REPORT OF GEOTECHNICAL INVESTIGATION
LANTERN CREST RIDGE II
11010 SUNSET TRAIL
SANTEE, CALIFORNIA

Prepared for

DEVELOPMENT CONTRACTOR, INC.
110 Town Center Parkway
Santee, California 92071

Prepared by

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Project No. SD528
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GROUP DELTA

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Development Contractor, Inc.
110 Town Center Parkway
Santee, California 92071

Attention: Mr. Michael Grant

SUBJECT: REPORT OF GEOTECHNICAL INVESTIGATION
Lantern Crest Ridge II
11010 Sunset Trail
Santee, California

Mr. Grant:

This report provides the results of our geotechnical investigation for the proposed Lantern Crest Ridge II development in Santee, California. This report was prepared in general accordance with our proposal dated March 27, 2017.

Site development appears to be conceptually feasible from a geotechnical perspective. However, several geotechnical constraints exist that will need to be addressed prior to development. Conclusions regarding the geologic conditions and geotechnical recommendations for design and construction are provided in this report.

We appreciate this opportunity to be of continued professional service. Feel free to contact the office with any questions or comments, or if you need anything else.

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1.0 INTRODUCTION

This report presents the results from our geotechnical investigation for the proposed Lantern Crest Ridge II development in Santee, California. The approximate location of the site is shown on the Site Location Map, Figure 1A. The site vicinity is shown in Figure 1B. Geotechnical Maps with aerial images and preliminary grading plans are shown in Figures 2A and 2B, respectively. The approximate locations of the exploratory boring, test pits and interpreted site geology are also shown in Figures 2A and 2B.

The purpose of this geotechnical investigation was to characterize geotechnical conditions at the site and to provide geotechnical recommendations for design and construction. The recommendations are based on our recent subsurface exploration and laboratory testing, geologic and engineering analyses, and our previous experience with similar geologic conditions.

1.1 Scope of Services

This report was prepared in general accordance with the provisions of the referenced proposal (GDC, 2017). In summary, we provided the following scope of services.

- Geologic reconnaissance of the general site conditions, and delineated the exploration locations for Underground Service Alert (USA) to identify existing utilities onsite.
- Subsurface exploration including one exploratory boring and seven exploratory test pits at the approximate locations shown in Figures 2A and 2B. Logs of these explorations are provided in Appendix A.
- Laboratory testing on selected soil samples collected from the explorations. The laboratory test results are summarized in Appendix B.
- Infiltration testing including two falling head infiltration tests at the approximate locations shown in Figures 2A and 2B. Infiltration test results and our assessment of infiltration feasibility are provided in Appendix C.
- Geologic and engineering analyses of the field and laboratory data to develop recommendations for design and construction.
- Preparing this report summarizing our findings, conclusions and preliminary geotechnical recommendations.

1.2 Site Description

The undeveloped property is located at 11010 Sunset Trail in Santee, California, as shown on the Site Location Map, Figure 1A. The 2.8-acre site is roughly rectangular, with an approximate centroid located at a longitude of 32.8337° north and latitude of 116.9593° west, as shown on the Site Vicinity Plan, Figure 1B. The site is accessed via Sunset Trail on the southern side of the site. The western boundary is bordered by multi- and single-family residential properties. The multifamily residential property has an approximately 10-foot tall crib retaining wall just west of the property boundary. The eastern boundary is undeveloped along the northern half, and has an existing multi-story senior living facility and approximately 15foot tall segmental retaining wall along the southern half. The northern boundary is also undeveloped.

There are no known existing structures or improvements on the site. During our site reconnaissance and subsurface investigation, most of the site was covered with a relatively heavy growth of weeds and grass.

The property generally slopes down to the west southwest at inclinations ranging from 2 to 1 (horizontal to vertical) in the northeastern portion of the site to 6 to 1 on the southern portion of the site. The northern portion is generally more rugged, with several natural drainages running approximately east-west across the site. The approximate locations of the natural drainages are shown on Figures 2A and 2B. Elevations range from a low of about 490 feet above mean sea level (MSL) on the southwest portion of the site, up to a high of about 580 feet MSL at the northeast corner.

1.3 Proposed Development

A site and preliminary grading plan prepared by POLARIS Development Consultants, Inc. (undated) showing the general layout of the proposed development is included as Figure 2B. We understand the site development may include a three-story residential building with a basement level and two single-story duplex structures supported on shallow foundations and on-grade slabs. Other site improvements will include retaining walls, asphalt concrete paved driveways, and parking areas, as well as Portland Cement Concrete sidewalks, flatwork, curbs, gutters, and driveways, a biofiltration basin, and a variety of subsurface utilities.

Cut and fill earthwork will be needed to create level building areas. Based on our review of the preliminary grading plan (POLARIS, undated), maximum cuts up to about 20 feet are proposed on the north side of the new building. Fill slopes with heights up 10 feet at inclinations of 2:1 (horizontal:vertical) are also proposed throughout the site. No permanent cut slopes are proposed at this time. Several geogrid reinforced segmental retaining walls are also proposed throughout the site with heights up to 18½ feet. Basement and site retaining walls up to about 20 feet high will also be constructed along the eastern side of the new building.

2.0 FIELD AND LABORATORY INVESTIGATION

The subsurface conditions were investigated by excavating one exploratory boring (B-1) and seven test pits (TP-1 through TP-7) at the approximate locations shown on Figures 2A & 2B. The boring was drilled on March 24, 2017, and the test pits were completed on May 27, 2017. The boring and test pits were excavated using a track-mounted limited access drill rig with six-inch diameter hollow-stem augers and a John Deere 410K backhoe with a 24-inch wide bucket, respectively. Each exploration encountered refusal at depths ranging from one to eight feet below the ground surface. Logs describing the subsurface conditions observed in these explorations are presented in Appendix A.

Representative bulk soil samples were collected from the test pits and relatively intact and disturbed driven samples were collected from the boring. The samples were transported to our laboratory for further visual examination and laboratory testing. Laboratory tests were completed to assist in classifying and correlating samples, and evaluating their physical and engineering properties. The laboratory tests included an evaluation of in-situ moisture content, particle size analysis, soil classification, corrosivity, expansion index, relationship between dry unit and moisture content (compaction curve), and R-value. Details of the laboratory testing program, including test results, are provided in Appendix B.

3.0 INFILTRATION TESTING

The feasibility for infiltration at site was evaluated by performing two infiltration tests (B-1A and B-1B) at the approximate locations shown on Figures 2A and 2B. The test holes were drilled in conjunction with Boring B-1 on March 24, 2017, using the track-mounted limited access drill rig equipped with six-inch diameter hollow stem augers. The depth of each infiltration test was approximately five feet. Following drilling, test holes were constructed by cleaning the sidewalls, placing about 2 inches of pea gravel at the bottom of the hole, placing a 5-foot long section of 4-inch diameter slotted PVC casing into the hole, and backfilling the annulus between the sidewalls and casing with pea gravel. The test holes were then pre-soaked with a water column of about 36 inches for a period of about 18 hours prior to the start of the infiltration test.

Infiltration testing was performed on March 25, 2017. Infiltration testing was conducted using the Borehole Percolation Test method (Riverside County Percolation Test, 2011) referenced in the City of Santee BMP Design Manual (2016). The results of infiltration testing are summarized below.

Test Hole	Stabilized Infiltration Rate inches/hour	Design Infiltration Rate* inches/hour
B-1A	0.13	0.06
B-1B	1.20	0.60
	Average	**0.33**

* Design infiltration rate adopted a factor of safety of 2.0.

The infiltration feasibility is discussed further in the Stormwater Infiltration section of this report, and the test data and completed Worksheet C.4-1 are attached in Appendix C.

4.0 GEOLOGY AND SUBSURFACE CONDITIONS

The site is in the mountain range foothills region of the Peninsular Ranges geomorphic province of southern California. The mountain ranges are underlain primarily by Mesozoic metamorphic rocks that were intruded by plutonic rocks of the southern California batholith. Specifically, the site is underlain by crystalline Granitic Rock, which is covered with variable depths of colluvium. The interpreted geologic conditions at the site are depicted on the Regional Geologic Map, Figure 3. Geologic conditions at the site are described below.

4.1 Granitic Rock

Early Cretaceous-age granitic rock (map symbol Kgr – undivided tonalite and granodiorite) is underlies the site, as shown on the Geologic Map, Figure 3. Granitic rock was encountered in all our explorations at the ground surface or underlying the colluvium at depths ranging from about one to four feet below existing grades.

The rock was typically observed to consist of decomposed to fresh granitic rock, which generally excavated as a silty fine to coarse sand (where decomposed to intensely weathered) with variable amounts of fresh rock fragments up to about 2 feet in diameter. The rock is generally gray, light brown, and orange-brown. The weathered rock was observed to have a relative density ranging from dense to very dense based on backhoe excavation effort and Standard Penetration Testing (SPT) conducted during drilling. Backhoe and auger refusal was encountered in all our explorations on hard rock at depths ranging from one to eight feet below existing grades.

In addition, several outcrops of unweathered granitic rock, boulders and core stones were observed at various portions of the site. The outcrops indicate an irregular surface of hard crystalline bedrock across the site. Corestones and outcrops may be boulder-sized and up to greater than 20 feet in diameter. The approximate locations of the observed outcrops are shown on Figures 2A and 2B.

4.2 Colluvium

Colluvium is soil that is transported down slope by the force of gravity. Colluvium covers the granitic rock throughout most of the site. Colluvium was encountered in all the explorations except for B-1, which encountered decomposed granitic rock at the ground surface. Colluvium was encountered at the existing ground surface and extended down to depths up to about four feet. The colluvium was observed to predominantly consist of reddish brown to brown silty sand with variable amounts of gravel, cobble and boulder sized rock fragments up to about two feet in diameter. The colluvium was observed to have a loose relative density based on backhoe excavation effort.

4.3 Groundwater

Groundwater was not observed in the explorations that extended to a maximum depth of 8 feet below existing grades. The State Water Resources Control Board website (GeoTracker, 2017), indicates groundwater elevations at the United States Border Patrol Station located at 225 Kenney Street in El Cajon (about 2,000 feet southwest of the site) ranged from approximately 360 to 384 feet above MSL from 2007 to 2016, which is more than 100 feet below existing grades at the site. Note that variations in rainfall, irrigation or site drainage conditions may create zones of wet soil or seepage. Such conditions are difficult to predict, and are typically mitigated if and where they occur.

5.0 GEOLOGIC HAZARDS

The site is not located within an area previously known for significant geologic hazards. Evidence of past landslides, liquefaction or active faulting at the site was not encountered in our geotechnical investigation or in our literature review. We anticipate the main geologic hazard will be the potential for strong ground shaking from an earthquake. Each of the geologic hazards is described below.

5.1 Ground Rupture

Ground rupture is not considered to be a substantial geologic hazard at the site. Ground rupture is the result of movement on an active fault reaching the ground surface. No indications of active faulting were found in our reconnaissance or literature review. The site is not located within a State of California Earthquake Fault Hazard Zone. An unnamed pre-quaternary age fault is mapped approximately 2½ kilometers (about 1½ miles) southwest of the site, and is labeled under the category of inactive, potentially active, or activity unknown. The nearest known active fault is part of the Rose Canyon fault zone that is about 23 km (about 14 miles) west of the site (USGS, 2008). The locations of known active faults within a 100 km (about 60 miles) radius of these site are shown on the Fault Location Map, Figure 4.

5.2 Seismicity

The United States Geologic Survey has developed an interactive website that provides Next Generation Attenuation (NGA) probabilistic seismic analyses based on the site location and shear wave velocity (USGS, 2009). Based on these analyses, and assuming an average shear wave velocity of 360 meters / second (m/s) at the site, we estimate that the peak ground accelerations (PGA) with a 2, 5 and 10 percent probability of being exceeded in a 50-year period at the site are approximately 0.38, 0.29g and 0.22g, respectively. These three risk levels are often referred to as the Maximum Considered (MCE), Upper Bound (UBE) and Design Basis Earthquakes (DBE), respectively. The shaking hazard may be mitigated by structural design of the buildings per California Building Code.

5.3 Soil Liquefaction and Seismic Compaction

Groundwater was not encountered and granitic rock underlies the site. Therefore, the potential for soil liquefaction and its secondary effects should be very low. Liquefaction is a phenomenon where loose, saturated coarse-grained soils lose their strength and acquire some mobility from the strong ground motion induced by earthquakes. The secondary effects of liquefaction include sand boils, settlement, reduced soil shear strength, lateral spreading, and global instability (flow slides) in areas with sloping ground.

The potential for seismic compaction should be low since loose unsaturated coarse grained soils will be removed and replaced as compacted fill. Seismic compaction is the settlement of loose unsaturated granular soils from strong ground shaking.

5.4 Landslides and Slope Instability

Evidence of ancient landslides or slope instabilities were not observed during our literature review, site reconnaissance, or subsurface exploration. Relatively steep rock slopes are present to the east of the site. These slopes appear to be stable and the risk for deep seated slope failure should be low. However, boulders or surficial weathered zones may be loosened during periods of heavy rain or earthquake and boulder stabilization or removal and rock fall mitigation should be considered in these areas.

5.5 Rockfall

Outcrops of hard rock and large boulders were observed on existing slopes above the development offsite to the East. Further evaluation should be considered to determine if any of the large rocks have a potential to fall downslope into the project as a result of weathering or during a seismic event. Rockfall hazards should be mitigated during earthwork construction and can include blasting or chemical splitting of large boulders, or mechanical removal of loose rock with earthmoving equipment.

5.6 Expansive Soils

Our expansion index testing performed on a representative sample of the onsite colluvium indicates a low potential for expansion. However, expansive clayey soils may be locally present in some of the colluvium. These materials should be selectively placed within fill and/or mixed with non-expansive soils during earthwork. They should not be a significant hazard at the site if appropriate earthwork practices are maintained.

5.7 Tsunamis, Seiches and Flooding

Given the distance between the subject site and the coast, and the elevation of the site above mean sea level (more than 400 feet), the potential for damage due to tsunamis or seiches is remote. The site is not located within a FEMA 100-year flood zone (FEMA, 2012), and consequently, the potential for flooding is low.

6.0 CONCLUSIONS

Site development appears to be conceptually feasible from a geotechnical perspective. However, several geotechnical constraints exist that will need to be addressed prior to development. Conclusions regarding the geotechnical conditions at the site are provided below and geotechnical recommendations for design and construction are provided in the following sections of this report.

- Colluvium underlies portions of the site. The colluvium was observed to be loose. All colluvium in development areas should be excavated and replaced as properly compacted fill.

- Complete removal of unsuitable soils may be difficult to accomplish along the property boundaries without extending the remedial grading off-site, or establishing structural setback zones within the site.

- Granitic rock was encountered throughout the site at the ground surface and underlying the colluvium. Excavations extending into granitic rock should be anticipated to be very difficult, and heavy ripping, rock hammering, and/or blasting may be needed.

- Soils generated from excavations should be suitable for reuse as properly compacted fill, provided the recommendations in this report are met. Screening and/or crushing of oversized materials, processing, and moisture conditioning should be anticipated.

- Shallow foundations are suitable for support of the planned structures. Recommendations for design are provided in the following sections.

- Several natural drainages cross the site. Flow within these drainages should be directed away from the proposed improvements in non-erodible drainage devices to a suitable outlet.

- The main geologic hazard at the site is the potential for strong ground shaking from an earthquake. The shaking hazard may be mitigated by structural design of the buildings per the applicable building code and stabilization or removal of any boulders observed in the ridge above the site that could be dislodged from strong ground shaking.

- The potential for full or partial infiltration has been assessed in accordance with the City of Santee BMP Design Manual (2016). Infiltration testing yielded a design average infiltration rate of about 0.3 inches per hour, assuming a factor of safety of 2.0. Our feasibility screening of the potential for on-site infiltration resulted in the "no infiltration" category. Worksheet C.4-1 is presented in Appendix C.

7.0 RECOMMENDATIONS

The remainder of this report presents recommendations for earthwork and the preliminary design for the proposed structures and associated improvements. These recommendations are based on empirical and analytical methods typical of the standards of practice in southern California. If these recommendations do not to appear to cover a specific feature of the project, please contact our office for additions or revisions.

7.1 Earthwork

Earthwork should be conducted per the requirements of the applicable California Building Code and Grading Ordinance for the City of Santee. The following recommendations are provided regarding specific aspects of the proposed earthwork. These recommendations should be considered subject to revision based on the conditions observed by the Geotechnical Engineer during earthwork.

7.1.1 Site Preparation

General site preparation should begin with the removal of deleterious and other unsuitable materials from the site. These materials include existing structures, foundations, slabs, trees, vegetation, trash, contaminated soil and demolition debris. Areas of the subgrade disturbed by demolition should be restored to the satisfaction of the Geotechnical Engineer during earthwork.

Existing subsurface utilities that will be abandoned should be removed and the excavations backfilled and compacted as described in the Fill Compaction section of this report. Alternatively, the abandoned pipes may be grouted with a two-sack sand-cement slurry under the observation of the Geotechnical Engineer.

7.1.2 Remedial Earthwork

All colluvium in development areas should be excavated and replaced as properly compacted fill. Removals should expose competent granitic rock material as determined in the field by the Geotechnical Engineer or their field designate. Removals should extend beyond the toe of fill slopes and the outer edge of improvements a minimum distance equal to a 1:1 line projected outward and down to an approved removal bottom or 5 feet, whichever is greater.

Removal depths should mostly range from one to five feet, although deeper removals may be needed. The removed soil that is free of deleterious and unsuitable material may be replaced as properly compacted fill. It should be noted that complete removal of unsuitable soils may be difficult to accomplish along the property boundaries without extending the remedial grading off-site, or establishing structural setback zones within the site.

7.1.3 Over-Excavation

Over-excavation of the cut area within buildings and other improvement pads should be considered where structures are supported by a shallow foundation that will straddle a transition from cut to fill. The engineering characteristics of materials in cut and fill may result in a high contrast in stiffness that could cause foundations to crack and display other forms of distress, depending on the type and rigidity of the shallow foundation system adopted.

In general, it is preferable to either deepen the foundations, or extend them deeper with a sand and cement slurry to bear entirely on competent rock. Otherwise, we recommend over-excavation of the cut area and replacement with properly compacted fill. Figure 5 provides recommendations for “shallow” and “deep” transitions.

Over-excavation and replacement with compacted fill should be considered as an alternative method of site preparation to ease pipeline and other utility installations. We recommend a minimum uniform over-excavation of one foot below the bottom level of the pipe bedding.

Excavations that require heavy ripping or blasting may create voids or uneven surfaces that should be filled with properly compacted granular soils.

7.1.4 Fill Materials

Except for surficial organic materials, the colluvium and properly processed material from granite rock excavations should be suitable for use as engineered fill, except where noted below.

- Clayey soils, where encountered, should be placed in deeper fills at least five feet below finished grade.
- Rock over six inches in the largest dimension will be generated from excavations in granite. This rock may be disposed of offsite or in nonstructural fill; crushed to less than six inches in maximum dimension for use in engineered fills; or placed individually per the recommendation of the Geotechnical Engineer or their field designate.

7.1.5 Import Soil

Imported fill sources should be observed prior to hauling onto the site to determine the suitability for use. In general, imported soil for common fill should consist of granular soil with a maximum particle size of less than three inches, a fines content of less than 35 percent passing the No. 200 sieve based on ASTM C136 and an Expansion Index less than 20 based on ASTM D4829.

Samples of all proposed import should be tested by the Geotechnical Engineer to evaluate the suitability of these soils for their proposed use. During earthwork, soil types may be encountered by the Contractor that do not appear to conform to those discussed within this report. The Geotechnical Engineer should evaluate the suitability of these soils for their proposed use.

7.1.6 Fill Compaction

All fill and backfill should be placed at slightly above optimum moisture content using equipment that can produce a uniformly compacted product. The minimum recommended relative compaction is 90 percent of the maximum dry density based on ASTM D1557.

A two-sack sand and cement slurry may also be used for structural fill as an alternative to compacted soil. It has been our experience that slurry is often useful in confined areas which may be difficult to access with typical compaction equipment. Samples of the slurry should be fabricated and tested for compressive strength during construction. A minimum 28-day compressive strength of 100 pounds per square inch (psi) is recommended for the sand and cement slurry.

7.1.7 Fill Slope Construction

The face of fill slopes should be compacted as the fill is placed to form the slope. Fill slopes and pads formed over a ground surface that slopes at more than a 5:1 (horizontal to vertical) gradient should be constructed entirely on formational materials (hard rock or approved weathered rock). An equipment width keyway should be provided at the base of the slope and benches should be formed to produce a level area to receive the fill. Benches should be wide enough to provide complete coverage by the compaction equipment during fill placement. Figure 6 illustrates these recommendations.

7.1.8 Cut Slope Construction

If blasting is performed near finished cut slope surfaces, it should be controlled to minimize the development of new cracks and/or expansion of existing discontinuities. Controlled blasting techniques, such as presplitting or smooth-wall blasting should be considered.

7.1.9 Excavation Characteristics

A rippability study has not been completed for this site. We understand from anecdotal information that previous earthwork at adjacent sites required heavy ripping and blasting. Note that backhoe and auger refusal on hard rock was encountered in our explorations as shallow as one foot below existing grades. Fresh rock exposures and corestones were also observed onsite. Cuts up to 20 feet into the granitic rock materials are shown on the preliminary grading plans. Consequently, excavations extending into granitic rock should be anticipated to be very difficult, and heavy ripping, rock hammering, and/or blasting may be needed.

7.1.10 Temporary Excavations

Temporary excavations are anticipated for the construction of the proposed retaining walls and underground utilities. All excavations should conform to Cal-OSHA guidelines.

