contractor's regularly scheduled and documented safety meetings.

Safety Vests: Safety vests are provided for, and are to be worn by GSI personnel, at all times, when they are working in the field.

Safety Flags: Two safety flags are provided to GSI field technicians; one is to be affixed to the vehicle when on site, the other is to be placed atop the spoil pile on all test pits.

Flashing Lights: All vehicles stationary in the grading area shall use rotating or flashing amber beacons, or strobe lights, on the vehicle during all field testing. While operating a vehicle in the grading area, the emergency flasher on the vehicle shall be activated.

In the event that the contractor's representative observes any of our personnel not following the above, we request that it be brought to the attention of our office.

Test Pits Location, Orientation, and Clearance

The technician is responsible for selecting test pit locations. A primary concern should be the technician's safety. Efforts will be made to coordinate locations with the grading contractor's authorized representative, and to select locations following or behind the established traffic pattern, preferably outside of current traffic. The contractor's authorized representative (supervisor, grade checker, dump man, operator, etc.) should direct excavation of the pit and safety during the test period. Of paramount concern should be the soil technician's safety, and obtaining enough tests to represent the fill.

Test pits should be excavated so that the spoil pile is placed away from oncoming traffic, whenever possible. The technician's vehicle is to be placed next to the test pit, opposite the spoil pile. This necessitates the fill be maintained in a driveable condition. Alternatively, the contractor may wish to park a piece of equipment in front of the test holes, particularly in small fill areas or those with limited access.

A zone of non-encroachment should be established for all test pits. No grading equipment should enter this zone during the testing procedure. The zone should extend approximately 50 feet outward from the center of the test pit. This zone is established for safety and to avoid excessive ground vibration, which typically decreases test results.

When taking slope tests, the technician should park the vehicle directly above or below the

test location. If this is not possible, a prominent flag should be placed at the top of the slope. The contractor's representative should effectively keep all equipment at a safe operational distance (e.g., 50 feet) away from the slope during this testing.

The technician is directed to withdraw from the active portion of the fill as soon as possible following testing. The technician's vehicle should be parked at the perimeter of the fill in a highly visible location, well away from the equipment traffic pattern. The contractor should inform our personnel of all changes to haul roads, cut and fill areas or other factors that may affect site access and site safety.

In the event that the technician's safety is jeopardized or compromised as a result of the contractor's failure to comply with any of the above, the technician is required, by company policy, to immediately withdraw and notify his/her supervisor. The grading contractor's representative will be contacted in an effort to affect a solution. However, in the interim, no further testing will be performed until the situation is rectified. Any fill placed can be considered unacceptable and subject to reprocessing, recompaction, or removal.

In the event that the soil technician does not comply with the above or other established safety guidelines, we request that the contractor bring this to the technician's attention and notify this office. Effective communication and coordination between the contractor's representative and the soil technician is strongly encouraged in order to implement the above safety plan.

Trench and Vertical Excavation

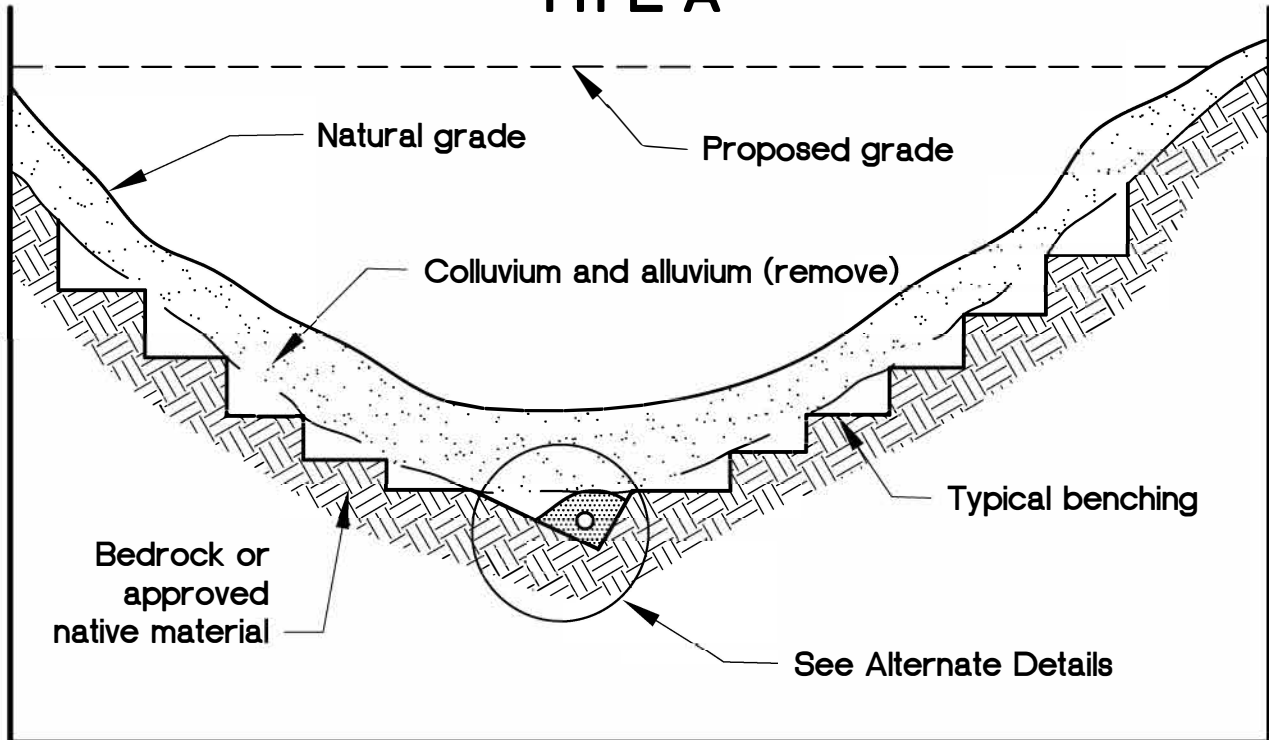
It is the contractor's responsibility to provide safe access into trenches where compaction testing is needed. Our personnel are directed not to enter any excavation or vertical cut which: 1) is 5 feet or deeper unless shored or laid back; 2) displays any evidence of instability, has any loose rock or other debris which could fall into the trench; or 3) displays any other evidence of any unsafe conditions regardless of depth.

All trench excavations or vertical cuts in excess of 5 feet deep, which any person enters, should be shored or laid back. Trench access should be provided in accordance with Cal/OSHA or state and local standards. Our personnel are directed not to enter any trench by being lowered or "riding down" on the equipment.

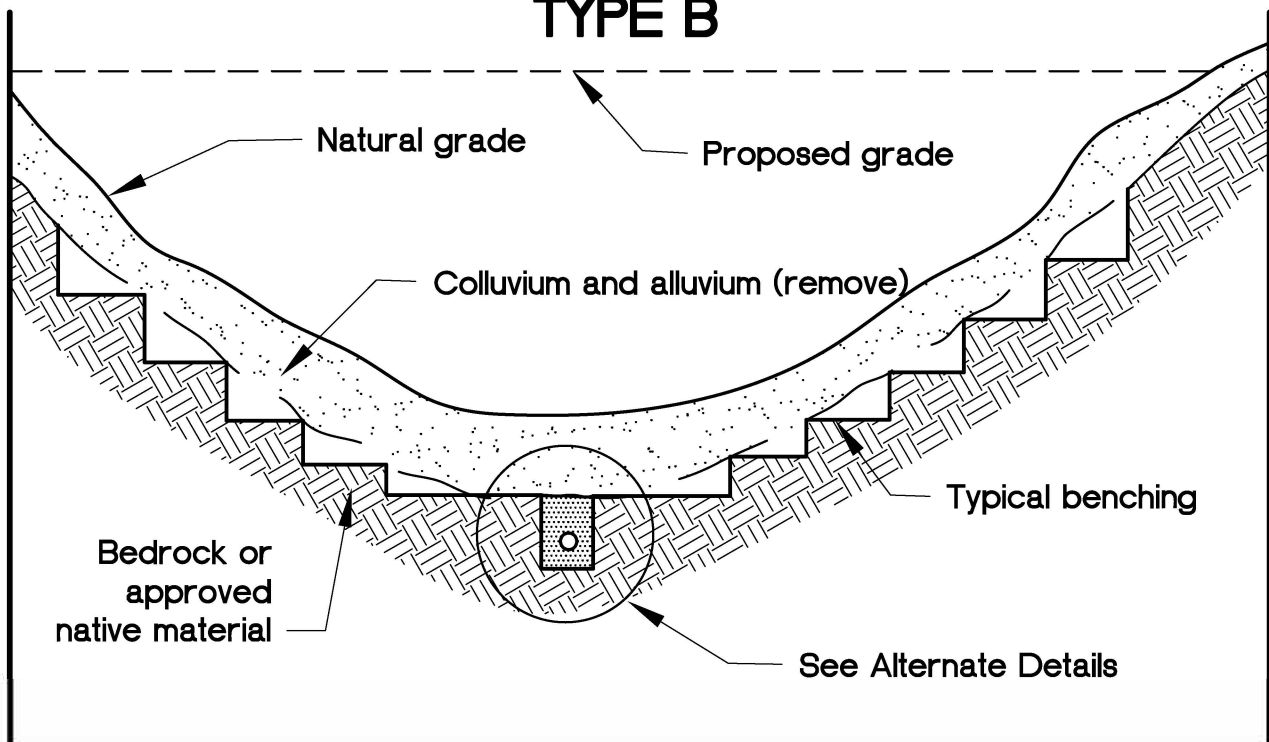
If the contractor fails to provide safe access to trenches for compaction testing, our company policy requires that the soil technician withdraw and notify his/her supervisor. The contractor's representative will be contacted in an effort to affect a solution. All backfill not tested due to safety concerns or other reasons could be subject to reprocessing or removal.

If GSI personnel become aware of anyone working beneath an unsafe trench wall or vertical excavation, we have a legal obligation to put the contractor and owner/developer on notice to immediately correct the situation. If corrective steps are not taken, GSI then has an obligation to notify Cal/OSHA or the proper controlling authorities.

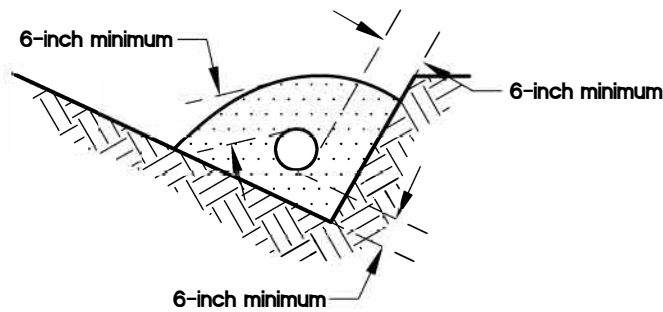
TYPE A



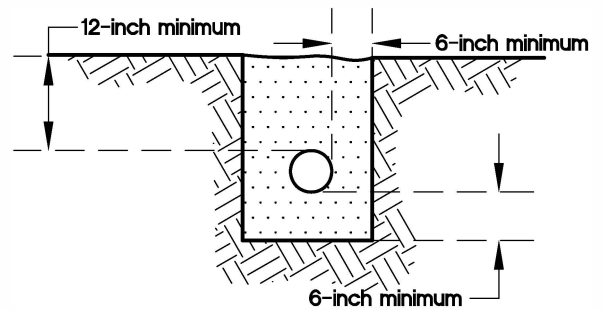
TYPE B



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



A-1



B-1

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

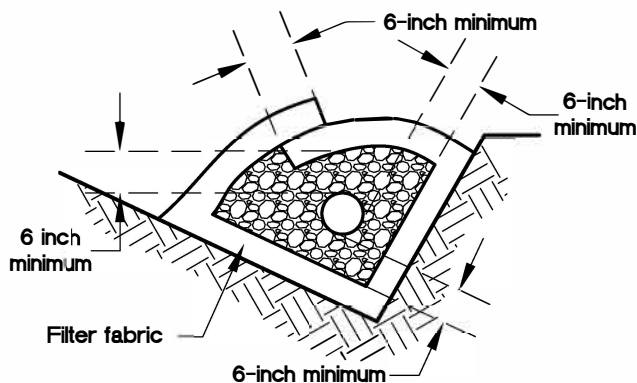
Perforated pipe: 6-inch-diameter ABS or PVC pipe or approved substitute with minimum 8 perforations ($\frac{1}{4}$ -inch diameter) per lineal foot in bottom half of pipe (ASTM D-2751, SDR-35, or ASTM D-1527, Schd. 40).

