UPDATED GEOTECHNICAL INVESTIGATION
PROSPECT ESTATES II DEVELOPMENT
SANTEE, CALIFORNIA

Prepared for

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

Prepared by

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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: **UPDATED GEOTECHNICAL INVESTIGATION**
Prospect Estates II Development
Santee, California

Mr. Grant:

The following report provides an updated geotechnical investigation for the Prospect Estates II residential development in the City of Santee, California. As part of this update, we have reviewed the referenced geotechnical report that was recently prepared for the southern portion of the site (GEI, 2016), as well as our findings from the Prospect Estates I investigation located immediately east of the site (GDC, 2016a). We have also completed a supplemental subsurface investigation in the northern portion of the site, including five additional borings, laboratory tests and analyses.

The following update provides preliminary geotechnical recommendations for site development. Note that geologic observation and additional laboratory testing will be needed during grading of the site in order to better characterize the depth and distribution of expansive soils, and develop the final geotechnical parameters for post-tension slab foundation design. Updated geotechnical recommendations should be provided in the as-graded report once the site grading is completed.

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

The following report provides the results of our updated geotechnical investigation for the proposed Prospect Estates II residential development in Santee, California. The location of the property is shown on the Site Location Map, Figure 1. The site vicinity is shown in more detail on the Exploration Plan, Figure 2. The approximate locations of the six borings previously excavated at the site, as well as the five supplemental borings conducted for this study are shown in Figure 2.

The purpose of this geotechnical investigation was to characterize the general geotechnical constraints across the entire site, and provide updated geotechnical recommendations for remedial grading and mitigation of the highly expansive soil conditions that are prevalent throughout the site. The recommendations provided herein are based on the subsurface explorations and laboratory test results, as well as engineering and geologic 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, 2016b). In summary, we provided the following scope of services.

- We conducted a geologic reconnaissance of the general site conditions, and reviewed the previous reports referenced in Section 8.0.

- We conducted a supplemental subsurface exploration in the northern portion of the site which included five exploratory borings at the approximate locations shown on the Exploration Plan, Figure 2. We also reviewed the findings of the previous subsurface exploration at the site which including six exploratory borings at the locations shown in Figure 2. Logs for all of the borings are provided in Appendix A.

- We conducted laboratory tests on selected soil samples collected from the borings including sieve analysis, Atterberg Limits, Expansion Index, pH, resistivity, soluble sulfate and chloride, direct shear and R-Value. The laboratory test results are presented in Appendix B. We also incorporated previous tests that we conducted for the Prospect Estates I development immediately east of the site (GDC, 2016c).

- We conducted engineering analyses using the field and laboratory data to help develop preliminary geotechnical recommendations for site preparation, remedial earthwork, post-tension slab foundation, pavement, and retaining wall design, soil reactivity, and site drainage and moisture protection.

- We prepared this report summarizing our findings, conclusions and preliminary geotechnical recommendations for remedial grading and site development.

1.2 Site Description

The approximate centroid of the site is located at a longitude of 32.8330° north and latitude of 117.0098° west, as shown on the Site Location Map, Figure 1. The property is situated southwest of the intersection between State Routes 52 and 125. The site may be accessed via Prospect Avenue, which forms the southern property boundary. The western and southeastern portions of the site are bordered by existing single family residential properties. The geologic conditions at the Prospect Estates I residential development, which is located immediately east of the subject site, are described in detail in the referenced geotechnical report (GDC, 2016a).

At the time of our subsurface investigation, the southern portion of the site was covered with a light growth of weeds and grass. Several residential structures currently occupy the northern portion of the site. These structures are surrounded by numerous trees and landscaping areas. The property slopes down gently towards the San Diego River and Mission Gorge to the north. Elevations on site range from a high of about 373 feet along Prospect Avenue, down to a low of about 345 feet along the northern property line.

1.3 Proposed Development

A preliminary site development plan showing the general layout of the proposed subdivision is shown on the Exploration Plan, Figure 2. We understand that site development may include 46 two-story single family residential buildings supported by post-tension slab foundations. Other site improvements will include asphalt concrete paved residential streets and cul-de-sacs, Portland cement concrete sidewalks and driveways, and a variety of subsurface utilities. A vegetated bio-retention basin and a small park area also proposed at the site, as shown in Figure 2.

We anticipate that site development will begin with the demolition of the existing structures, and the removal of the landscaping vegetation and other improvements in the northern portion of the property. Remedial grading will be conducted to remove and compact the compressible alluvium, remediate cut/fill transitions, and provide a minimum 3-foot thick cap of relatively low expansion soil throughout the surface of the site (EI<70). Cut and fill grading will also be needed to create the level building pad areas, with cut and fill depths typically on the order of 5 feet or less. A variety of retaining walls are also anticipated to accommodate the grade changes.

2.0 FIELD AND LABORATORY INVESTIGATION

The field investigation program included a visual and geologic reconnaissance of the site, and the advancement of five supplemental exploratory borings on May 16th, 2017. We also reviewed the findings of six exploratory borings excavated at the site by others on November 18th, 2015. The maximum depth of exploration at the site was about 17½ feet below surrounding grades. The approximate locations of all of the exploratory borings are shown on the Exploration Plan, Figure 2. Logs for all of these borings are provided in the figures of Appendix A (GEI, 2016).

Various soil samples were collected from the borings for laboratory testing and analysis. The testing program included gradation analysis and Atterberg Limits to help classify the site soils using the Unified Soil Classification System (USCS). Expansion Index, pH, resistivity, sulfate and chloride tests were conducted to help evaluate the soil expansion and corrosion potential. Direct shear tests were conducted to help estimate the in-situ soil strength. R-Value tests were conducted to aid in preliminary pavement section design. The laboratory test results are in Appendix B.

3.0 GEOLOGY AND SUBSURFACE CONDITIONS

The site is located within the coastal plain section of the Peninsular Ranges geomorphic province of southern California. The coastal plain generally consists of subdued landforms underlain by marine sedimentary formations. Specifically, the site is underlain by Granitic Rock and the Friars Formation, which are covered with a variable depth of alluvium and undocumented fill throughout the site. The general geologic conditions at the site are depicted on the Regional Geologic Map, Figure 3. The geologic conditions encountered in the explorations are described in detail below.

3.1 Granitic Rock

The published geologic maps for San Diego County indicate that the site is underlain at depth by Granitic Rock (Map Symbol – Kgr). The granite is described as Tonalite, or quartz diorite. As observed in the supplemental borings, as well as our test pit excavations for Prospect Estates I, the upper portion of the Tonalite has been completely weathered into silty and clayey sand (Unified Soil Classification Symbols – SM and SC). The weathered granite is underlain by fresh granitic rock.

Corrected SPT blow counts (N_{60}) collected within the Granitic Rock ranged from 14 to 100 or more. In general, the intensely weathered upper portion of the Granitic Rock had corrected SPT blow counts ranging from 14 to 38, and averaging 22, which indicates a medium dense condition for a sand. By comparison, the mildly weathered Granitic Rock at depths greater than 10 feet below grade typically had SPT blow counts over 100, indicating a very dense condition.

Our previous laboratory tests indicate that the silty sand (SM) generated by excavations into the weathered granite (SM) typically has a very low expansion potential, with Expansion Indices ranging from 0 to 6. By comparison, the clayey sand (SC) generated from the weathered granite is moderately expansive, with an Expansion Index of 51 in the one sample we tested (see Figure B-2 in Appendix B). According to the referenced report, the weathered granite at the site may have a low to medium expansion potential, with Expansion Indices ranging from 37 to 55 (GEI, 2016). Soils generated by excavations into the weathered granite may be suitable for use in the relatively low expansion (EI<70) soil cap recommended throughout the site.

3.2 Friars Formation

The Eocene-age Friars Formation (Map Symbol - Tf) is believed to overlie the Granitic Rock in the southern portion of the site, as shown on the Regional Geologic Map, Figure 3. The Friars Formation was encountered in all six of the previous exploratory borings conducted in the southern portion of the site (GEI, 2016). However, Friars Formation was not encountered in our five supplemental borings located in the northern portion of the site.

As shown in the boring logs, and observed in our previous test pit excavations for the Prospect Estates I development to the east, the Friars Formation typically consists of sandy lean claystone (CL) and fat claystone (CH), with lesser amounts of clayey sandstone (SC). The formation is light olive to yellow brown in color, and hard. One Atterberg Limit test was conducted on a sample of the Friars Formation as part of the referenced study (GEI, 2016). This test indicated a high plasticity with a Liquid Limit of 54, and a Plasticity Index of 29. Three Expansion Index (EI) tests were also previously conducted on samples of the Friars Formation. These tests indicated a *high* expansion potential, with Expansion Indices ranging from 93 to 129, as shown in Figure B-2.

3.3 Alluvium

Young alluvium (Map Symbol - Qya) covers the Granitic Rock in the northern portion of the site, and the Friars Formation in the southern portion of the site. In our five supplemental exploratory borings, the alluvium generally ranged in thickness from 3 to 6½ feet. However, the alluvium extended down to a maximum depth of 15 feet below grade in the previous Boring B-6 (GEI, 2016). Note that this unit was described as “slopewash” in the GEI report. The alluvium is considered to be both potentially compressible and highly expansive, and should not be used for the direct support of the new building foundations and slabs, or heave sensitive concrete sidewalks and driveways.

The surficial alluvium in the five supplemental borings we conducted in the northern portion of the site generally classified as sandy fat clay (CH), which graded to sandy lean clay (CL) with increased depth. The previous boring logs prepared by GEI in the southern portion of the site indicate that the alluvium in that area generally consisted of sandy lean clay (CL) and clayey sand (SC). The alluvium was typically dark brown to dark gray in color, and very stiff to hard in consistency.

A total of nine Expansion Index tests have been conducted on samples of the alluvium collected from the Prospect Estates I and II developments, as summarized in Figure B-2 in Appendix B. The tests indicate that the fat clay (CH) alluvium is *highly* to *very highly* expansive, with an Expansion Index ranging from 120 to 149, and averaging 134. The two lean clay (CL) alluvium samples that were tested both had Expansion Indices of 91, which indicates a *high* expansion potential. The tests also indicate that the clayey sand (SC) at the site has a *low* to *medium* expansion potential, with an Expansion Index ranging from 51 to 64, and averaging 56.

3.4 Fill

Available aerial photographs indicate that undocumented fill was placed throughout the southern portion of the site in the spring of 2003 in order to create level areas for baseball fields. Up to seven feet of undocumented fill was encountered in the previous exploratory borings in that area (GEI, 2016). Minor pockets of undocumented fill were also observed in the northern portion of the site. As shown on the boring logs, the undocumented fill generally consists of silty or clayey sand (SM or SC) with roughly 29 to 44 percent fines. Several feet of poorly graded gravel (GP) was also encountered within the fill in previous Boring B-6.

The boring logs from the GEI report indicate that the fill soils are medium dense, although no blow counts were taken within the fill to justify this assumption (the blow counts in Boring B-5 were inflated by the presence of a large rock in the sampler). The fill is considered to be potentially compressible and unsuitable for support of new fill or foundation loads. Expansion Index tests in the sandy fill varied from 59 to 61, which indicates a “medium” expansion potential.

3.5 Groundwater

No groundwater was encountered in any of the borings completed at the site. However, it should be pointed out that the borings only extended to a maximum depth of 17½ feet below site grades. Consequently, the borings may not have been deep enough to encounter the regional groundwater table. Groundwater levels may also fluctuate over time due to changes in the water surface elevation and flow rate within the San Diego River to the north, as well as variations in rainfall, irrigation or site drainage conditions.

4.0 GEOLOGIC HAZARDS

The subject 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 the recent geotechnical investigation or in our literature review. However, two landslides are mapped within the Friars Formation immediately south of the site, as shown in Figure 3. We anticipate that the main geologic hazards at the site will be associated with the potential for strong ground shaking due to a seismic event on a distant active fault. Each of the geologic hazards is described below.

4.1 Ground Rupture

Ground rupture is the result of movement on an active fault reaching the ground surface. The locations of known active faults within a 100 km radius of the site are shown on the Fault Location Map, Figure 4. The nearest known active fault is located within the Rose Canyon fault zone, about 18 km southwest of the site. The site is not located within an Alquist-Priolo Earthquake Fault Zone. No indications of active faulting were found in our site reconnaissance or literature review. Consequently, ground rupture is not considered to be a substantial geologic hazard at the site.

4.2 Seismicity

The centroid of the site is roughly located at latitude 32.8330° north and longitude 117.0098° west. 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 using an average shear wave velocity of 365 m/s for 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 are approximately 0.38, 0.28g and 0.21g, respectively. These three risk levels are often referred to as the Maximum Considered (MCE), Upper Bound (UBE) and Design Basis Earthquakes (DBE), respectively. By comparison, the Design and MCE level peak ground accelerations from the 2016 California Building Code (CBC) are 0.25g and 0.36g, respectively, as shown in Table 1.

4.3 Liquefaction and Dynamic Settlement

Liquefaction involves the sudden loss in strength of a saturated, cohesionless soil (sand and non-plastic silts) caused by the build-up of pore water pressure during cyclic loading, such as that produced by an earthquake. This increase in pore water pressure can temporarily transform the soil into a fluid mass, resulting in sand boils, settlement and lateral ground deformations. Typically, liquefaction occurs in areas where there are loose to medium dense sands and silts, and where the depth to groundwater is less than 50 feet from the ground surface. In summary, three simultaneous conditions are required for liquefaction:

- Historic high groundwater within 50 feet of the ground surface
- Liquefiable soils such as loose to medium dense sands
- Strong shaking, such as that caused by an earthquake

Groundwater was not encountered in any of the borings conducted at the site. Furthermore, we have recommended that all of the compressible soils be excavated and replaced as a compacted fill during site development. Given the absence of shallow groundwater, the clayey nature of the site soils, and the dense nature of the underlying Friars Formation and Granitic Rock, the potential for liquefaction and dynamic settlement to adversely affect the development is considered to be low.

4.4 Landslides and Lateral Spreads

Evidence of ancient landslides or slope instabilities was not observed during our literature review or reconnaissance. However, two landslides are mapped within the Friars Formation immediately south of the site. Provided that the site is graded in accordance with our recommendations, it is our opinion that slope instability should not adversely affect the planned development. Earthwork excavations should be observed by the geotechnical consultant during grading.

4.5 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 340 feet), the potential for damage due to tsunamis or seiches is considered to be remote. The site is not located within a FEMA 100-year flood zone, as shown in Figure 5. Based on our previous experience with the Prospect Estates I development immediately east of the site, we understand that the areas along the northern of the site (below an elevation of 339 feet), may be subjected to inundation along the San Diego River associated with the failure of either the San Vicente Dam or El Capitan Dam to the east (GDC, 2016a). However, we understand that the finish grades for the subject site have an elevation of 340 feet or higher. Consequently, the potential for flooding at the site is considered to be low.

5.0 CONCLUSIONS

Site development appears to be feasible from a geotechnical perspective. However, there are several geotechnical constraints which will need to be addressed prior to development.

- The site is underlain by up to about 15 feet of undocumented fill and alluvium. These soils are considered to be compressible, and unsuitable for the direct support of the new buildings and improvements. All undocumented fill and alluvium throughout the site should be excavated and replaced as compacted fill prior to site development.

- Highly expansive clays were encountered at the site. The detrimental effects of expansive soil heave may be mitigated by blending lower expansion soils with the higher expansive soils, lime stabilization of the expansive clay, or a combination of these methods. The intent would be to cap the building and improvement areas throughout the site with at least three feet of low to medium expansion potential material (EI<70). This subgrade preparation should be combined with structurally robust post-tensioned slab foundations.

- Post-tensioned slab foundations are suitable for support of the planned residential structures. However, additional laboratory testing and geotechnical analyses will be needed in order to develop geotechnical parameters for use in post-tension slab design that reflect the actual as-graded soil conditions. The preliminary post-tension slab design parameters provided in this update report should be verified or revised once the site is fine graded.

- The soil resistivity test results indicate that the site soils are corrosive to metals. The sulfate content testing indicates a negligible potential for sulfate attack of concrete. Soil corrosivity should be further evaluated during fine grading of the site.

- The development includes a bio-retention basin in the northwest corner of the site that is intended to promote on site infiltration of storm water runoff. The potential for full or partial infiltration has been assessed in accordance with the City of Santee BMP Design Manual dated February 2016. A feasibility screening of the potential for on-site infiltration is presented as Worksheet C.4-1 in Appendix C. The on-site soils are not considered suitable for either full or partial infiltration.

- There are no known active faults located beneath the subject site, and the potential for ground rupture to adversely impact the development is remote. Other geologic hazards that may impact site development are primarily associated with the potential for strong ground shaking from an earthquake on the Rose Canyon fault zone. The shaking hazard may be mitigated by structural design in accordance with the applicable building code.

6.0 RECOMMENDATIONS

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

6.1 Plan Review

We recommend that grading, foundation and improvement plans be reviewed by Group Delta prior to construction. Substantial changes in the development may occur from the design concepts used for this update. Such changes may require additional evaluation, which may result in modification of the remedial grading recommendations provided in this report.