The design and construction of temporary slopes, as well their maintenance and monitoring during construction, is the responsibility of the Contractor. The Contractor should have a Competent Person evaluate the soil or rock conditions encountered during excavation to determine permissible temporary slope inclinations and other measures as required by California OSHA (OSHA). Based on the existing data interpreted from site reconnaissance and subsurface exploration, the following OSHA Soil Types may be assumed for planning purposes. Note that slopes that exceed 20 feet in height require specific analysis by a registered Civil Engineer.

Geologic Unit	Cal/OSHA Soil Type
Fill and Colluvium	Type C
Weathered Granitic Rock	Type A[1]

1. Not subject to vibration, no fracturing, fissuring of dip into face of excavation.

The Contractor should note the materials encountered in construction excavations could vary significantly across the site. The above assessment of OSHA Soil Types for temporary slopes is based on preliminary engineering classifications of material encountered in widely spaced explorations. The Contractor's Competent Person should observe temporary slopes at regular intervals to assess their need for maintenance and stability.

7.2 Seismic Design Criteria

The USGS mapped spectral ordinates S_S and S_1 equal 0.875 and 0.338, respectively. For a Site Class C, the Site Coefficients F_a and F_v are equal to 1.150 and 1.461, respectively. The design level spectral ordinates S_{DS} and S_{D1} equal 0.613 and 0.329, respectively. The 2016 CBC Design and MCE Spectra for a Site Class C are provided in Table 1.

7.3 Shallow Foundations

Shallow foundations may be used for the planned structures, supported as recommended in the Over-Excavation section of this report. Design recommendations are provided below.

7.3.1 Allowable Vertical Bearing Capacity

Shallow foundations (strip and spread footings) founded entirely on properly compacted fill or relatively undisturbed rock may be designed using the following design parameters. Foundations should not transition between compacted fill and rock unless a Geotechnical Engineer provides specific recommendations for such placement.

- Allowable Bearing: 3,000 / 5,000 lbs/ft^2 (fill / weathered rock)
  (allow a ⅓ increase for short-term wind or seismic loads).
- Minimum Footing Width: 18 inches (continuous)
  24 inches (square/rectangular)
- Minimum Footing Depth: 24 inches below lowest adjacent soil grade
- Minimum Reinforcement: Per Structural Engineer

The above allowable vertical bearing pressures are net values and do not include the weight of the footing. Adjacent footings founded at different elevations should be located such that they do not surcharge each other. The slope from bearing level to bearing level should be flatter than 1 to 1 (horizontal to vertical).

7.3.2 Settlement

Provided all the subgrade for shallow building foundations is prepared as recommended in the Earthwork sections of this report, we estimate that the total and differential settlement of the new shallow foundations will be less than 1 inch and ¾ inch in 40 feet, respectively. Settlement should occur when building loads are applied.

7.3.3 Lateral Resistance

Lateral loads against structures may be resisted by friction between the bottoms of footings and slabs and the soil, and passive pressure from the portion of vertical foundation members embedded into fill or formational materials. A coefficient of friction of 0.35 and a passive pressure of 350 / 135 lbs/ft^2 per foot of embedment may be used for “infinite” level ground in front of the footing or wall and ground in front of the footing or wall that descend at 2:1 (horizontal to vertical) respectively. The upper foot of soil generating passive pressures should be neglected in lateral resistance calculations.

7.3.4 Slope Setback

All foundations should be setback from any descending slope at least eight feet. The setback should be measured horizontally from the outside bottom edge of the footing to the slope face. The horizontal setback may be reduced by deepening the foundation to achieve the recommended setback distance projected from the footing bottom to the face of the slope. Proposed foundations closer than 8 feet to a descending slope should be reviewed on a case by case basis.

Note that the outer few feet of all slopes are susceptible to gradual down-slope movements due to slope creep, which can affect hardscape such as concrete slabs. Settlement sensitive structures should not be constructed within five feet of the slope top without specific review by the Geotechnical Engineer.

7.4 On-Grade Slabs

Slab thickness, control joints, and reinforcement should be designed by the Structural Engineer and should conform to the requirements of the current California Building Code. We recommend a minimum slab thickness of 5 inches. The subgrade is anticipated to be predominately sandy soils with a low expansion potential.

7.4.1 Moisture Protection for On-Grade Slabs

Concrete slabs constructed on grade ultimately cause the moisture content to rise in the underlying soil. This results from continued capillary rise and the termination of normal evapotranspiration. Because normal concrete is permeable, the moisture will eventually penetrate the slab. Excessive moisture may cause mildewed carpets, lifting or discoloration of floor tiles, or similar problems. To decrease the likelihood of problems related to damp slabs, suitable moisture protection measures should be used where moisture sensitive floor coverings, equipment, or other factors warrant.

The Architect should specify an appropriate moisture barrier based on the allowable moisture transmission rate for the flooring. This may require a “vapor barrier” or a “vapor retarder”. The American Concrete Institute (ACI) provides detailed recommendations for moisture protection systems (ACI 302.1R-04). ACI defines a “vapor retarder” as having a minimum thickness of 10-mil, and a water transmission rate of less than 0.3 perms when tested per ASTM E96. ACI defines a "vapor barrier" as having a water transmission rate of 0.01 perms or less (such as a 15 mil StegoWrap). The vapor membrane should be constructed in accordance with ASTM E1643 and E1745 guidelines. All laps or seams should be overlapped at least 6 inches or per the manufacturer recommendations. Joints and penetrations should be sealed with pressure sensitive tape, or the manufacturer's adhesive. The vapor membrane should be protected from puncture, and repaired per the manufacturer's recommendations if damaged.

The vapor membrane is typically placed over 4 inches of granular material. The material should consist of a clean, fine graded sandy soil with roughly 10 to 30 percent passing the No. 100 sieve. The sand should not be contaminated with clay, silt, or organic material. Based on the particle size distributions curves presented in Appendix B, selectively mined and processed onsite soils should be able to meet the gradation specifications. The sand should be proof-rolled prior to placing the vapor membrane. Based on current ACI recommendations, the concrete slab should be placed directly over the vapor membrane. The common practice of placing sand over the vapor membrane may increase moisture transmission through the slab, because it provides a reservoir for bleed water from the concrete to collect. The sand placed over the vapor membrane may also move during placement, resulting in an irregular slab thickness. When placing concrete directly on an impervious membrane, it should be noted that finishing delays may occur. Care should be taken to assure that a low water to cement ratio is used, and that the concrete is moist cured in accordance with ACI guidelines.

7.5 Earth-Retaining Structures

Formation of the site is expected to require free standing gravity and/or cantilever retaining walls that could be constructed with masonry block, cast-in-place reinforced concrete and/or as Segmental Retaining Walls (SRW) with geogrid reinforcement. Permanent subterranean walls for structures are expected to be cast-in-place reinforced concrete walls constructed within a temporary excavation.

As previously noted, existing retaining walls are located along the northwestern and southeastern boundaries of the site. Structural setbacks, temporary shoring, and/or deepened footings may be needed to prevent decreasing passive pressures at the toe of the existing upslope walls and surcharging existing downslope walls with new upslope retaining walls.

The following preliminary geotechnical parameters are provided for design:

- Cantilever retaining walls that yield at the top at least ½ percent of the wall height may be designed using the active earth pressures shown in Figure 7A.
- Permanent subterranean walls that are restrained from lateral movement may be designed for an at-rest earth pressures shown in Figure 7B.
- Foundations for retaining walls can be designed using the recommendations in the Shallow Foundations section of this report.

The above parameters assume the following:

1. Walls will retain properly processed, placed and compacted coarse grained soils meeting the recommendation in the Import Soils section of this report.
2. All retaining walls have a vertical back.
3. Surcharges within a 1:1 plane extending back and up from the base of the wall should be accounted for in the wall design. Depending on whether the wall is cantilevered or restrained, 30% or 50%, respectively, of the maximum surcharge load should be used to develop a rectangular pressure distribution. Lateral loads for line or point loads can be provided on an as needed basis.
4. No hydrostatic pressures. All retaining walls should contain adequate backdrains to relieve hydrostatic pressures. Typical wall drain details are shown Figure 8.
5. Compaction within four feet of the wall will be completed with light hand-held or equivalent equipment; the lateral pressures would be higher if heavy equipment is used for soil compaction next to the walls.
6. Existing or proposed water bearing utilities, surface conditions that could promote infiltration (e.g., irrigated landscaping) behind walls, and seeps encountered during construction may require additional subsurface drainage. An inclined drainage system along the wall backcut, such as shown in Figure 9, should be considered in such circumstances.

7.5.1 Seismic Wall Design

The California Building Code requires seismic design for all earth retaining structures over six feet in height. The following seismic pressure and increments are recommended. Note the seismic increment has been added to the active earth pressure to the develop the seismic pressure.

- The seismic pressure and the seismic pressure increment for cantilever retaining walls may be idealized as a triangular pressure distribution as shown in Figure 7A.
- The seismic pressure and the seismic pressure increment for permanent subterranean walls may be idealized as a triangular pressure distribution as shown in Figure 7B.

7.5.2 Segmental Retaining Walls

Proprietary segmental retaining wall (SRW) systems should be designed in accordance with the National Concrete Masonry Association (NCMA), Design Manual for Segmental Retaining Walls 3rd Edition, or similar methodologies.

SRWs should be constructed entirely on prepared rock. Based on the proposed location of the retaining wall and the sloping grades around the wall base, a 15-foot wide and 2-foot deep keyway excavated into competent granitic rock materials should be provided at the base of the wall. The keyway should be graded at an inclination of about two percent into the slope, and a collector drain should be provided along the lowest portion of the keyway (Figure 9). Benches should be formed to produce a level area to receive the fill. Benches should be wide enough to provide complete coverage by the compaction equipment during fill placement and to provide clearance for the length of geogrid reinforcement specified by the wall designer.

The fill used in the reinforced and retained zones must be granular soils with a Plasticity Index of less than 6 that meet the gradation requirements specified by NCMA for walls less than 20 feet in height, as shown in the table below. Most of the on-site soils should be suitable. Soils meeting the plasticity and gradation requirements for the reinforced and retained zones may be assumed to have a minimum friction angle of 30^o with a total unit weight of about 135 lb/ft^3. The soil used should be sampled and tested by the Geotechnical Engineer to confirm that the recommended minimum soil parameters are obtained.

Sieve Size	(% Passing)
1 inch	100
No. 4	100 - 20
No. 40	0-60
No. 200	0-35

Where there are sloping ground conditions below the SRW, the minimum embedment depth to the top of the gravel leveling pad should be H/7, where H is the retained height of the SRW.

Typical SRW drainage details are shown in Figure 9, which assumes soil in the reinforced zones possesses 0 to 15 percent passing the No. 200 sieve. Otherwise, gravel drainage fill will be needed immediately behind the SRW. In addition, it may be necessary to selectively "mine" on-site soils or use import soils to meet the 0 to 15 percent passing the No. 200 sieve criterion.

The global stability of SRWs should be assessed by the Geotechnical Engineer as part of their review of the SRW calculations and plans.

7.6 Exterior Surface Improvements

Alternatives are provided for exteriors concrete slabs (e.g., sidewalks) and pavements comprising Asphalt Concrete (AC) and Portland Cement Concrete (PCC). Recommendations for interlocking concrete pavers can be provided on an as-needed basis if considered as part of the exterior surface improvements. Note the following items that apply to these alternatives:

- The upper 12 inches of pavement subgrade be scarified immediately prior to constructing the pavements, brought to optimum moisture, and compacted to at least 95 percent of the maximum dry density per ASTM D1557.

- Aggregate base, where specified below, should also be compacted to 95 percent of the maximum dry density based on ASTM D1557. Aggregate base should conform to the Standard Specifications for Public Works Construction (*SSPWC),* Section 200-2.

- An R-Value test completed on a soil sample from TP-08 resulted in an equilibrium resistance value of 68. Given the Caltrans recommendation to use an R-value of no larger than 50 (Caltrans, 2016), an R-Value of 50 was assumed for preliminary design. R-Value tests should be conducted on samples of the actual pavement subgrade soil immediately prior to establishing finish subgrade.

7.6.1 Exterior Concrete Slabs

Exterior slabs (sidewalks or similar) should be at least 4 inches thick. Crack control joints should be placed on a maximum spacing of 10-foot centers, each way, for slabs, and on 5-foot centers for sidewalks. The potential for differential movements across the control joints may be reduced by using steel reinforcement. Typical reinforcement for exterior slabs would consist of 6x6 W2.9/W2.9 welded wire fabric placed securely at mid-height of the slab.

7.6.2 Asphalt Concrete

Asphalt concrete pavement design was conducted in general accordance with the Caltrans Design Method (Topic 608.4). Traffic Indices of 5.0, 6.0 and 7.0 were assumed for preliminary design purposes. The project civil engineer should review the assumed traffic levels to determine if and where they are appropriate. Based on an assumed R-Value of 50, the following pavement sections would apply.

Pavement Type	Traffic Index	Asphalt Section	Base Section (R~50)
Passenger Car Parking	5.0	3 Inches	4 Inches
Truck Traffic Areas	6.0	3 Inches	5 Inches
Heavy Traffic Areas	7.0	4 Inches	5 Inches

Asphalt concrete should conform to Section 400-4 of the SSPWC and should be compacted to between 91 and 97 percent of the maximum theoretical density per Caltrans Section 39 requirements.

7.6.3 Portland Cement Concrete Pavements

Concrete pavement design was conducted in general accordance with the simplified design procedure of the Portland Cement Association (1984). This methodology is based on a 20-year design life. For design, it was assumed that aggregate interlock would be used for load transfer across control joints. The subgrade materials were assumed to provide "medium" support. Based on these assumptions noted above, and using the same traffic indices presented previously, we recommend that the PCC pavement sections at the site consist of at least 6 inches of concrete placed over 6 inches of compacted aggregate base. For heavier traffic areas (Traffic Index of 7.0), at least 7 inches of concrete over 6 inches of aggregate base is recommended. Concrete should have a modulus of rupture (MR) of 650 psi or greater. Crack control joints should be constructed for all PCC pavements on a maximum spacing of 10 feet, each way. Concentrated truck traffic areas, such as trash truck aprons and loading docks, should be reinforced with number 4 bars on 18-inch centers, each way.

7.7 Pipelines

The development will include a variety of pipelines such as water, storm drain and sewer systems. Geotechnical aspects of pipeline design include lateral earth pressures for thrust blocks, modulus of soil reaction, and pipe bedding. Each of these parameters is discussed separately below.

7.7.1 Thrust Blocks

Lateral resistance for thrust blocks may be determined by a passive pressure value of 250 lbs/ft^2 per foot of embedment, assuming a triangular distribution. This value may be used for thrust blocks embedded into compacted fill soils as well as formational materials.

7.7.2 Modulus of Soil Reaction

The modulus of soil reaction (E') is used to characterize the stiffness of soil backfill placed along the sides of buried flexible pipelines. For evaluating deflection due to the load associated with trench backfill over the pipe, a value of 1,200 lbs/in^2 is recommended for pipes up 5 feet deep, and 1,800 lbs/in^2 for pipes deeper than 5 feet (Hartley & Duncan, 1987). These values assume properly compacted backfill (relative compaction of 90% or more as evaluated by ASTM D1557) and granular bedding material is placed around the pipe.

7.7.3 Pipe Bedding

Typical pipe bedding as specified in the *Standard Specifications for Public Works Construction* may be used. As a minimum, we recommend that pipes be supported on at least 4 inches of granular bedding material such as minus ¾-inch crushed rock or decomposed granite. Where pipeline or trench excavations exceed a 15 percent gradient, we do not recommend that open graded rock be used for bedding or backfill because of the potential for piping and internal erosion. For sloping utilities, we recommend that coarse sand or sand-cement slurry be used for the bedding and pipe zone. The slurry should consist of a 2-sack mix having a slump no greater than 5 inches.

7.8 Reactive Soils

To evaluate the sulfate exposure of concrete in contact with the site soils, soils samples were tested for water-soluble sulfate content, as shown in Figure B-3. The test results indicate that the on-site soils have a *negligible* potential for sulfate attack based on commonly accepted criteria. The sulfate content of the finish grade soils should be determined during final grading.

To evaluate the reactivity of the site soils with buried metals, soil samples were tested for pH, resistivity and chloride contents were determined (see Figure B-3). These tests suggest that the on-site soils are *corrosive* to buried metals, based on the resistivity test results. Typical corrosion control measures should be incorporated into design, such as providing minimum clearances between reinforcing steel and soil, or sacrificial anodes for buried metal structures. The chloride content, resistivity, and pH of the finish grade soils should be determined during final grading.

7.9 Grading Plan Design

Fill slopes should be designed at a 2:1 (horizontal to vertical) ratio. Cut slopes formed in granite rock can be designed at 1.5:1. Steeper cut slopes may be possible depending on the condition and orientation of rock defects (e.g., joints and fractures) and the use of local rock slope stabilization measures (e.g., rock bolts and anchors). Cut slopes in colluvium should be avoided or designed at a 2:1 ratio with a stability fill. A stability fill is constructed by over-excavating the cut slope face by 10 feet horizontally and replacing the excavation with compacted fill that is benched into the cut face during fill placement. A Geotechnical Engineer should re-evaluate as necessary the fill and cut slope configuration adopted for the final design.

The grading plans should provide notes and/or details for keying and benching, over-excavation, selective fill placement, canyon subdrains and other earthwork considerations. A Geotechnical Engineer should review the 40-scale grading plans and help develop these notes and details.

7.10 Surface Drainage

Foundation and slab performance depends greatly on the ability of surface runoff to adequately drain from the site. Several natural drainages were observed on the northern portion of the site, as shown in Figures 2A & 2B. Flow from these drainages should be directed away from foundations, retaining walls, slopes, and other settlement sensitive structures using appropriate non-erodible drainage devices that tie in to a suitable outlet.

The ground surface should be graded so that water flows rapidly away from the structures and tops of slopes without ponding. The surface gradient needed to achieve this may depend on the prevailing landscaping. Planters should be built so that water will not seep into the foundation, slab, or pavement areas. If roof drains are used, the drainage should be channeled by pipe to storm drains, or discharge at least 10 feet from buildings. Irrigation should be limited to the minimum needed to sustain landscaping. Excessive irrigation, surface water, water line breaks, or rainfall may cause perched groundwater to develop within the underlying soil.

All slopes are susceptible to surficial slope failure and erosion given substantial wetting of the slope face. The surficial slope stability may be enhanced by providing proper site drainage. The site should be graded so that water from the surrounding areas is not able to flow over the tops of the slopes. Diversion structures should be provided where necessary. Surface runoff should be confined to gunite-lined swales or other appropriate devices to reduce the potential for erosion.

Slopes should be planted with vegetation that will increase their stability. A Landscape Architect should be consulted to develop a specific planting palate suitable to maintain the stability of slope surfaces.

7.11 Stormwater Infiltration

The potential for full or partial infiltration has been assessed in accordance with the City of Santee BMP Design Manual (2016). Infiltration testing yielded a design average infiltration rate of about 0.3 inches per hour, assuming a factor of safety of 2.0. However, impermeable granitic rock materials were encountered at a depth of about 8 feet below existing grades. Due to this relatively shallow rock condition, there is a potential for infiltrated stormwater to daylight in adjacent properties that are down gradient from the proposed basin. Accordingly, our feasibility screening of the potential for on-site infiltration resulted in the "no infiltration" category. Worksheet C.4-1 followed by our field percolation testing data and results are presented in Appendix C.

8.0 ADDITIONAL GEOTECHNICAL SERVICES

Development of the project may require the additional geotechnical services listed below:

- Performance of a geophysical evaluation to evaluate rippability of the onsite granitic rock.
- Updating recommendations for design changes.
- Reviewing the civil, structural, retaining wall and landscape drawing packages for compatibility with the recommendations provided in the geotechnical report.
- Reviewing the 40-scale grading plans and help develop notes and/or details for keying and benching, over-excavation, selective fill placement, canyon subdrains and other earthwork considerations.
- Responding to comments by the reviewing agencies.
- Finalizing the geotechnical report as needed for building permits.

9.0 CONSTRUCTION OBSERVATION AND TESTING

The restoration of subgrade disturbed by demolition, the preparation and subgrade for hardscaping and building improvements, and the placement of engineered fill should be performed under the observation and testing services of a Geotechnical Engineer and their field designate. Tests should be taken to determine the in-place moisture and relative compaction of engineered fill.

All foundation and concrete slab subgrade soils should be observed by a Geotechnical Engineer or their field designate prior to placement of steel and concrete to observe that the subgrade is satisfactory. Excavations should be properly dimensioned and free of soft, loose or disturbed soils.

An As-Built Geotechnical Report should be prepared following the completion of all geotechnically significant forms of construction. The report should be prepared per local guidelines.

10.0 LIMITATIONS

This report was prepared using the degree of care and skill ordinarily exercised, under similar circumstances, by reputable Geotechnical Engineers practicing in similar localities. No warranty, express or implied, is made as to the conclusions and professional opinions included in this report. The findings of this report are valid as of the present date. However, changes in the condition of a property can occur with the passage of time, whether due to natural processes or the work of man on this or adjacent properties. In addition, changes in applicable or appropriate standards of practice may occur from legislation or the broadening of knowledge. Accordingly, the findings of this report may be invalidated wholly or partially by changes outside our control. Therefore, this report is subject to review and should not be relied upon after a period of three years.