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

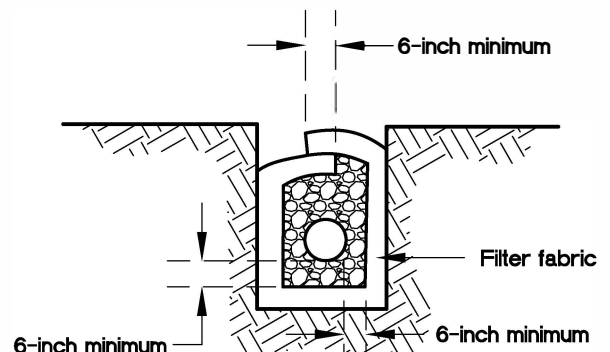
FILTER MATERIAL

<u>Sieve Size</u>	<u>Percent Passing</u>
1 inch	100
$\frac{3}{4}$ inch	90-100
$\frac{3}{8}$ 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

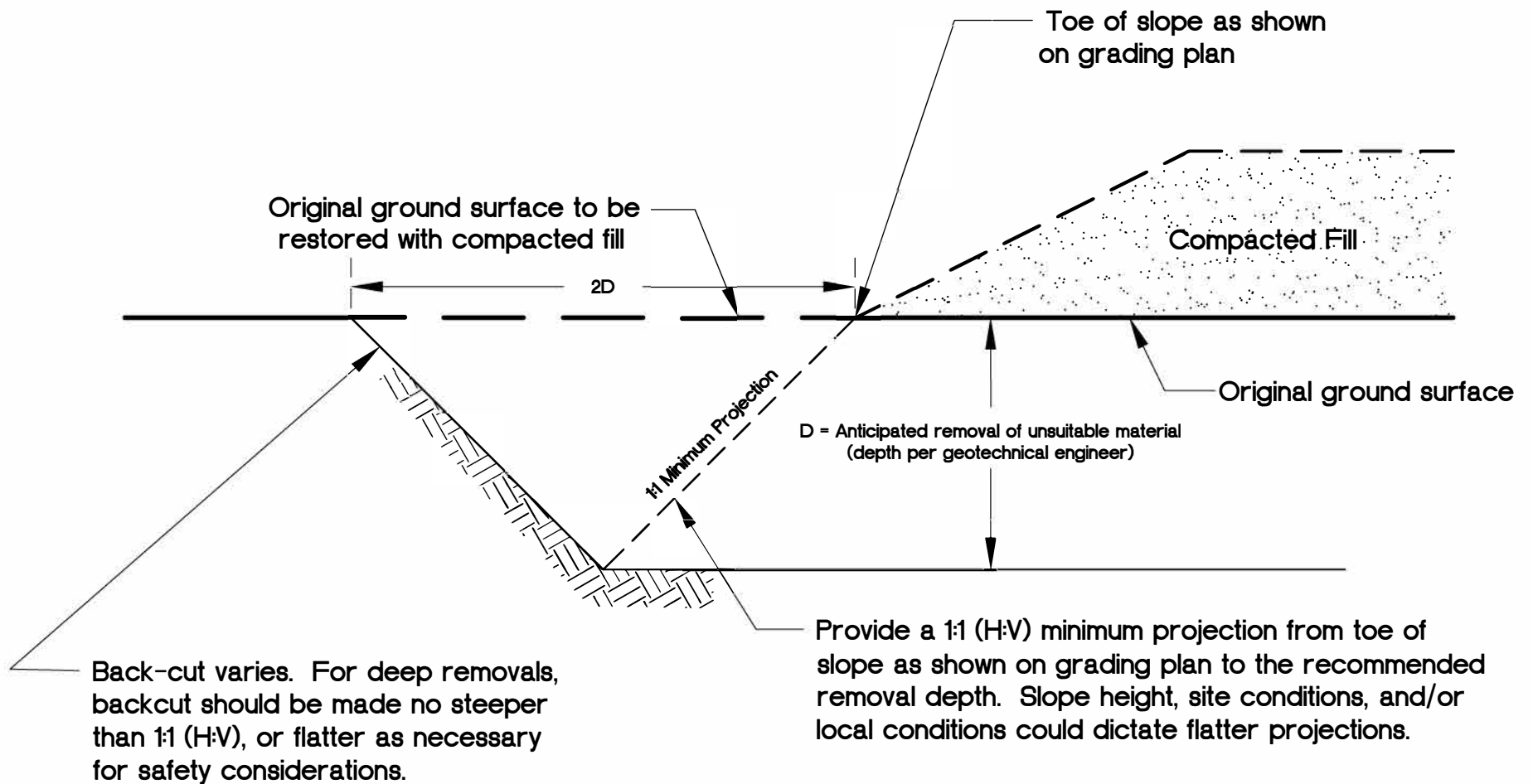
Gravel Material: 9 cubic feet per lineal foot.

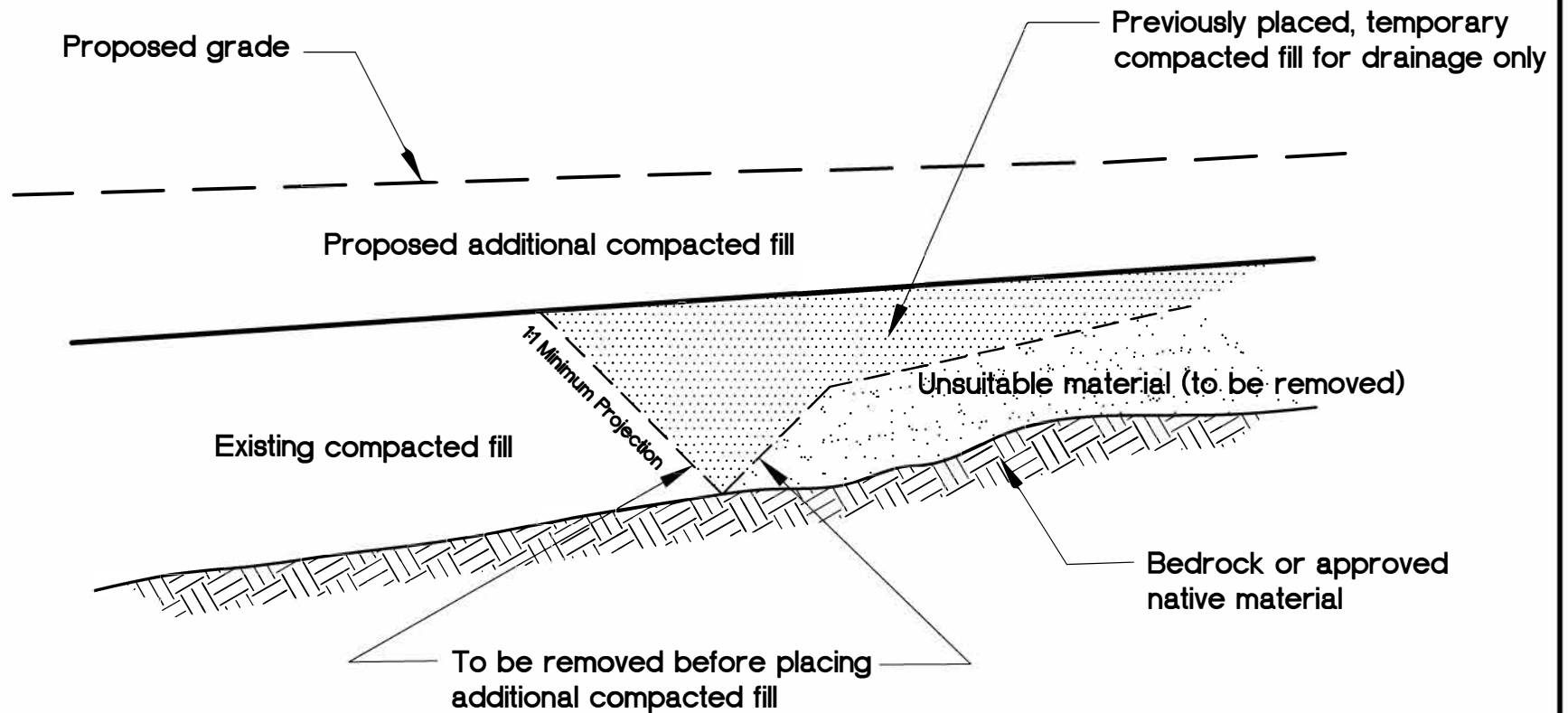
Perforated Pipe: See Alternate 1

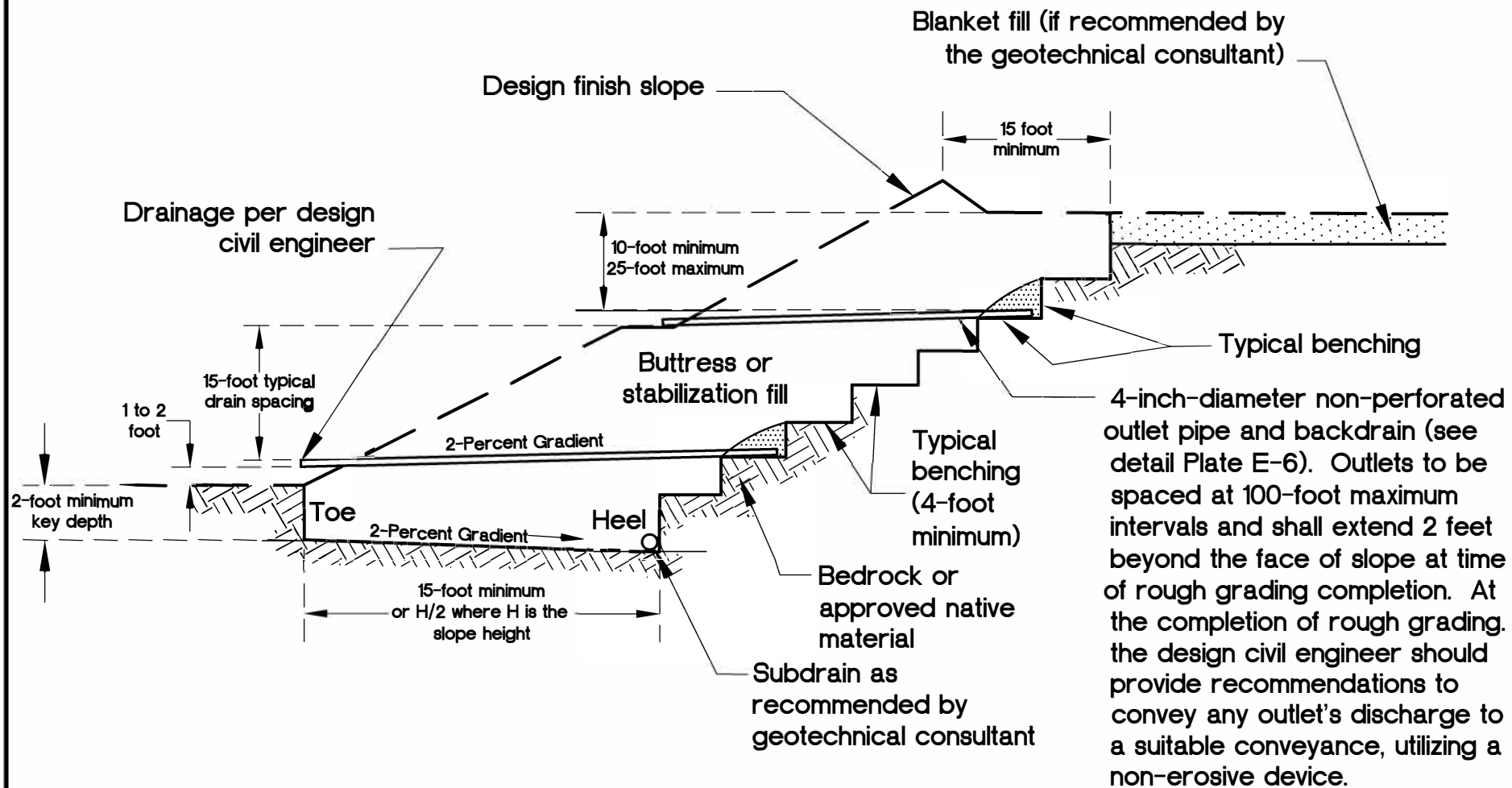
Gravel: Clean $\frac{3}{4}$ -inch rock or approved substitute.