6.2 Excavation and Grading Observation

Foundation and grading excavations should be observed by Group Delta Consultants. During grading, Group Delta Consultants should provide observation and testing services continuously. Such observations are considered essential to identify field conditions that differ from those anticipated by this investigation, to adjust designs to the actual field conditions, and to determine that the remedial grading is accomplished in general accordance with the recommendations presented in this report. Our recommendations are contingent upon Group Delta Consultants providing these services. Our personnel should perform sufficient testing of fill and backfill during grading and improvement operations to support our professional opinion as to compliance with the compaction recommendations.

6.3 Earthwork

Grading and earthwork should be conducted in general accordance with 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 construction.

6.3.1 Site Preparation

General site preparation should begin with the removal of deleterious materials from throughout the site. Deleterious materials include existing pavements, foundations, slabs-on-grade, and other demolition debris, as well as vegetation, trees, trash and contaminated soil. Existing subsurface utilities that will be abandoned should be removed and the excavations backfilled and compacted as described below. Alternatively, abandoned pipes may be grouted with a two-sack sand-cement slurry under the observation of Group Delta Consultants.

6.3.2 Compressible Soils

The undocumented fill and alluvium throughout the site is considered to be compressible and should be completely excavated and replaced as a uniformly compacted fill in all areas that will be developed. Removals should expose competent formational material as determined in the field by our personnel during grading. In general, alluvium and fill removals are anticipated to be on the order of 4 to 6 feet deep, although removals of 10 to 15 feet or more will be needed in some portions of the site (see GEI Borings B-5 and B-6). The removed soil that is free of deleterious material may be replaced as a uniformly compacted fill to the proposed plan elevations. It should be noted that complete removal of the compressible soils may be difficult to accomplish along the property boundaries without extending the remedial grading off-site.

6.3.3 Expansive Soils

We recommend mitigating expansive soil heave by selectively grading the site so that soils with relatively low potential for expansion are used within the upper three feet of subgrade below all single family residential buildings that will be supported with post-tensioned slab foundations, as well as the surrounding concrete sidewalks and driveways. For preliminary design, we recommend targeting an Expansion Index of 70 or less (EI<70). The soil used for the three-foot cap could be derived from cut excavations within the on-site sands, including the weathered granite and sandy portions of the fill and alluvium, or by using imported sand or lime stabilized on-site clay. This process combined with post-tensioned slab foundation design can accommodate an increased potential expansion since the design will use the specific as-graded expansion profile. The current Post-Tensioning Institute design method estimates differential swell based on comprehensive laboratory testing of soil samples obtained from the as-graded subgrade. The design method includes an evaluation of the potential expansion within the upper nine feet of the soil profile.

6.3.4 Building Areas

Residential structures should not straddle cut/fill transitions, due to the potential for adverse differential settlement. Typical transition conditions are depicted in Figure 6. These conditions include lots with cut/fill transitions, as well as transitions between shallow and deep fills. Our recommended remediation measures are also shown in Figure 6.

For both cut/fill and deep fill transition conditions, we recommend that remedial earthwork consist of excavating the formational materials beneath the building pad, and replacing them as uniformly compacted fill. The minimum depth of the recommended over-excavation should be equal to a H/2, where "H" is equal to the greatest depth of fill underlying the proposed structure. The depth of the over-excavation should not be less than 3 feet, and does not need to extend deeper than 10 feet below pad grades. Note that the over-excavation should extend at least 10 feet horizontally beyond the proposed building envelopes. The over-excavated building pads should be brought back to plan grade with compacted fill prepared as recommended in Section 6.3.5. The upper 3 feet of soil should consist of relatively low expansion material (EI<70), as discussed in Section 6.3.3.

6.3.5 Fill Compaction

All fill and backfill should be placed at slightly above optimum moisture content using equipment that is capable of producing a uniformly compacted product. The minimum recommended relative compaction is 90 percent of the maximum dry density based on ASTM D1557. Sufficient observation and testing should be performed by Group Delta Consultants so that an opinion can be rendered as to the compaction achieved. Rocks or concrete fragments greater than 6 inches in dimension should not be used in structural fill.

Imported fill sources should be observed prior to hauling onto the site to determine the suitability for use. In general, imported fill materials should consist of granular soil with 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 the proposed import should be tested by Group Delta Consultants in order to evaluate the suitability of these soils for their proposed use

During grading operations, soil types may be encountered by the contractor that do not appear to conform to those discussed within this report. Group Delta Consultants should be notified to evaluate the suitability of these soils for their proposed use.

6.3.6 Surface Drainage

Slope, foundation and slab performance depends greatly on how well surface runoff drains from the site. This is true both during construction and over the entire life of the structure. The ground surface around structures should be graded so that water flows rapidly away from the structures and tops of slopes without ponding. The surface gradient needed to achieve this may depend on the prevailing landscape.

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. Should excessive irrigation, surface water intrusion, water line breaks, or unusually high rainfall occur, saturated zones or “perched” groundwater may develop within the soil.

6.3.7 Slope Stability

A fine grading plan has not yet been developed for the property. We anticipate that various cut or fill slopes may be needed for the new development. We recommend that permanent cut and fill slopes be inclined no steeper than 2:1 (horizontal to vertical). Fills over sloping ground should be constructed entirely on prepared bedrock. In areas where the ground surface slopes at more than a 5:1 gradient, it should be benched 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.

In general, all slopes are subject to some creep, whether the slopes are natural or man-made. Slope creep is the very slow, down-slope movement of the near surface soil along the slope face. The degree and depth of the movement is influenced by soil type and the moisture conditions. This movement is typical in slopes and is not considered a hazard. However, it may affect structures built on or near the slope face. We recommend that settlement-sensitive structures not be located within 5 feet of the top of the slopes without specific evaluation by Group Delta Consultants.

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. We recommend that slopes be planted with vegetation that will increase their stability. Ice plant is generally not recommended. We recommend that vegetation include woody plants, along with ground cover. All plants should be adapted for growth in semi-arid climates with little or no irrigation. A landscape architect should be consulted in order to develop a specific planting palate suitable for slope stabilization.

6.3.8 Excavation Characteristics

All the geotechnical borings were drilled to a depth of 15 feet using a 6-inch diameter hollow stem auger on a truck mounted rotary drill rig or a track mounted limited access rig. Excavations are not expected to exceed this depth.

6.3.9 Temporary Excavations

Temporary excavations are anticipated throughout the site, such as for the removal of the existing deleterious materials, trenches for the proposed utilities, and remedial grading in building pad areas. All excavations should conform to Cal-OSHA guidelines. Temporary slopes at the site should be inclined no steeper than 1:1 (horizontal to vertical) for heights up to 20 feet. Higher temporary slopes should be evaluated by Group Delta on a case by case basis during grading operations. Temporary excavations that encounter seepage or other potentially adverse conditions should also be evaluated by the geotechnical consultant on a case-by-case basis during grading. Remedial measures may include dewatering, shoring or flattening the temporary slope.

6.4 Preliminary Foundation Recommendations

The design of the foundation system should be performed by the structural engineer, and should incorporate the geotechnical parameters provided in the as-graded geotechnical report prepared after site grading is completed. The design of foundations will be controlled by the expansion potential of the near surface soils. Because of the selective grading we have recommended, we anticipate that soils having an Expansion Index of no greater than 70 (EI<70) will be present in the upper three feet of the foundation influence zone for these structures. Based on the anticipated soil conditions, and the expected magnitude of the new structural loads, we anticipate that the lightly loaded residential structures at the site may be supported by post-tensioned slab foundations. Preliminary post-tension slab foundation design parameters are provided below.

6.4.1 Post-Tension Slab Foundations

Provided that remedial grading is conducted per our recommendations, the residential lots at the site should be underlain by three or more feet of sandy compacted fill (EI<70) over highly to very highly expansive clay. The following preliminary post-tension slab foundation design parameters are considered applicable to buildings that will be underlain by such conditions. Note that these recommendations should be considered preliminary, and subject to revision based on the conditions observed by Group Delta Consultants during grading of the site. The final foundation design parameters should be provided in the as-graded geotechnical report.

Moisture Variation, em:	*Center Lift:*	*7.9 feet*
	Edge Lift:	*4.1 feet*
Differential Swell, ym:	*Center Lift:*	*1.8 inches*
	Edge Lift:	*3.0 inches*
Allowable Bearing:	*2,000 psf at slab subgrade*	

6.4.2 Settlement

Provided that remedial grading is conducted as recommended, total and differential settlement of the proposed structures is generally not expected to exceed one inch and ¾-inch in 40 feet, respectively. The potential for settlement should be better defined in the as-graded geotechnical report prepared after the site is fine graded.

6.4.3 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 0.30 and a passive pressure of 250 psf per foot of depth may be used.

6.4.4 Slope Setback

As a minimum, all foundations should be setback from any descending slope at least 8 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. Note that the outer few feet of all slopes are susceptible to gradual down-slope movements due to slope creep. This will affect hardscape such as concrete slabs. We recommend that settlement sensitive structures not be constructed within 5 feet of the slope top without specific review by Group Delta.

6.4.5 Seismic Design

Structures should be designed in general accordance with the applicable seismic provisions of the 2016 California Building Code (CBC). Based on our current understanding of the site conditions, it is our opinion that a 2016 CBC Site Class C may be assumed for the entire site. The USGS mapped spectral ordinates S_S and S_1 equal 0.874 and 0.340, respectively. For a Site Class C, the acceleration and velocity coefficients F_a and F_v equal 1.050 and 1.460, respectively, and the spectral design parameters S_{DS} and S_{D1} equal 0.612 and 0.331, respectively. The MCE spectral parameters S_{MS} and S_{M1} equal 0.918 and 0.496, respectively. The peak ground acceleration (PGA) from the 2016 CBC Design Spectrum for Site Class C may be taken as 40 percent of S_{DS} or 0.245g. The 2016 CBC Design and MCE Acceleration Response Spectra for Site Class C are shown in Table 1.

6.5 On-Grade Slabs

Building slabs-on-grade should be at least 5½-inches thick. The actual slab thickness, control joints, and reinforcement should be designed by the post-tension structural engineer and should conform to the requirements of the current CBC. The on-site soils are anticipated to be predominately clayey with a high to very high expansion potential. Expansive clays have the potential to swell or shrink in response to changes in moisture. These volume changes can result in damage to slabs and hardscape. In order to reduce the potential for damage associated with soil expansion, we have recommended that at least three feet of low to medium expansion soils (EI<70) be placed directly beneath all heave sensitive concrete slabs on-grade, including buildings, sidewalks and driveways. Post-tension slab foundations are also recommended to further reduce the damage potential.

6.5.1 Moisture Protection for 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 most common moisture barriers in southern California consist of two inches of clean sand covered by 'visqueen' plastic sheeting. Two inches of sand are placed over the plastic to decrease concrete curing problems. It has been our experience that such systems will transmit approximately 6 to 12 pounds of moisture per 1000 square feet per day. The architect should

review the estimated moisture transmission rates, since these values may be excessive for some applications, such as sheet vinyl, wood flooring, vinyl tiles, or carpeting with impermeable backings that use water soluble adhesives. Sheet vinyl may develop discoloration or adhesive degradation due to excessive moisture. Wood flooring may swell and dome if exposed to excessive moisture. 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 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 often placed over 4 inches of granular material. The materials should be 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. The sand should be proof-rolled prior to placing the vapor membrane.

Based on current ACI recommendations, concrete 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 prior to concrete 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.

6.5.2 Exterior Slabs

The near surface soils observed during our field investigation primarily consisted of lean and fat clay (CL and CH) with a *high* to *very high* expansion potential. The Expansion Index (EI) test results are shown in Figure B-2 in Appendix B. Exterior slabs and sidewalks 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.

It should be noted that the exterior slab recommendations assume that the upper three feet of exterior slab subgrade incorporates select soil with an Expansion Index of 70 or less (EI<70), as discussed in Section 6.3.3. Note that even with this select fill cap, some movement of the exterior slabs should be anticipated. One inch of differential movement across the control joints would not be considered unusual for the site conditions, and more may occur (particularly if the exterior slabs were to be constructed directly on the highly expansive on-site clays). The potential for differential movements across the control joints may be reduced by using steel reinforcement. Typical reinforcement for exterior slabs and sidewalks would consist of 6x6 W2.9/W2.9 welded wire fabric placed securely at mid-height of the slab.

6.5.3 Reactive Soils

To assess the sulfate exposure of concrete in contact with the site soils, samples were tested for water-soluble sulfate content, as shown in Figure B-3 in Appendix B. These tests indicate that the on-site soils may have a *negligible* potential for sulfate attack based on common criteria. The sulfate content of the finish grade soils within the building pad areas should be confirmed by the project geotechnical consultant during fine grading.

In order to assess the reactivity of the site soils with buried metals, the pH, resistivity and chloride contents were also determined (see Figure B-3). These tests suggest that the on-site soils are *corrosive* to buried metals. Typical corrosion control measures should be incorporated into design, such as providing minimum clearances between reinforcing steel and soil, or sacrificial anodes (where needed) for buried metal structures. A corrosion consultant may be contacted for specific corrosion control recommendations for the planned site development.

6.6 Earth-Retaining Structures

Backfilling retaining walls with expansive soil can increase lateral pressures well beyond normal active pressures. We recommend that retaining walls be backfilled with soil that has an Expansion Index of 20 or less for a horizontal distance behind the wall that is equal to the height of the wall. The on-site soil generally does not meet this criterion. Imported soil will be needed for wall backfill.

Retaining wall backfill should be compacted to at least 90 percent relative compaction based on ASTM D1557. Backfill should not be placed until the retaining walls have achieved adequate strength. Heavy compaction equipment, which could cause distress to the walls, should not be used. For wall design, an allowable bearing capacity of 2,000 lbs/ft^2, a coefficient of friction of 0.30, and a passive pressure of 250 psf per foot of depth is recommended.

Cantilever retaining walls with level granular non-expansive backfill may be designed using an active earth pressure approximated by an equivalent fluid pressure of 35 lbs/ft^3. The active pressure should be used for walls free to yield at the top at least ½ percent of the wall height. Walls that are restrained so that such movement is not permitted, or walls with 2:1 sloping backfill that are free to yield, should be designed for an earth pressure approximated by an equivalent fluid pressure of 55 lbs/ft^3. These pressures do not include seepage forces or surcharges. All retaining walls should contain backdrains to relieve hydrostatic pressures. Typical retaining wall backdrain details are shown Figure 7.

6.7 Preliminary Pavement Design

Alternatives are provided for asphalt concrete and Portland cement concrete pavements. In each case, the upper 12 inches of pavement subgrade be scarified immediately prior to constructing the pavements, brought to about optimum moisture, and compacted to at least 95 percent of the maximum dry density per ASTM D1557. Aggregate base should also be compacted to 95 percent of the maximum dry density. Aggregate base should conform to the Standard Specifications for Public Works Construction (SSPWC*)*, Section 200-2. Asphalt concrete should conform to Section 400-4 of the *SSPWC* and should be compacted to between 91 and 97 percent relative compaction based on the Maximum Theoretical (or Rice) density.

6.7.1 Asphalt Concrete

Asphalt concrete pavement design was conducted in general accordance with the Caltrans Design Method (Topic 608.4). Two samples of the subgrade soil collected from our supplemental borings were tested for R-Value in general accordance with CT301. The test results are presented in Figures B-5.1 and B-5.2 in Appendix B. Both tests indicated an R-Value of less than 5. For the preliminary pavement sections provided herein, a minimum R-Value of 5 was assumed due to the predominately clayey nature of the on-site site soils. Additional R-Value tests may be conducted on samples of the actual pavement subgrade soil once the site is fine graded.

Traffic Indices of 5.0 through 7.0 were assumed for preliminary design purposes. The project civil engineer should review these Traffic Indices and determine which may apply to the various streets proposed for the development. Based on the minimum R-Value of 5, and the assumed range of Traffic Indices, the following preliminary pavement sections would apply.

PAVEMENT TYPE	TRAFFIC INDEX	ASPHALT SECTION	BASE SECTION
Local Street	5.0	3 Inches	10 Inches
Collector Streets	6.0	4 Inches	12 Inches
Industrial Streets	7.0	4 Inches	16 Inches

6.7.2 Portland Cement Concrete

Concrete pavement design was conducted in general accordance with the simplified design procedure of the Portland Cement Association. 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 “low” support. Based on these assumptions, 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.

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.

6.8 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.

6.8.1 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.

6.8.2 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 the purpose of evaluating deflection due to the load associated with trench backfill over the pipe, a value of 1,500 lbs/in^2 is recommended for the general conditions, assuming granular bedding material is placed around the pipe.

6.8.3 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 disintegrated 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.

6.9 Infiltration Assessment

A bioretention basin is proposed in the northwest corner of the site near the location of Boring B-1, as shown on the Exploration Plan, Figure 2. The surficial soils in this area (and throughout most of the site) generally consist of sandy lean or fat clay (CL or CH). The fines content (percent passing #200 sieve) of these soils range from 67 to 78 percent, averaging 70 percent. The clay content is typically over 50 percent.

Our previous experience with permeability and infiltration testing of fine grained clayey soils indicates that even partial infiltration will not be feasible at the site, as shown on Worksheet C.4-1 in Appendix C. Provide below are two references that correlate fines content by way of USCS Soil

Type. These correlations provide further justification that the site is not suitable for full or partial infiltration.