11.0 REFERENCES

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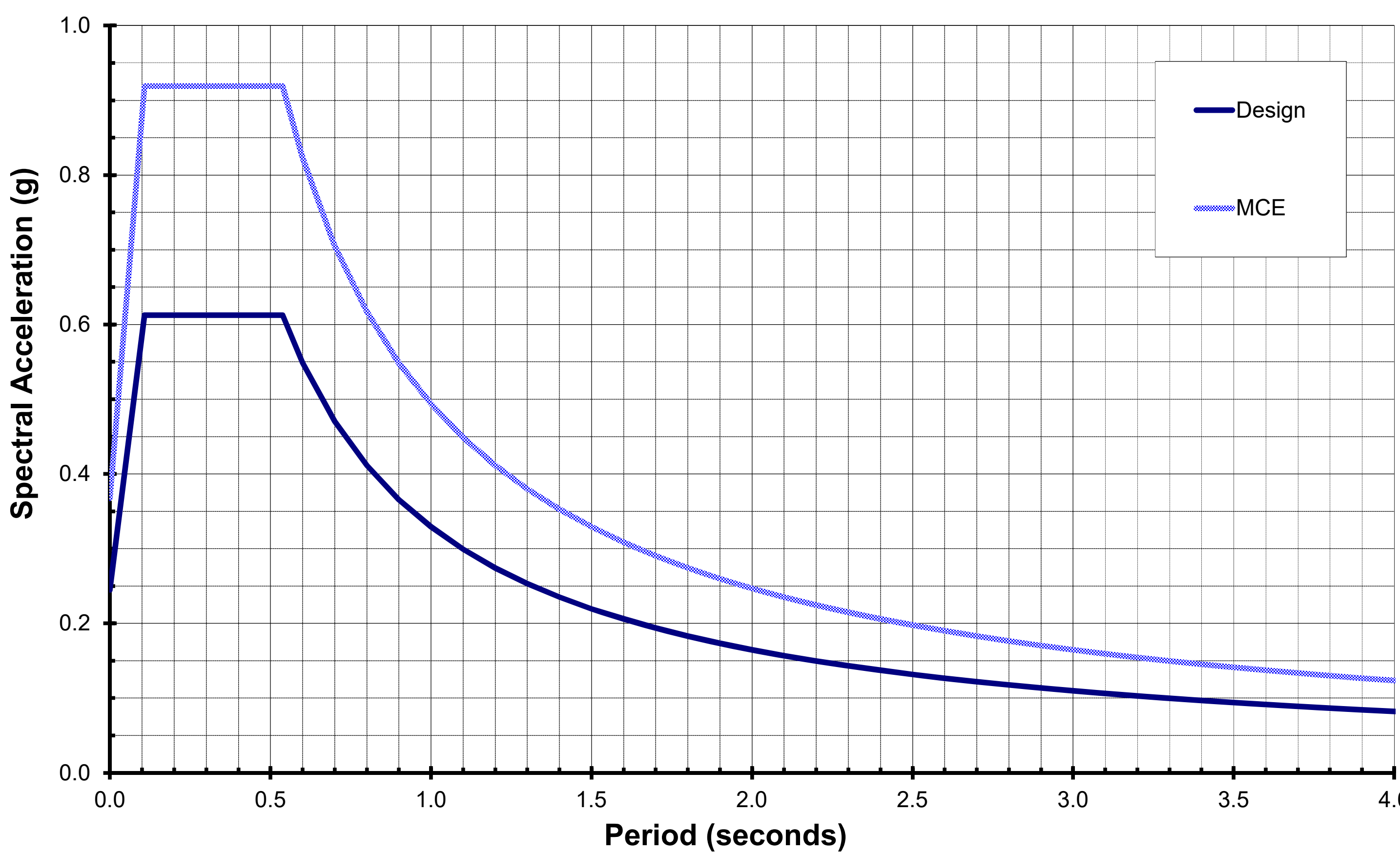
TABLE 1 - 2016 CBC ACCELERATION RESPONSE SPECTRA

INPUT	S_S=	0.875	g = short period (0.2 sec) mapped spectral response acceleration MCE Site Class B (**CBC 2016 Fig. 1613.3.1(1) or USGS Ground Motion Calculator**)
	S_1=	0.338	g = 1.0 sec period mapped spectral response acceleration MCE Site Class B (**CBC 2016 Fig. 1613.3.1(2) or USGS Ground Motion Calculator**)
	Site Class=	C	= Site Class definition based on **ASCE 7-10 Table 20.3-1**
	F_a=	1.050	= Site Coefficient applied to S_s to account for soil type (**CBC 2016 Table 1613.3.3(1)**)
	F_v=	1.461	= Site Coefficient applied to S_1 to account for soil type (**CBC 2016 Table 1613.3.3(2)**)
	T_L=	8.00	sec = Long Period Transition Period (**ASCE 7-10 Figure 22-12**)
OUTPUT	S_{MS}=	0.919	= site class modified short period (0.2 sec) MCE spectral response acceleration = F_a x S_s (**CBC 2016 Eqn. 16-37**)
	S_{M1}=	0.494	= site class modified 1.0 sec period MCE spectral response acceleration = F_v x S_1 (**CBC 2016 Eqn. 16-38**)
	S_{DS}=	0.613	= site class modified short period (0.2 sec) Design spectral response acceleration = 2/3 x S_{MS} (**CBC 2016 Eqn. 16-39**)
	S_{D1}=	0.329	= site class modified 1.0 sec period Design spectral response acceleration = 2/3 x S_{M1} (**CBC 2016 Eqn. 16-40**)
	T_0=	0.107	sec = 0.2 S_{D1}/S_{DS} = Control Period (left end of peak) for ARS Curve (Section 11.4.5 ASCE 7-10)
	T_S=	0.537	sec = S_{D1}/S_{DS} = Control Period (right end of peak) for ARS Curve (Section 11.4.5 ASCE 7-10)

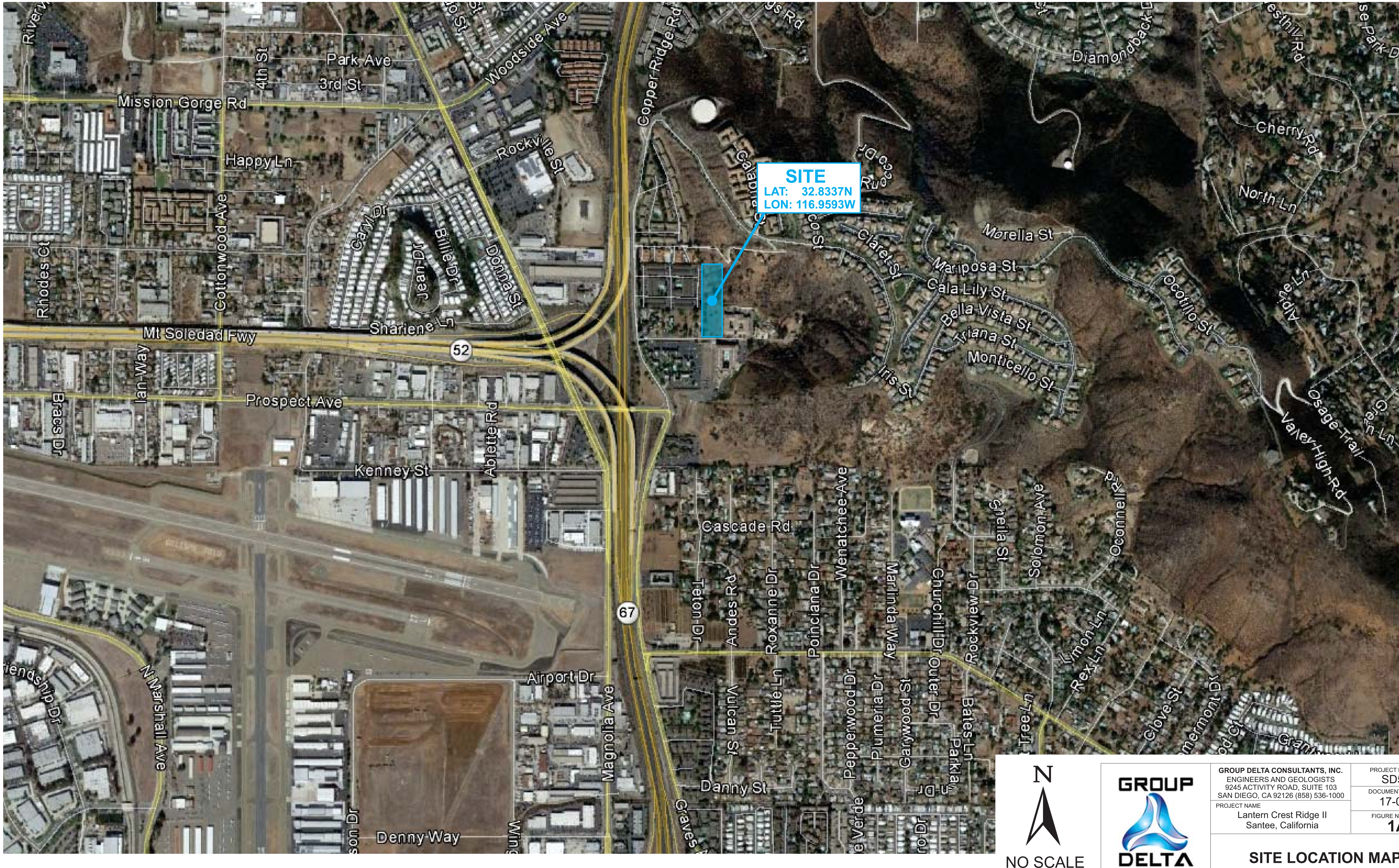
Site Latitude:	32.8337
Site Longitude:	-116.9593
Seismic Design Category:	D

SPECTRUM CALCULATION

T (seconds)	Design Sa (g)	MCE Sa (g)
0.000	0.245	0.368
0.107	0.613	0.919
0.537	0.613	0.919
0.600	0.549	0.823
0.700	0.470	0.705
0.800	0.412	0.617
0.900	0.366	0.549
1.000	0.329	0.494
1.100	0.299	0.449
1.200	0.274	0.412
1.300	0.253	0.380
1.400	0.235	0.353
1.500	0.219	0.329
1.600	0.206	0.309
1.700	0.194	0.290
1.800	0.183	0.274
1.900	0.173	0.260
2.000	0.165	0.247
2.100	0.157	0.235
2.200	0.150	0.224
2.300	0.143	0.215
2.400	0.137	0.206
2.500	0.132	0.198
2.600	0.127	0.190
2.700	0.122	0.183
2.800	0.118	0.176
2.900	0.114	0.170
3.000	0.110	0.165
3.100	0.106	0.159
3.200	0.103	0.154
3.300	0.100	0.150
3.400	0.097	0.145
3.500	0.094	0.141
3.600	0.091	0.137
3.700	0.089	0.133
3.800	0.087	0.130
3.900	0.084	0.127
4.000	0.082	0.123
5.000	0.066	0.099



FIGURES



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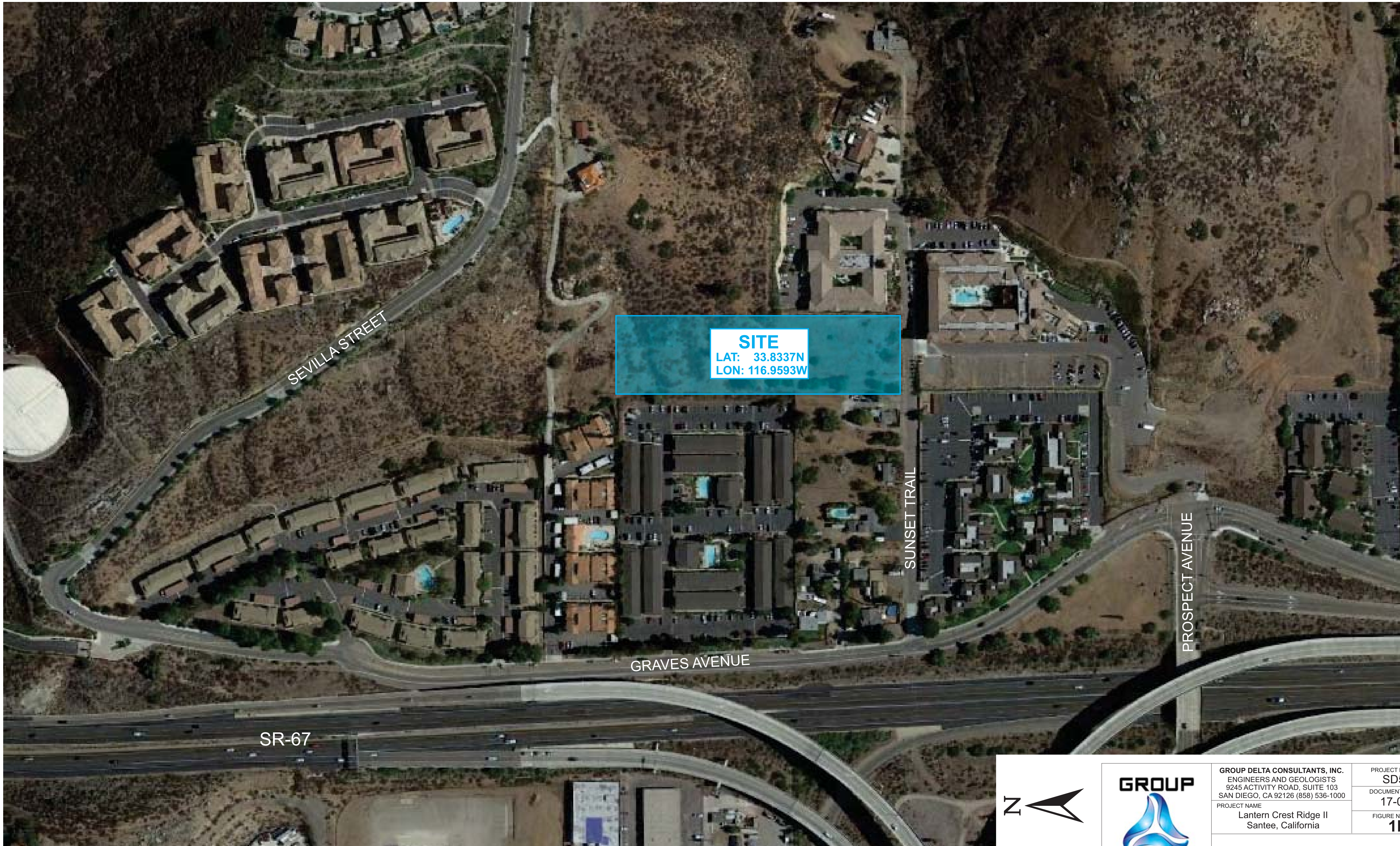
PROJECT NAME	Lantern Crest Ridge II Santee, California
PROJECT NUMBER	SD528
DOCUMENT NUMBER	17-0057
FIGURE NUMBER	1A

SITE LOCATION MAP



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DOCUMENT NUMBER
17-0057

FIGURE NUMBER
1B

SITE VICINITY PLAN

LEGEND:

B-1	Boring Location		Drainage Location	Qcol	Colluvium		Site Boundary
TP-7	Test Pit Location	?	Geologic Contact (Queried where uncertain)	Kgr	Granitic Rock (Buried where circled)		

N

SCALE: 1" = 50'

0 25 50 100

REFERENCE: Google, Inc. (2016). Google Earth Pro application, Imagery date: November 8.

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DOCUMENT NUMBER
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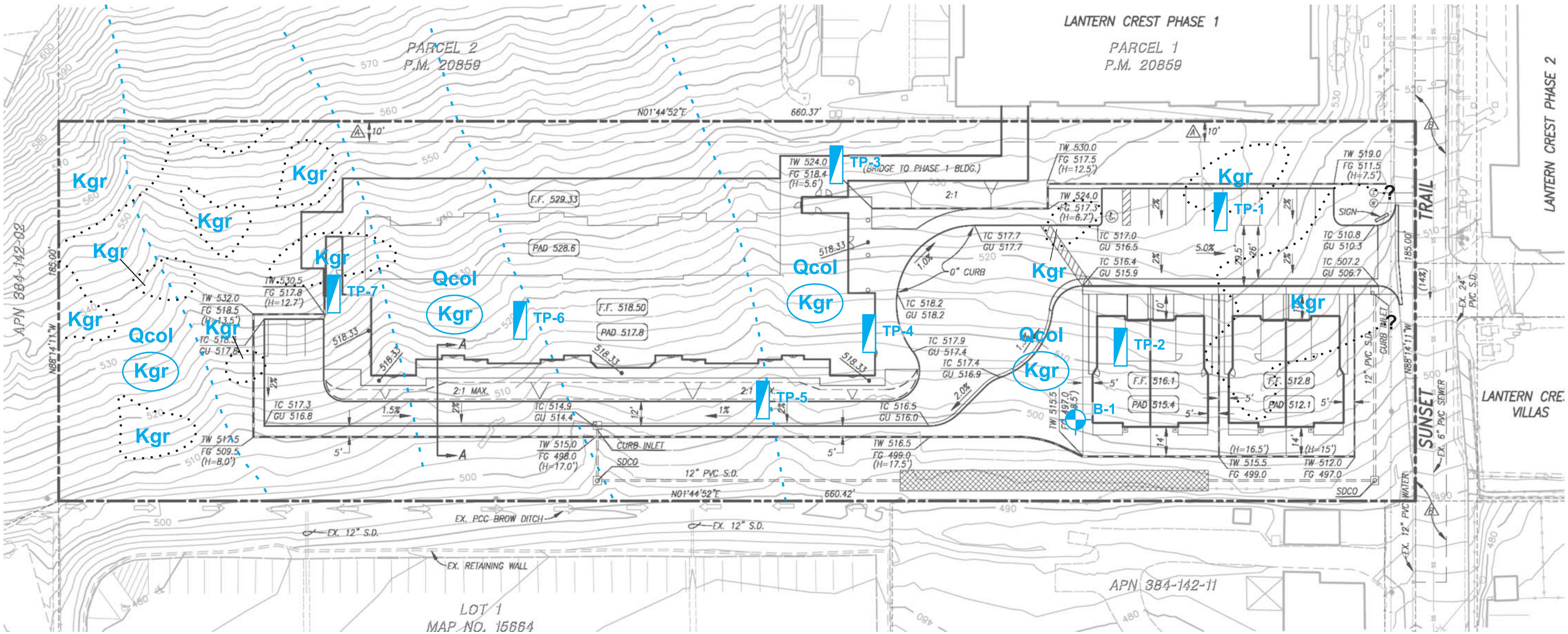
FIGURE NUMBER
2A

GEOTECHNICAL MAP



LEGEND:

B-1 Boring Location

TP-7 Test Pit Location

Drainage Location

?
Geologic Contact (Queried where uncertain)

Qcol Colluvium

Kgr Granitic Rock (Buried where circled)

SCALE: 1" = 50'

0 25 50 100

REFERENCE: POLARIS Development Consultants, Inc. (Undated). Site Plan & Preliminary Grading Plan, Lantern Crest Ridge II.

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DOCUMENT NUMBER
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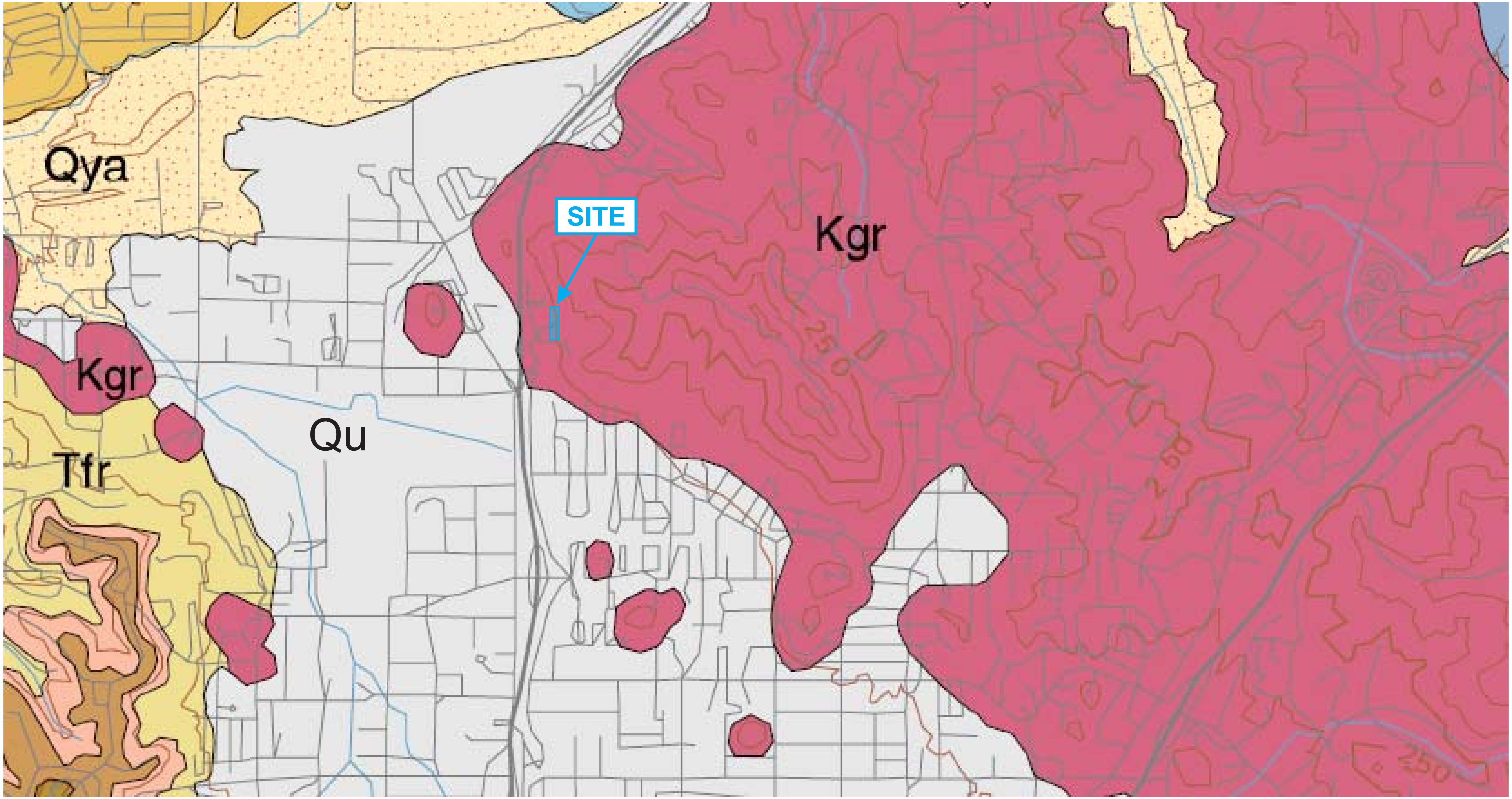
FIGURE NUMBER
2B

GEOTECHNICAL MAP



EXPLANATION:

Kgr Approximate location of weathered granitic rock (undivided tonalite and granodiorite).