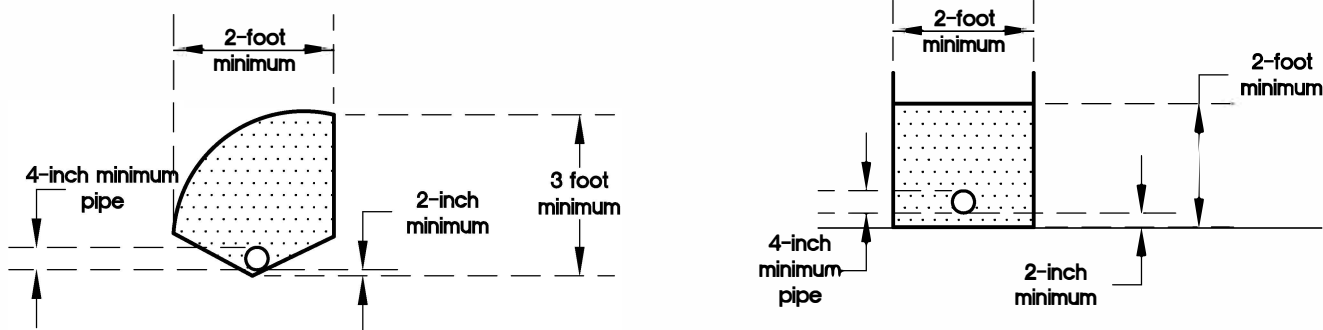
Filter Fabric: Mirafi 140 or approved substitute.

ALTERNATE 2: PERFORATED PIPE, GRAVEL, AND FILTER FABRIC









Filter Material: Minimum of 5 cubic feet per lineal foot of pipe or 4 cubic feet per lineal 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 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.

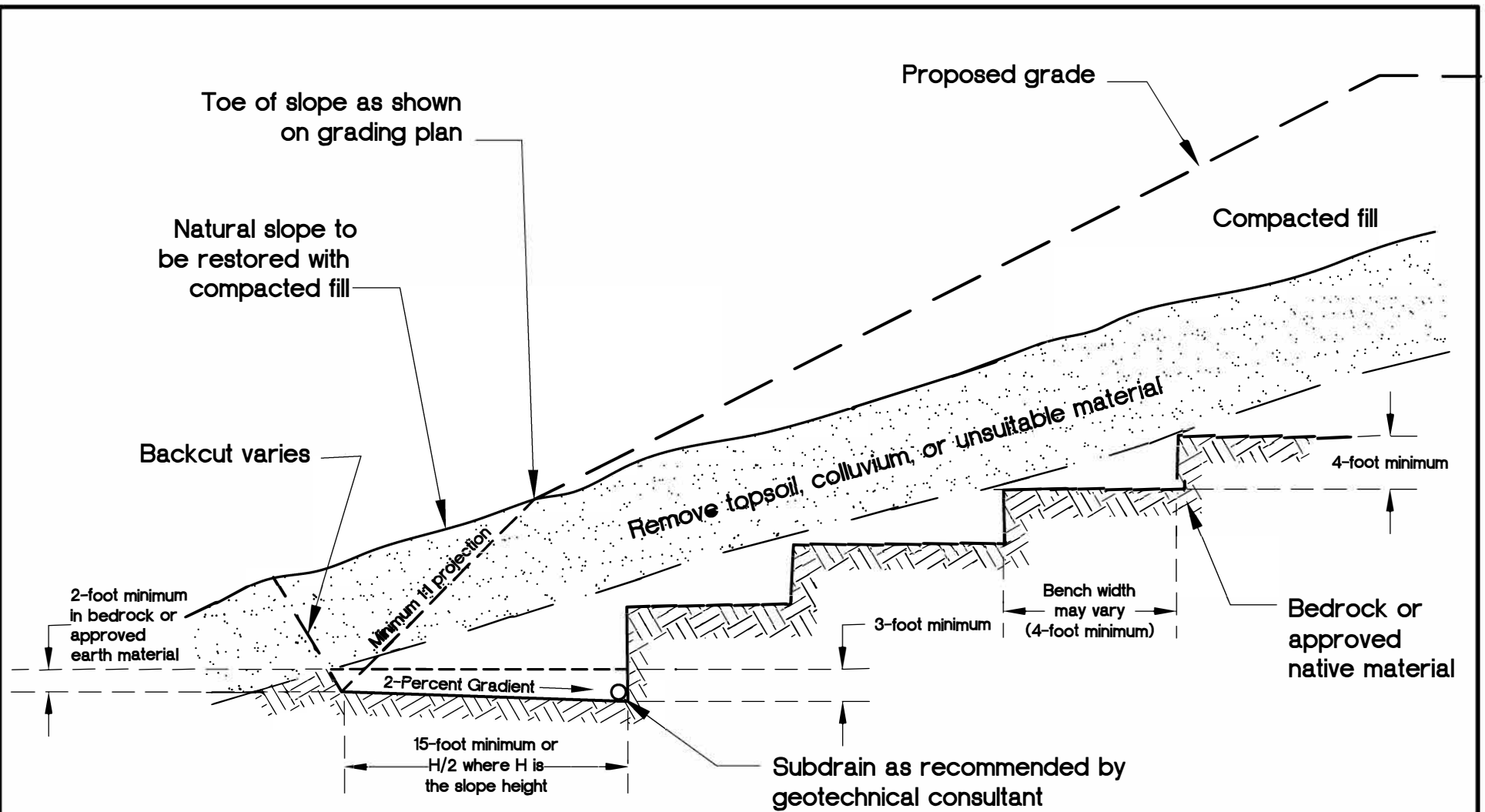
- Notes:**
1. Trench for outlet pipes to be backfilled and compacted with onsite soil.
 2. Backdrains and lateral drains shall be located at elevation of every bench drain. First drain located at elevation just above lower lot grade. Additional drains may be required at the discretion of the geotechnical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

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

<u>Sieve Size</u>	<u>Percent Passing</u>
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

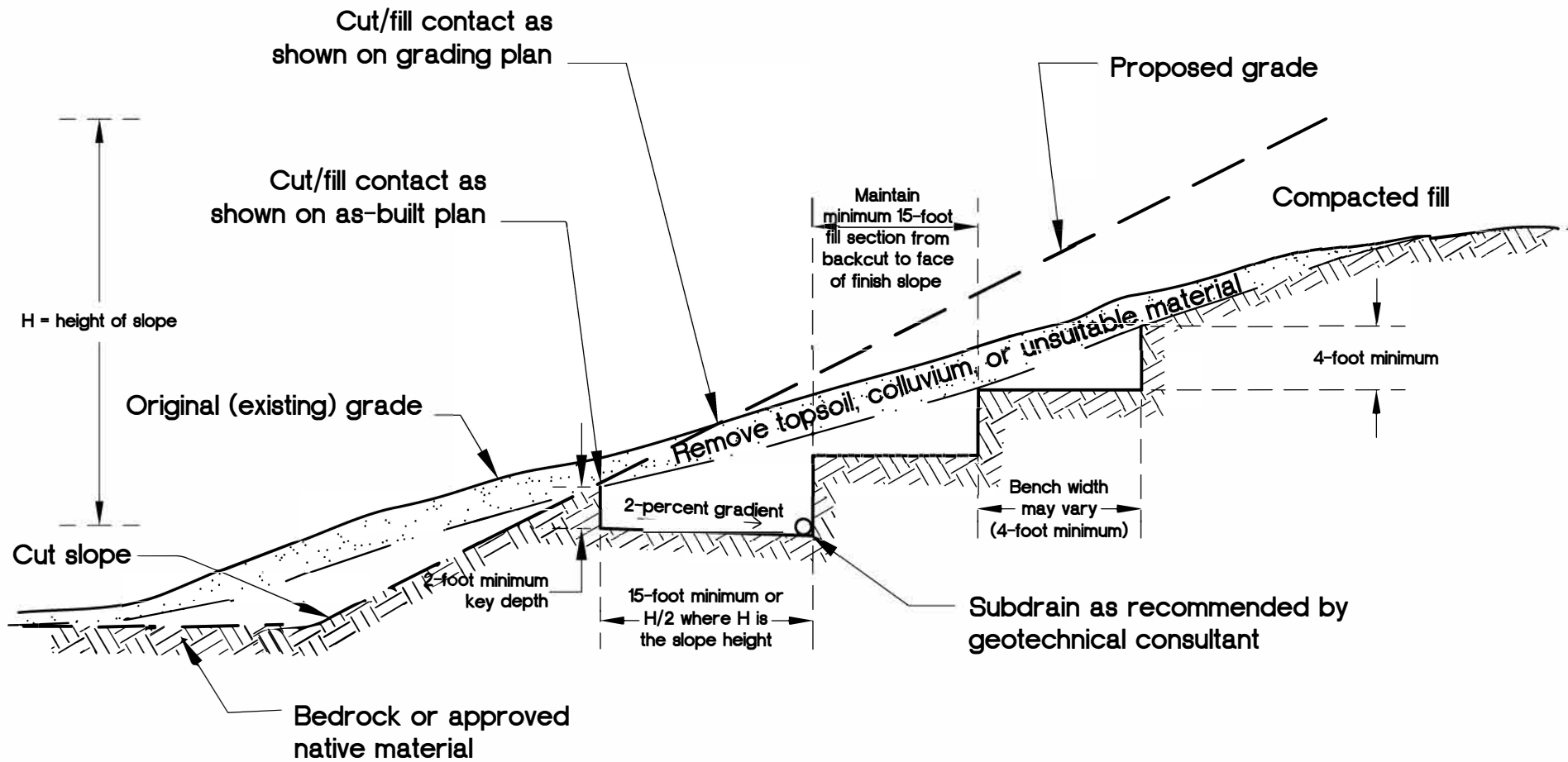
Gravel shall be of the following specification or an approved equivalent.