- Terzaghi and Peck (1967) provide a correlation of permeability to soil type. Their correlations indicate "impervious soils, e.g., homogeneous clay", which best represents the soils at the site, as having "practically impervious" drainage characteristics with an estimated coefficient of permeability of 1 X 10^{-7} to 1 X 10^{-9} inches per hour.
- Hough (1957) as reproduced in Hunt (1986) provides a correlation of permeability to soil type. Their correlation for "clay (30 to 50% clay sizes)", which reasonably represents the soils at the site, estimates a coefficient of permeability of 1 X 10^{-4} inches per hour.

The above correlations are corroborated with the permeability test results for several samples of sandy lean clay (CL) that were previously evaluated by Group Delta Consultants at other sites and are presented in Appendix C. These tests show that the saturated permeability of a typical lean clay is essentially impermeable (see Figures C-1.1 to C-1.3 in Appendix C). Fat clays will have an even lower permeability.

We also note the City of Santee BMP Manual (2016) indicate that soils with relatively high fines content are undesirable for infiltration, as summarized below:

- D.5.2 Site Suitability Considerations for Selection of an Infiltration Factor of Safety: Predominant soil texture/percent fines – soil texture and the percent of fines can influence the potential for clogging. *Finer grained soils may be more susceptible to clogging.*
- Table D.5-1: Suitability Assessment Related Considerations for Infiltration Facility Safety Factors - *Silty and clayey soils with significant fines are a "High Concern"*.

7.0 LIMITATIONS

This report was prepared using the degree of care and skill ordinarily exercised, under similar circumstances, by reputable geotechnical consultants 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.

8.0 REFERENCES

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Kennedy, M. P., and Tan, S. S. (2005). *Geologic Map of the San Diego 30'x60' Quadrangle, California*: California Geologic Survey, Scale 1:100,000.

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Terzaghi, K and Peck B. (1967). *Soil Mechanics in Engineering Practice*, John Wiley & Sons, Second Edition.

TABLES

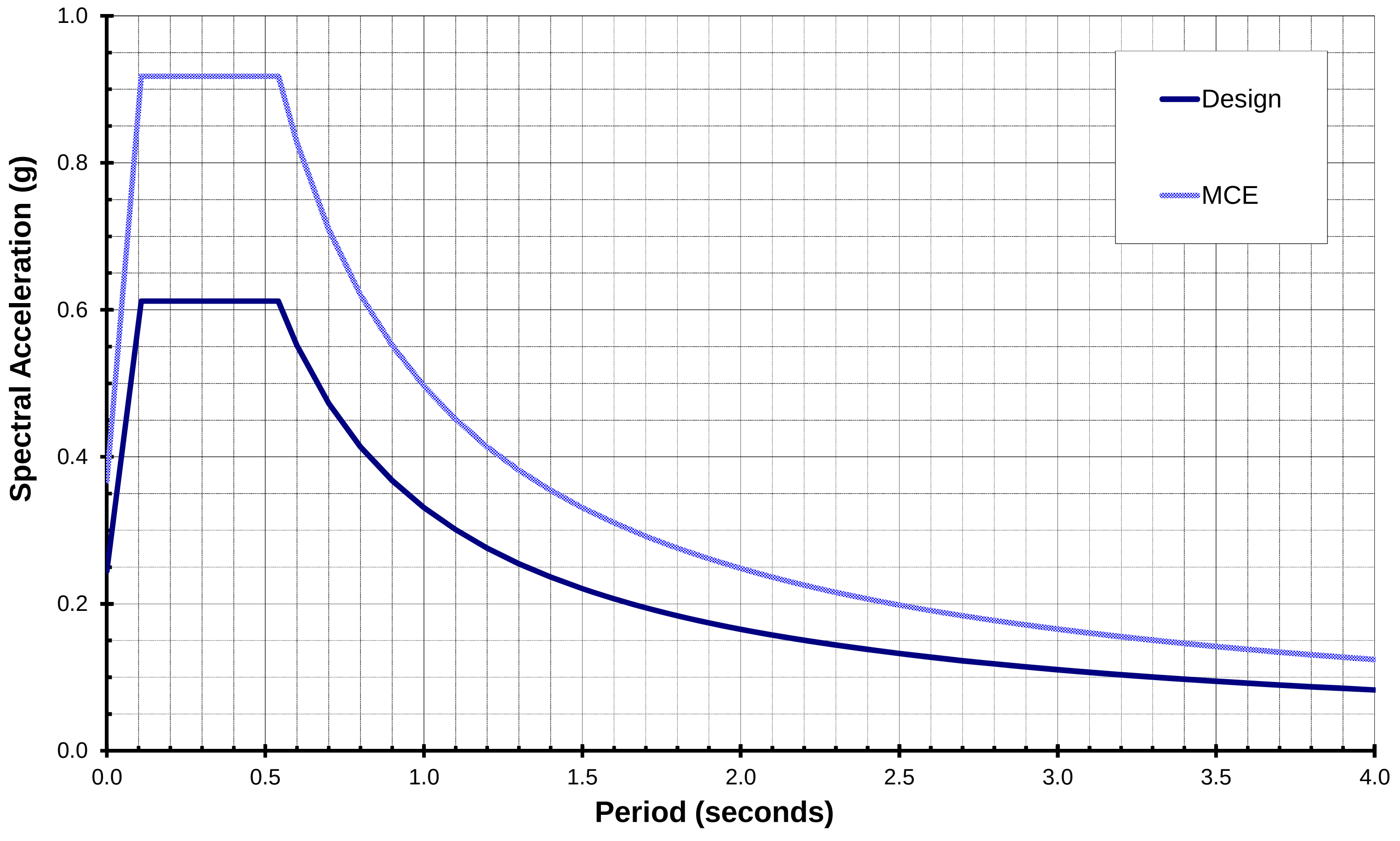
TABLE 1 - 2016 CBC ACCELERATION RESPONSE SPECTRA

INPUT	S_S=	0.874	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=	0.340	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 **CBC 2016 Table 1613.3.2**
	F_a=	1.050	= Site Coefficient applied to S_s to account for soil type (**CBC 2016 Table 1613.3.3(1)**)
	F_v=	1.460	= 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 22-16**)
OUTPUT	S_{MS}=	0.918	= site class modified short period (0.2 sec) MCE spectral response acceleration = F_a x S_s (**CBC 2016 Eqn. 16-37**)
	S_{M1}=	0.496	= site class modified 1.0 sec period MCE spectral response acceleration = F_v x S_1 (**CBC 2016 Eqn. 16-38**)
	S_{DS}=	0.612	= site class modified short period (0.2 sec) Design spectral response acceleration = 2/3 x S_{MS} (**CBC 2016 Eqn. 16-39**)
	S_{D1}=	0.331	= site class modified 1.0 sec period Design spectral response acceleration = 2/3 x S_{M1} (**CBC 2016 Eqn. 16-40**)
	T_0=	0.108	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=	0.541	sec = S_{D1}/S_{DS} = Control Period (right end of peak) for ARS Curve (Section 11.4.5 ASCE 7-10)

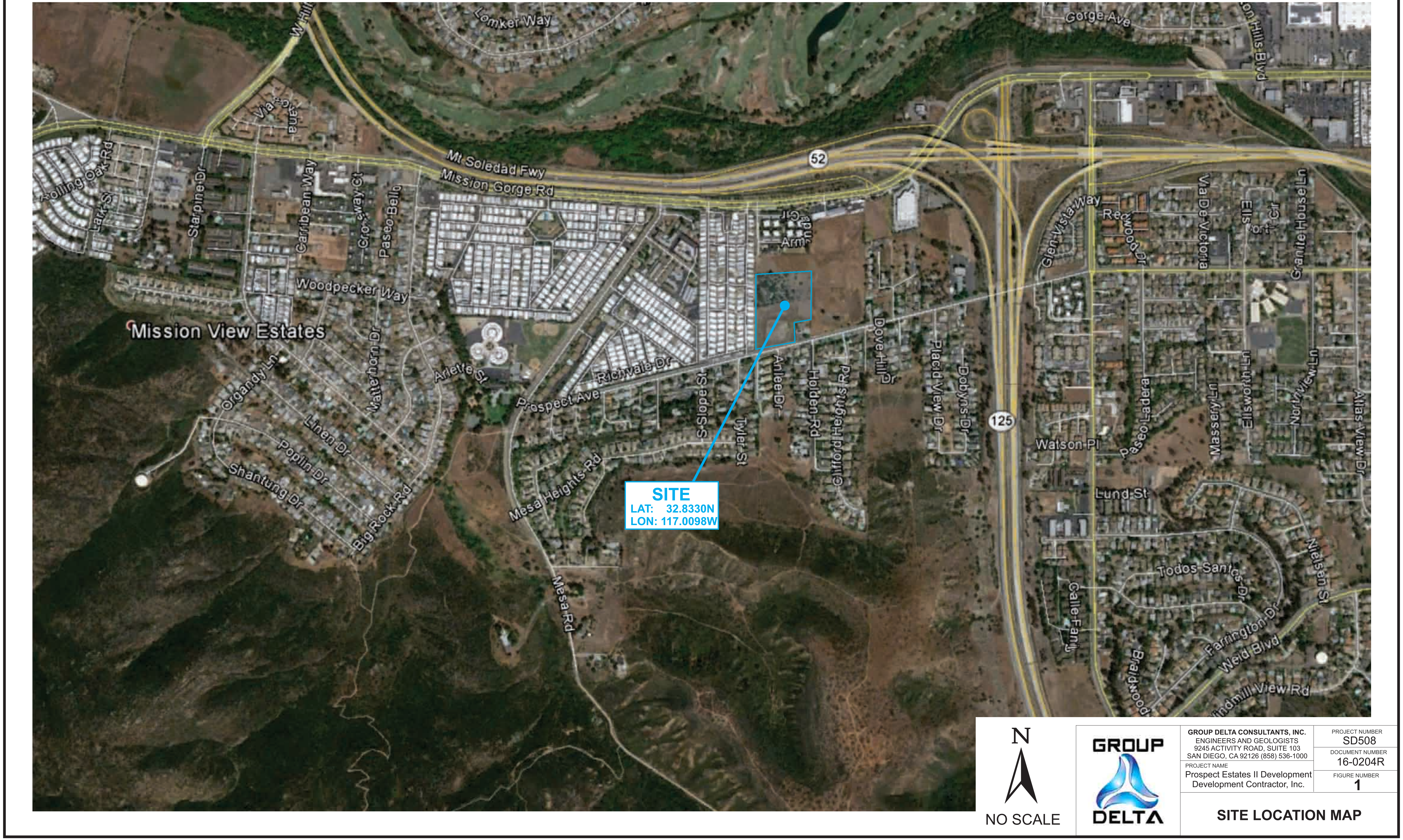
Site Latitude:	32.8330
Site Longitude:	-117.0098
Seismic Design Category:	D

SPECTRUM CALCULATION

T (seconds)	Design Sa (g)	MCE Sa (g)
0.000	0.245	0.367
0.108	0.612	0.918
0.541	0.612	0.918
0.600	0.552	0.827
0.700	0.473	0.709
0.800	0.414	0.621
0.900	0.368	0.552
1.000	0.331	0.496
1.100	0.301	0.451
1.200	0.276	0.414
1.300	0.255	0.382
1.400	0.236	0.355
1.500	0.221	0.331
1.600	0.207	0.310
1.700	0.195	0.292
1.800	0.184	0.276
1.900	0.174	0.261
2.000	0.165	0.248
2.100	0.158	0.236
2.200	0.150	0.226
2.300	0.144	0.216
2.400	0.138	0.207
2.500	0.132	0.199
2.600	0.127	0.191
2.700	0.123	0.184
2.800	0.118	0.177
2.900	0.114	0.171
3.000	0.110	0.165
3.100	0.107	0.160
3.200	0.103	0.155
3.300	0.100	0.150
3.400	0.097	0.146
3.500	0.095	0.142
3.600	0.092	0.138
3.700	0.089	0.134
3.800	0.087	0.131
3.900	0.085	0.127
4.000	0.083	0.124
4.000	0.083	0.124



FIGURES



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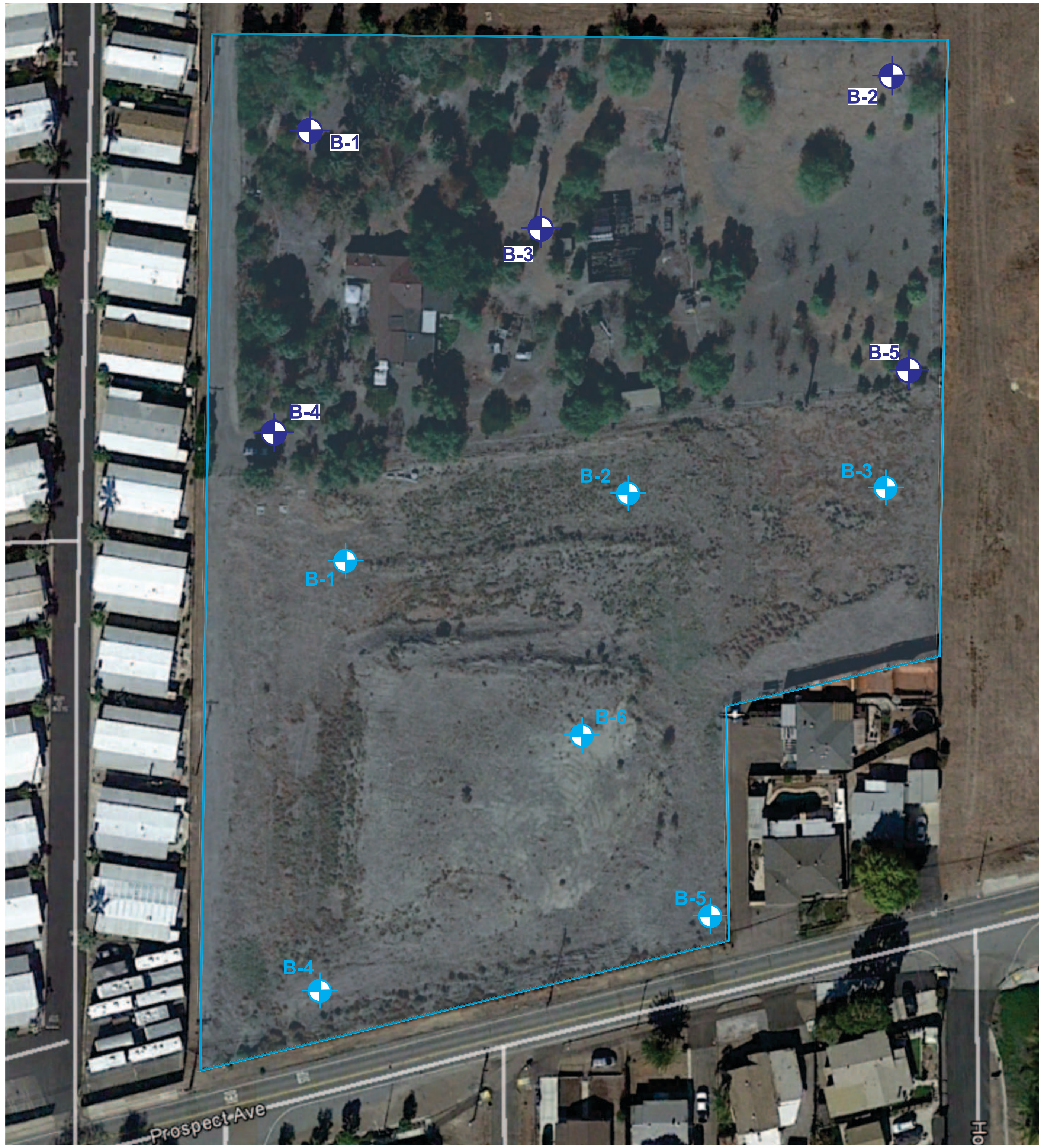
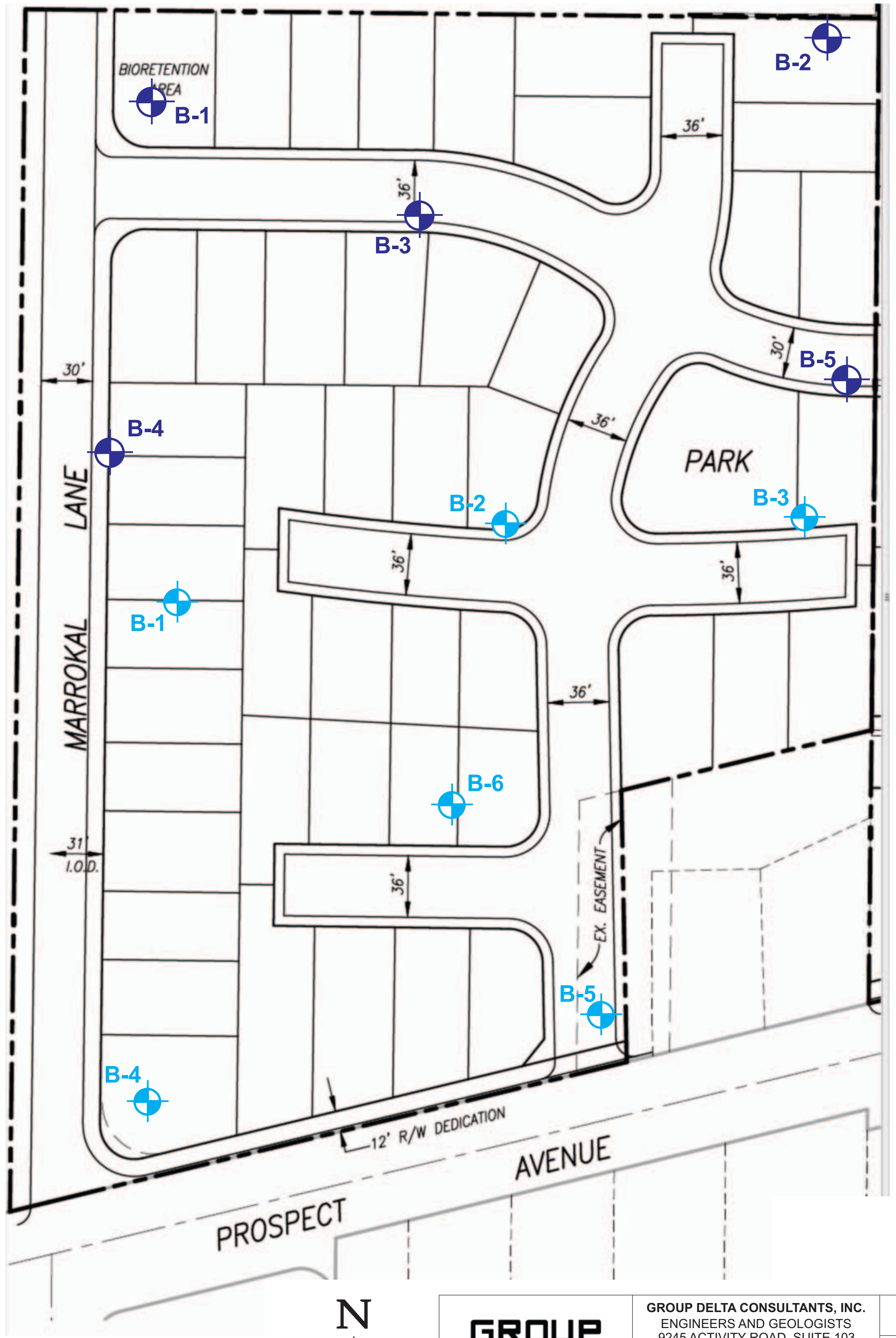
DOCUMENT NUMBER
16-0204R

FIGURE NUMBER
1

SITE LOCATION MAP





EXPLANATION:

B-5 ⊕	Approximate locations of the five supplemental exploratory borings (GDC, 2017).
B-6 ⊕	Approximate locations of the six previous borings by others (GEI, 2016).