Qu Approximate location of mapped colluvium/alluvium (undivided)

REFERENCE: Todd, V.R. (2004). Geologic Map of the El Cajon 30' x 60' Quadrangle, Scale 1:100,000

N

NO SCALE

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DOCUMENT NUMBER
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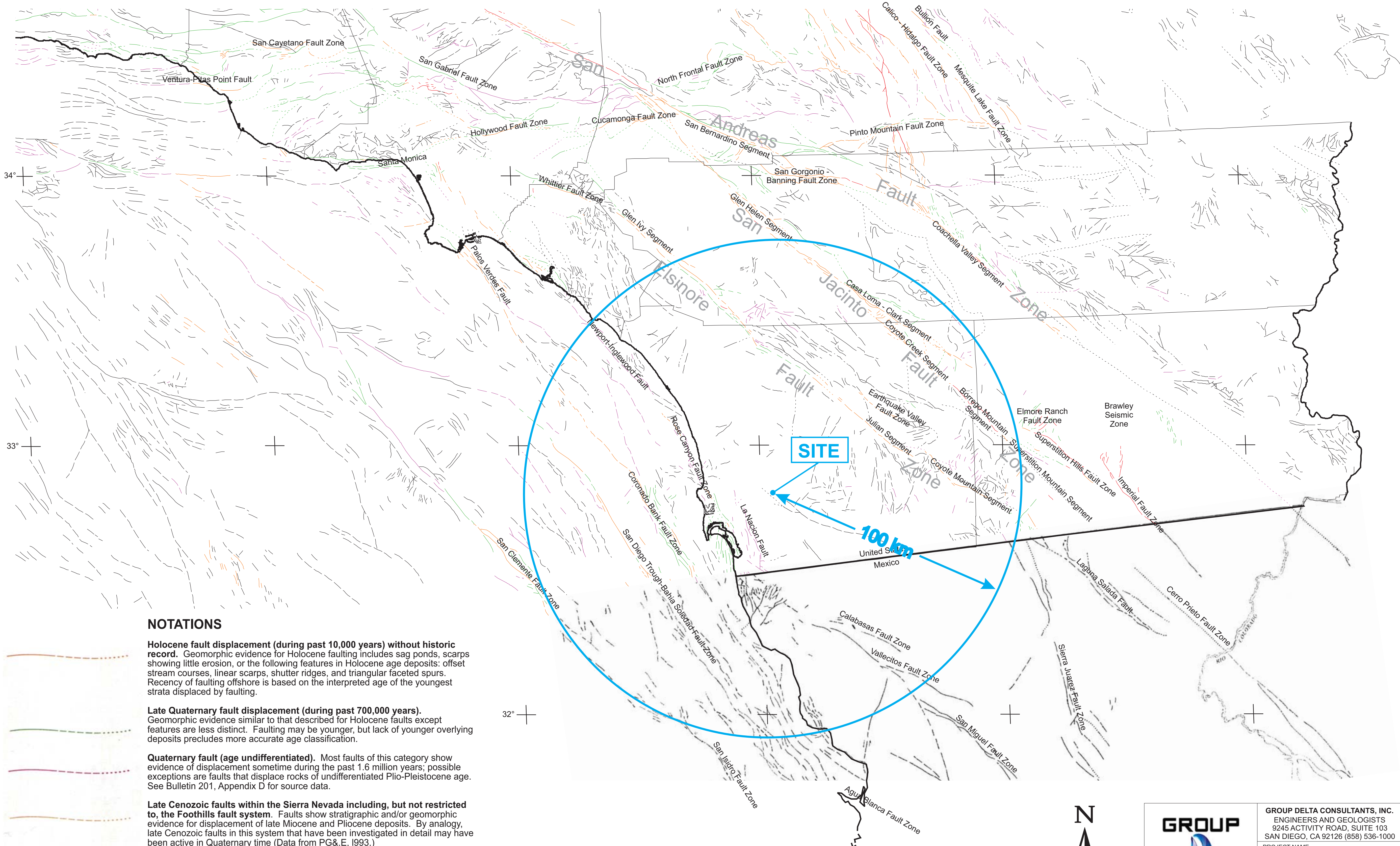
FIGURE NUMBER
3

REGIONAL GEOLOGIC MAP



NOTATIONS

Holocene fault displacement (during past 10,000 years) without historic record. Geomorphic evidence for Holocene faulting includes sag ponds, scarps showing little erosion, or the following features in Holocene age deposits: offset stream courses, linear scarps, shutter ridges, and triangular faceted spurs. Recency of faulting offshore is based on the interpreted age of the youngest strata displaced by faulting.

Late Quaternary fault displacement (during past 700,000 years). Geomorphic evidence similar to that described for Holocene faults except features are less distinct. Faulting may be younger, but lack of younger overlying deposits precludes more accurate age classification.

Quaternary fault (age undifferentiated). Most faults of this category show evidence of displacement sometime during the past 1.6 million years; possible exceptions are faults that displace rocks of undifferentiated Plio-Pleistocene age. See Bulletin 201, Appendix D for source data.

Late Cenozoic faults within the Sierra Nevada including, but not restricted to, the Foothills fault system. Faults show stratigraphic and/or geomorphic evidence for displacement of late Miocene and Pliocene deposits. By analogy, late Cenozoic faults in this system that have been investigated in detail may have been active in Quaternary time (Data from PG&E, I993.)

Pre-Quaternary fault (older than 1.6 million years) or fault without recognized Quaternary displacement. Some faults are shown in this category because the source of mapping used was of reconnaissance nature, or was not done with the object of dating fault displacements. Faults in this category are not necessarily inactive.

0 10 20 30 40 50 60

SCALE (KM)

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PROJECT NAME
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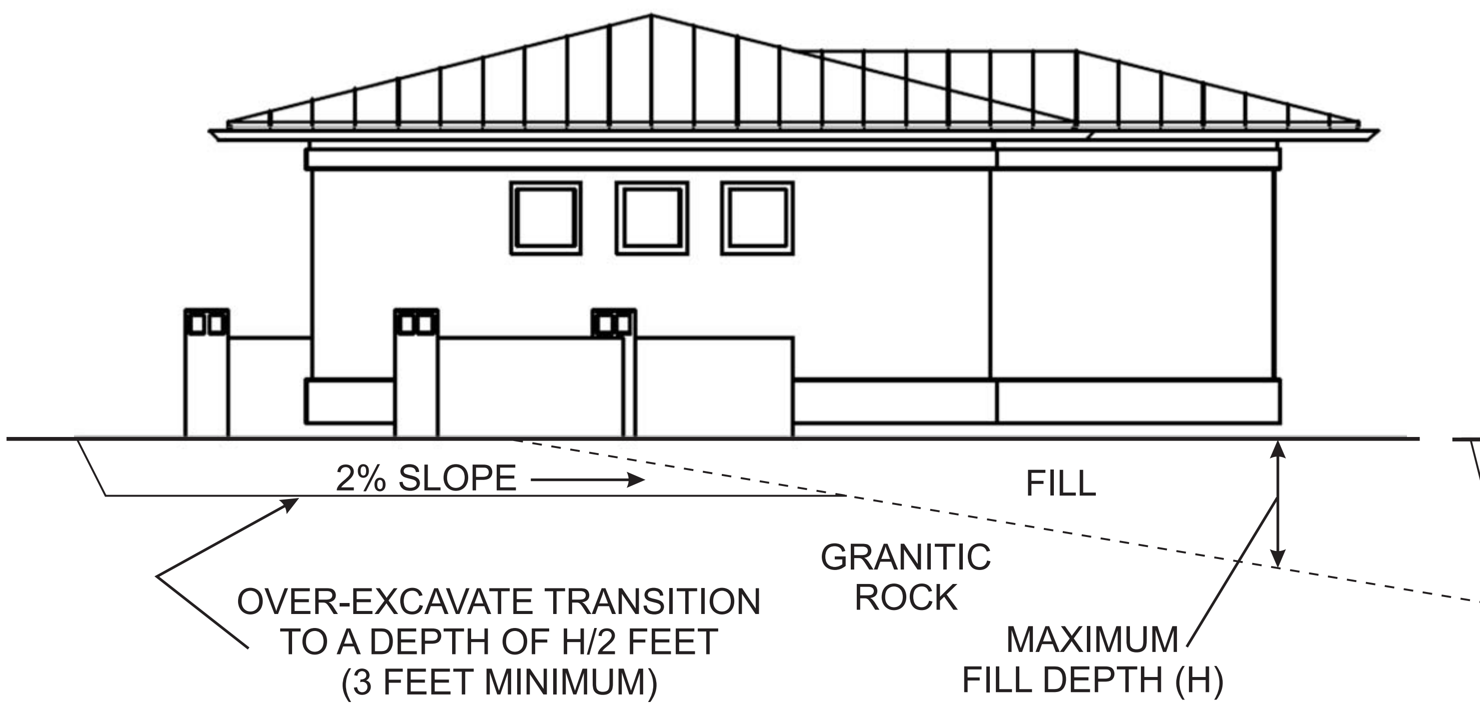
Santee, California

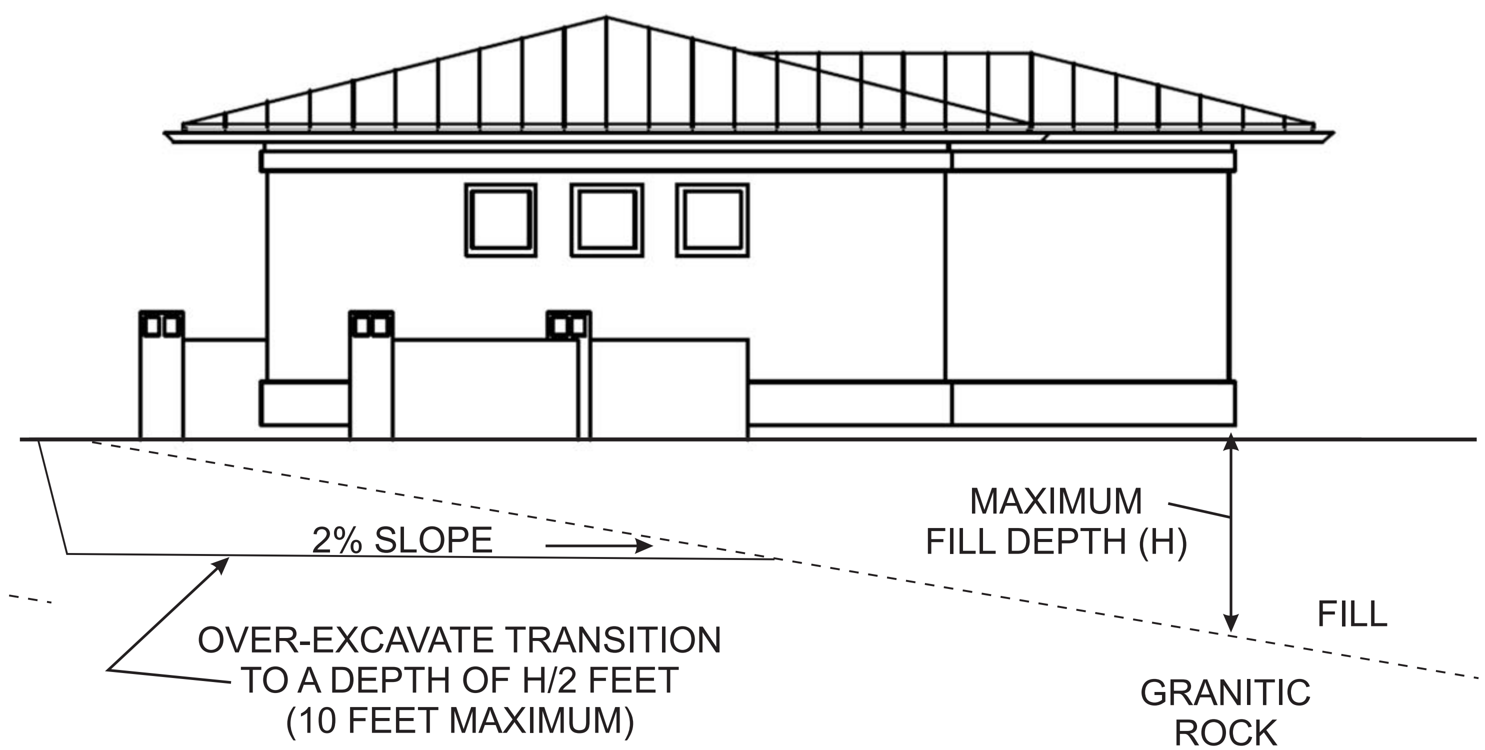
FIGURE NUMBER
4

FAULT LOCATION MAP

TYPICAL CUT/FILL TRANSITION



TYPICAL DEEP FILL TRANSITION



NOTES

1) Structures should not cross cut/fill nor deep fill transitions, due to the potential for adverse differential movement.

2) For building pads underlain by both cut/fill and deep fill transitions, the cut portion of the pads should be over-excavated to a depth of H/2, where H is equal to the greatest depth of fill beneath the foundations.

3) Over-excavations should extend at least 3 feet below bottom of foundation, and do not need to extend more than 10 feet below bottom of foundation.

4) Over-excavations should extend at least 10 feet beyond the perimeters of the building foundations, including any isolated column footings.

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PROJECT NUMBER
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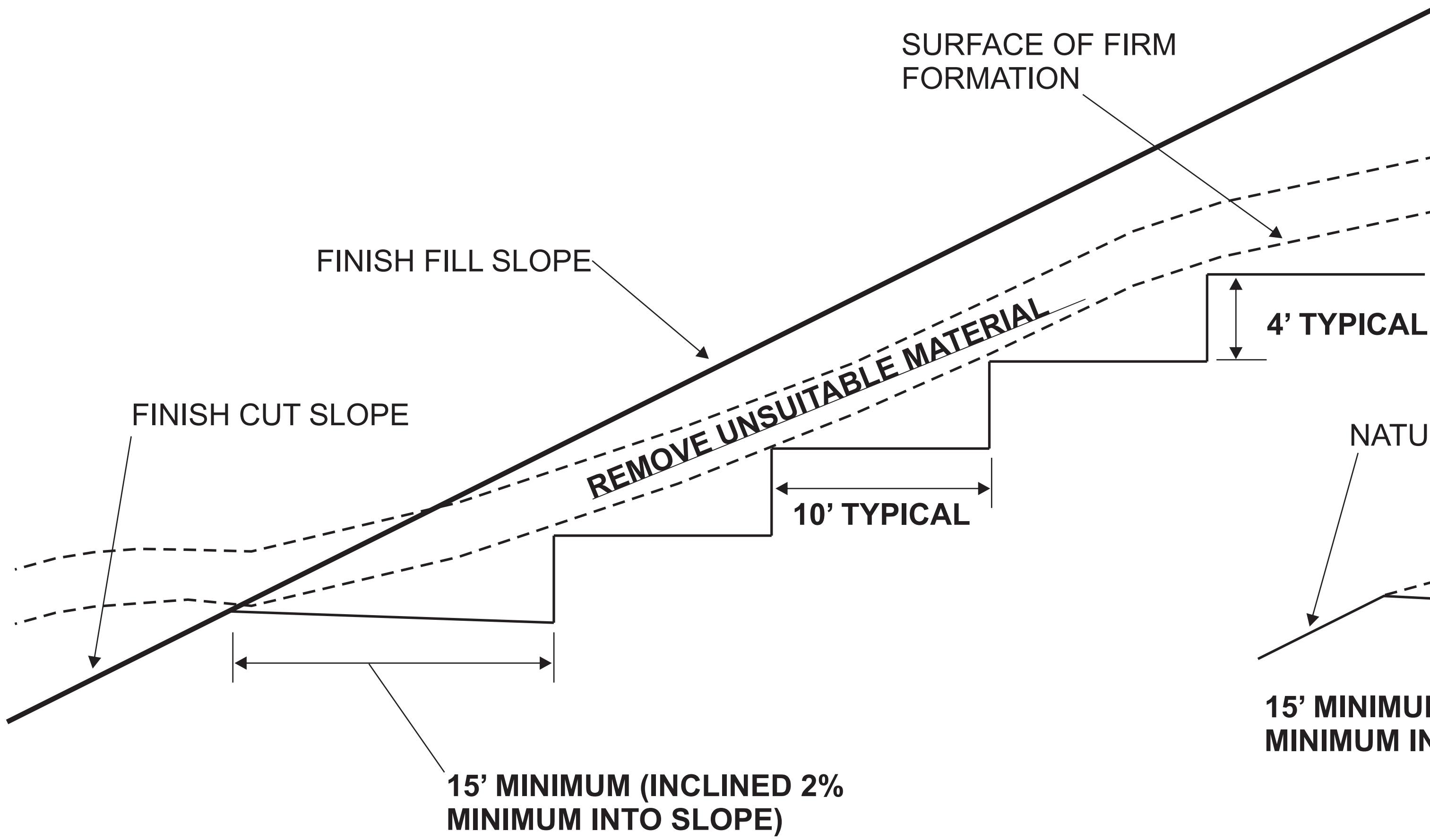
DOCUMENT NUMBER
17-0057

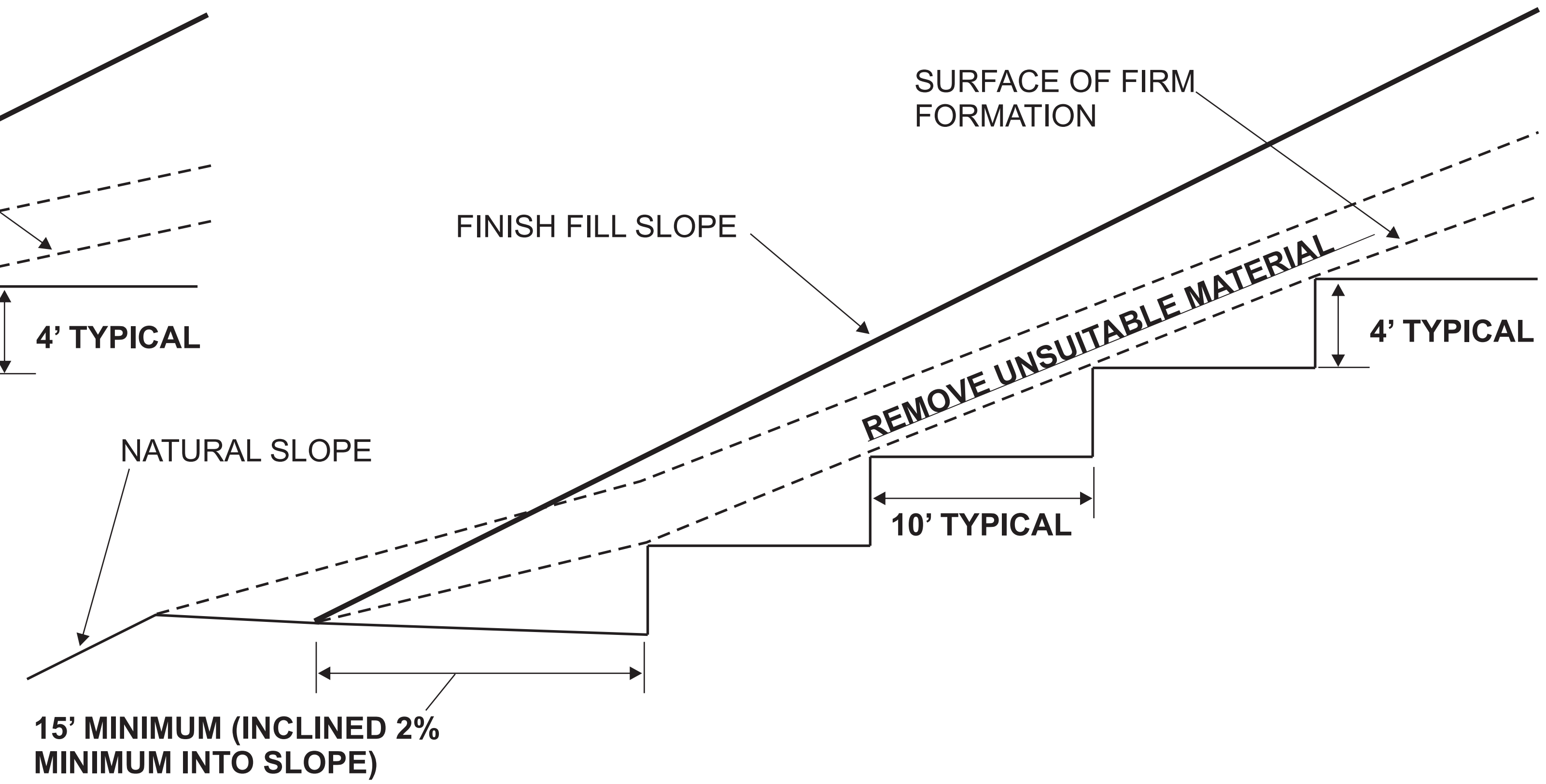
FIGURE NUMBER
5

TYPICAL TRANSITION DETAILS

FILL OVER CUT SLOPE



FILL OVER NATURAL SLOPE



NOTES

1) Where the existing ground surface slopes at more than a 5:1 gradient, benches should be constructed to provide level areas for fill placement.