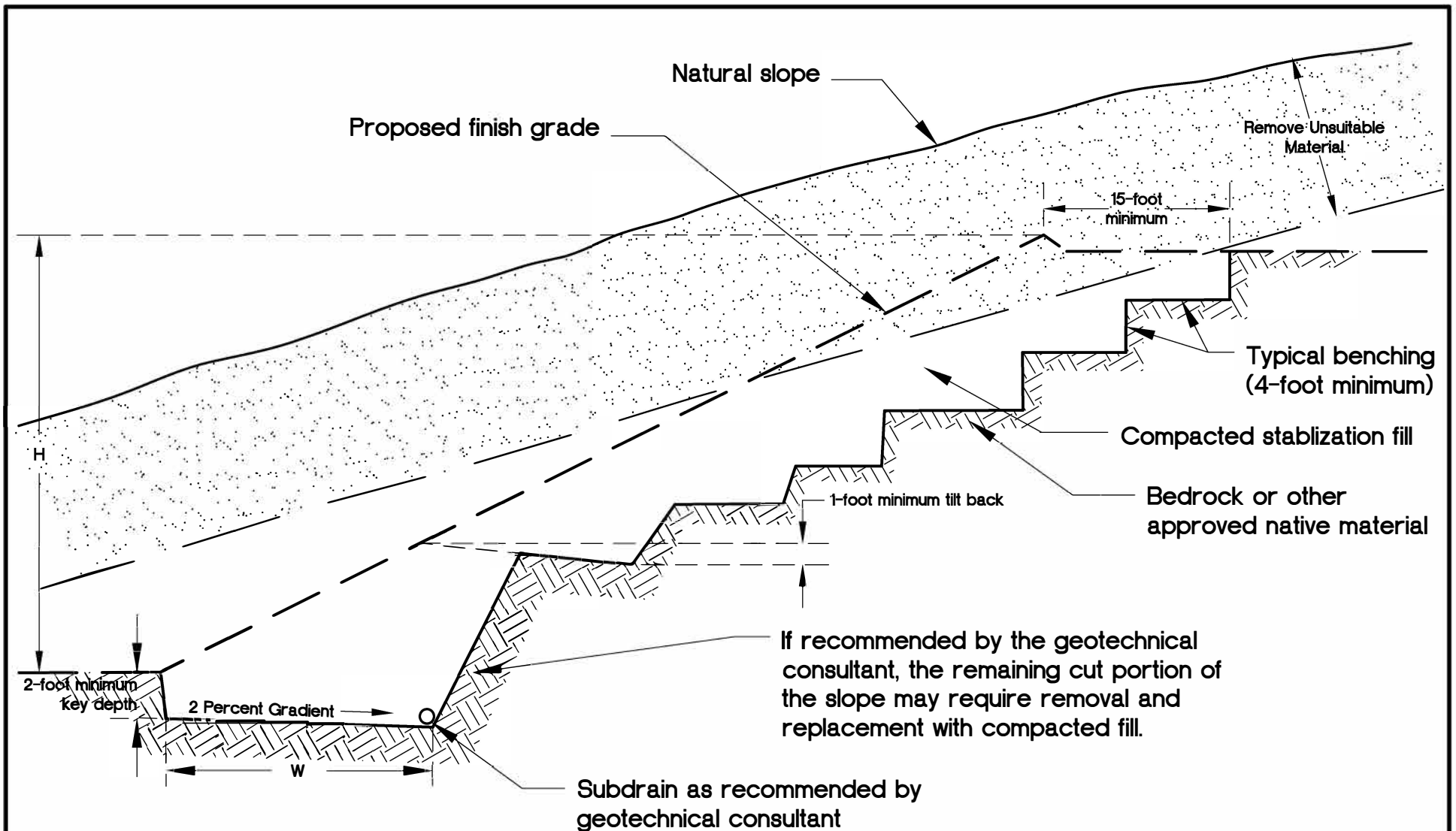
<u>Sieve Size</u>	<u>Percent Passing</u>
1½ inch	100
No. 4	50
No. 200	8



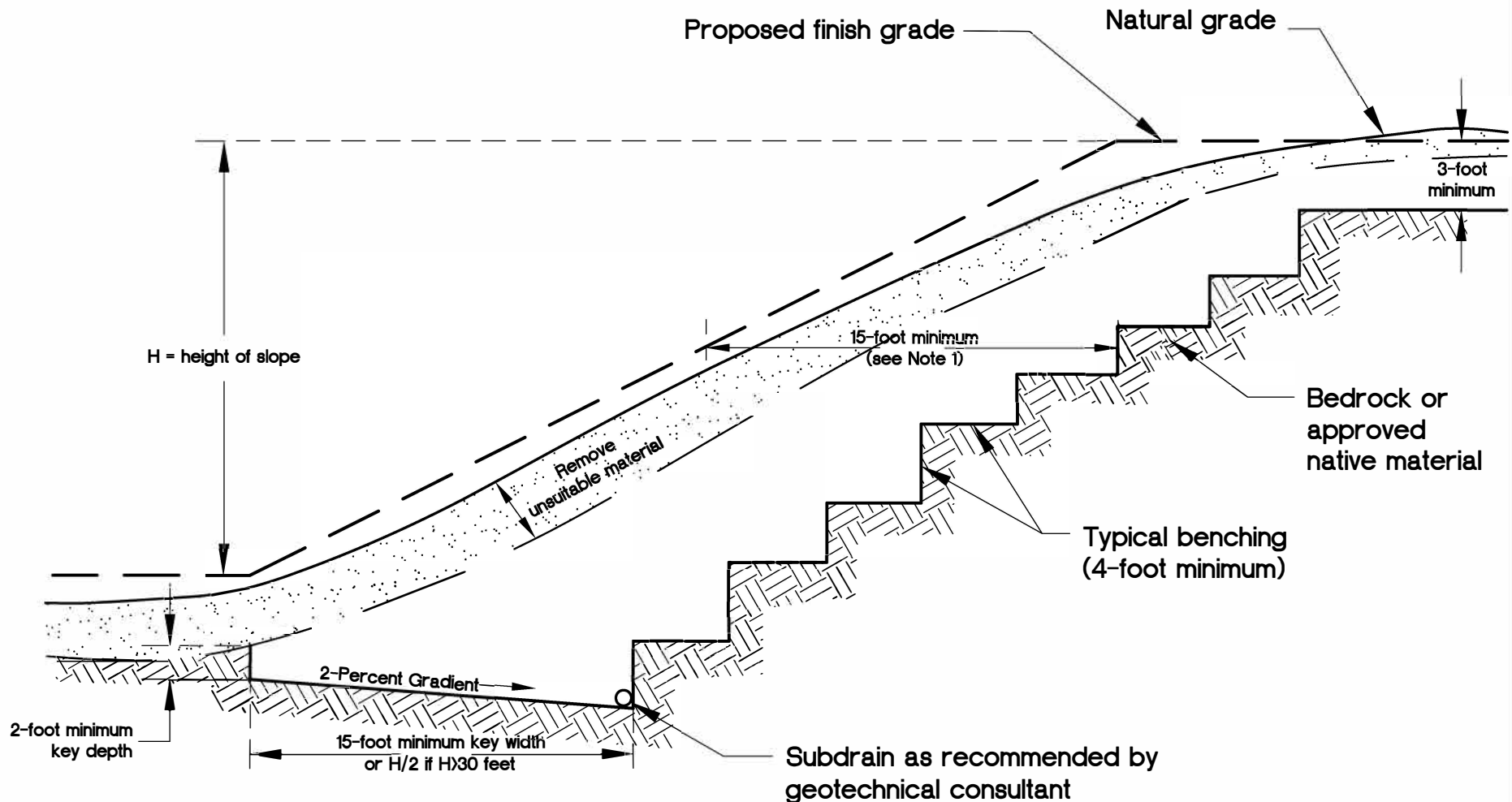
NOTES:

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.