REFERENCE: Polaris Development Consultants (2017). Tentative Map for Prospect Estates II + Sheffer Property, 46 Lots + Pak Site, April 22.

N

NO SCALE

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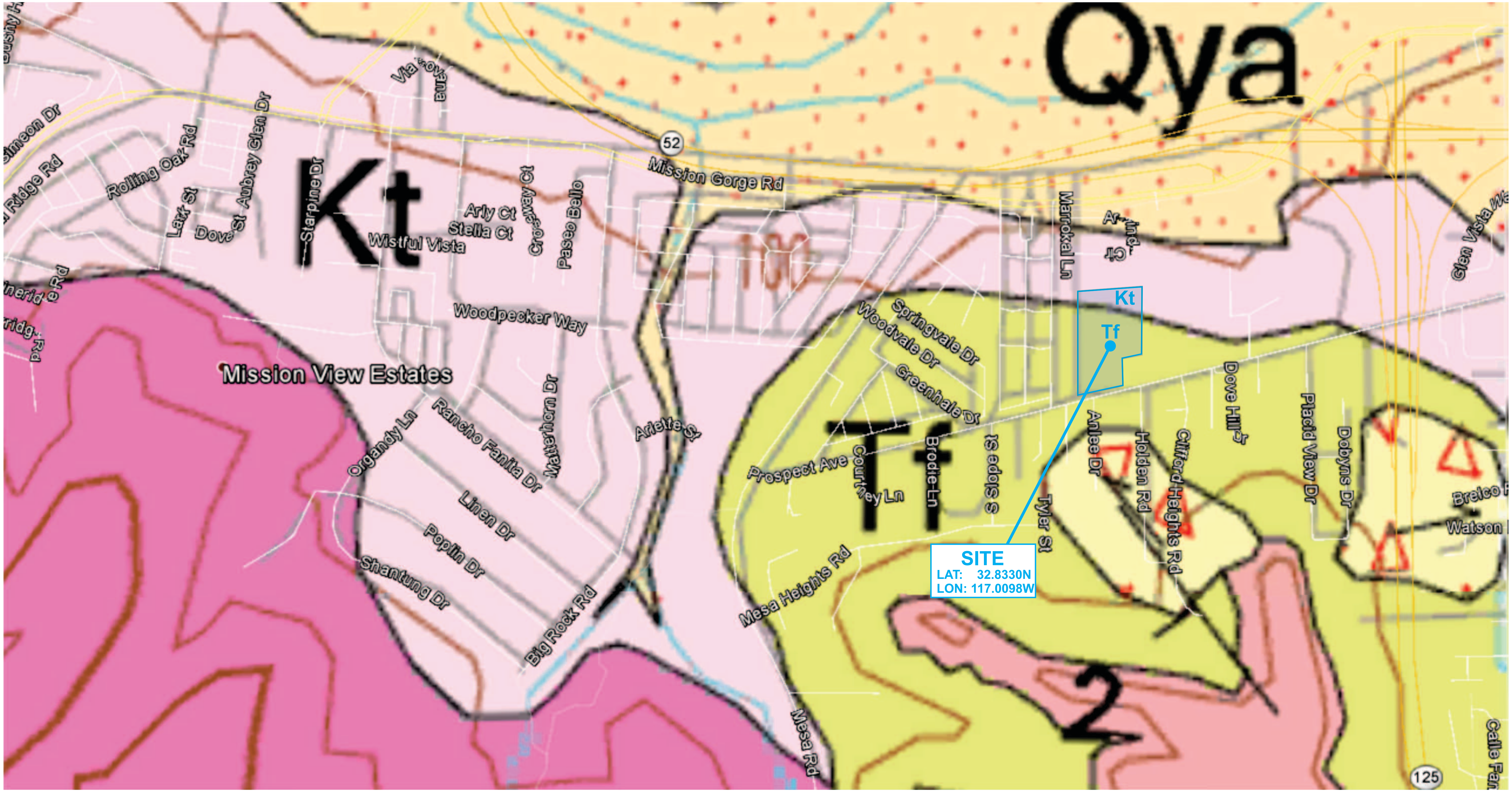
FIGURE NUMBER
2

EXPLORATION PLAN



EXPLANATION:

Kt	Approximate location of weathered granitic rock (tonalite).
Tf	Approximate location of Friars Formation (sandstone and claystone)

REFERENCE: Kennedy & Tan (2005). Geologic Map of the San Diego 30' x 60' Quadrangle, Scale 1:100,000

N

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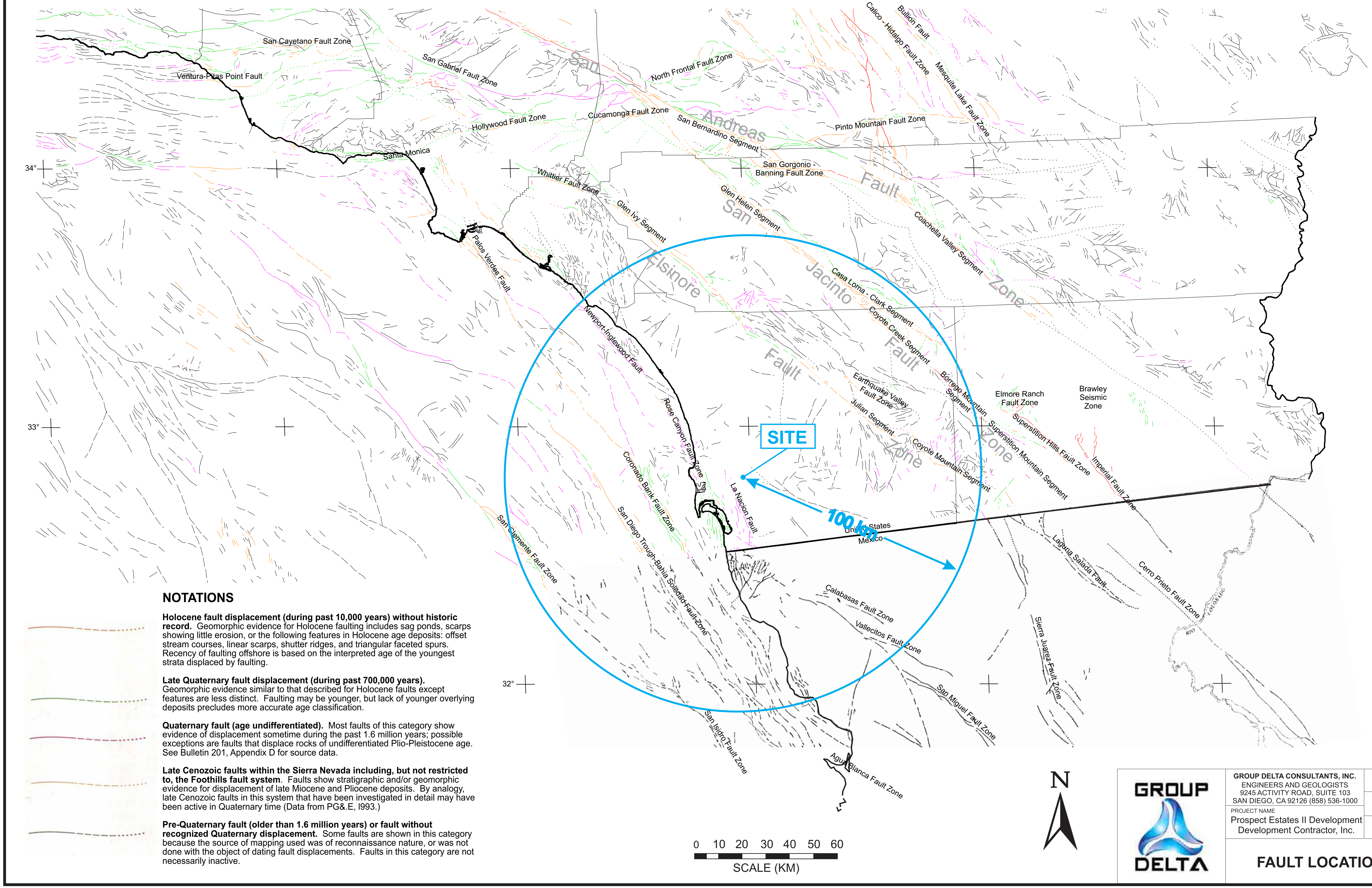
FIGURE NUMBER
3

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.

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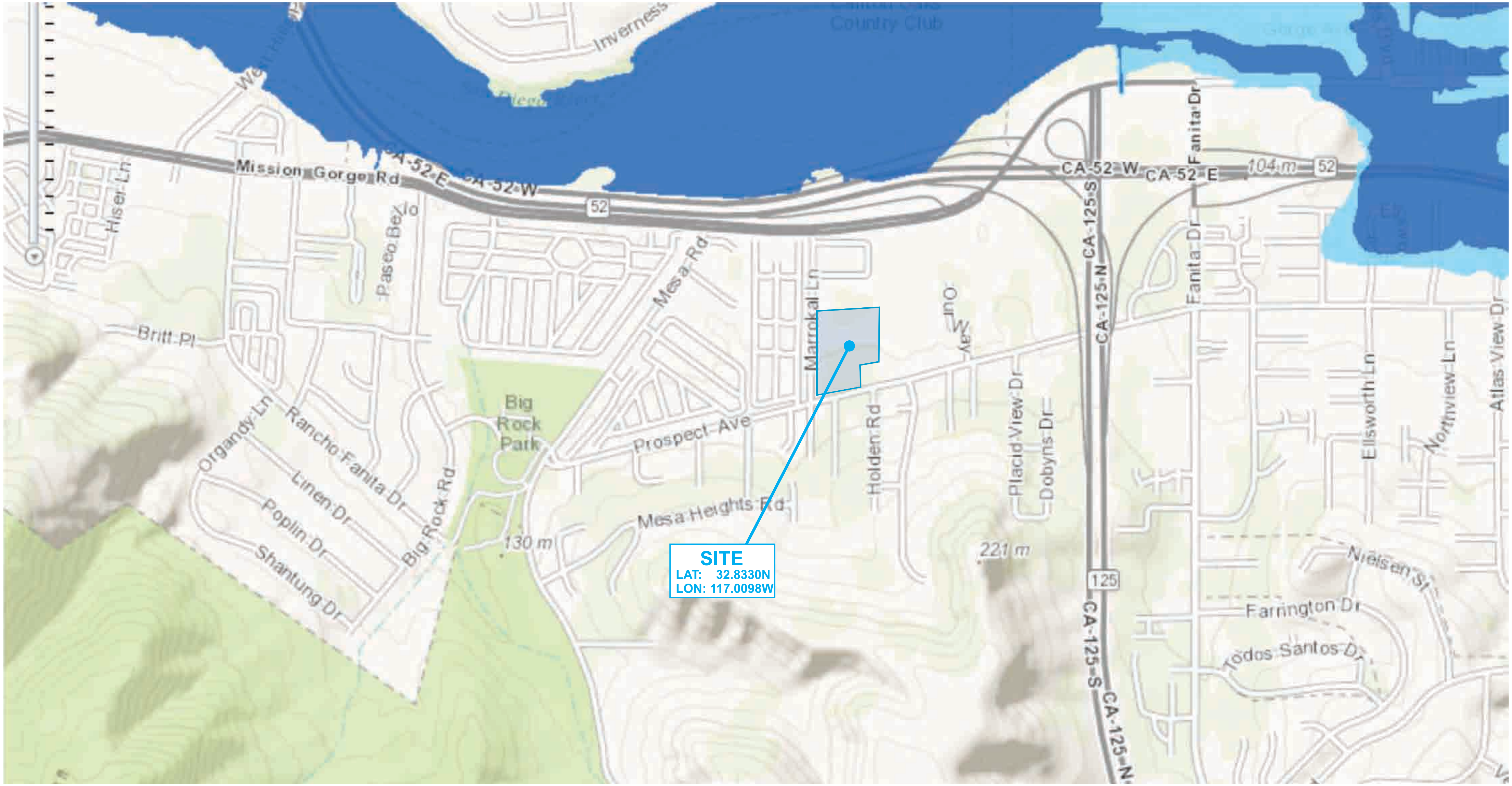
FIGURE NUMBER
4

FAULT LOCATION MAP



EXPLANATION:

🟦 Approximate location of the FEMA 100-year Floodplain.

🟦 Approximate location of the FEMA 500-Year Floodplain.

REFERENCE: California Emergency Management Agency (2015). Hazard Mitigation Web Portal, Fema 100 and 500-year Floodplains.