2) Benches should be wide enough to provide complete coverage by the compaction equipment.

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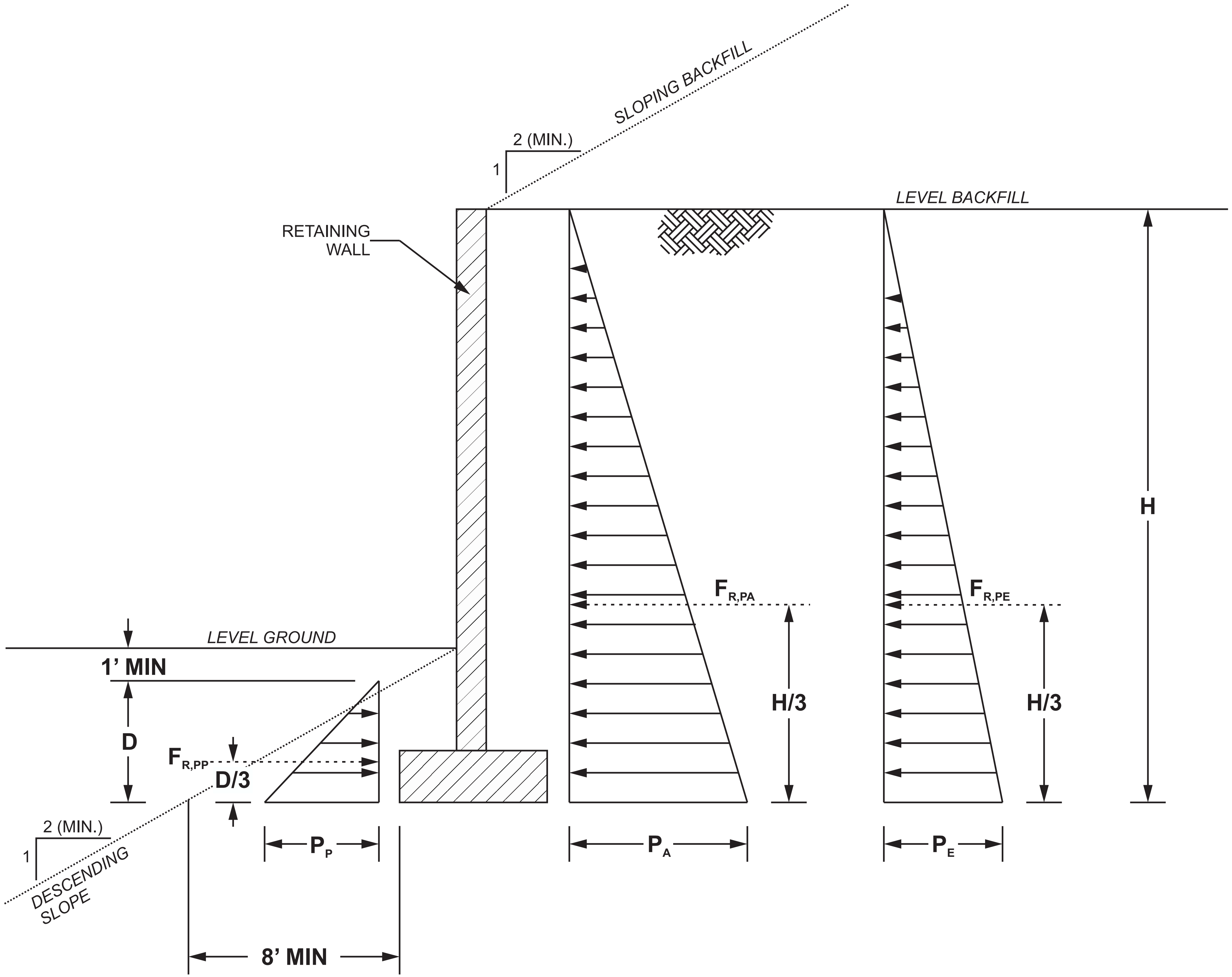
PROJECT NAME Lantern Crest Ridge II Santee, California	PROJECT NUMBER SD528 DOCUMENT NUMBER 17-0057 FIGURE NUMBER 6

SLOPE GRADING DETAILS



NOTES:

1. PASSIVE PRESSURES MAY BE INCREASED BY ⅓ DURING SEISMIC LOADING.
2. ASSUMES NO HYDROSTATIC PRESSURE.
3. SURCHARGES FROM CONSTRUCTION EQUIPMENT, EXCAVATED SOIL OR CONSTRUCTION MATERIALS ARE NOT INCLUDED.
4. SEISMIC INCREMENT LATERAL EARTH PRESSURE (P_E) IS BASED ON A PEAK GROUND ACCELERATION OF 0.35g.
5. SEISMIC INCREMENT LATERAL EARTH PRESSURE (P_E) IS CALCULATED USING THE RECOMMENDATIONS OF MONONOBE AND MATSUO (1929), OKABE (1926) AND AL ATIK AND SITAR (2009).
6. H AND D ARE MEASURED IN FEET.
7. PRESSURE DISTRIBUTION ASSUMES GRANULAR SOIL MATERIALS COMPACTED AS RECOMMENDED IN THE GEOTECHNICAL REPORT.

LATERAL EARTH PRESSURES

LATERAL EARTH PRESSURE TYPE	EQUIVALENT FLUID PRESSURE (PSF)	
ACTIVE, P_A	LEVEL BACKFILL	2:1 SLOPING BACKFILL
	40H	65H
SEISMIC INCREMENT, P_E*	12H	
PASSIVE, P_P	LEVEL GROUND	2:1 DESCENDING SLOPE
	350D	135D

*SEISMIC PRESSURE, $P_{AE} = P_A + P_E$

NOT TO SCALE

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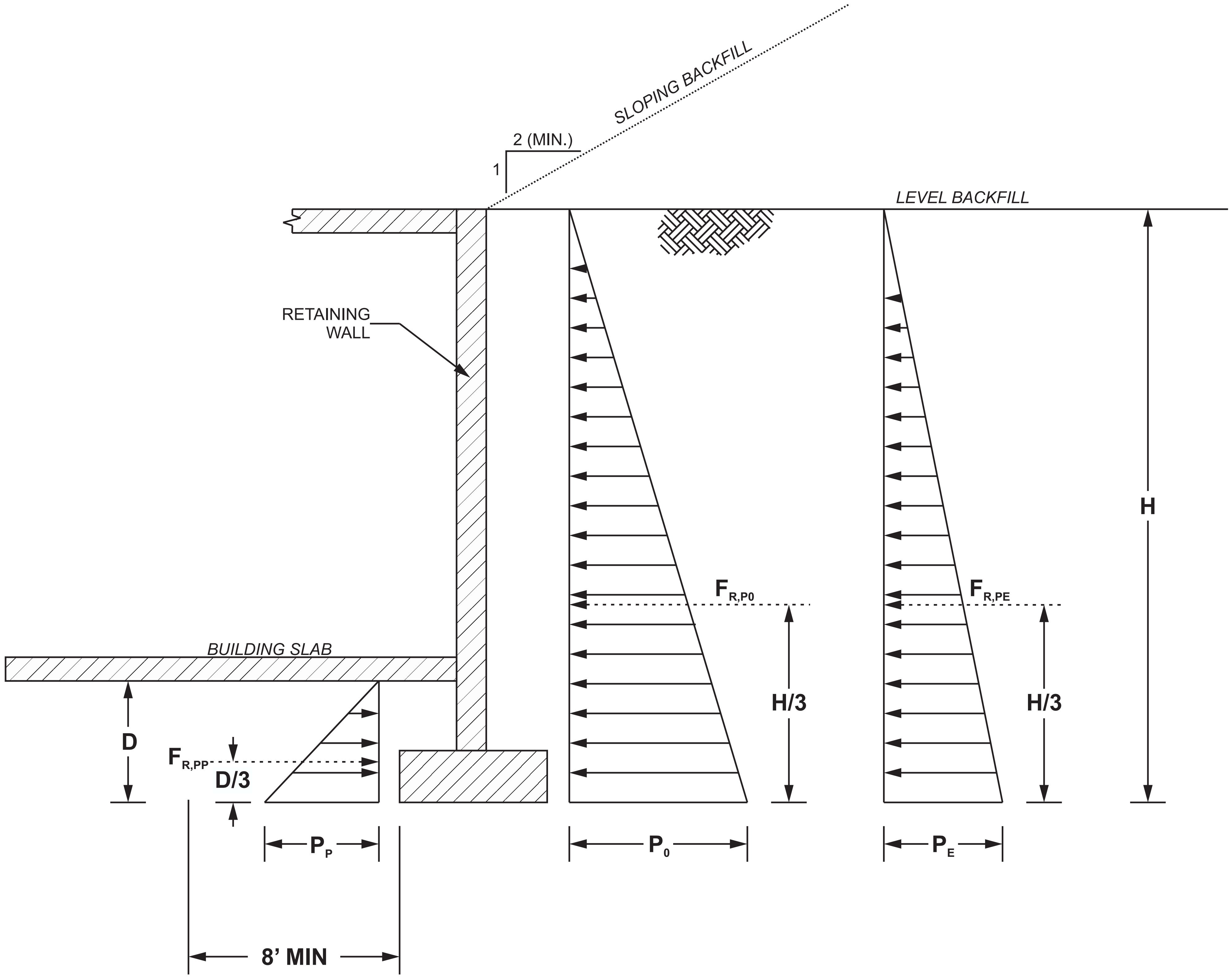
PROJECT NAME
Lantern Crest Ridge II
Santee, California

PROJECT NUMBER
SD528
DOCUMENT NUMBER
17-0057
FIGURE NUMBER
7A

LATERAL EARTH PRESSURES FOR YIELDING RETAINING WALLS



NOTES:

1. PASSIVE PRESSURES MAY BE INCREASED BY ⅓ DURING SEISMIC LOADING.
2. ASSUMES NO HYDROSTATIC PRESSURE.
3. SURCHARGES FROM CONSTRUCTION EQUIPMENT, EXCAVATED SOIL OR CONSTRUCTION MATERIALS ARE NOT INCLUDED.
4. SEISMIC INCREMENT LATERAL EARTH PRESSURE (P_E) IS BASED ON A PEAK GROUND ACCELERATION OF 0.35g.
5. SEISMIC INCREMENT LATERAL EARTH PRESSURE (P_E) IS CALCULATED USING THE RECOMMENDATIONS OF MONONOBE AND MATSUO (1929), OKABE (1926) AND AL ATIK AND SITAR (2009).
6. H AND D ARE MEASURED IN FEET.
7. PRESSURE DISTRIBUTION ASSUMES GRANULAR SOIL MATERIALS COMPACTED AS RECOMMENDED IN THE GEOTECHNICAL REPORT.

LATERAL EARTH PRESSURES

LATERAL EARTH PRESSURE TYPE	EQUIVALENT FLUID PRESSURE (PSF)	
AT-REST, P_0	LEVEL BACKFILL	2:1 SLOPING BACKFILL
	61H	89H
SEISMIC INCREMENT, P_E*	5H	
PASSIVE, P_P	LEVEL GROUND	2:1 DESCENDING SLOPE
	350D	135D

*SEISMIC PRESSURE, $P_{AE} = P_0 + P_E$

NOT TO SCALE

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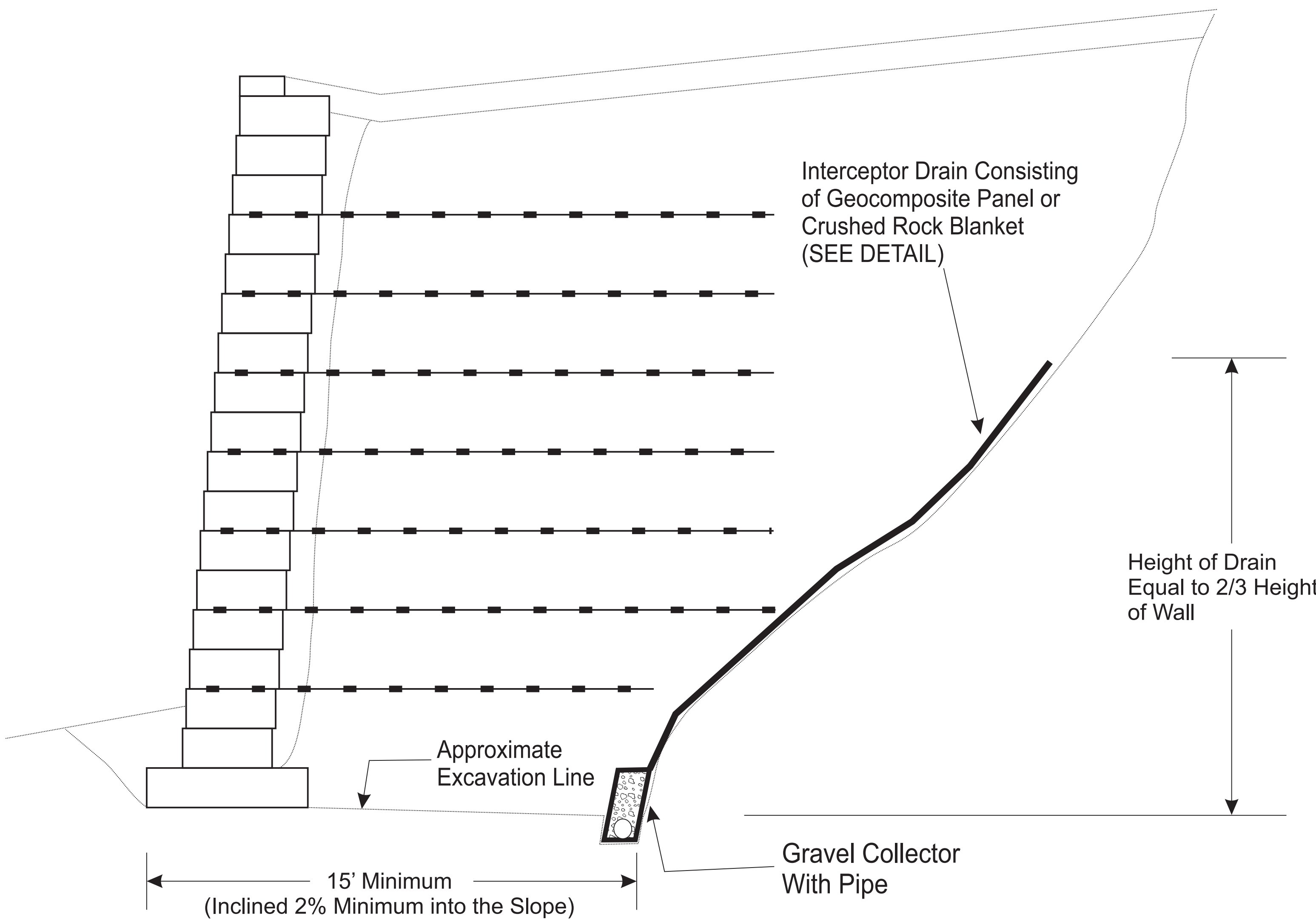
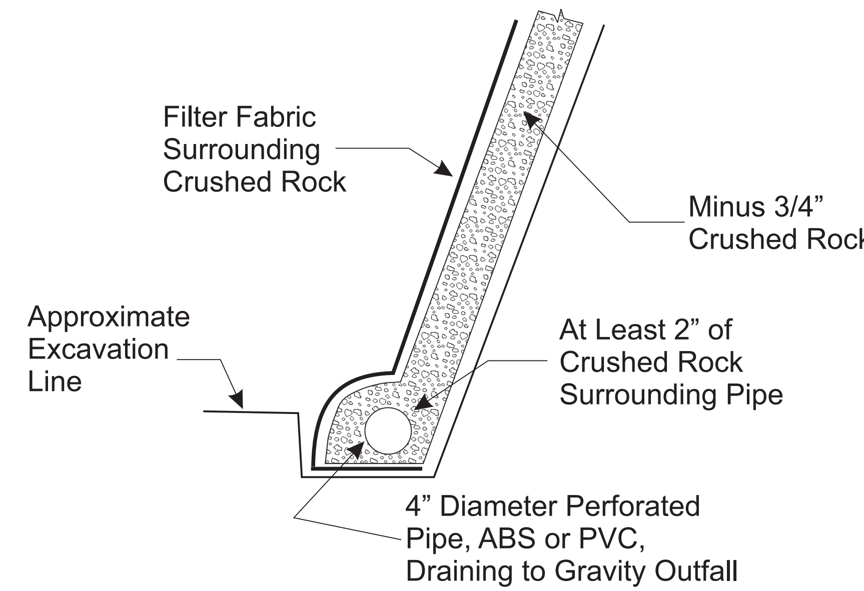
PROJECT NUMBER: SD528
DOCUMENT NUMBER: 17-0057
PROJECT NAME: Lantern Crest Ridge II, Santee, California
FIGURE NUMBER: 7B

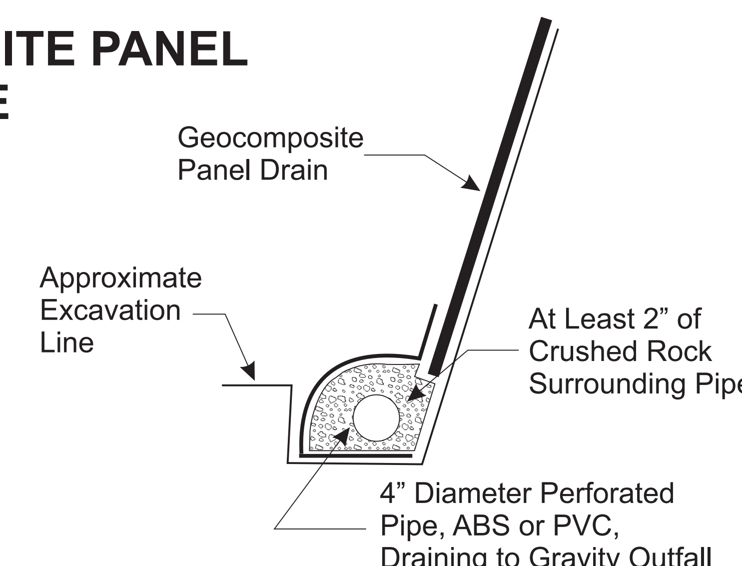
LATERAL EARTH PRESSURES FOR RESTRAINED RETAINING WALLS



ROCK BLANKET ALTERNATIVE



GEOCOMPOSITE PANEL ALTERNATIVE



NOTES

1) Perforated pipe should outlet through a solid pipe to a free gravity outfall. Perforated pipe and outlet pipe should have a fall of at least 1%.

2) Geocomposite panel drain should consist of Miradrain 6000, J-Drain 400, Supac DS-15, or approved similar product.

3) Filter fabric should consist of Mirafi 140N, Supac 5NP, Amoco 4599, or similar approved fabric. Filter fabric should be overlapped at least 6-inches.