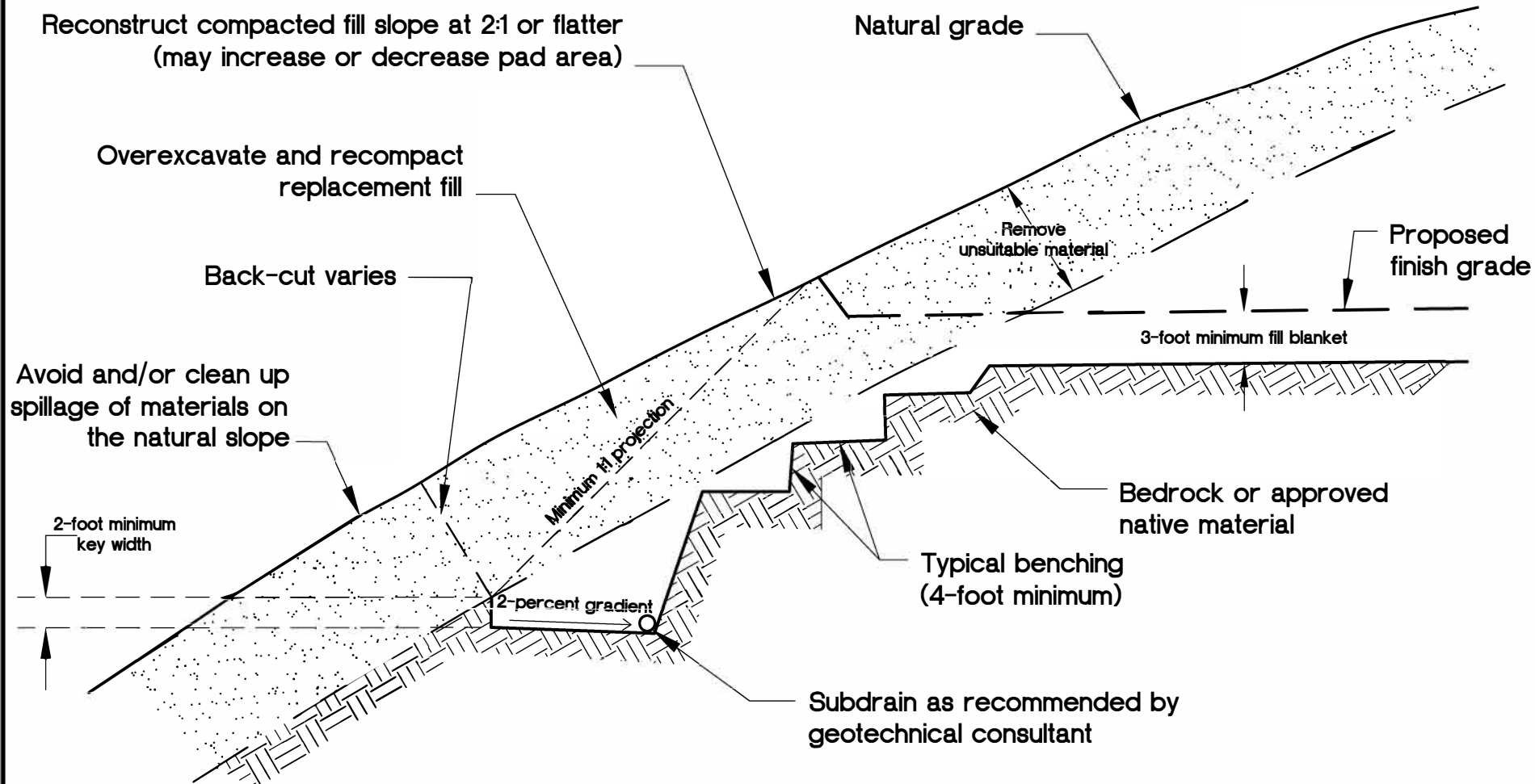




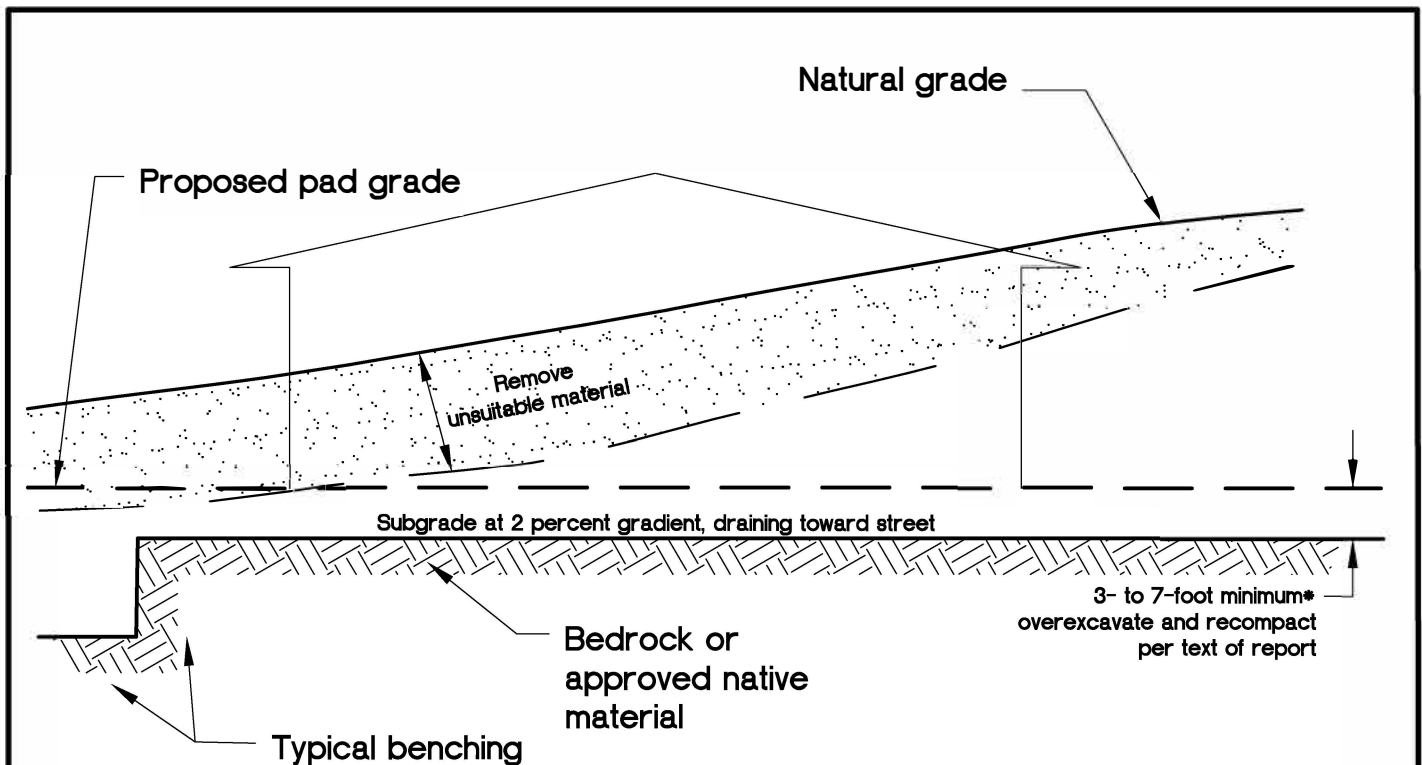
- NOTES:
1. Subdrains may be required as specified by the geotechnical consultant.
 2. W shall be equipment width (15 feet) for slope heights less than 25 feet. For slopes greater than 25 feet, W shall be evaluated by the geotechnical consultant. At no time, shall W be less than $H/2$, where H is the height of the slope.



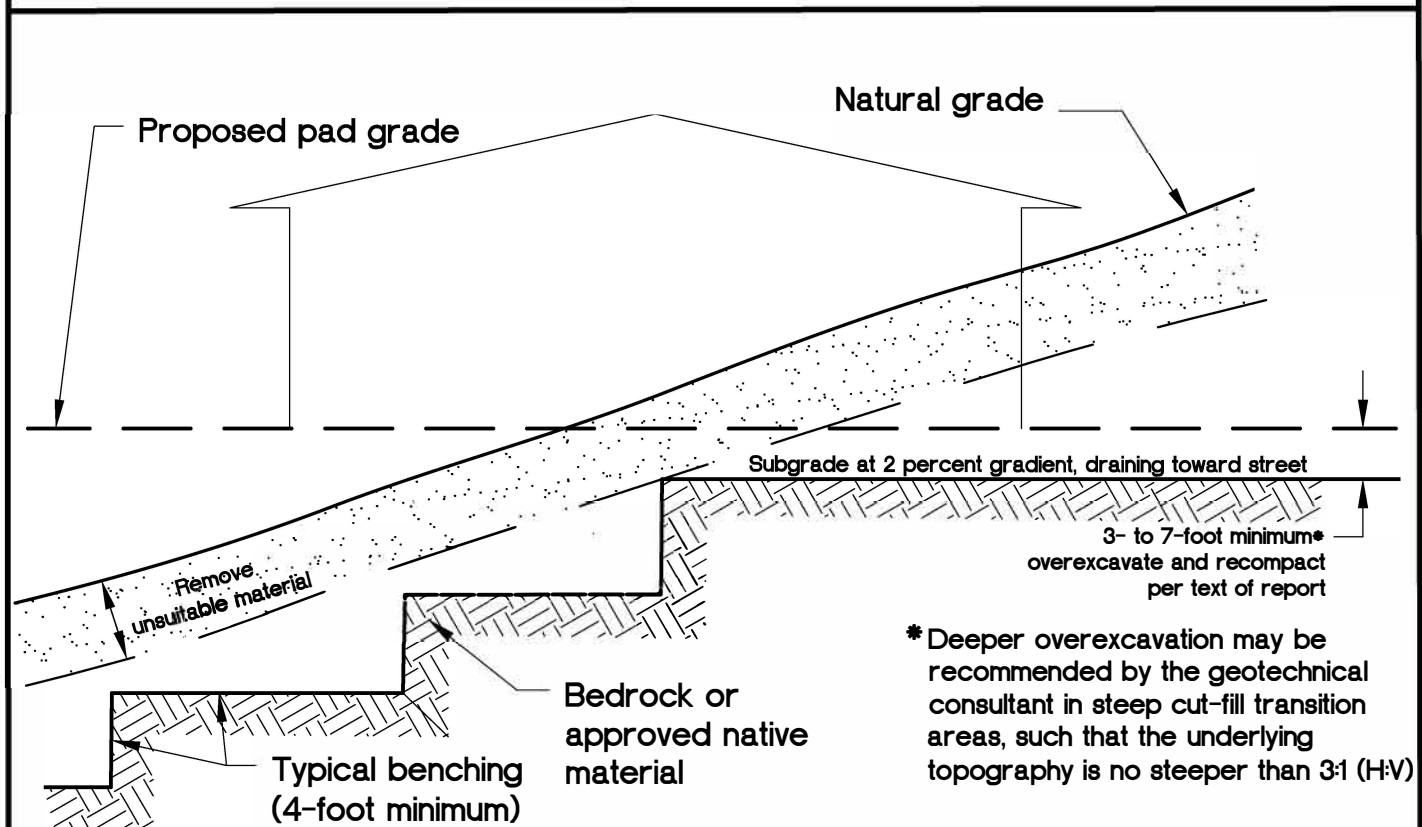
- NOTES:
1. 15-foot minimum to be maintained from proposed finish slope face to backcut.
 2. The need and disposition of drains will be evaluated by the geotechnical consultant based on field conditions.
 3. Pad overexcavation and recompaction should be performed if evaluated to be necessary by the geotechnical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



- 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.



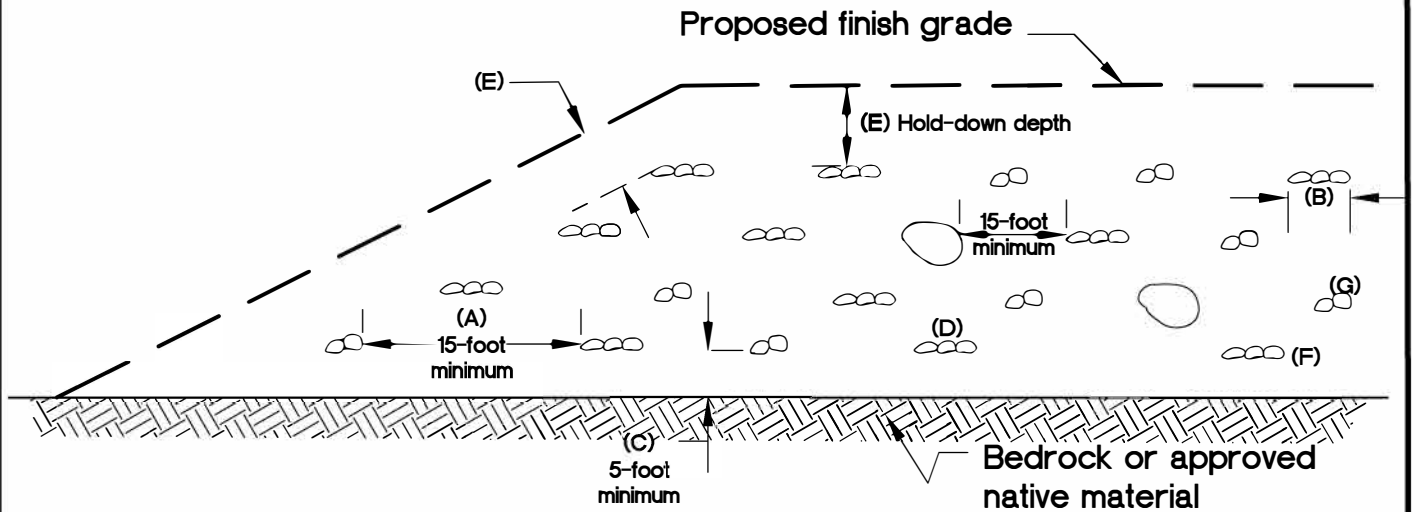
CUT LOT OR MATERIAL-TYPE TRANSITION



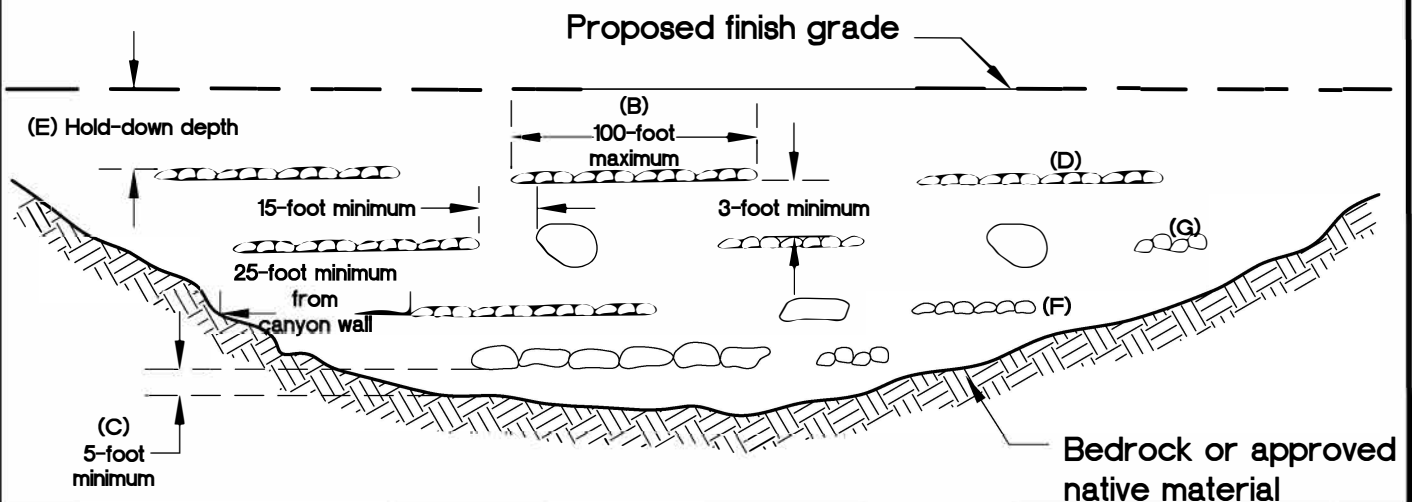
* Deeper overexcavation may be recommended by the geotechnical consultant in steep cut-fill transition areas, such that the underlying topography is no steeper than 3:1 (H:V)