N

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FIGURE NUMBER
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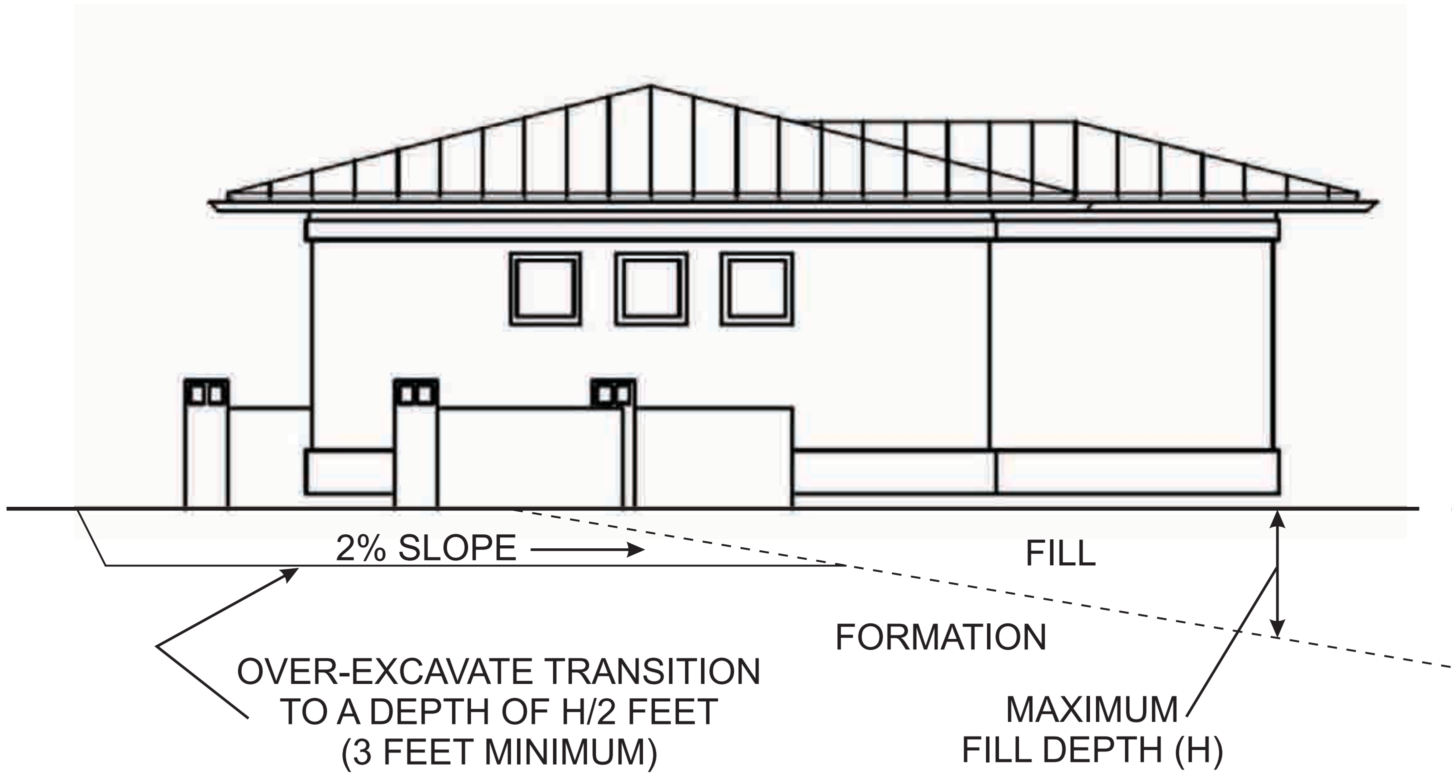
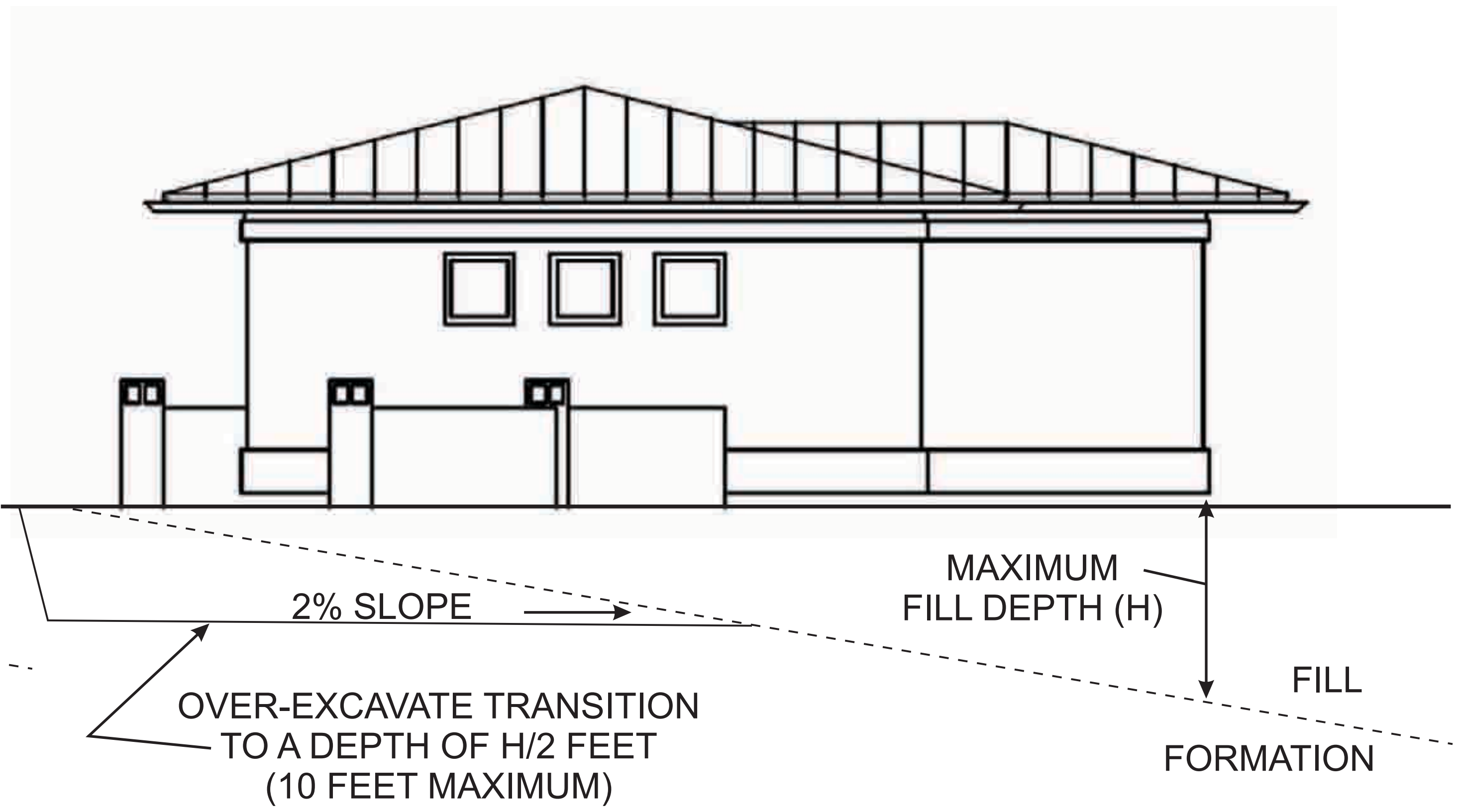
FEMA FLOOD MAP

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 building.

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

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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DOCUMENT NUMBER
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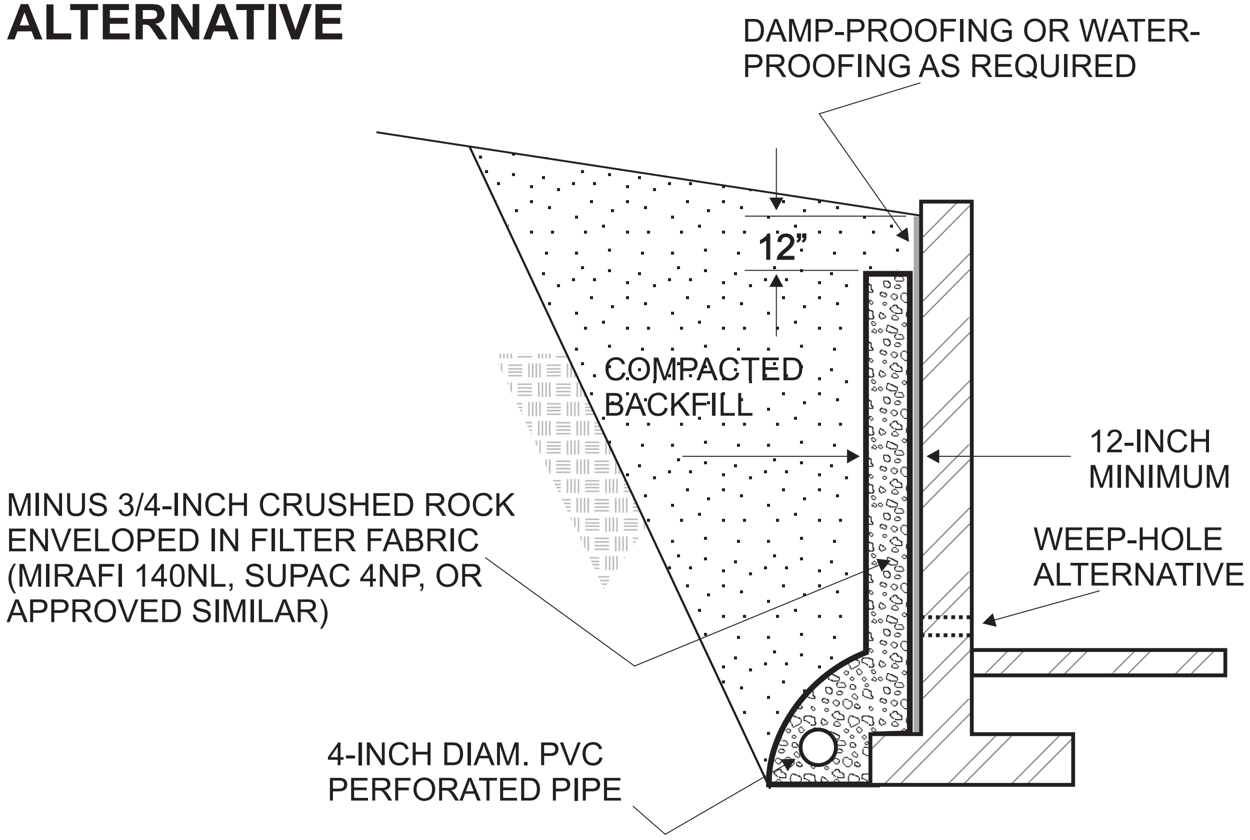
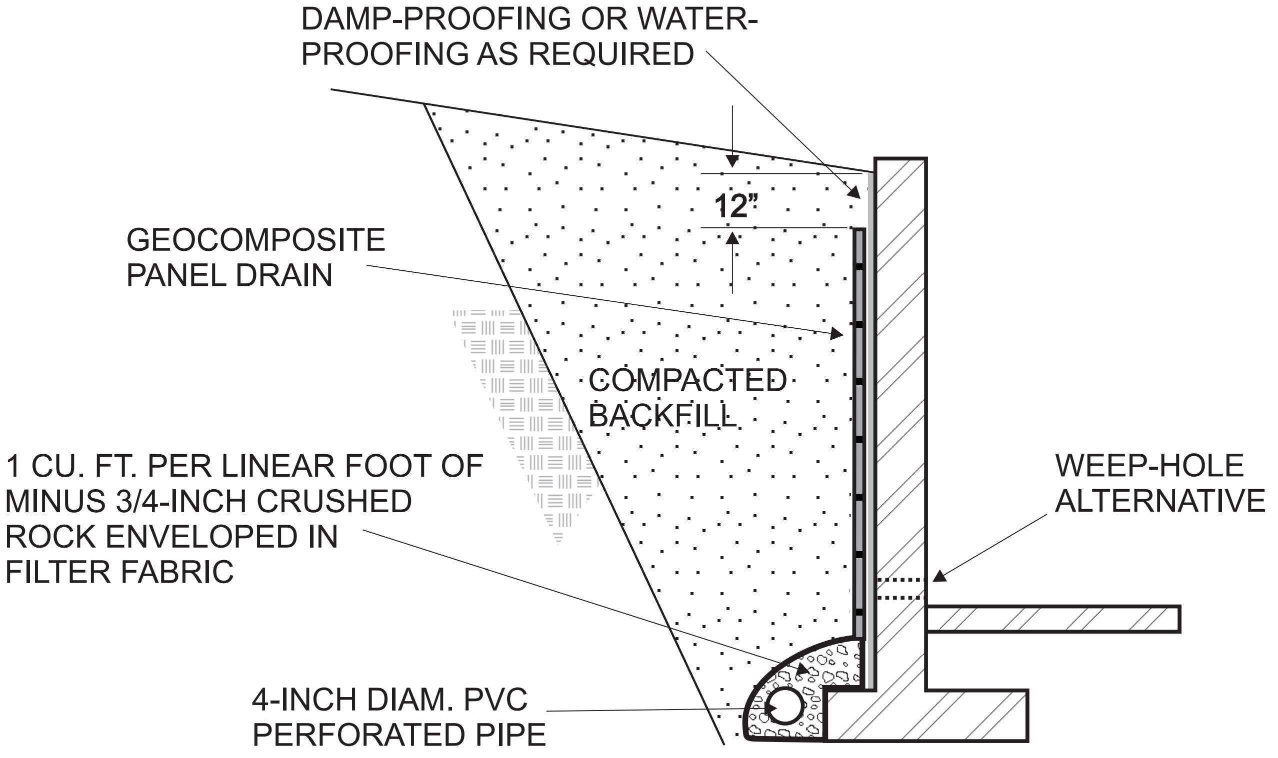
FIGURE NUMBER
6

TYPICAL TRANSITION DETAILS

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
SD508

DOCUMENT NUMBER
16-0204

FIGURE NUMBER
7

WALL DRAIN DETAILS

APPENDIX A
FIELD EXPLORATION

APPENDIX A

FIELD EXPLORATION

Our supplemental subsurface exploration program included a visual and geologic reconnaissance of the site, and the advancement of five exploratory borings on May 16th, 2017. The maximum depth of exploration was about 16 feet below grade. The approximate locations of these five borings are shown on the Exploration Plan, Figure 2. The locations of six borings previously conducted at the site by others are also shown in Figure 2 (GEI, 2016). Logs of our five recent borings are provided in Figures A-1 through A-5, immediately following the Boring Record Legends. Logs of the six borings previously conducted by others are attached as a separate Appendix A (GEI, 2016).

The supplemental exploratory borings were conducted by Pacific Drilling Company using the track mounted Mole drill rig to account for the limited site access. Drive samples were collected from the borings using a Standard Cat-Head with an assumed Energy Transfer Ratio (ETR) of 60 percent. Disturbed samples were collected from the borings 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 where needed to approximate the standard 60 percent ETR, as shown on the logs (N_{60}). Bulk samples were also collected from the borings at selected intervals.

The boring locations were determined by visually estimating, pacing and taping distances from landmarks shown on the Exploration Plan, Figure 2. The locations 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		Required	Optional
		Field	Lab		
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. SD508

Prospect Estates II Update
Development Contractor, Inc.

BORING RECORD LEGEND #1

GROUP SYMBOLS AND NAMES

Graphic / Symbol	Group Names
GW	Well-graded GRAVEL Well-graded GRAVEL 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
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
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
GC	CLAYEY GRAVEL CLAYEY GRAVEL with SAND
GC-GM	SILTY, CLAYEY GRAVEL SILTY, CLAYEY GRAVEL with SAND
SW	Well-graded SAND Well-graded SAND with GRAVEL
SP	Poorly graded SAND Poorly graded SAND with GRAVEL
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)
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)
SM	SILTY SAND SILTY SAND with GRAVEL
SC	CLAYEY SAND CLAYEY SAND with GRAVEL
SC-SM	SILTY, CLAYEY SAND SILTY, CLAYEY SAND with GRAVEL
PT	PEAT
	COBBLES COBBLES and BOULDERS BOULDERS

Graphic / Symbol	Group Names
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
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
ML	SILT SILT with SAND SILT with GRAVEL SANDY SILT SANDY SILT with GRAVEL GRAVELLY SILT GRAVELLY SILT 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
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
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
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
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
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
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

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. SD508

Prospect Estates II Update
Development Contractor, Inc.

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. SD508

Prospect Estates II Update
Development Contractor, Inc.

BORING RECORD LEGEND #3

LEGEND OF ROCK MATERIALS

⊠	IGNEOUS ROCK
☰	SEDIMENTARY ROCK
▨	METAMORPHIC ROCK

BEDDING SPACING

Description	Thickness/Spacing
Massive	Greater than 10 ft
Very Thickly Bedded	3 ft - 10 ft
Thickly Bedded	1 ft - 3 ft
Moderately Bedded	4 in - 1 ft
Thinly Bedded	1 in - 4 in
Very Thinly Bedded	1/4 in - 1 in
Laminated	Less than 1/4 in

WEATHERING DESCRIPTORS FOR INTACT ROCK

	Diagnostic Features					
	Chemical Weathering-Discoloration-Oxidation		Mechanical Weathering and Grain Boundary Conditions	Texture and Leaching		
Description	Body of Rock	Fracture Surfaces		Texture	Leaching	General Characteristics
Fresh	No discoloration, not oxidized	No discoloration or oxidation	No separation, intact (tight)	No change	No leaching	Hammer rings when crystalline rocks are struck.
Slightly Weathered	Discoloration or oxidation is limited to surface of, or short distance from, fractures; some feldspar crystals are dull	Minor to complete discoloration or oxidation of most surfaces	No visible separation, intact (tight)	Preserved	Minor leaching of some soluble minerals	Hammer rings when crystalline rocks are struck. Body of rock not weakened.
Moderately Weathered	Discoloration or oxidation extends from fractures usually throughout; Fe-Mg minerals are "rusty"; feldspar crystals are "cloudy"	All fracture surfaces are discolored or oxidized	Partial separation of boundaries visible	Generally preserved	Soluble minerals may be mostly leached	Hammer does not ring when rock is struck. Body of rock is slightly weakened.
Intensely Weathered	Discoloration or oxidation throughout; all feldspars and Fe-Mg minerals are altered to clay to some extent; or chemical alteration produces in situ disaggregation, grain boundary conditions	All fracture surfaces are discolored or oxidized; surfaces friable	Partial separation, rock is friable; in semi-arid conditions, granitics are disaggregated	Texture altered by chemical disintegration (hydration, argillation)	Leaching of soluble minerals may be complete	Dull sound when struck with hammer; usually can be broken with moderate to heavy manual pressure or by light hammer blow without reference to planes of weakness such as incipient or hairline fractures or veinlets. Rock is significantly weakened.
Decomposed	Discolored of oxidized throughout, but resistant minerals such as quartz may be unaltered; all feldspars and Fe-Mg minerals are completely altered to clay		Complete separation of grain boundaries (disaggregated)	Resembles a soil; partial or complete remnant rock structure may be preserved; leaching of soluble minerals usually complete		Can be granulated by hand. Resistant minerals such as quartz may be present as "stringers" or "dikes".

PERCENT CORE RECOVERY (REC)

$$\frac{\Sigma \text{ Length of the recovered core pieces (in.)}}{\text{Total length of core run (in.)}} \times 100$$

ROCK QUALITY DESIGNATION (RQD)

$$\frac{\Sigma \text{ Length of intact core pieces } \geq 4 \text{ in.}}{\text{Total length of core run (in.)}} \times 100$$

RQD* indicates soundness criteria not met.