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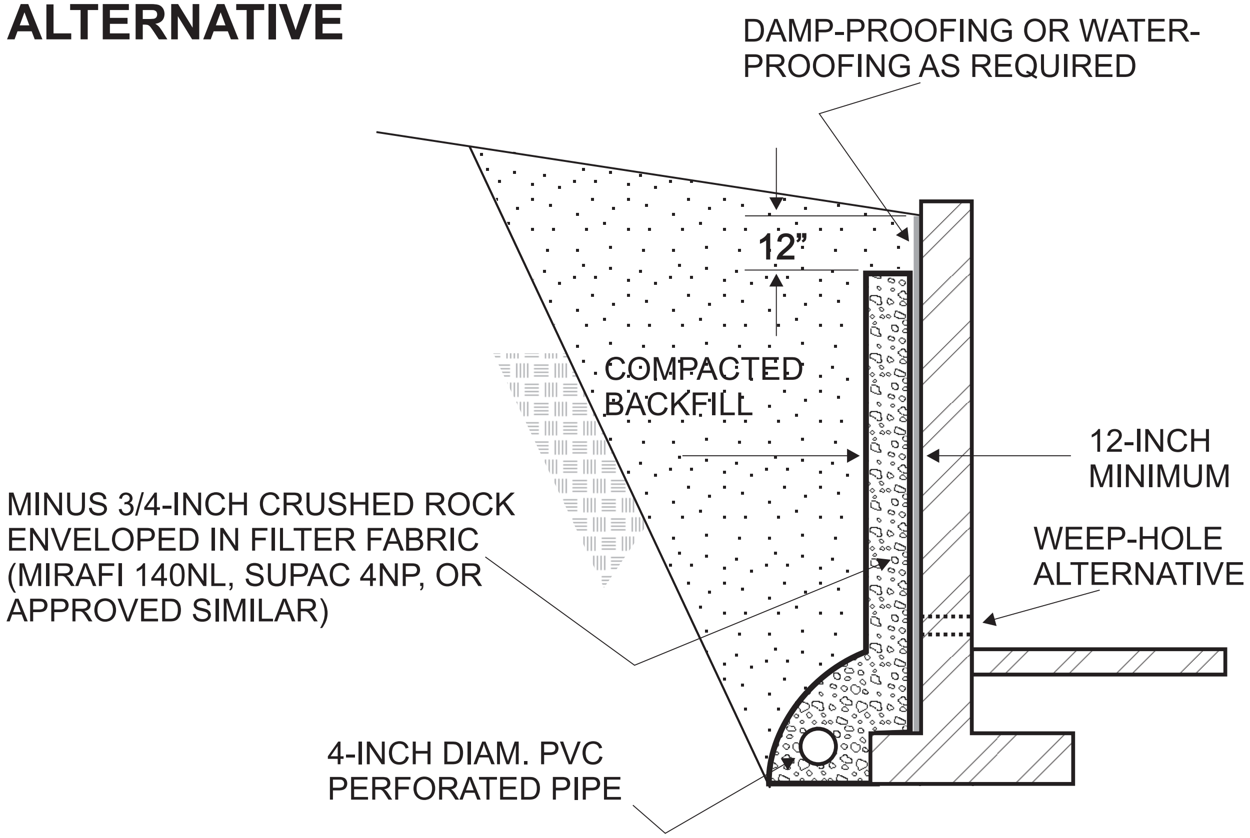
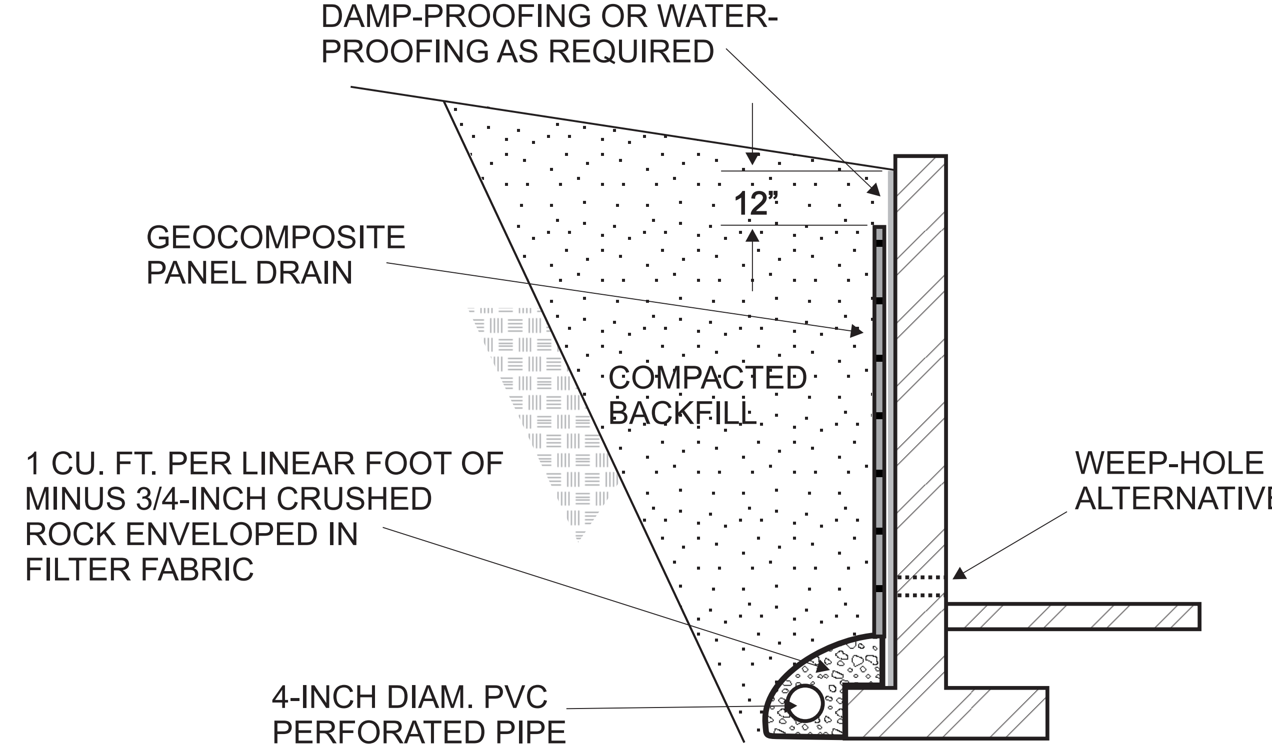
	DOCUMENT NUMBER 17-0057
PROJECT NAME Lantern Crest Ridge II Santee, California	FIGURE NUMBER 8
SEGMENTAL WALL DRAINS	

ROCK AND FABRIC ALTERNATIVE



PANEL DRAIN ALTERNATIVE



NOTES

1) Perforated pipe should outlet through a solid pipe to a free gravity outfall. Perforated pipe and outlet pipe should have a fall of at least 1%.

2) As an alternative to the perforated pipe and outlet, weep-holes may be constructed. Weep-holes should be at least 2 inches in diameter, spaced no greater than 8 feet, and be located just above grade at the bottom of wall.

3) Filter fabric should consist of Mirafi 140N, Supac 5NP, Amoco 4599, or similar approved fabric. Filter fabric should be overlapped at least 6-inches.

4) Geocomposite panel drain should consist of Miradrain 6000, J-Drain 400, Supac DS-15, or approved similar product.

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PROJECT NUMBER SD528
DOCUMENT NUMBER 17-0057
PROJECT NAME Lantern Crest Ridge II, Santee, California
FIGURE NUMBER 9

WALL DRAIN DETAILS

APPENDIX A
FIELD EXPLORATION

APPENDIX A

FIELD EXPLORATION

Our subsurface exploration program included a visual and geologic reconnaissance of the site, drilling of one soil boring on March 24, 2017, and the excavation of seven exploratory test pits on May 27, 2017. The maximum depth of exploration was about 8 feet below surrounding grades. The approximate locations of the boring and test pits are shown in Figures 2A and 2B. Logs of the explorations are provided in Figures A-1 through A-8, immediately following the Boring Record Legends.

The exploratory boring was drilled using a FRASTE track-mounted limited access drill rig equipped with 6-inch diameter hollow-stem augers. Drive samples were collected from the boring using an automatic hammer with an average Energy Transfer Ratio (ETR) of about 83 percent for the FRASTE rig. Disturbed samples were collected from the boring using a 2-inch outside diameter Standard Penetration Test (SPT) sampler. Less disturbed samples were collected using a 3-inch outside diameter ring lined sampler (a modified California sampler). These samples were sealed in plastic bags, labeled, and returned to the laboratory for testing. For each sample, the number of blows needed to drive the sampler 12 inches was recorded on the logs. The field blow counts (N) were normalized to approximate a standard 60 percent ETR, as shown on the logs (N_{60}). The exploratory test pits were excavated using a John Deere 310K backhoe with a 24-inch wide bucket. Bulk samples were collected from the test pits at selected intervals.

Note that the exploration locations were determined by visually estimating, pacing and taping distances from landmarks shown in Figures 2A and 2B. The locations shown should not be considered more accurate than is implied by the method of measurement used and the scale of the map. The lines designating the interface between differing soil materials on the logs may be abrupt or gradational. Further, soil conditions at locations between the excavations may be substantially different from those at the specific locations we explored. It should be noted that the passage of time may also result in changes in the soil conditions reported in the logs.

SOIL IDENTIFICATION AND DESCRIPTION SEQUENCE

Sequence	Identification Components	Refer to Section: Field	Refer to Section: Lab	Required	Optional
1	Group Name	2.5.2	3.2.2	●	
2	Group Symbol	2.5.2	3.2.2	●	
	Description Components				
3	Consistency of Cohesive Soil	2.5.3	3.2.3	●	
4	Apparent Density of Cohesionless Soil	2.5.4		●	
5	Color	2.5.5		●	
6	Moisture	2.5.6		●	
7	Percent or Proportion of Soil	2.5.7	3.2.4	●	⬬
	Particle Size	2.5.8	2.5.8	●	⬬
	Particle Angularity	2.5.9			○
	Particle Shape	2.5.10			○
8	Plasticity (for fine-grained soil)	2.5.11	3.2.5		○
9	Dry Strength (for fine-grained soil)	2.5.12			○
10	Dilatency (for fine-grained soil)	2.5.13			○
11	Toughness (for fine-grained soil)	2.5.14			○
12	Structure	2.5.15			○
13	Cementation	2.5.16		●	
14	Percent of Cobbles and Boulders	2.5.17		●	
	Description of Cobbles and Boulders	2.5.18		●	
15	Consistency Field Test Result	2.5.3		●	
16	Additional Comments	2.5.19			○

Describe the soil using descriptive terms in the order shown

Minimum Required Sequence:

USCS Group Name (Group Symbol); Consistency or Density; Color; Moisture; Percent or Proportion of Soil; Particle Size; Plasticity (optional).

⬬ = optional for non-Caltrans projects

Where applicable:

Cementation; % cobbles & boulders; Description of cobbles & boulders; Consistency field test result

REFERENCE: Caltrans Soil and Rock Logging, Classification, and Presentation Manual (2010).

HOLE IDENTIFICATION

Holes are identified using the following convention:

H – YY – NNN

Where:

H: Hole Type Code

YY: 2-digit year

NNN: 3-digit number (001-999)

Hole Type Code and Description

Hole Type Code	Description
A	Auger boring (hollow or solid stem, bucket)
R	Rotary drilled boring (conventional)
RC	Rotary core (self-cased wire-line, continuously-sampled)
RW	Rotary core (self-cased wire-line, not continuously sampled)
P	Rotary percussion boring (Air)
HD	Hand driven (1-inch soil tube)
HA	Hand auger
D	Driven (dynamic cone penetrometer)
CPT	Cone Penetration Test
O	Other (note on LOTB)

Description Sequence Examples:

SANDY lean CLAY (CL); very stiff; yellowish brown; moist; mostly fines; some SAND, from fine to medium; few gravels; medium plasticity; PP=2.75.

Well-graded SAND with SILT and GRAVEL and COBBLES (SW-SM); dense; brown; moist; mostly SAND, from fine to coarse; some fine GRAVEL; few fines; weak cementation; 10% GRANITE COBBLES; 3 to 6 inches; hard; subrounded.

Clayey SAND (SC); medium dense, light brown; wet; mostly fine sand,; little fines; low plasticity.

GROUP DELTA

Project No. SD528

Lantern Crest Ridge II
Santee, California

BORING RECORD LEGEND #1

GROUP SYMBOLS AND NAMES

Graphic / Symbol	Group Names	Graphic / Symbol	Group Names
GW	Well-graded GRAVEL Well-graded GRAVEL with SAND	CL	Lean CLAY Lean CLAY with SAND Lean CLAY with GRAVEL SANDY lean CLAY SANDY lean CLAY with GRAVEL GRAVELLY lean CLAY GRAVELLY lean CLAY with SAND
GP	Poorly graded GRAVEL Poorly graded GRAVEL with SAND		
GW-GM	Well-graded GRAVEL with SILT Well-graded GRAVEL with SILT and SAND	CL-ML	SILTY CLAY SILTY CLAY with SAND SILTY CLAY with GRAVEL SANDY SILTY CLAY SANDY SILTY CLAY with GRAVEL GRAVELLY SILTY CLAY GRAVELLY SILTY CLAY with SAND
GW-GC	Well-graded GRAVEL with CLAY (or SILTY CLAY) Well-graded GRAVEL with CLAY and SAND (or SILTY CLAY and SAND)		
GP-GM	Poorly graded GRAVEL with SILT Poorly graded GRAVEL with SILT and SAND	ML	SILT SILT with SAND SILT with GRAVEL SANDY SILT SANDY SILT with GRAVEL GRAVELLY SILT GRAVELLY SILT with SAND
GP-GC	Poorly graded GRAVEL with CLAY (or SILTY CLAY) Poorly graded GRAVEL with CLAY and SAND (or SILTY CLAY and SAND)		
GM	SILTY GRAVEL SILTY GRAVEL with SAND	OL	ORGANIC lean CLAY ORGANIC lean CLAY with SAND ORGANIC lean CLAY with GRAVEL SANDY ORGANIC lean CLAY SANDY ORGANIC lean CLAY with GRAVEL GRAVELLY ORGANIC lean CLAY GRAVELLY ORGANIC lean CLAY with SAND
GC	CLAYEY GRAVEL CLAYEY GRAVEL with SAND		
GC-GM	SILTY, CLAYEY GRAVEL SILTY, CLAYEY GRAVEL with SAND	OL	ORGANIC SILT ORGANIC SILT with SAND ORGANIC SILT with GRAVEL SANDY ORGANIC SILT SANDY ORGANIC SILT with GRAVEL GRAVELLY ORGANIC SILT GRAVELLY ORGANIC SILT with SAND
SW	Well-graded SAND Well-graded SAND with GRAVEL		
SP	Poorly graded SAND Poorly graded SAND with GRAVEL	CH	Fat CLAY Fat CLAY with SAND Fat CLAY with GRAVEL SANDY fat CLAY SANDY fat CLAY with GRAVEL GRAVELLY fat CLAY GRAVELLY fat CLAY with SAND
SW-SM	Well-graded SAND with SILT Well-graded SAND with SILT and GRAVEL		
SW-SC	Well-graded SAND with CLAY (or SILTY CLAY) Well-graded SAND with CLAY and GRAVEL (or SILTY CLAY and GRAVEL)	MH	Elastic SILT Elastic SILT with SAND Elastic SILT with GRAVEL SANDY elastic SILT SANDY elastic SILT with GRAVEL GRAVELLY elastic SILT GRAVELLY elastic SILT with SAND
SP-SM	Poorly graded SAND with SILT Poorly graded SAND with SILT and GRAVEL		
SP-SC	Poorly graded SAND with CLAY (or SILTY CLAY) Poorly graded SAND with CLAY and GRAVEL (or SILTY CLAY and GRAVEL)	OH	ORGANIC fat CLAY ORGANIC fat CLAY with SAND ORGANIC fat CLAY with GRAVEL SANDY ORGANIC fat CLAY SANDY ORGANIC fat CLAY with GRAVEL GRAVELLY ORGANIC fat CLAY GRAVELLY ORGANIC fat CLAY with SAND
SM	SILTY SAND SILTY SAND with GRAVEL		
SC	CLAYEY SAND CLAYEY SAND with GRAVEL	OH	ORGANIC elastic SILT ORGANIC elastic SILT with SAND ORGANIC elastic SILT with GRAVEL SANDY elastic ELASTIC SILT SANDY ORGANIC elastic SILT with GRAVEL GRAVELLY ORGANIC elastic SILT GRAVELLY ORGANIC elastic SILT with SAND
SC-SM	SILTY, CLAYEY SAND SILTY, CLAYEY SAND with GRAVEL		
PT	PEAT	OL/OH	ORGANIC SOIL ORGANIC SOIL with SAND ORGANIC SOIL with GRAVEL SANDY ORGANIC SOIL SANDY ORGANIC SOIL with GRAVEL GRAVELLY ORGANIC SOIL GRAVELLY ORGANIC SOIL with SAND
	COBBLES COBBLES and BOULDERS BOULDERS		

FIELD AND LABORATORY TESTING

C	Consolidation (ASTM D 2435)
CL	Collapse Potential (ASTM D 5333)
CP	Compaction Curve (CTM 216)
CR	Corrosion, Sulfates, Chlorides (CTM 643; CTM 417; CTM 422)
CU	Consolidated Undrained Triaxial (ASTM D 4767)
DS	Direct Shear (ASTM D 3080)
EI	Expansion Index (ASTM D 4829)
M	Moisture Content (ASTM D 2216)
OC	Organic Content (ASTM D 2974)
P	Permeability (CTM 220)
PA	Particle Size Analysis (ASTM D 422)
PI	Liquid Limit, Plastic Limit, Plasticity Index (AASHTO T 89, AASHTO T 90)
PL	Point Load Index (ASTM D 5731)
PM	Pressure Meter
R	R-Value (CTM 301)
SE	Sand Equivalent (CTM 217)
SG	Specific Gravity (AASHTO T 100)
SL	Shrinkage Limit (ASTM D 427)
SW	Swell Potential (ASTM D 4546)
UC	Unconfined Compression - Soil (ASTM D 2166) Unconfined Compression - Rock (ASTM D 2938)
UU	Unconsolidated Undrained Triaxial (ASTM D 2850)
UW	Unit Weight (ASTM D 4767)

SAMPLER GRAPHIC SYMBOLS

- Standard Penetration Test (SPT)
- Standard California Sampler
- Modified California Sampler (2.4" ID, 3" OD)
- Shelby Tube
- Piston Sampler
- NX Rock Core
- HQ Rock Core
- Bulk Sample
- Other (see remarks)

DRILLING METHOD SYMBOLS

- Auger Drilling
- Rotary Drilling
- Dynamic Cone or Hand Driven
- Diamond Core

WATER LEVEL SYMBOLS

- First Water Level Reading (during drilling)
- Static Water Level Reading (after drilling, date)

Definitions for Change in Material

Term	Definition	Symbol
Material Change	Change in material is observed in the sample or core and the location of change can be accurately located.	
Estimated Material Change	Change in material cannot be accurately located either because the change is gradational or because of limitations of the drilling and sampling methods.	
Soil / Rock Boundary	Material changes from soil characteristics to rock characteristics.	

REFERENCE: Caltrans Soil and Rock Logging, Classification, and Presentation Manual (2010).

GROUP DELTA

Project No. SD528

Lantern Crest Ridge II
Santee, California

BORING RECORD LEGEND #2

CONSISTENCY OF COHESIVE SOILS

Description	Shear Strength (tsf)	Pocket Penetrometer, PP, Measurement (tsf)	Torvane, TV, Measurement (tsf)	Vane Shear, VS, Measurement (tsf)
Very Soft	Less than 0.12	Less than 0.25	Less than 0.12	Less than 0.12
Soft	0.12 - 0.25	0.25 - 0.5	0.12 - 0.25	0.12 - 0.25
Medium Stiff	0.25 - 0.5	0.5 - 1	0.25 - 0.5	0.25 - 0.5
Stiff	0.5 - 1	1 - 2	0.5 - 1	0.5 - 1
Very Stiff	1 - 2	2 - 4	1 - 2	1 - 2
Hard	Greater than 2	Greater than 4	Greater than 2	Greater than 2

APPARENT DENSITY OF COHESIONLESS SOILS

Description	SPT N_{60} (blows / 12 inches)
Very Loose	0 - 5
Loose	5 - 10
Medium Dense	10 - 30
Dense	30 - 50
Very Dense	Greater than 50

MOISTURE

Description	Criteria
Dry	No discernable moisture
Moist	Moisture present, but no free water
Wet	Visible free water

PERCENT OR PROPORTION OF SOILS

Description	Criteria
Trace	Particles are present but estimated to be less than 5%
Few	5 - 10%
Little	15 - 25%
Some	30 - 45%
Mostly	50 - 100%

PARTICLE SIZE

Description		Size (in)
Boulder		Greater than 12
Cobble		3 - 12
Gravel	Coarse	3/4 - 3
	Fine	1/5 - 3/4
Sand	Coarse	1/16 - 1/5
	Medium	1/64 - 1/16
	Fine	1/300 - 1/64
Silt and Clay		Less than 1/300

CEMENTATION

Description	Criteria
Weak	Crumbles or breaks with handling or little finger pressure.
Moderate	Crumbles or breaks with considerable finger pressure.
Strong	Will not crumble or break with finger pressure.

REFERENCE: Caltrans Soil and Rock Logging, Classification, and Presentation Manual (2010), with the exception of consistency of cohesive soils vs. N_{60}.

Plasticity

Description	Criteria
Nonplastic	A 1⁄8-in. thread cannot be rolled at any water content.
Low	The thread can barely be rolled and the lump cannot be formed when drier than the plastic limit.
Medium	The thread is easy to roll and not much time is required to reach the plastic limit. The thread cannot be rerolled after reaching the plastic limit. The lump crumbles when drier than the plastic limit.
High	It takes considerable time rolling and kneading to reach the plastic limit. The thread can be rerolled several times after reaching the plastic limit. The lump can be formed without crumbling when drier than the plastic limit.

CONSISTENCY OF COHESIVE SOILS

Description	SPT N_{60} (blows/12 inches)
Very Soft	0 - 2
Soft	2 - 4
Medium Stiff	4 - 8
Stiff	8 - 15
Very Stiff	15 - 30
Hard	Greater than 30

Ref: Peck, Hansen, and Thornburn, 1974, "Foundation Engineering," Second Edition.

Note: Only to be used (with caution) when pocket penetrometer or other data on undrained shear strength are unavailable. Not allowed by Caltrans Soil and Rock Logging and Classification Manual, 2010.

GROUP DELTA

Project No. SD528

Lantern Crest Ridge II
Santee, California

BORING RECORD LEGEND #3

BORING RECORD

PROJECT NAME	PROJECT NUMBER	BORING
Lantern Crest Ridge II	SD528	**B-1**

SITE LOCATION	START	FINISH	SHEET NO.
10110 Sunset Trail, Santee, California	3/24/2017	3/24/2017	1 of 1

DRILLING COMPANY	DRILLING METHOD	LOGGED BY	CHECKED BY
Pacific Drilling	Hollow Stem Auger	T. Latimer	R. Stroop

DRILLING EQUIPMENT	BORING DIA. (in)	TOTAL DEPTH (ft)	GROUND ELEV (ft)	DEPTH/*ELEV.* GROUND WATER (ft)
FRASTE PL-G	6	8	500	▼ N/A / *na*

SAMPLING METHOD	NOTES
Hammer: 140 lbs., Drop: 30 in.	ETR ~ 83%, N_{60} ~ 83/60 * N ~ 1.38 * N

DEPTH (feet)	ELEVATION (feet)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS / 6 IN)	BLOW/FT "N"	N_{60}	MOISTURE (%)	DRY DENSITY (pcf)	OTHER TESTS	DEPTH (feet)	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
												GRANITIC ROCK (Kgr): GRANTIC ROCK; decomposed; very dense; gray-brown; moist; fine to coarse grained; some fines; trace mica.
			R-1	24 50 (5")	50/(5")	46/(5")			PA			(68% Sand; 32% Fines)
												GRANTIC ROCK; decomposed to intensely weathered; very dense; light brown to orange-brown; moist; fine to coarse grained; some fines; trace mica.
			S-2	8 17 50	67	93						
5	495									5		
												Terminated in hard granitic rock (refusal).
												Total Depth: 8 feet No groundwater encountered.