CUT-FILL LOT (DAYLIGHT TRANSITION)

VIEW NORMAL TO SLOPE FACE



VIEW PARALLEL TO SLOPE FACE



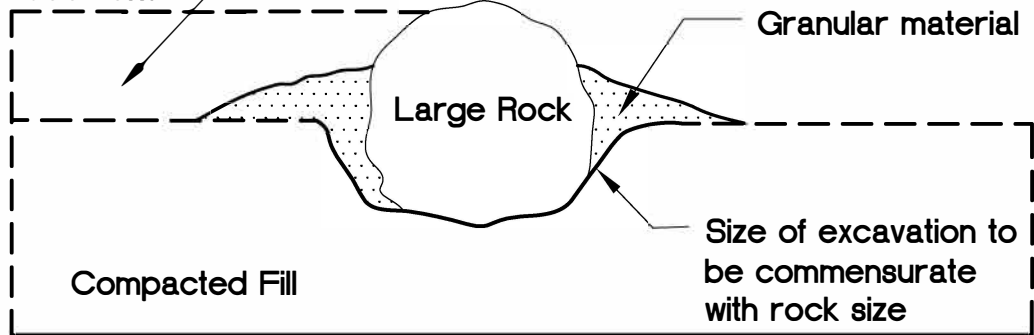
NOTES:

- One equipment width or a minimum of 15 feet between rows (or windrows).
- Height and width may vary depending on rock size and type of equipment. Length of windrow shall be no greater than 100 feet.
- If approved by the geotechnical consultant, windrows may be placed directly on competent material or bedrock, provided adequate space is available for compaction.
- 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.
- Clear area for utility trenches, foundations, and swimming pools; Hold-down depth as specified in text of report, subject to governing agency approval.
- All fill over and around rock windrow shall be compacted to at least 90 percent relative compaction or as recommended.
- 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

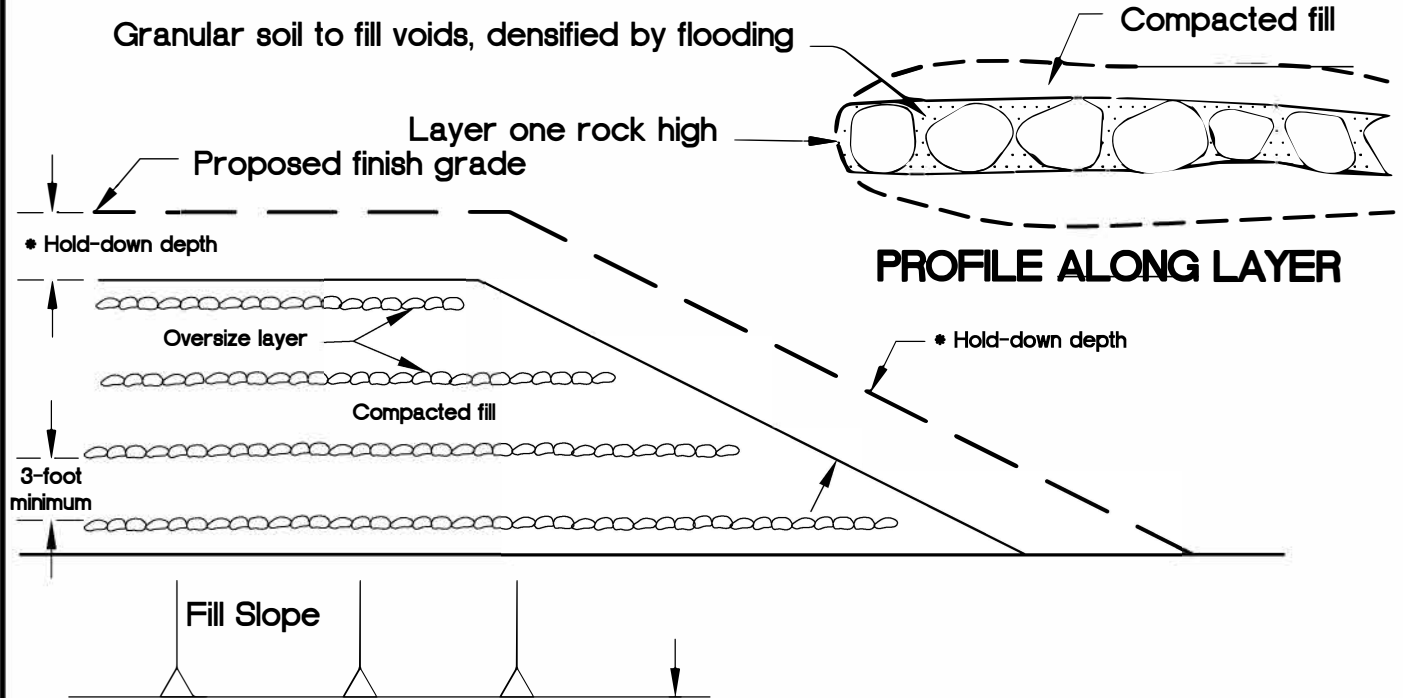
ROCK DISPOSAL PITS

Fill lifts compacted over rock after embedment

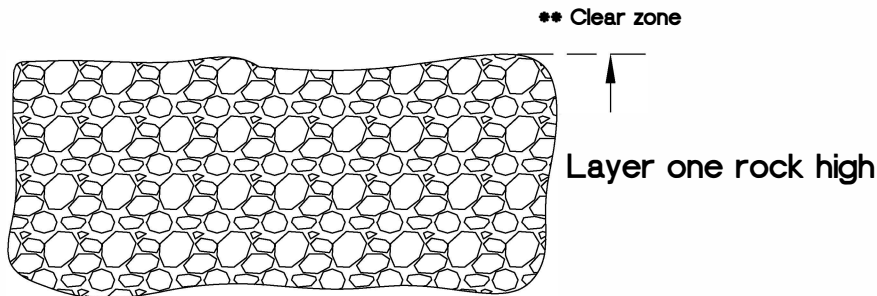


ROCK DISPOSAL LAYERS

Granular soil to fill voids, densified by flooding



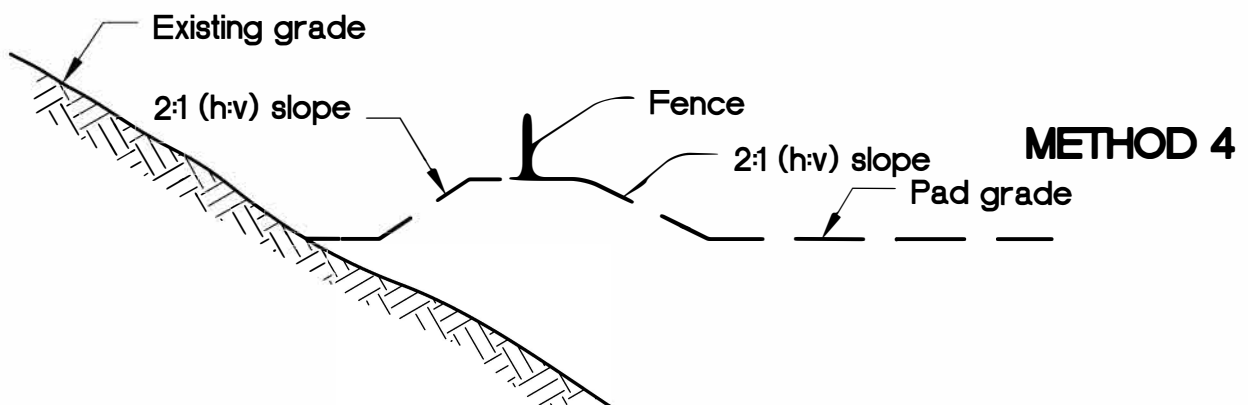
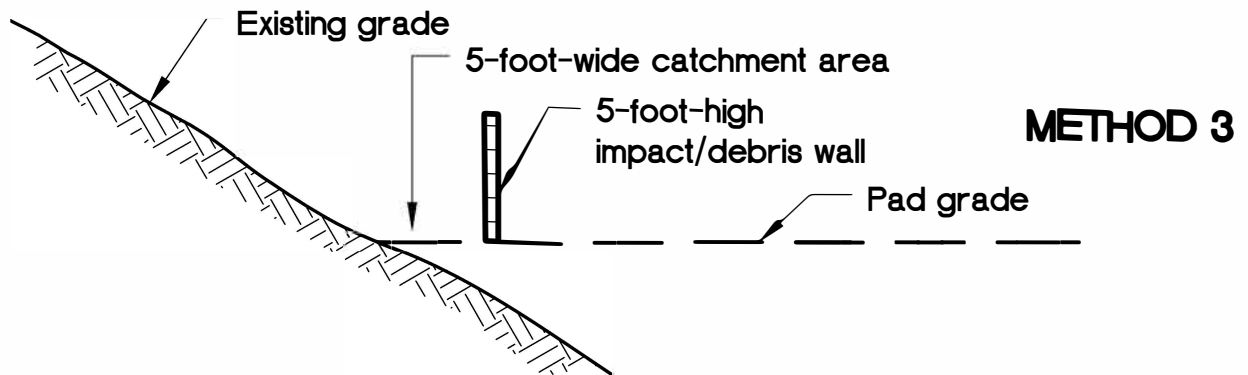
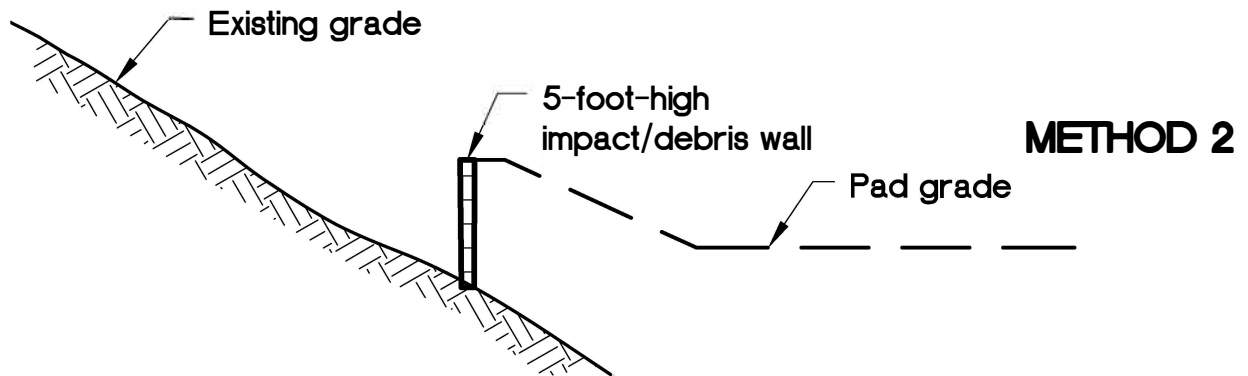
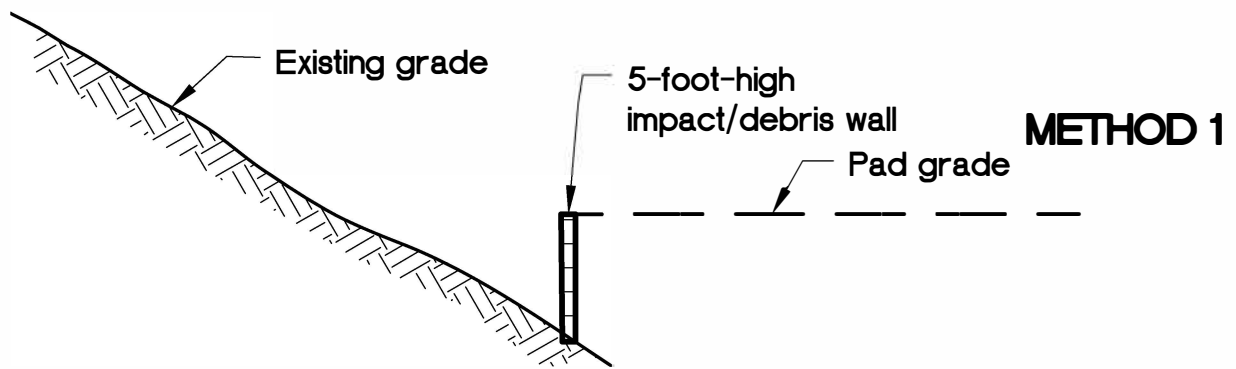
TOP VIEW