ROCK HARDNESS

Description	Criteria
Extremely Hard	Cannot be scratched with a pocketknife or sharp pick. Can only be chipped with repeated heavy hammer blows
Very Hard	Cannot be scratched with a pocketknife or sharp pick. Breaks with repeated heavy hammer blows.
Hard	Can be scratched with a pocketknife or sharp pick with difficulty (heavy pressure). Breaks with heavy hammer blows.
Moderately Hard	Can be scratched with a pocketknife or sharp pick with light or moderate pressure. Breaks with moderate hammer blows
Moderately Soft	Can be grooved 1/16 in. deep with a pocketknife or sharp pick with moderate or heavy pressure. Breaks with light hammer blow or heavy manual pressure.
Soft	Can be grooved or gouged easily with a pocketknife or sharp pick with light pressure, can be scratched with fingernail. Breaks with light to moderate manual pressure.
Very Soft	Can be readily indented, grooved or gouged with fingernail, or carved with a pocketknife. Breaks with light manual pressure.

FRACTURE DENSITY

Description	Observed Fracture Density
Unfractured	No fractures
Very Slightly Fractured	Core lengths greater than 3 ft.
Slightly Fractured	Core lengths mostly from 1 to 3 ft.
Moderately Fractured	Core lengths mostly 4 in. to 1 ft.
Intensely Fractured	Core lengths mostly from 1 to 4 in.
Very Intensely Fractured	Mostly chips and fragments.

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

GROUP DELTA

Project No. SD408

Prospect Estates II Update
Development Contractor, Inc.

BORING RECORD LEGEND #4

BORING RECORD

PROJECT NAME DCI - Prospect Estates II
PROJECT NUMBER SD508
BORING **B-01**
SITE LOCATION Marrokal Lane, Santee, CA
START 5/16/2017
FINISH 5/16/2017
SHEET NO. 1 of 1
DRILLING COMPANY Pacific Drilling Company
DRILLING METHOD Hollow Stem Auger
LOGGED BY TSL
CHECKED BY MAF
DRILLING EQUIPMENT Limited Access Tracked Rig (Mole)
BORING DIA. (in) 6
TOTAL DEPTH (ft) 15.5
GROUND ELEV (ft) 347
DEPTH/*ELEV.* GROUND WATER (ft) ▼ N/A / *na*
SAMPLING METHOD Standard Cat-Head
NOTES ETR ~ 60%, N_{60} ~ 60/60 * N ~ N_{SPT} ~ N_{CAL} / 1.5

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
	345	▨	B-1						PA PI CR EI		▨	**ALLUVIUM (Qal):** SANDY FAT CLAY (CH); dark brown; moist; mostly fines; some SAND; high plasticity. (0% Gravel; 33% Sand; 67% Fines) (LL~63; PL~16; PI~47) Rounded GRAVEL (2" diameter).
		⧗	R-2-2 R-2-1	15 26 40	66	44	24.2	93			▨	SANDY LEAN CLAY (CL) interbedded with CLAYEY SAND (SC). LEAN CLAY (CL); hard; very light green-gray to white; moist; mostly fines; medium plasticity (PP = 4½+). CLAYEY SAND (SC); dense; orange-brown; moist; mostly fine SAND; some fines; low plasticity.
5	340	☒	S-3	15 16 22	38	38				5	░	**GRANITIC ROCK (Kgr):** Decomposed; (SILTY SAND (SM); dense; light orange-brown; moist; mostly fine to medium SAND; some fines; nonplastic).
10	335	☒	S-4-2 S-4-1	22 25 36	61	61	23.4			10	▨	Decomposed; (CLAYEY SAND (SC); very dense; light green-gray to white; moist; mostly fine SAND, some fines; low plasticity. Thin interbedds of SILTY SAND (SM); orange; moist; mostly fine SAND; some fines; nonplastic).
15		☒	S-5	50 (6")	100	100				15	▩	Decomposed; (SILTY SAND (SM); very dense; light orange-brown; moist; mostly fine SAND; some fines; nonplastic).
	330											Bottom of boring at 15½ feet below grade. Groundwater not encountered during drilling. Backfilled 5/18/17.

GROUP DELTA CONSULTANTS, INC.
9245 Activity Road, Suite 103
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.

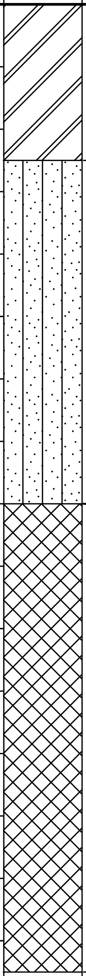
FIGURE

A-1

BORING RECORD

PROJECT NAME DCI - Prospect Estates II
PROJECT NUMBER SD508
BORING **B-02**
SITE LOCATION Marrokal Lane, Santee, CA
START 5/16/2017
FINISH 5/16/2017
SHEET NO. 1 of 1
DRILLING COMPANY Pacific Drilling Company
DRILLING METHOD Hollow Stem Auger
LOGGED BY TSL
CHECKED BY MAF
DRILLING EQUIPMENT Limited Access Tracked Rig (Mole)
BORING DIA. (in) 6
TOTAL DEPTH (ft) 15.5
GROUND ELEV (ft) 344
DEPTH/*ELEV.* GROUND WATER (ft) ⊻ N/A / *na*
SAMPLING METHOD Standard Cat-Head
NOTES ETR ~ 60%, N_{60} ~ 60/60 * N ~ N_{SPT} ~ N_{CAL} / 1.5

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)	DESCRIPTION AND CLASSIFICATION
		▩	B-1						PA PI EI		**ALLUVIUM (Qal):** SANDY FAT CLAY (CH); dark brown; moist; mostly fines; some fine SAND; high plasticity. (1% Gravel; 31% Sand; 68% Fines) (LL~66; PL~22; PI~44)
		⧗	R-2-2 R-2-1	14 9 16	25	17	11.7	106			**GRANITIC ROCK (Kgr):** Decomposed; (SILTY SAND (SM); medium dense; dark orange-brown; moist; mostly fine SAND; some fines; nonplastic).
	340										
5										5	
		⊠	S-3	5 5 10	15	15	17.8				Decomposed; (Interbedded with CLAYEY SAND (SC); orange-brown; moist; mostly fine SAND; some fines; low plasticity).
	335										Decomposed; (SILTY SAND (SM); very dense; orange gray; moist; mostly fine to medium SAND; nonplastic).
10		⊠	S-4	50 (5")	120	120				10	
	330										
15		⊠	S-5	50 (6")	100	100				15	Gray.
											Bottom of boring at 15½ feet below grade. Groundwater not encountered during drilling. Backfilled 5/18/17.
	325										



GROUP DELTA CONSULTANTS, INC.
9245 Activity Road, Suite 103
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-2

BORING RECORD

PROJECT NAME DCI - Prospect Estates II
PROJECT NUMBER SD508
BORING **B-03**
SITE LOCATION Marrokal Lane, Santee, CA
START 5/16/2017
FINISH 5/16/2017
SHEET NO. 1 of 1
DRILLING COMPANY Pacific Drilling Company
DRILLING METHOD Hollow Stem Auger
LOGGED BY TSL
CHECKED BY MAF
DRILLING EQUIPMENT Limited Access Tracked Rig (Mole)
BORING DIA. (in) 6
TOTAL DEPTH (ft) 15.5
GROUND ELEV (ft) 347
DEPTH/*ELEV.* GROUND WATER (ft) ▼ N/A / *na*
SAMPLING METHOD Standard Cat-Head
NOTES ETR ~ 60%, N_{60} ~ 60/60 * N ~ N_{SPT} ~ N_{CAL} / 1.5

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
			B-1						PA PI EI			**ALLUVIUM (Qal):** SANDY FAT CLAY (CH); dark brown; moist; mostly fines; some fine to medium SAND; high plasticity. (0% Gravel; 32% Sand; 68% Fines) (LL~70; PL~15; PI~55)
	345		R-2-2 R-2-1	15 20 35	55	37	22.4	102				Hard. PP = 4½+
5			S-3	14 15 16	31	31				5		**GRANITIC ROCK (Kgr):** Decomposed; (SANDY SILT (ML); hard; very light yellow-green gray; moist; mostly fines; some fine to medium SAND; low plasticity).
	340											Decomposed; (SILTY SAND (SM); very dense; light gray; moist; mostly fine to coarse SAND; some fines; nonplastic).
10			S-4	50 (5")	120	120				10		
	335											
15			S-5	50 (6")	100	100	22.9			15		Decomposed; (Fine to coarse SILTY SAND (SM)).
	330											Bottom of boring at 15½ feet below grade. Groundwater not encountered during drilling. Backfilled 5/18/17.

GROUP DELTA CONSULTANTS, INC.
9245 Activity Road, Suite 103
San Diego, CA 92126

FIGURE
A-3

BORING RECORD

PROJECT NAME DCI - Prospect Estates II
PROJECT NUMBER SD508
BORING **B-04**
SITE LOCATION Marrokal Lane, Santee, CA
START 5/16/2017
FINISH 5/16/2017
SHEET NO. 1 of 1
DRILLING COMPANY Pacific Drilling Company
DRILLING METHOD Hollow Stem Auger
LOGGED BY TSL
CHECKED BY MAF
DRILLING EQUIPMENT Limited Access Tracked Rig (Mole)
BORING DIA. (in) 6
TOTAL DEPTH (ft) 16
GROUND ELEV (ft) 354
DEPTH/*ELEV.* GROUND WATER (ft) ▼ N/A / *na*
SAMPLING METHOD Standard Cat-Head
NOTES ETR ~ 60%, N_{60} ~ 60/60 * N ~ N_{SPT} ~ N_{CAL} / 1.5

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
		▨									░	**FILL:** SILTY SAND (SM); brown; dry; mostly SAND; some fines; nonplastic.
		▨	B-1						PA PI R		▧	**ALLUVIUM (Qal):** SANDY FAT CLAY (CH); dark brown; very stiff; moist; mostly fines; some fine SAND; high plasticity. (2% Gravel; 31% Sand; 67% Fines) (LL~73; PL~32; PI~41) PP = 2¾
		⧗	R-2-2 R-2-1	8 10 10	20	13	20.2	83			▧	
	350										▧	SANDY LEAN CLAY (CL); very stiff; light yellow-brown; moist; mostly fines; medium plasticity. Stiff to very stiff; very light yellow. PP = 1½
5		☒	S-3	5 6 8	14	14				5	░	**GRANITIC ROCK (Kgr):** Decomposed; (SILTY SAND (SM); medium dense; light yellow to light orange; moist; mostly fine SAND; some fines; nonplastic).
	345										▩	Decomposed; (SILTY SAND (SM); very dense; light yellow green-gray; moist; mostly fine to coarse SAND; few fine GRAVEL; nonplastic).
10		☒	S-4	50 (6")	100	100	21.2			10	▩	
	340										▩	
15		☒	S-5	25 50 (5")	85	85				15	▩	
												Bottom of boring at 16 feet below grade. Groundwater not encountered during drilling. Backfilled 5/18/17.
	335											

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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-4

BORING RECORD

PROJECT NAME DCI - Prospect Estates II
PROJECT NUMBER SD508
BORING **B-05**
SITE LOCATION Marrokal Lane, Santee, CA
START 5/16/2017
FINISH 5/16/2017
SHEET NO. 1 of 1
DRILLING COMPANY Pacific Drilling Company
DRILLING METHOD Hollow Stem Auger
LOGGED BY TSL
CHECKED BY MAF
DRILLING EQUIPMENT Limited Access Tracked Rig (Mole)
BORING DIA. (in) 6
TOTAL DEPTH (ft) 15.5
GROUND ELEV (ft) 353
DEPTH/*ELEV.* GROUND WATER (ft) ▼ N/A / *na*
SAMPLING METHOD Standard Cat-Head
NOTES ETR ~ 60%, N_{60} ~ 60/60 * N ~ N_{SPT} ~ N_{CAL} / 1.5

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
		▩	B-1						PA PI CR R		▨	**ALLUVIUM (Qal):** FAT CLAY WITH SAND (CH); dark brown; moist; mostly fines; little fine SAND; high plasticity. (1% Gravel; 21% Sand; 78% Fines) (LL~71; PL~19; PI~52) Very stiff. PP = 3¾
	350	⧗	R-2-2 R-2-1	9 14 18	32	21	24.5	97	DS		▨	LEAN CLAY WITH SAND (CL); hard; orange-brown; moist; mostly fines; few to little fine to medium SAND; medium plasticity. PP = 4 to 4½+ Interbedded with SANDY SILT (ML); hard; light green-gray; moist; mostly fines; some fine SAND; nonplastic.
5		☒	S-3	8 9 9	18	18				5	▨	
	345										▥	**GRANITIC ROCK (Kgr):** Decomposed; (SILTY SAND (SM); very dense; light green-gray; moist; mostly fine to medium SAND; some fines; nonplastic).
10		☒	S-4	50 (5")	120	120	5.7			10	▩	Decomposed; (SILTY SAND (SM); very dense; light green-gray; moist; mostly fine to medium SAND; some fines; nonplastic).
	340										▩	Decomposed; (Fine to coarse SAND; little fine GRAVEL).
15		☒	S-5	50 (6")	100	100				15	▩	
	335											Bottom of boring at 15½ feet below grade. Groundwater not encountered during drilling. Backfilled 5/18/17.

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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-5

APPENDIX A

BORING LOGS (GEI, 2016)

EQUIPMENT **Truck-mounted Hollow Stem Drill Rig**	DIMENSION & TYPE OF EXCAVATION **6-inch diameter Boring**	DATE LOGGED **11-18-15**
SURFACE ELEVATION **± 354' Mean Sea Level**	GROUNDWATER/ SEEPAGE DEPTH **Not Encountered**	LOGGED BY **JAB**

DEPTH (feet)	SYMBOL	SAMPLE	FIELD DESCRIPTION AND CLASSIFICATION		IN-PLACE MOISTURE (%)	IN-PLACE DRY DENSITY (pcf)	OPTIMUM MOISTURE (%)	MAXIMUM DRY DENSITY (pcf)	DENSITY (% of M.D.D.)	EXPAN. + CONSOL. - (%)	EXPANSION INDEX	BLOW COUNTS/FT.	SAMPLE O.D. (INCHES)
			DESCRIPTION AND REMARKS (Grain size, Density, Moisture, Color)	U.S.C.S.									
1			**CLAYEY SAND**, fine- to coarse-grained, with gravel to 3" in diameter and cobble to 8" in diameter (~5%). Medium dense. Moist. Dark brown. **FILL (Qaf)**	SC									
2 3		■	**CLAYEY SAND**, fine- to coarse-grained, with trace gravel to 3/4" in diameter; medium plasticity. Medium dense. Moist. Very dark gray. **SLOPEWASH (Qsw)** -- 39% passing #200 sieve. -- increase in gravel content.	SC	9.7	112.5						45	3"
4 5 6		☒	**SANDY CLAY**, fine- to medium-grained; high plasticity. Very stiff. Slightly moist. Red. **FRIARS FORMATION (Tf)** -- 51% passing #200 sieve. LL=54. PL=25. PI=29.	CH							116		
7 8		▨	**SANDY CLAY**, fine-grained, with some caliche. Very stiff. Moist. Light green-gray. **FRIARS FORMATION (Tf)**	CL								28	2"
9			Bottom @ 8.5'										

- ▼ PERCHED WATER TABLE
- ☒ BULK BAG SAMPLE
- 1 IN-PLACE SAMPLE
- ■ MODIFIED CALIFORNIA SAMPLE
- S NUCLEAR FIELD DENSITY TEST
- ▨ STANDARD PENETRATION TEST

JOB NAME
Prospect Estates II

SITE LOCATION
NE of Prospect Ave. and Marrokal Lane, Santee, CA

JOB NUMBER
15-10926

FIGURE NUMBER
IIIa

REVIEWED BY
WDH

Geotechnical Exploration, Inc.

LOG No.
B-1

EQUIPMENT	DIMENSION & TYPE OF EXCAVATION	DATE LOGGED
Truck-mounted Hollow Stem Drill Rig	**6-inch diameter Boring**	**11-18-15**
SURFACE ELEVATION	GROUNDWATER/ SEEPAGE DEPTH	LOGGED BY
± 355' Mean Sea Level	**Not Encountered**	**JAB**

DEPTH (feet)	SYMBOL	SAMPLE	FIELD DESCRIPTION AND CLASSIFICATION: DESCRIPTION AND REMARKS (Grain size, Density, Moisture, Color)	U.S.C.S.	IN-PLACE MOISTURE (%)	IN-PLACE DRY DENSITY (pcf)	OPTIMUM MOISTURE (%)	MAXIMUM DRY DENSITY (pcf)	DENSITY (% of M.D.D.)	EXPAN. + CONSOL. - (%)	EXPANSION INDEX	BLOW COUNTS/FT.	SAMPLE O.D. (INCHES)
2		☒	**CLAYEY SAND**, fine- to coarse-grained, with gravel to 3" in diameter and cobble to 6" in diameter (~5%). Medium dense. Moist. Dark brown. **FILL (Qaf)** -- 44% passing #200 sieve.	SC							61		
4		■	**CLAYEY SAND**, fine- to coarse-grained, with trace gravel to 3/4" in diameter; medium plasticity. Medium dense. Moist. Dark gray. **SLOPEWASH (Qsw)**	SC									
6		■	**SANDY CLAY**, fine-grained with some caliche; high plasticity. Hard. Moist. Light green-gray with red-brown. **FRIARS FORMATION (Tf)**	CH								76	3"
8 10		☒ ▨										45	2"
12			Bottom @ 11.5'										

▼ PERCHED WATER TABLE

☒ BULK BAG SAMPLE

1 IN-PLACE SAMPLE

■ MODIFIED CALIFORNIA SAMPLE

S NUCLEAR FIELD DENSITY TEST

▨ STANDARD PENETRATION TEST

JOB NAME
Prospect Estates II

SITE LOCATION
NE of Prospect Ave. and Marrokal Lane, Santee, CA

JOB NUMBER
15-10926

REVIEWED BY **WDH**

Geotechnical Exploration, Inc.