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FIGURE

A-1

BORING RECORD

PROJECT NAME	PROJECT NUMBER	BORING
Lantern Crest Ridge II	SD528	TP-1

SITE LOCATION	START	FINISH	SHEET NO.
10110 Sunset Trail, Santee, California	4/27/2017	4/27/2017	1 of 1

DRILLING COMPANY	DRILLING METHOD	LOGGED BY	CHECKED BY
West Tech	Test Pit	J. Sanders	R. Stroop

DRILLING EQUIPMENT	BORING DIA. (in)	TOTAL DEPTH (ft)	GROUND ELEV (ft)	DEPTH/*ELEV.* GROUND WATER (ft)
JD 410K Backhoe with 24-inch Bucket	24	4	524	▼ N/A / *na*

SAMPLING METHOD	NOTES
Shovel	

DEPTH (feet)	ELEVATION (feet)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS / 6 IN)	BLOW/FT "N"	N_{60}	MOISTURE (%)	DRY DENSITY (pcf)	OTHER TESTS	DEPTH (feet)	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
												COLLUVIUM (Qcol): Silty SAND (SM); loose; light brown to reddish brown; moist; mostly fine to coarse sand; little fines and gravel- to cobble-sized rock fragments; nonplastic.
												GRANITIC ROCK (Kgr): GRANTIC ROCK; intensely weathered; reddish brown to gray; moist; fine to coarse grained. Terminated in hard granitic rock.
	520											
												Total Depth: 4 feet No groundwater encountered.
5										5		
	515											

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San Diego, CA 92126

BORING RECORD

PROJECT NAME: Lantern Crest Ridge II	PROJECT NUMBER: SD528	BORING: **TP-2**	
SITE LOCATION: 10110 Sunset Trail, Santee, California	START: 4/27/2017	FINISH: 4/27/2017	SHEET NO.: 1 of 1
DRILLING COMPANY: West Tech	DRILLING METHOD: Test Pit	LOGGED BY: J. Sanders	CHECKED BY: R. Stroop
DRILLING EQUIPMENT: JD 410K Backhoe with 24-inch Bucket	BORING DIA. (in): 24	TOTAL DEPTH (ft): 7	GROUND ELEV (ft): 512
DEPTH/*ELEV.* GROUND WATER (ft): ⊻ N/A / *na*			
SAMPLING METHOD: Shovel	NOTES:		

DEPTH (feet)	ELEVATION (feet)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS / 6 IN)	BLOW/FT "N"	N_{60}	MOISTURE (%)	DRY DENSITY (pcf)	OTHER TESTS	DEPTH (feet)	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
												COLLUVIUM (Qcol): Silty SAND (SM); loose; reddish brown; dry; mostly fine to coarse sand; little fines; nonplastic.
			B-1				4.8					Moist.
	510											
												GRANITIC ROCK (Kgr): GRANTIC ROCK; intensely weathered; reddish brown to gray; moist; fine to coarse grained; little rock fragments up to 12 inches in diameter.
			B-2				4.8		PA CR MAX R			(2% Gravel; 84% Sand; 14% Fines)
5										5		Harder excavation; intensely to moderately weathered.
												Terminated in hard granitic rock.
	505											
												Total Depth: 7 feet No groundwater encountered.

GDC_LOG_BORING_MMX_SOIL_SD SD528 LOGS.GPJ GDCLOG.GDT 5/24/17

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San Diego, CA 92126

THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF THE ACTUAL CONDITIONS ENCOUNTERED.

FIGURE A-3

BORING RECORD

PROJECT NAME	Lantern Crest Ridge II
PROJECT NUMBER	SD528
BORING	TP-3
SITE LOCATION	10110 Sunset Trail, Santee, California
START	4/27/2017
FINISH	4/27/2017
SHEET NO.	1 of 1
DRILLING COMPANY	West Tech
DRILLING METHOD	Test Pit
LOGGED BY	J. Sanders
CHECKED BY	R. Stroop
DRILLING EQUIPMENT	JD 410K Backhoe with 24-inch Bucket
BORING DIA. (in)	24
TOTAL DEPTH (ft)	4.5
GROUND ELEV (ft)	534
DEPTH/*ELEV.* GROUND WATER (ft)	▼ N/A / *na*
SAMPLING METHOD	Shovel
NOTES	

DEPTH (feet)	ELEVATION (feet)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS / 6 IN)	BLOW/FT "N"	N_{60}	MOISTURE (%)	DRY DENSITY (pcf)	OTHER TESTS	DEPTH (feet)	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
												COLLUVIUM (Qcol): Silty SAND (SM); loose; reddish brown; moist; mostly fine to coarse sand; little fines; few gravel and rock fragments; nonplastic.
	530											**GRANITIC ROCK (Kgr):** GRANTIC ROCK; decomposed to intensely weathered; reddish brown; moist; fine to coarse grained; rock fragments up to 24 inches in diameter. Harder excavation; intensely to moderarely weathered; gray to light brown. Terminated in hard granitic rock.
5										5		Total Depth: 4.5 feet No groundwater encountered.
	525											

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FIGURE A-4

BORING RECORD

PROJECT NAME: Lantern Crest Ridge II			PROJECT NUMBER: SD528		BORING: **TP-4**
SITE LOCATION: 10110 Sunset Trail, Santee, California			START: 4/27/2017	FINISH: 4/27/2017	SHEET NO.: 1 of 1
DRILLING COMPANY: West Tech	DRILLING METHOD: Test Pit		LOGGED BY: J. Sanders		CHECKED BY: R. Stroop
DRILLING EQUIPMENT: JD 410K Backhoe with 24-inch Bucket	BORING DIA. (in): 24	TOTAL DEPTH (ft): 6	GROUND ELEV (ft): 510	DEPTH/*ELEV.* GROUND WATER (ft): ▼ N/A / *na*	
SAMPLING METHOD: Shovel	NOTES:				

DEPTH (feet)	ELEVATION (feet)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS / 6 IN)	BLOW/FT "N"	N_{60}	MOISTURE (%)	DRY DENSITY (pcf)	OTHER TESTS	DEPTH (feet)	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
												COLLUVIUM (Qcol): Silty SAND (SM); loose; reddish brown; dry; mostly fine to coarse sand; little fines; few gravel; nonplastic.
												Moist.
												GRANITIC ROCK (Kgr): GRANTIC ROCK; decomposed to intensely weathered; reddish brown to gray; moist; fine to coarse grained.
												Harder excavation; intensely to moderately weathered; rock fragments up to 24 inches in diameter
5	505									5		
												Terminated in hard granitic rock.
												Total Depth: 6 feet No groundwater encountered.

GDC_LOG_BORING_MMX_SOIL_SD SD528 LOGS.GPJ GDCLOG.GDT 5/24/17

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San Diego, CA 92126

FIGURE A-5

BORING RECORD

PROJECT NAME	Lantern Crest Ridge II
PROJECT NUMBER	SD528
BORING	**TP-5**
SITE LOCATION	10110 Sunset Trail, Santee, California
START	4/27/2017
FINISH	4/27/2017
SHEET NO.	1 of 1
DRILLING COMPANY	West Tech
DRILLING METHOD	Test Pit
LOGGED BY	J. Sanders
CHECKED BY	R. Stroop
DRILLING EQUIPMENT	JD 410K Backhoe with 24-inch Bucket
BORING DIA. (in)	24
TOTAL DEPTH (ft)	7
GROUND ELEV (ft)	502
DEPTH/*ELEV.* GROUND WATER (ft)	▼ N/A / *na*
SAMPLING METHOD	Shovel
NOTES	

DEPTH (feet)	ELEVATION (feet)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS / 6 IN)	BLOW/FT "N"	N_{60}	MOISTURE (%)	DRY DENSITY (pcf)	OTHER TESTS	DEPTH (feet)	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
		Bulk	B-1				8.8		PA CR EI			**<u>COLLUVIUM (Qcol)</u>:** Silty SAND (SM); loose; reddish brown; dry; mostly fine to coarse sand; little fines; trace gravel; nonplastic; with rock fragments up to 24 inches in diameter. (1% Gravel; 55% Sand; 44% Fines)
	500											**<u>GRANITIC ROCK (Kgr)</u>:** GRANTIC ROCK; decomposed to intensely weathered; reddish brown to gray; dry; fine to coarse grained; with rock fragments up to 12 inches in diameter. Moist.
												Harder excavation; intensely to moderately weathered.
5										5		
												Difficult excavation; moderately weathered with fresh rock fragments.
	495											Terminated in hard granitic rock.
												Total Depth: 7 feet No groundwater encountered.

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FIGURE A-6

BORING RECORD

PROJECT NAME	PROJECT NUMBER	BORING
Lantern Crest Ridge II	SD528	**TP-6**

SITE LOCATION	START	FINISH	SHEET NO.
10110 Sunset Trail, Santee, California	4/27/2017	4/27/2017	1 of 1

DRILLING COMPANY	DRILLING METHOD	LOGGED BY	CHECKED BY
West Tech	Test Pit	J. Sanders	R. Stroop

DRILLING EQUIPMENT	BORING DIA. (in)	TOTAL DEPTH (ft)	GROUND ELEV (ft)	DEPTH/*ELEV.* GROUND WATER (ft)
JD 410K Backhoe with 24-inch Bucket	24	1	518	⊻ N/A / *na*

SAMPLING METHOD	NOTES
Shovel	

DEPTH (feet)	ELEVATION (feet)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS / 6 IN)	BLOW/FT "N"	N_{60}	MOISTURE (%)	DRY DENSITY (pcf)	OTHER TESTS	DEPTH (feet)	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
												COLLUVIUM (Qcol): Silty SAND (SM); loose; brown to reddish brown; moist; mostly fine to coarse sand; little fines and gravel- to cobble-sized rock fragments; nonplastic. Terminated on hard granitic rock.
												Total Depth: 1 foot No groundwater encountered.
	515											
5										5		
	510											

GDC_LOG_BORING_MMX_SOIL_SD SD528 LOGS.GPJ GDCLOG.GDT 5/24/17

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San Diego, CA 92126

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FIGURE

A-7

BORING RECORD

PROJECT NAME	PROJECT NUMBER	BORING
Lantern Crest Ridge II	SD528	TP-7

SITE LOCATION	START	FINISH	SHEET NO.
10110 Sunset Trail, Santee, California	4/27/2017	4/27/2017	1 of 1

DRILLING COMPANY	DRILLING METHOD	LOGGED BY	CHECKED BY
West Tech	Test Pit	J. Sanders	R. Stroop

DRILLING EQUIPMENT	BORING DIA. (in)	TOTAL DEPTH (ft)	GROUND ELEV (ft)	DEPTH/*ELEV.* GROUND WATER (ft)
JD 410K Backhoe with 24-inch Bucket	24	6	532	▼ N/A / *na*

SAMPLING METHOD	NOTES
Shovel	

DEPTH (feet)	ELEVATION (feet)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS / 6 IN)	BLOW/FT "N"	N_{60}	MOISTURE (%)	DRY DENSITY (pcf)	OTHER TESTS	DEPTH (feet)	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
												COLLUVIUM (Qcol): Silty SAND (SM); loose; reddish brown; dry; mostly fine to coarse sand; little fines; few gravel; nonplastic; few angular rock fragments.
	530											
												Moist.
												GRANITIC ROCK (Kgr): GRANTIC ROCK; intensely weathered; reddish brown; moist; fine to coarse grained.
5										5		Becomes light brown to gray.
												Terminated in hard granitic rock.
	525											Total Depth: 6 feet No groundwater encountered.

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FIGURE A-8

APPENDIX B

LABORATORY TESTING

APPENDIX B

LABORATORY TESTING

Laboratory testing was conducted in a manner consistent with the level of care and skill ordinarily exercised by members of the profession currently practicing under similar conditions and in the same locality. No warranty, express or implied, is made as to the correctness or serviceability of the test results, or the conclusions derived from these tests. Where a specific laboratory test method has been referenced, such as ASTM or Caltrans, the reference only applies to the specified laboratory test method, which has been used only as a guidance document for the general performance of the test and not as a "Test Standard". A brief description of the various tests performed for this project follows.

Classification: Soils were visually classified per the Unified Soil Classification System as established by the American Society of Civil Engineers per ASTM D2487. The soil classifications are shown on the exploration logs in Appendix A.

Particle Size Analysis: Particle size analyses were performed in accordance with ASTM D422 and were used to supplement visual classifications. The test results are shown in the exploration logs in Appendix A and in Figures B-1.1 and B-1.3.

Expansion Index: The expansion potential of a selected soil sample was estimated in general accordance with the laboratory procedures outlined in ASTM test method D4829. The test results are summarized in Figure B-2. Figure B-2 also presents common criteria for evaluating the expansion potential based on the expansion index.

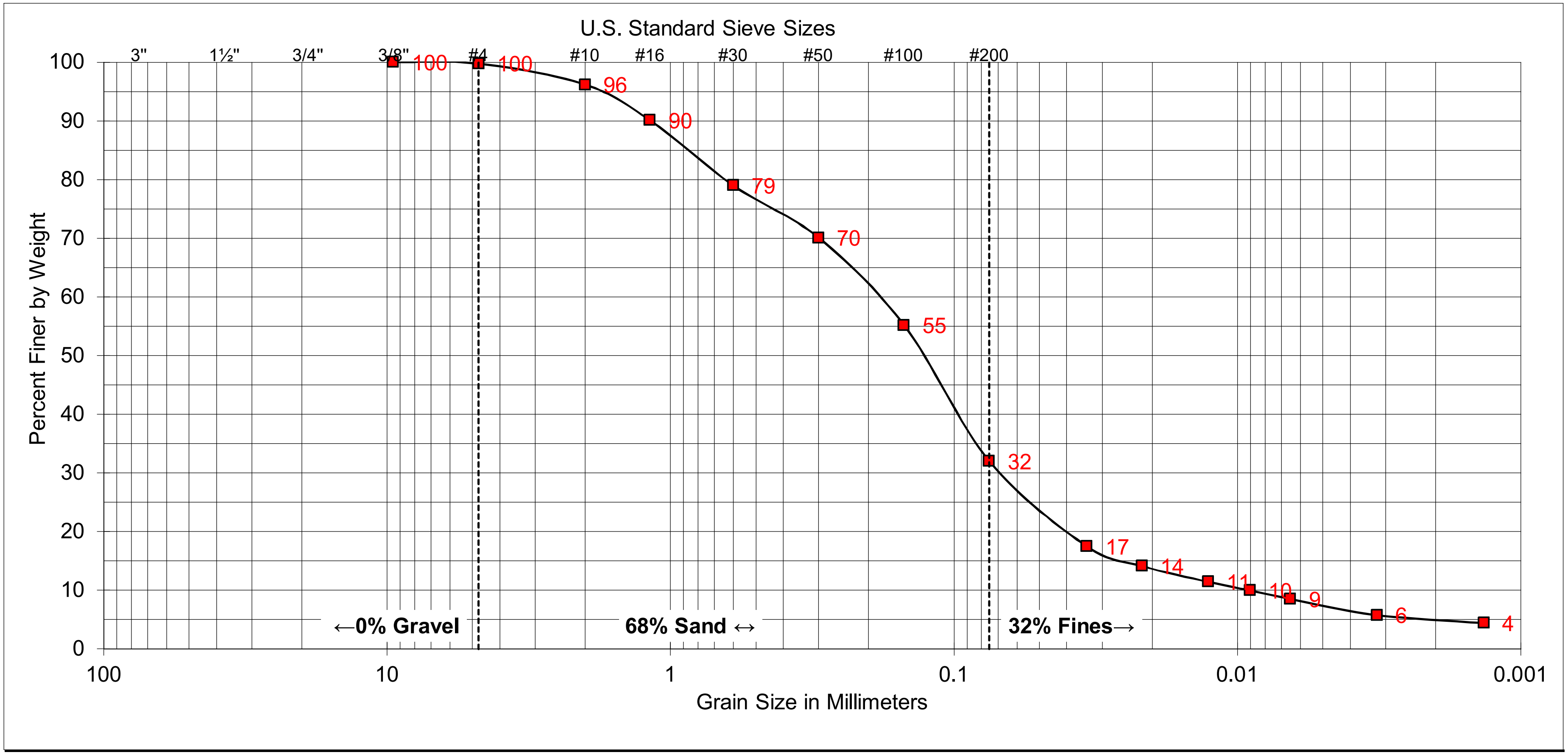
pH and Resistivity: To assess the potential for reactivity with buried metals, selected soil samples were tested for pH and minimum resistivity using Caltrans test method 643. The corrosivity test results are summarized in Figure B-3.

Sulfate Content: To assess the potential for reactivity with concrete, selected soil samples were tested for water soluble sulfate. The sulfate was extracted from the soil under vacuum using a 10:1 (water to dry soil) dilution ratio. The extracted solution was tested for water soluble sulfate in general accordance with ASTM D516. The test results are also presented in Figure B-3, along with common criteria for evaluating soluble sulfate content.

Chloride Content: Soil samples were also tested for water soluble chloride. The chloride was extracted from the soil under vacuum using a 10:1 (water to dry soil) dilution ratio. The extracted solution was then tested for water soluble chloride using a calibrated ion specific electronic probe. The test results are also shown in Figure B-3.

Modified Proctor Density Test: A modified Proctor density test was performed on a representative bulk soil sample in general accordance with ASTM D1557. The maximum modified Proctor density and optimum moisture content are shown on Figure B-4.

R-Value: R-Value tests were performed on selected samples of the subgrade soils collected from the site in general accordance with CTM 301. The test results are shown in Figure B-4.



COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND
GRAVEL		SAND			CLAY

SAMPLE	
BORING NUMBER:	B-1
SAMPLE DEPTH:	3' - 3.5'

UNIFIED SOIL CLASSIFICATION: SM

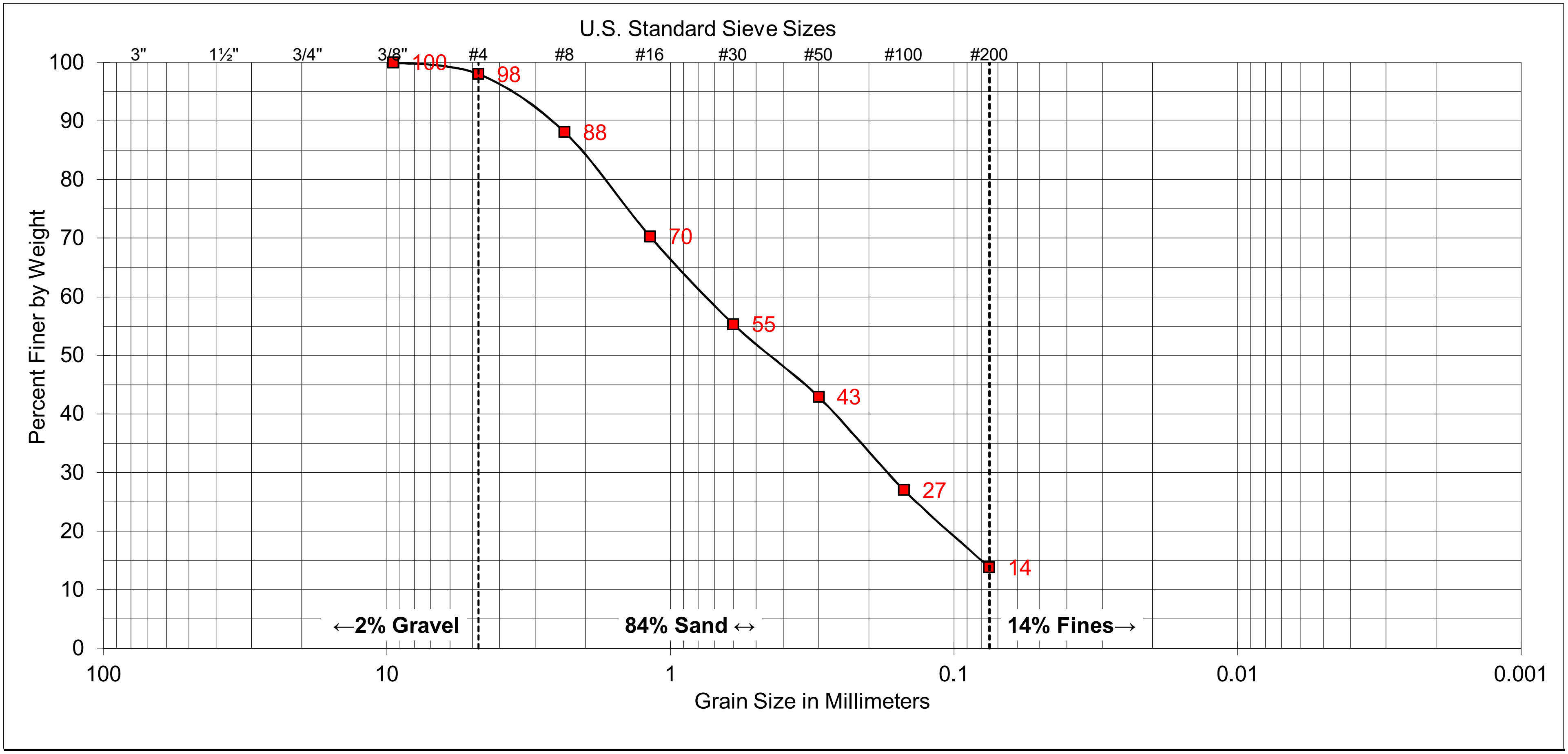
DESCRIPTION: SILTY SAND

ATTERBERG LIMITS	
LIQUID LIMIT:	--
PLASTIC LIMIT:	--
PLASTICITY INDEX:	--

GROUP DELTA

SOIL CLASSIFICATION

Document No. 17-0057
Project No. SD528
FIGURE B-1.1



COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND CLAY
GRAVEL		SAND			

SAMPLE	
BORING NUMBER:	TP-2
SAMPLE DEPTH:	4' - 5'