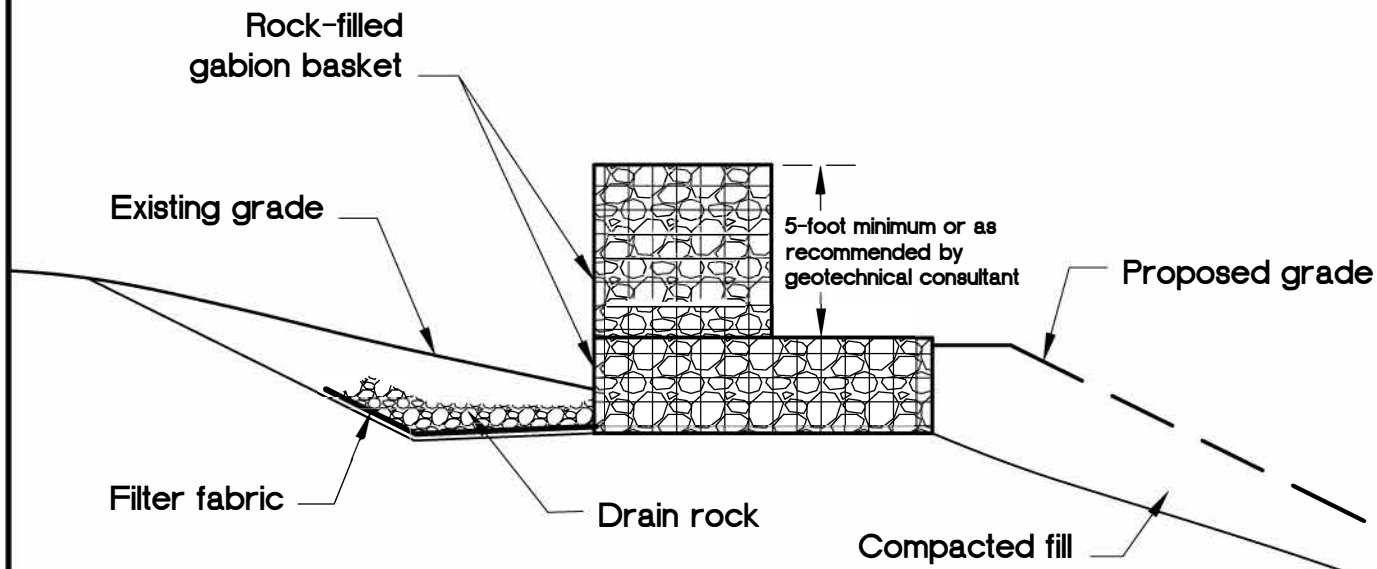
* Hold-down depth or below lowest utility as specified in text of report, subject to governing agency approval.

** 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



NOT TO SCALE

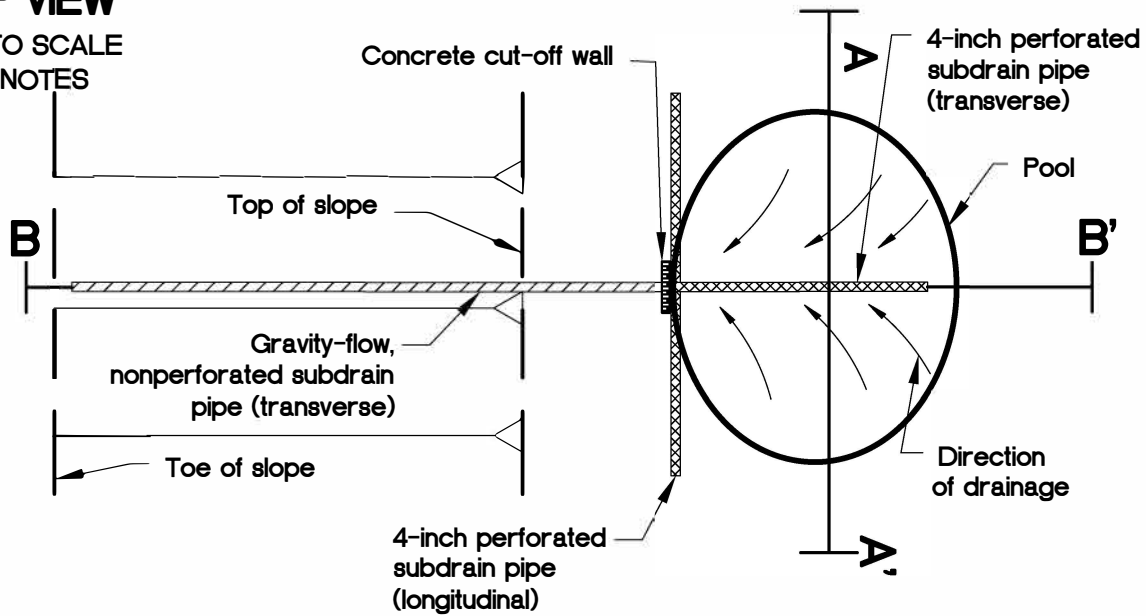


Gabion impact or diversion wall should be constructed at the base of the ascending slope subject to rock fall. Walls need to be constructed with high segments that sustain impact and mitigate potential for overtopping, and low segment that provides channelization of sediments and debris to desired depositional area for subsequent clean-out. Additional subdrain may be recommended by geotechnical consultant.

From GSA, 1987

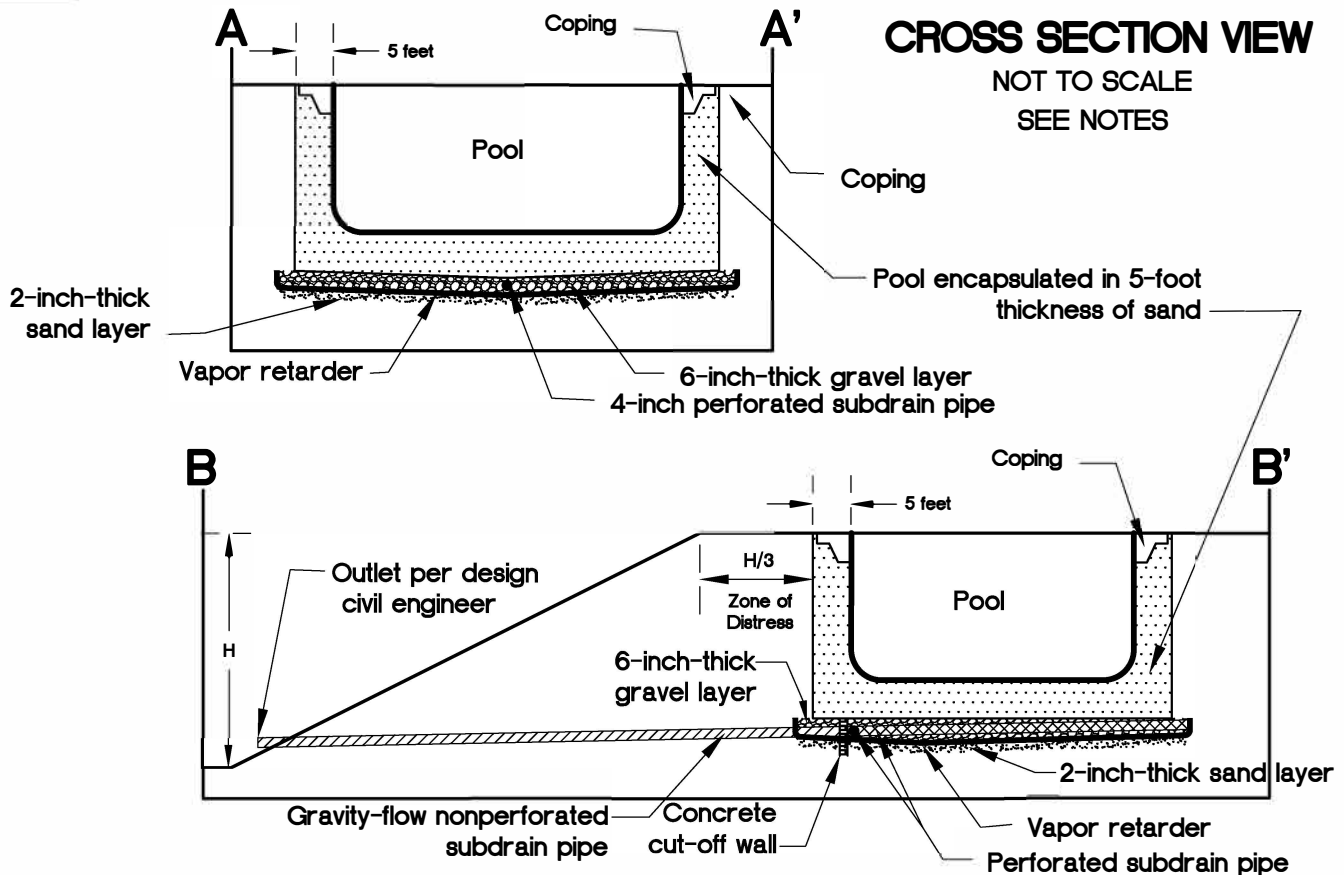
MAP VIEW

NOT TO SCALE
SEE NOTES



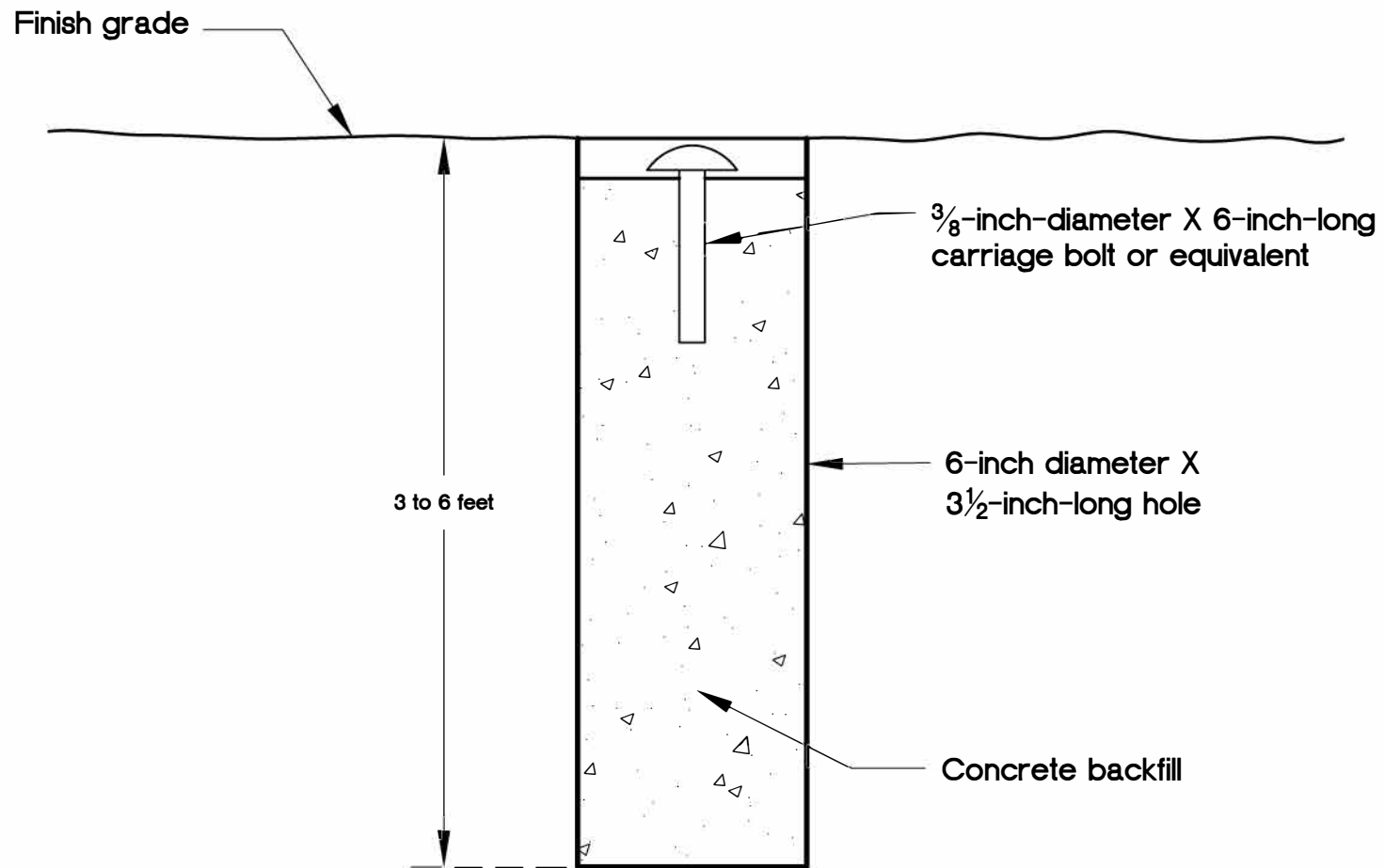
CROSS SECTION VIEW

NOT TO SCALE
SEE NOTES

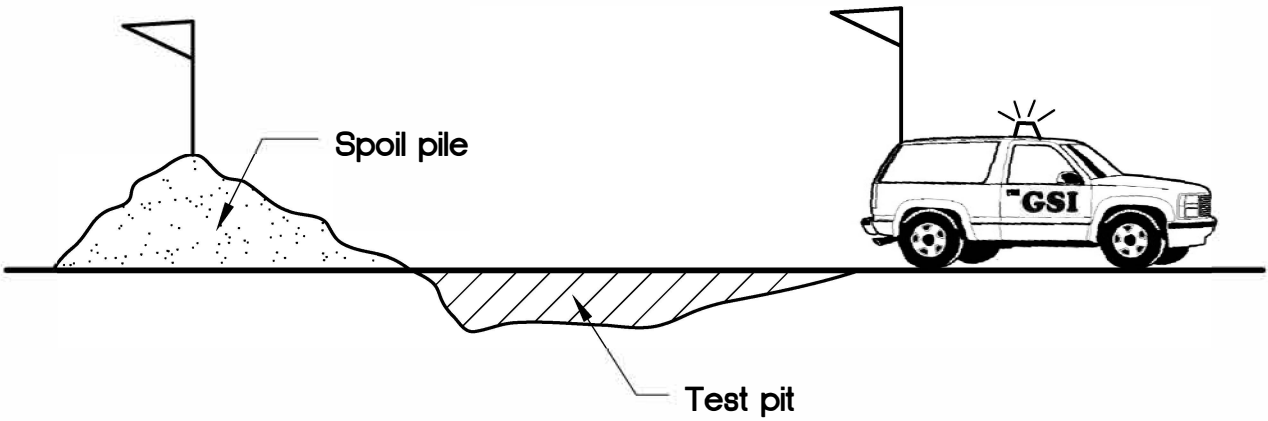


NOTES:

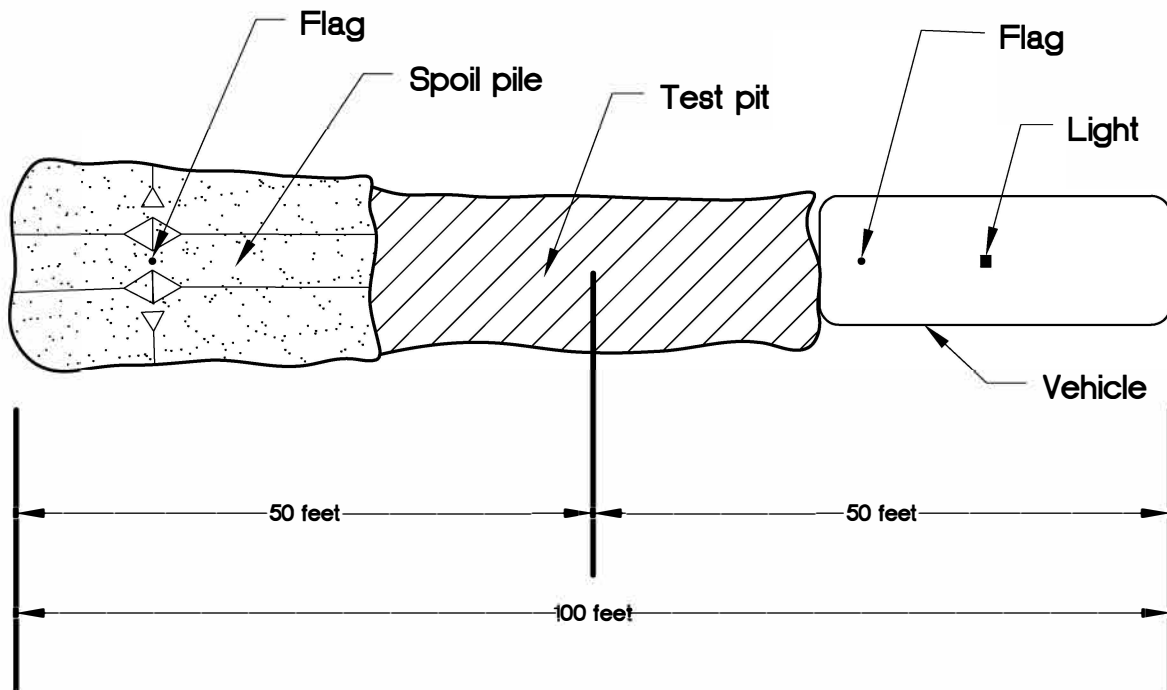
1. 6-inch-thick, clean gravel ($\frac{3}{4}$ to $1\frac{1}{2}$ inch) sub-base encapsulated in Mirafi 140N or equivalent, underlain by a 15-mil vapor retarder, with 4-inch-diameter perforated pipe longitudinal connected to 4-inch-diameter perforated pipe transverse. Connect transverse pipe to 4-inch-diameter nonperforated pipe at low point and outlet or to sump pump area.
2. Pools on fills thicker than 20 feet should be constructed on deep foundations; otherwise, distress (tilting, cracking, etc.) should be expected.
3. Design does not apply to infinity-edge pools/spas.

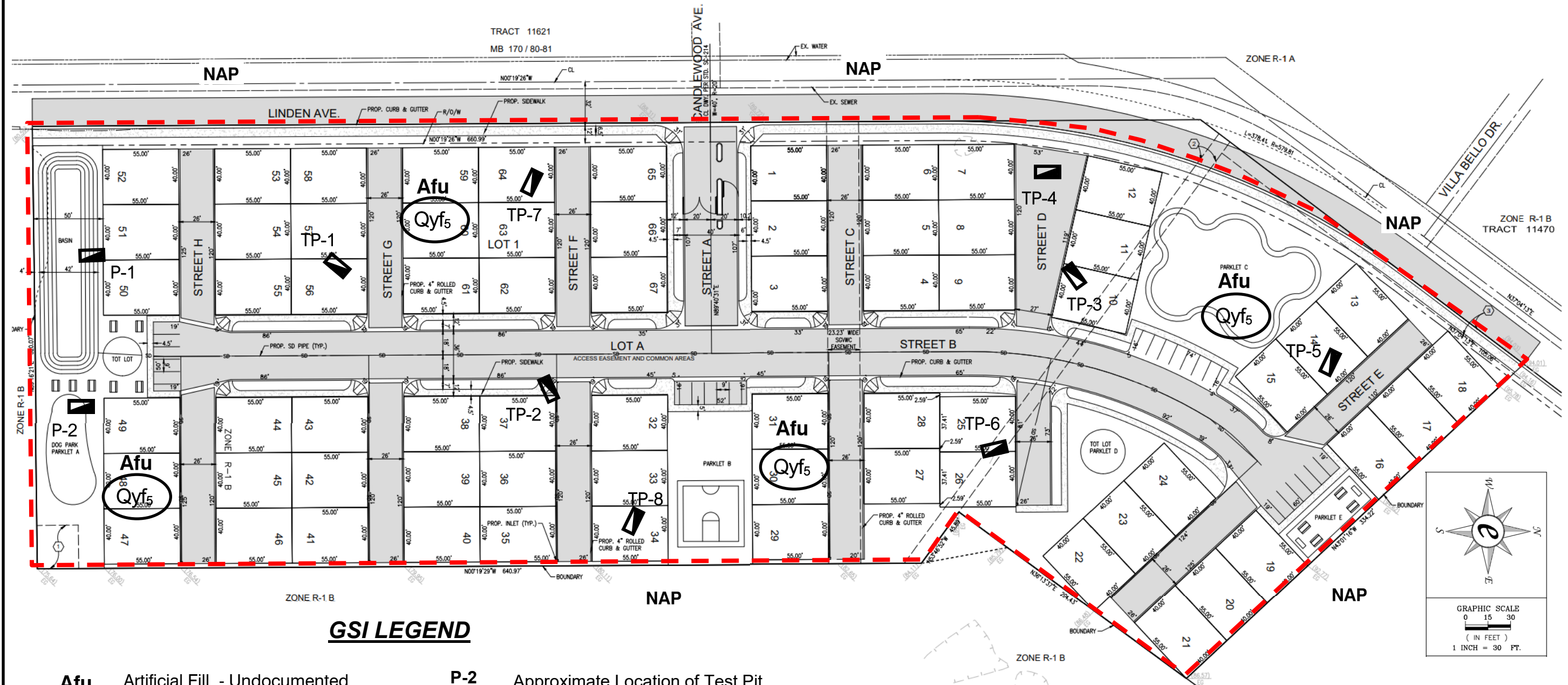


SIDE VIEW



TOP VIEW





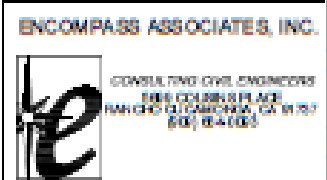
GSi LEGEND

- | | | | |
|-------------|---|------------|---|
| Afu | Artificial Fill - Undocumented. | P-2 | Approximate Location of Test Pit for Percolation Feasibility Testing. |
| Qyf5 | Quaternary-age Young Alluvial-Fan Deposits, Unit 5, Circled Where Buried. | | Approximate Area Under the Purview of this Report. |
| TP-8 | Approximate Location of Exploratory Test Pit. | NAP | Not A Part of this Study. |

ALL LOCATIONS ARE APPROXIMATE

This document or e-file is not part of the Construction Documents and should not be relied upon as being an accurate depiction of design.

Figure adapted from the Tentative Tract Map by Encompass Associates, Inc., plotted March 13, 2025.



TENTATIVE TRACT NO. 20793

TENTATIVE TRACT MAP



Plate 1

GEOTECHNICAL MAP

W.O. 8972-A-IE	DATE: 09/25	SCALE: See Bar Scale
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