UNIFIED SOIL CLASSIFICATION: SM

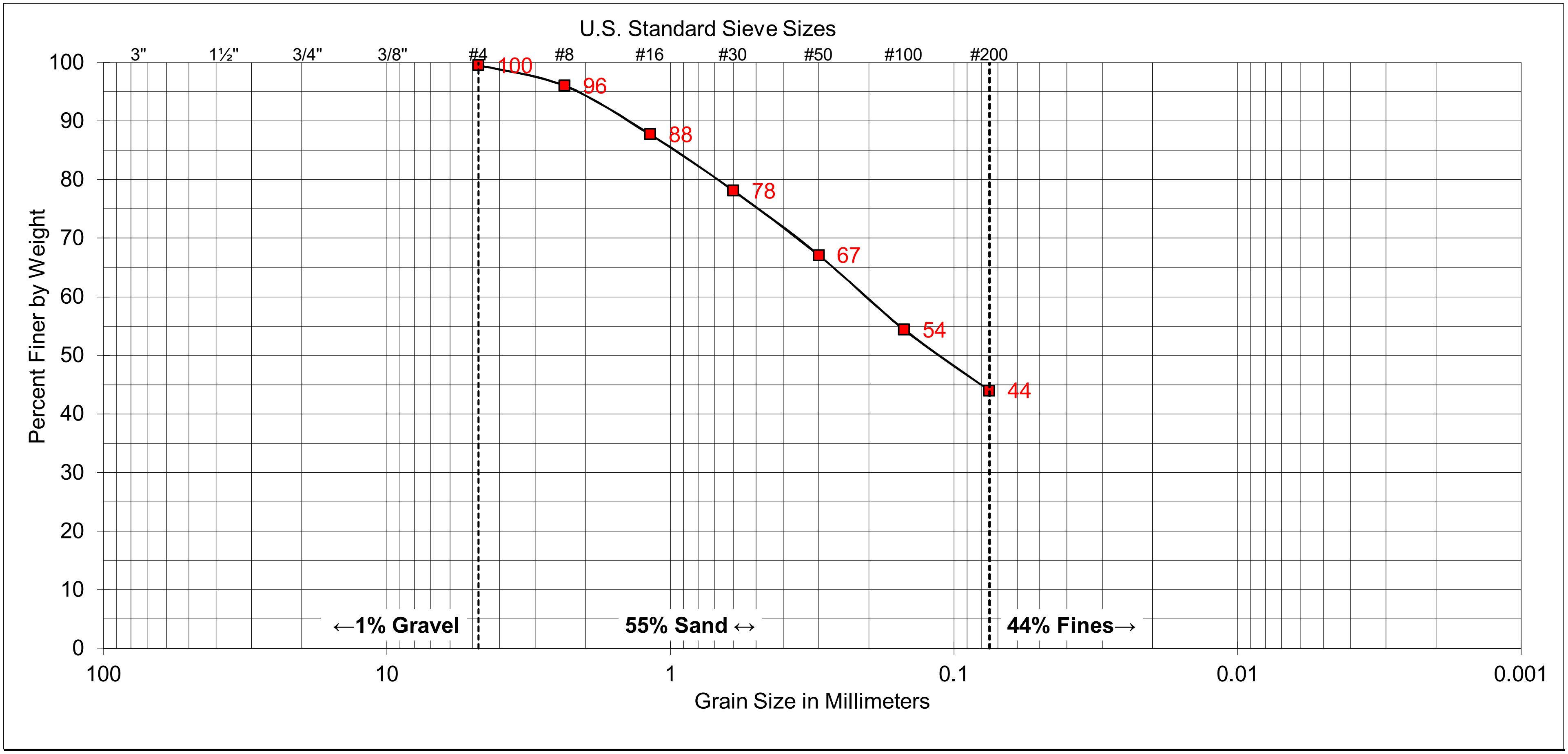
DESCRIPTION: SILTY SAND

ATTERBERG LIMITS	
LIQUID LIMIT:	--
PLASTIC LIMIT:	--
PLASTICITY INDEX:	--

GROUP DELTA

SOIL CLASSIFICATION

Document No. 17-0057
Project No. SD528
FIGURE B-1.2



COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND
GRAVEL		SAND			CLAY

SAMPLE	
BORING NUMBER:	TP-5
SAMPLE DEPTH:	0' - 2'

UNIFIED SOIL CLASSIFICATION: SM

DESCRIPTION: SILTY SAND

ATTERBERG LIMITS	
LIQUID LIMIT:	--
PLASTIC LIMIT:	--
PLASTICITY INDEX:	--

GROUP DELTA

SOIL CLASSIFICATION

Document No. 17-0057
Project No. SD528
FIGURE B-1.3

EXPANSION TEST RESULTS
(ASTM D4829)

SAMPLE	DESCRIPTION	EXPANSION INDEX
TP-5 @ 0'– 2'	**COLLUVIUM**: Reddish brown silty SAND (SM)	44

EXPANSION INDEX	POTENTIAL EXPANSION
0 to 20	Very low
21 to 50	Low
51 to 90	Medium
91 to 130	High
Above 130	Very High

CORROSIVITY TEST RESULTS
(ASTM D516, CTM 643)

SAMPLE NO.	pH	RESISTIVITY [OHM-CM]	SULFATE CONTENT [%]	CHLORIDE CONTENT [%]
TP-2 @ 4' – 5'	7.7	4,880	<0.01	<0.01
TP-5 @ 0' – 2'	7.5	1,160	<0.01	0.01

SULFATE CONTENT [%]	SULFATE EXPOSURE	CEMENT TYPE
0.00 to 0.10	Negligible	-
0.10 to 0.20	Moderate	II, IP(MS), IS(MS)
0.20 to 2.00	Severe	V
Above 2.00	Very Severe	V plus pozzolan

SOIL RESISTIVITY [OHM-CM]	GENERAL DEGREE OF CORROSIVITY TO FERROUS METALS
0 to 1,000	Very Corrosive
1,000 to 2,000	Corrosive
2,000 to 5,000	Moderately Corrosive
5,000 to 10,000	Mildly Corrosive
Above 10,000	Slightly Corrosive

CHLORIDE (Cl) CONTENT [%]	GENERAL DEGREE OF CORROSIVITY TO METALS
0.00 to 0.03	Negligible
0.03 to 0.15	Corrosive
Above 0.15	Severely Corrosive

MODIFIED PROCTOR DENSITY TEST
(ASTM D1557)

SAMPLE NO.	DESCRIPTION	MAXIMUM DRY DENSITY [lb/ft^3]	OPTIMUM MOISTURE [%]
TP-2 @ 4' – 5'	**Excavated Granitic Rock**: Reddish Brown Silty SAND (SM).	135.2	8.5

R-VALUE TEST RESULTS
(CTM 301)

SAMPLE NO.	DESCRIPTION	R-VALUE
TP-2 @ 4' – 5'	**Excavated Granitic Rock**: Reddish Brown Silty SAND (SM).	76

APPENDIX C

INFILTRATION FEASIBILITY ASSESSMENT

Worksheet 0-1: Categorization of Infiltration Feasibility Condition

<table>
<tr><th colspan="2">Categorization of Infiltration Feasibility Condition</th><th colspan="2">Worksheet C.4-1</th></tr>
<tr><td colspan="4"><u>Part 1 - Full Infiltration Feasibility Screening Criteria</u>
Would infiltration of the full design volume be feasible from a physical perspective without any undesirable consequences that cannot be reasonably mitigated?</td></tr>
<tr><td>Criteria</td><td>Screening Question</td><td>Yes</td><td>No</td></tr>
<tr><td>1</td><td>Is the estimated reliable infiltration rate below proposed facility locations greater than 0.5 inches per hour? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2 and Appendix D.</td><td></td><td>✓</td></tr>
<tr><td colspan="4">Provide basis: The average design infiltration rate from field testing is 0.3 inches/hour, as shown in the attached field infiltration test results.

Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.</td></tr>
<tr><td>2</td><td>Can infiltration greater than 0.5 inches per hour be allowed without increasing risk of geotechnical hazards (slope stability, groundwater mounding, utilities, or other factors) that cannot be mitigated to an acceptable level? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2.</td><td></td><td>✓</td></tr>
<tr><td colspan="4">Provide basis: Please see the answer to Criteria 1.

Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.</td></tr>
</table>

<table>
<tr><th colspan="4">Worksheet C.4-1 Page 2 of 4</th></tr>
<tr><th>Criteria</th><th>Screening Question</th><th>Yes</th><th>No</th></tr>
<tr><td>3</td><td>Can infiltration greater than 0.5 inches per hour be allowed without increasing risk of groundwater contamination (shallow water table, storm water pollutants or other factors) that cannot be mitigated to an acceptable level? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.</td><td></td><td>✓</td></tr>
<tr><td colspan="4">Provide basis: Please see the answer to Criteria 1.

Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.</td></tr>
<tr><td>4</td><td>Can infiltration greater than 0.5 inches per hour be allowed without causing potential water balance issues such as change of seasonality of ephemeral streams or increased discharge of contaminated groundwater to surface waters? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.</td><td></td><td>✓</td></tr>
<tr><td colspan="4">Provide basis: Please see the answer to Criteria 1.

Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.</td></tr>
<tr><td>Part 1 Result*</td><td colspan="2">If all answers to rows 1 - 4 are "Yes" a full infiltration design is potentially feasible. The feasibility screening category is Full Infiltration

If any answer from row 1-4 is "No", infiltration may be possible to some extent but would not generally be feasible or desirable to achieve a "full infiltration" design. Proceed to Part 2</td><td></td></tr>
</table>

*To be completed using gathered site information and best professional judgment considering the definition of MEP in the MS4 Permit. Additional testing and/or studies may be required by City Engineer to substantiate findings.

Worksheet C.4-1 Page 3 of 4

Part 2 – Partial Infiltration vs. No Infiltration Feasibility Screening Criteria

Would infiltration of water in any appreciable amount be physically feasible without any negative consequences that cannot be reasonably mitigated?

Criteria	Screening Question	Yes	No
5	**Do soil and geologic conditions allow for infiltration in any appreciable rate or volume?** The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2 and Appendix D.		✓

Provide basis: Refusal on hard granitic rock was encountered at a depth of 8 feet below ground surface.

Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability and why it was not feasible to mitigate low infiltration rates.

Criteria	Screening Question	Yes	No
6	**Can Infiltration in any appreciable quantity be allowed without increasing risk of geotechnical hazards (slope stability, groundwater mounding, utilities, or other factors) that cannot be mitigated to an acceptable level**? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2.	✓	

Provide basis: See the Group Delta Consultants report dated June 14, 2017.

Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability and why it was not feasible to mitigate low infiltration rates.

Worksheet C.4-1 Page 4 of 4			
Criteria	Screening Question	**Yes**	**No**
7	**Can Infiltration in any appreciable quantity be allowed without posing significant risk for groundwater related concerns (shallow water table, storm water pollutants or other factors)?** The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.	✓	
Provide basis: See the Group Delta Consultants report dated June 14, 2017. Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability and why it was not feasible to mitigate low infiltration rates.			
8	**Can infiltration be allowed without violating downstream water rights**? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.	✓	
Provide basis: No known downstream water rights have been identified, and there are no bodies of water immediately downstream from the site Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability and why it was not feasible to mitigate low infiltration rates.			
Part 2 Result*	If all answers from row 1-4 are yes then partial infiltration design is potentially feasible. The feasibility screening category is **Partial Infiltration.** If any answer from row 5-8 is no, then infiltration of any volume is considered to be **infeasible** within the drainage area. The feasibility screening category is **No Infiltration.**	No Infiltration	

*To be completed using gathered site information and best professional judgment considering the definition of MEP in the Permit. Additional testing and/or studies may be required by Agency/Jurisdictions to substantiate findings

BOREHOLE PERCOLATION TEST DATA SHEET
Storm Water Infilitration

Project Name: Lantern Crest **Job Number:** SD528 **Tested By:** TSL

Test Hole No: B-1A **Date Drilled:** 3/24/2017 **Date Tested:** 3/25/2017

Drilling Method: Hollow Stem Auger **Borehole Radius:** 3 inches

Depth of Hole as Drilled: 5 ft **Casing Stick-up:** 0.95 ft **Test Depth:** 2-5 ft

Reading Number	Time	Time Interval (min.)	Total Depth of Hole (ft.)	Initial Depth of Water (ft.)	Final Depth of Water (ft.)	Change in Water Level (in.)	Percolation Rate (in./min.)	Infiltration Rate (in./hour)*
Presoak	13:45 14:10	0:25	4.23	1.60	2.31	8.52	0.34	0.53
Presoak	14:10 14:35	0:25	4.23	2.31	2.39	0.96	0.04	0.07
Presoak	14:35 15:00	0:25	4.23	2.39	2.46	0.84	0.03	0.07
Presoak	15:00 15:25	0:25	4.17	2.46	2.58	1.44	0.06	0.12
0	15:25 8:35	17:10	4.21	2.58	EMPTY			
1	8:35 9:05	0:30	4.21	1.49	1.61	1.44	0.05	0.06
2	9:05 9:35	0:30	4.21	1.61	1.74	1.56	0.05	0.07
3	9:35 10:05	0:30	4.21	1.74	1.86	1.44	0.05	0.07
4	10:05 10:35	0:30	4.21	1.86	1.97	1.32	0.04	0.07
5	10:35 11:05	0:30	4.21	1.97	2.06	1.08	0.04	0.06
6	11:05 11:35	0:30	4.21	2.06	2.16	1.20	0.04	0.07
7	11:36 12:06	0:30	4.21	1.90	2.01	1.32	0.04	0.07
8	12:06 12:36	0:30	4.21	2.01	2.10	1.08	0.04	0.06
9	12:36 13:06	0:30	4.21	2.10	2.18	0.96	0.03	0.05

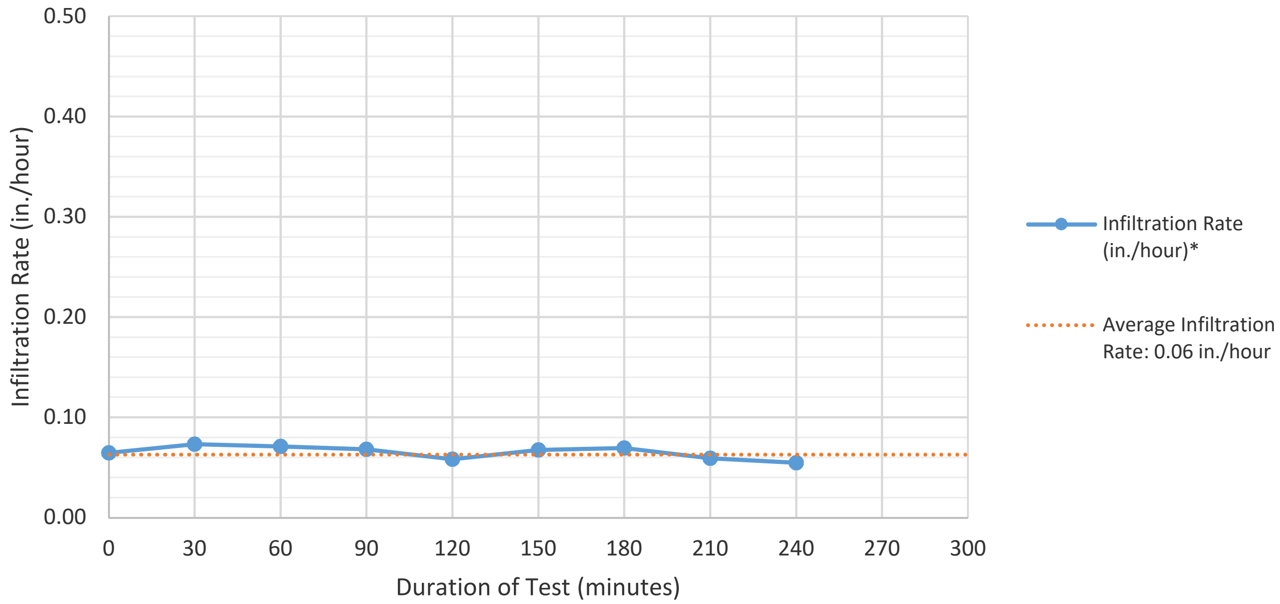
*Factor of Safety of 2 was used to calculate final values.

GROUP DELTA

BOREHOLE PERCOLATION TEST B-1A

Project No. SD528
Document No. 17-0057
FIGURE NO. C-1A

Borehole Percolation Test Results



GROUP DELTA

BOREHOLE PERCOLATION TEST
B-1A

Project No. SD528
Document No. 17-0057
FIGURE NO. C-1B

BOREHOLE PERCOLATION TEST DATA SHEET
Storm Water Infiltration

Project Name:	Lantern Crest	**Job Number:**	SD528	**Tested By:**	TSL
Test Hole No:	B-1B	**Date Drilled:**	3/24/2017	**Date Tested:**	3/25/2017
Drilling Method:	Hollow Stem Auger	**Borehole Radius:**	3 inches		
Depth of Hole as Drilled:	5 ft	**Casing Stick-up:**	0.5 ft	**Test Depth:**	2-5 ft

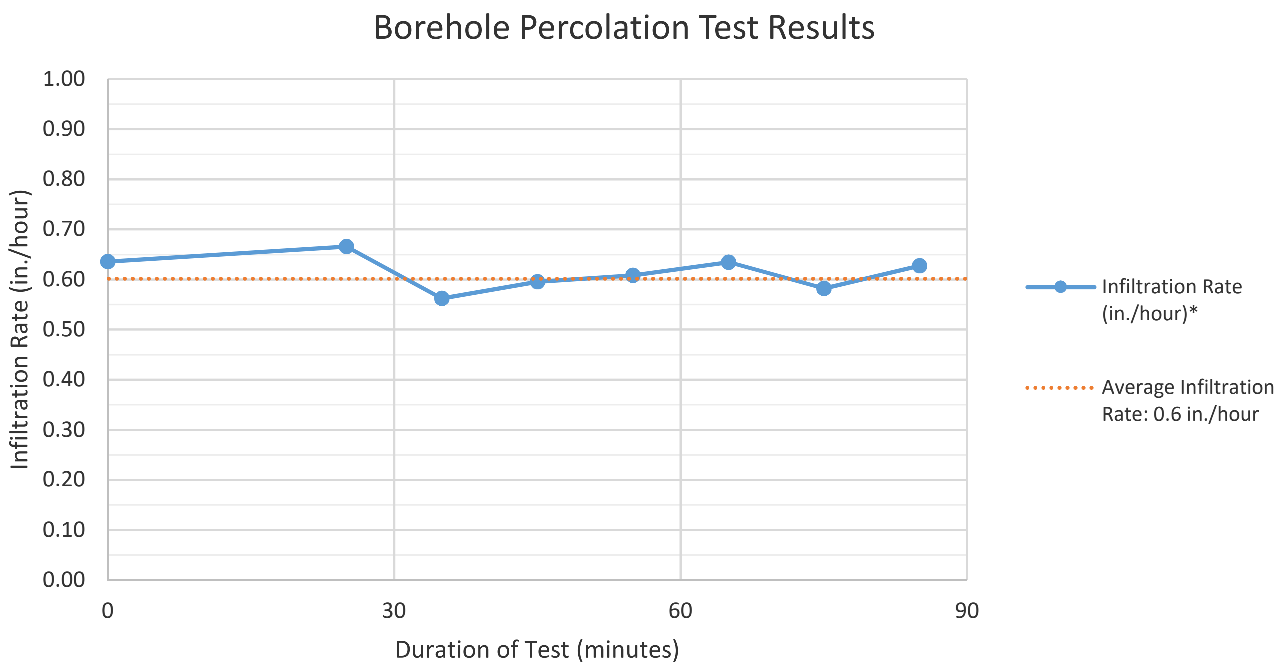
Reading Number	Time	Time Interval (min.)	Total Depth of Hole (ft.)	Initial Depth of Water (ft.)	Final Depth of Water (ft.)	Change in Water Level (in.)	Percolation Rate (in./min.)	Infiltration Rate (in./hour)*
Presoak	13:51 14:16	0:25	4.40	1.65	2.33	8.16	0.33	0.48
Presoak	14:16 14:41	0:25	4.40	2.33	3.10	9.24	0.37	0.77
Presoak	14:41 15:06	0:25	4.40	3.10	3.62	6.24	0.25	0.80
Presoak	15:06 15:31	0:25	4.38	3.62	3.97	4.20	0.17	0.89
Presoak	9:40 11:40	2:00	4.39	1.33	3.97	31.68	0.26	0.53
0	11:40 12:05	0:25	4.38	2.44	3.06	7.44	0.30	0.64
1	12:05 12:30	0:25	4.38	2.55	3.16	7.32	0.29	0.67
2	12:32 12:42	0:10	4.38	2.38	2.63	3.00	0.30	0.56
3	12:42 12:52	0:10	4.38	2.49	2.74	3.00	0.30	0.60
4	12:52 13:02	0:10	4.38	2.53	2.78	3.00	0.30	0.61
5	13:03 13:13	0:10	4.38	2.53	2.79	3.12	0.31	0.63
6	13:14 13:24	0:10	4.38	2.53	2.77	2.88	0.29	0.58
7	13:24 13:34	0:10	4.38	2.51	2.77	3.12	0.31	0.63

*Infiltration rate calculated using the Porchet Method. Factor of Safety of 2 was used to calculate final values.

GROUP DELTA

BOREHOLE PERCOLATION TEST B-1B

Project No. SD528
Document No. 17-0057
FIGURE NO. C-2A



BOREHOLE PERCOLATION TEST B-1B

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FIGURE NO. C-2B