FIGURE NUMBER
IIIb

LOG No.
B-2

EXPLORATION LOG 10926 PROSPECT.GPJ GEO_EXPL.GDT 2/9/16

EQUIPMENT	DIMENSION & TYPE OF EXCAVATION	DATE LOGGED
Truck-mounted Hollow Stem Drill Rig	**6-inch diameter Boring**	**11-18-15**
SURFACE ELEVATION	GROUNDWATER/ SEEPAGE DEPTH	LOGGED BY
± 353' Mean Sea Level	**Not Encountered**	**JAB**

DEPTH (feet)	SYMBOL	SAMPLE	FIELD DESCRIPTION AND CLASSIFICATION DESCRIPTION AND REMARKS (Grain size, Density, Moisture, Color)	U.S.C.S.	IN-PLACE MOISTURE (%)	IN-PLACE DRY DENSITY (pcf)	OPTIMUM MOISTURE (%)	MAXIMUM DRY DENSITY (pcf)	DENSITY (% of M.D.D.)	EXPAN. + CONSOL. - (%)	EXPANSION INDEX	BLOW COUNTS/FT.	SAMPLE O.D. (INCHES)
0–3		☒ ■	**SANDY CLAY**, fine- to coarse-grained, with trace gravel to 3/4" in diameter; medium plasticity. Very stiff. Moist. Dark gray. **SLOPEWASH (Qsw)** -- 69% passing #200 sieve.	CL							91	28	3"
3–7			**SANDY CLAY**, fine-grained with some caliche. Hard. Moist. Light green-gray. **FRIARS FORMATION (Tf)**	CH									
7–8.5		▨	**CLAYEY SAND**, fine- to medium-grained, with some caliche. Very dense. Moist. Light green-gray with red-brown. **FRIARS FORMATION (Tf)** -- 38% passing #200 sieve.	SC								56	2"
9			Bottom @ 8.5'										

▼ PERCHED WATER TABLE

☒ BULK BAG SAMPLE

1 IN-PLACE SAMPLE

■ MODIFIED CALIFORNIA SAMPLE

S NUCLEAR FIELD DENSITY TEST

▨ STANDARD PENETRATION TEST

JOB NAME
Prospect Estates II

SITE LOCATION
NE of Prospect Ave. and Marrokal Lane, Santee, CA

JOB NUMBER
15-10926

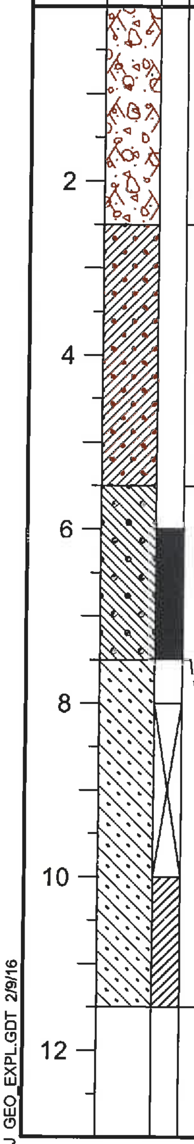
REVIEWED BY
WDH

Geotechnical Exploration, Inc.

FIGURE NUMBER
IIIc

LOG No.
B-3

EQUIPMENT	DIMENSION & TYPE OF EXCAVATION	DATE LOGGED
Truck-mounted Hollow Stem Drill Rig	**6-inch diameter Boring**	**11-18-15**
SURFACE ELEVATION	GROUNDWATER/ SEEPAGE DEPTH	LOGGED BY
± 367' Mean Sea Level	**Not Encountered**	**JAB**



FIELD DESCRIPTION AND CLASSIFICATION DESCRIPTION AND REMARKS (Grain size, Density, Moisture, Color)	U.S.C.S.	IN-PLACE MOISTURE (%)	IN-PLACE DRY DENSITY (pcf)	OPTIMUM MOISTURE (%)	MAXIMUM DRY DENSITY (pcf)	DENSITY (% of M.D.D.)	EXPAN. + CONSOL. - (%)	BLOW COUNTS/FT.	SAMPLE O.D. (INCHES)
CLAYEY SAND, fine- to coarse-grained, with gravel to 3" in diameter and cobble to 6" in diameter (~5%). Medium dense. Moist. Red-brown to brown. **FILL (Qaf)**	SC								
SANDY CLAY, fine- to coarse-grained, with trace gravel to 3/4" in diameter; medium plasticity. Very stiff. Moist. Very dark gray. **SLOPEWASH (Qsw)** -- sampler refusal on rock.	CL								
CLAYEY SAND, fine- to coarse-grained. Dense. Moist. Light green-gray. **FRIARS FORMATION (Tf)** -- 16% passing #200 sieve. @ 7.5' - abundant caliche transitioning to **CLAYSTONE**	SC							61	3"
SANDY CLAY, fine-grained, with some caliche; medium plasticity. Hard. Moist. Light green-gray with pale red inclusions. **FRIARS FORMATION (Tf)** Bulk bag sample from 8'- 11'. -- 57% passing #200 sieve.	CL							35	2"
Bottom @ 11.5'									

▼	PERCHED WATER TABLE
☒	BULK BAG SAMPLE
1	IN-PLACE SAMPLE
■	MODIFIED CALIFORNIA SAMPLE
S	NUCLEAR FIELD DENSITY TEST
▨	STANDARD PENETRATION TEST

JOB NAME
Prospect Estates II

SITE LOCATION
NE of Prospect Ave. and Marrokal Lane, Santee, CA

JOB NUMBER	REVIEWED BY	LOG No.
15-10926	**WDH**	**B-4**
FIGURE NUMBER	GEI Geotechnical Exploration, Inc.	
IIId		

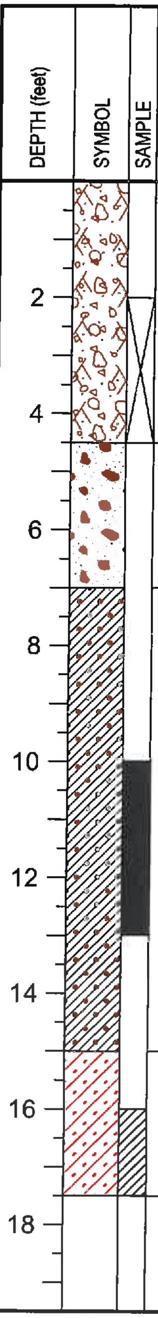
EQUIPMENT **Truck-mounted Hollow Stem Drill Rig**	DIMENSION & TYPE OF EXCAVATION **6-inch diameter Boring**	DATE LOGGED **11-18-15**
SURFACE ELEVATION **± 369' Mean Sea Level**	GROUNDWATER/ SEEPAGE DEPTH **Not Encountered**	LOGGED BY **JAB**

DEPTH (feet)	SYMBOL	SAMPLE	FIELD DESCRIPTION AND CLASSIFICATION DESCRIPTION AND REMARKS (Grain size, Density, Moisture, Color)	U.S.C.S.	IN-PLACE MOISTURE (%)	IN-PLACE DRY DENSITY (pcf)	OPTIMUM MOISTURE (%)	MAXIMUM DRY DENSITY (pcf)	DENSITY (% of M.D.D.)	EXPAN. + CONSOL. - (%)	BLOW COUNTS/FT.	SAMPLE O.D. (INCHES)
 2 4		■	**CLAYEY SAND**, fine- to coarse-grained, with gravel to 2" in diameter. Medium dense. Slightly moist. Light yellow-brown. **FILL (Qaf)** -- rock in sampler from 2.5'- 3'. Bulk bag sample from 1.5'- 3.5'. -- 29% passing #200 sieve.	SC							70/ 11"	3"
6 8 10		■	**SANDY CLAY**, fine- to coarse-grained sand, with trace gravel to 3/4" in diameter, moderate organic odor w/ trace organics; medium plasticity. Very stiff. Moist. Very dark gray. **SLOPEWASH (Qsw)**	CL							31	3"
12 14 16		▨ ☒ ▨	**SANDY CLAY**, fine-grained, with trace caliche; medium plasticity. Very stiff. Moist. Light yellow-brown w/ friable light green-gray formational inclusions. **FRIARS FORMATION (Tf)** Bulk bag sample from 11'- 14'.	CL							26 28	2" 2"
18			Bottom @ 16.5'									

- ▼ PERCHED WATER TABLE
- ☒ BULK BAG SAMPLE
- 1 IN-PLACE SAMPLE
- ■ MODIFIED CALIFORNIA SAMPLE
- S NUCLEAR FIELD DENSITY TEST
- ▨ STANDARD PENETRATION TEST

JOB NAME **Prospect Estates II**		
SITE LOCATION **NE of Prospect Ave. and Marrokal Lane, Santee, CA**		
JOB NUMBER **15-10926**	REVIEWED BY **WDH**	LOG No. **B-5**
FIGURE NUMBER **IIIe**	**Geotechnical Exploration, Inc.**	

EQUIPMENT	DIMENSION & TYPE OF EXCAVATION	DATE LOGGED
Truck-mounted Hollow Stem Drill Rig	**6-inch diameter Boring**	**11-18-15**
SURFACE ELEVATION	GROUNDWATER/ SEEPAGE DEPTH	LOGGED BY
± 367' Mean Sea Level	**Not Encountered**	**JAB**



FIELD DESCRIPTION AND CLASSIFICATION DESCRIPTION AND REMARKS (Grain size, Density, Moisture, Color)	U.S.C.S.	IN-PLACE MOISTURE (%)	IN-PLACE DRY DENSITY (pcf)	OPTIMUM MOISTURE (%)	MAXIMUM DRY DENSITY (pcf)	DENSITY (% of M.D.D.)	EXPAN. + CONSOL. - (%)	EXPANSION INDEX	BLOW COUNTS/FT.	SAMPLE O.D. (INCHES)
CLAYEY SAND, fine- to coarse-grained, with gravel to 3" in diameter and cobble to 8" in diameter (~5%). Medium dense. Slightly moist. Brown. **FILL (Qaf)** -- 36% passing #200 sieve.	SC							59		
GRAVEL, poorly graded, predominantly 3/4" in diameter, with fine- to coarse-grained clayey sand and silt. Medium dense. Dry. Light gray. **FILL (Qaf)**	GP									
SANDY CLAY, fine- to coarse-grained sand, with trace gravel, minor organic odor with trace organics; medium plasticity. Very stiff. Moist. Very dark gray. **SLOPEWASH (Qsw)** -- no sample recovery due to rock.	CL								37	3"
Bulk bag sample from 10'- 13'.									29	3"
SANDY CLAY, fine-grained; medium plasticity. Hard. Moist. Yellow. **FRIARS FORMATION (Tf)**	CL								33	2"
Bottom @ 17.5'										

▼ PERCHED WATER TABLE

☒ BULK BAG SAMPLE

1 IN-PLACE SAMPLE

■ MODIFIED CALIFORNIA SAMPLE

S NUCLEAR FIELD DENSITY TEST

▨ STANDARD PENETRATION TEST

JOB NAME
Prospect Estates II

SITE LOCATION
NE of Prospect Ave. and Marrokal Lane, Santee, CA

JOB NUMBER
15-10926

FIGURE NUMBER
IIIf

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WDH

Geotechnical Exploration, Inc.

LOG No.
B-6

APPENDIX B
LABORATORY TESTING

APPENDIX B

LABORATORY TESTING

Laboratory testing was conducted by Group Delta Consultants 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 according to the Unified Soil Classification System as established by the American Society of Civil Engineers per ASTM D2487. The soil classifications are shown on the boring 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 Figures B-1.1 to B-1.5.

Expansion Index: The expansion potential of selected soil samples was estimated in general accordance with ASTM D4829. The test results are summarized in Figure B-2, along with a summary of previous expansion index tests that have been conducted near the site. Figure B-2 also presents common criteria for evaluating the expansion potential based on the expansion index.

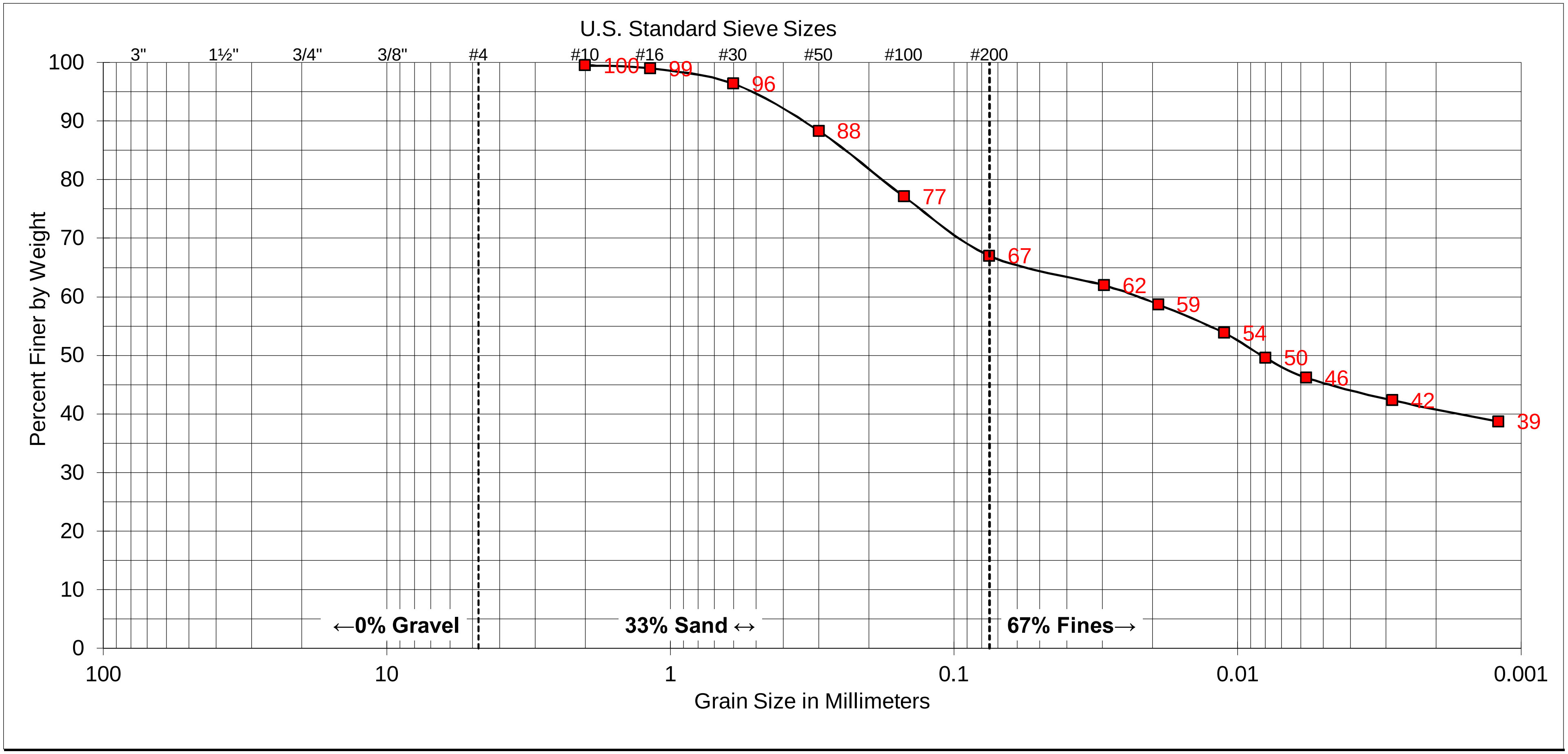
pH and Resistivity: To assess the potential for reactivity with buried metals, selected soil samples were tested for pH and minimum resistivity using Caltrans test method 643. The corrosivity test results are summarized in Figure B-3, along with previous corrosion tests we conducted on site.

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.

Chloride Content: The extracted solution from the sulfate test was also tested for water soluble chloride using a calibrated ion specific electronic probe. The results are also shown in Figure B-3.

Direct Shear: The shear strength of a selected sample was assessed using direct shear testing performed in general accordance with ASTM D3080. The test results are shown in Figure B-4.

R-Value: R-Value tests were performed on selected samples of the on-site soils in general accordance with CTM 301. The test results are shown in Figures B-5.1 and B-5.2.



COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND CLAY
GRAVEL		SAND			

SAMPLE

BORING NUMBER: B-1

SAMPLE DEPTH: 0' - 3'

UNIFIED SOIL CLASSIFICATION: CH

DESCRIPTION: SANDY FAT CLAY

ATTERBERG LIMITS

LIQUID LIMIT: 63

PLASTIC LIMIT: 16

PLASTICITY INDEX: 47

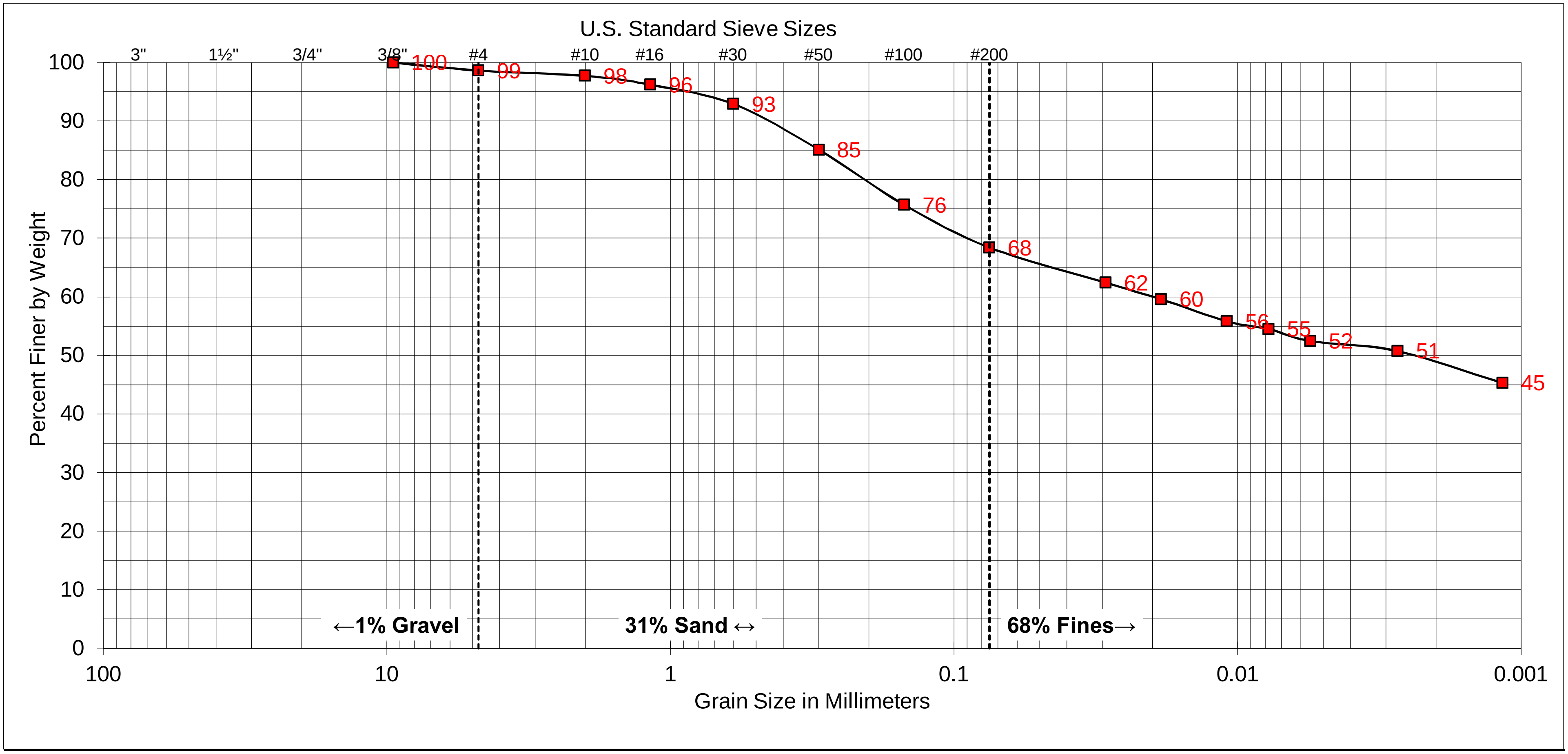
GROUP DELTA

SOIL CLASSIFICATION

Document No. 16-0204R

Project No. SD508

FIGURE B-1.1



COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND
GRAVEL		SAND			CLAY

SAMPLE	
BORING NUMBER:	B-2
SAMPLE DEPTH:	0' - 3'

UNIFIED SOIL CLASSIFICATION: CH

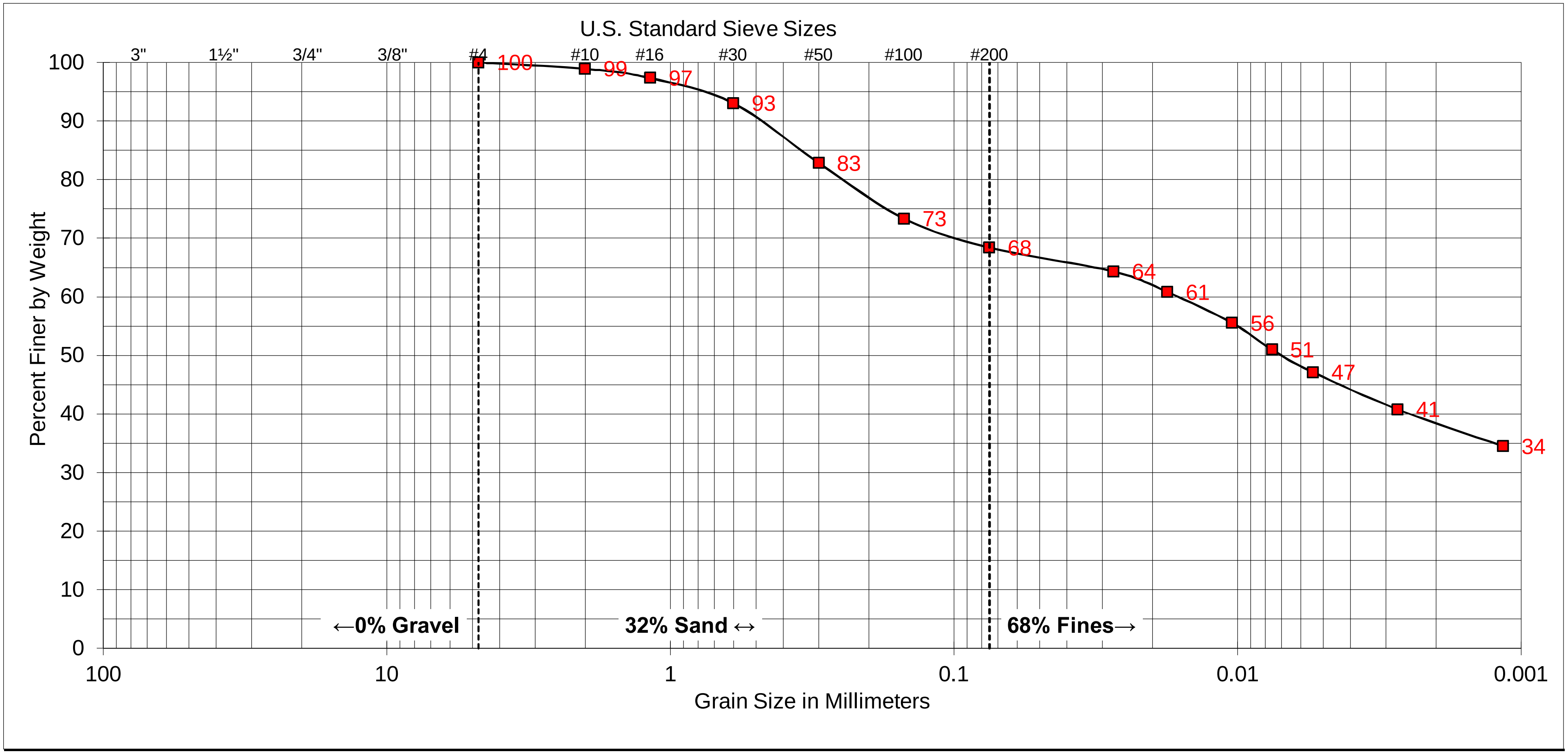
DESCRIPTION: SANDY FAT CLAY

ATTERBERG LIMITS	
LIQUID LIMIT:	66
PLASTIC LIMIT:	22
PLASTICITY INDEX:	44

GROUP DELTA

SOIL CLASSIFICATION

Document No. 16-0204R
Project No. SD508
FIGURE B-1.2



COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND
GRAVEL		SAND			CLAY

SAMPLE	
BORING NUMBER:	B-3
SAMPLE DEPTH:	0' - 3'

UNIFIED SOIL CLASSIFICATION: CH

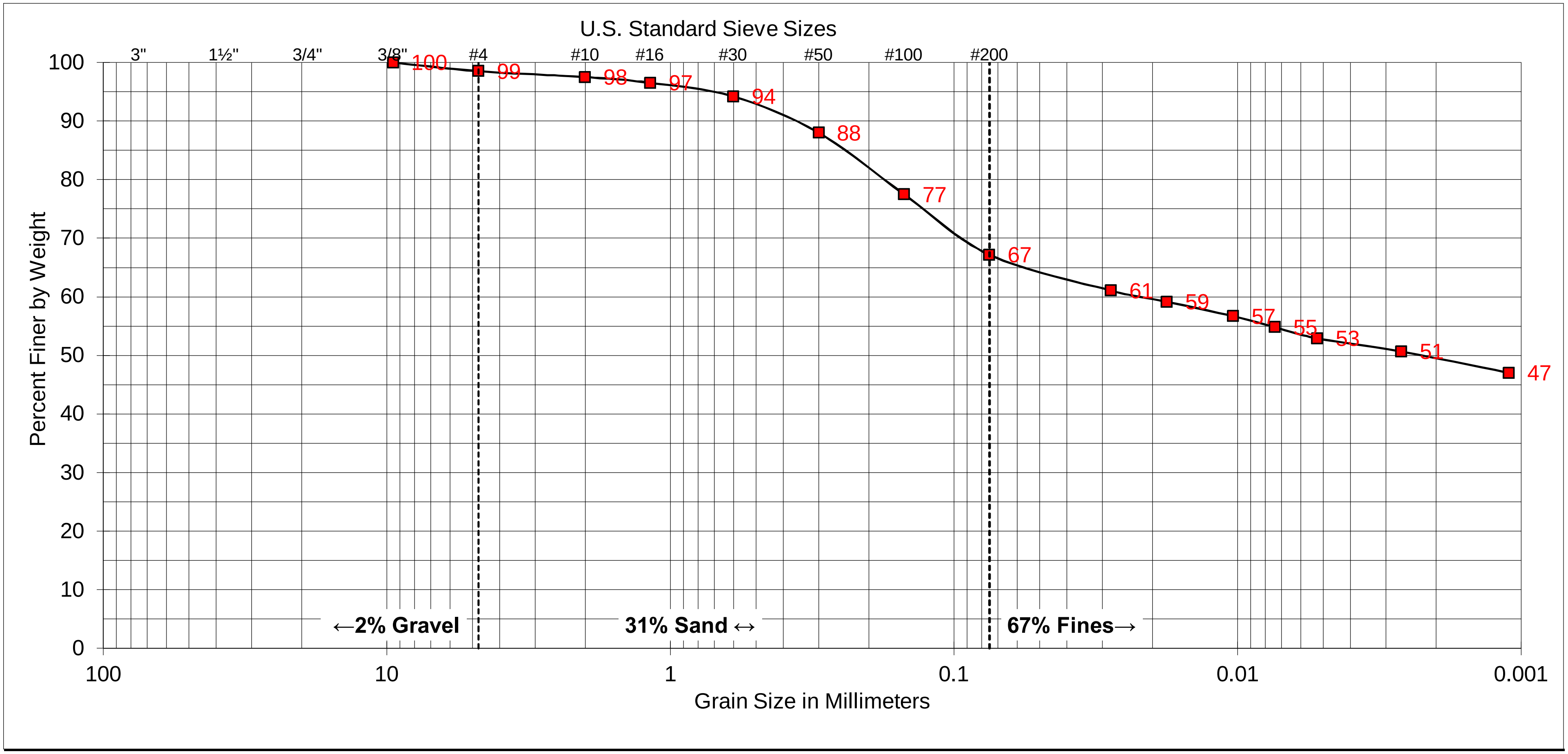
DESCRIPTION: SANDY FAT CLAY

ATTERBERG LIMITS	
LIQUID LIMIT:	70
PLASTIC LIMIT:	15
PLASTICITY INDEX:	55

GROUP DELTA

SOIL CLASSIFICATION

Document No. 16-0204R
Project No. SD508
FIGURE B-1.3



COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND CLAY
GRAVEL		SAND			

SAMPLE

BORING NUMBER: B-4

SAMPLE DEPTH: 0' - 3'

UNIFIED SOIL CLASSIFICATION: CH

DESCRIPTION: SANDY FAT CLAY

ATTERBERG LIMITS

LIQUID LIMIT: 73

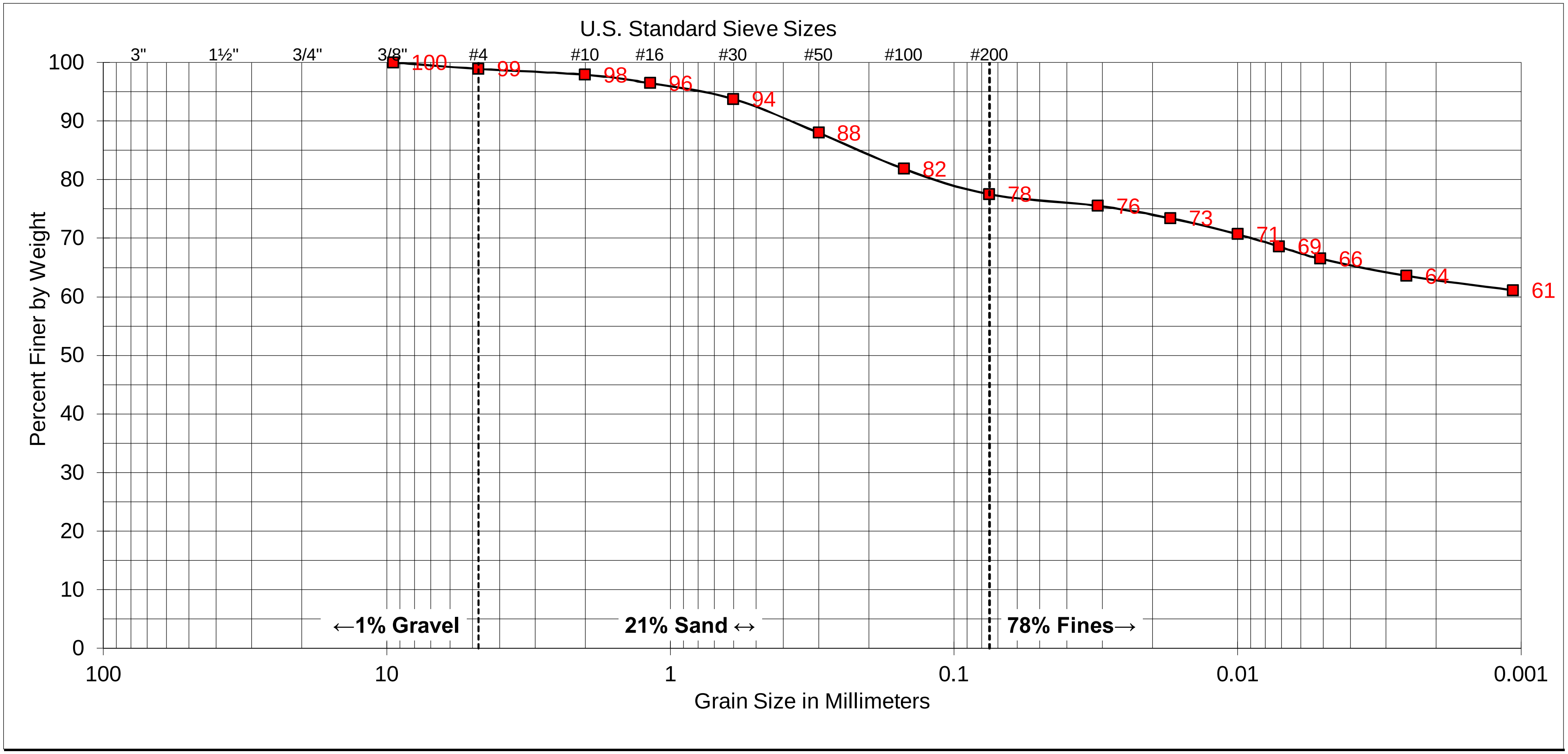
PLASTIC LIMIT: 32

PLASTICITY INDEX: 41

GROUP DELTA

SOIL CLASSIFICATION

Document No. 16-0204R
Project No. SD508
FIGURE B-1.4



COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND CLAY
GRAVEL		SAND			

SAMPLE

BORING NUMBER: B-5

SAMPLE DEPTH: 0' - 3'

UNIFIED SOIL CLASSIFICATION: CH

DESCRIPTION: FAT CLAY WITH SAND

ATTERBERG LIMITS

LIQUID LIMIT: 71

PLASTIC LIMIT: 19

PLASTICITY INDEX: 52

GROUP DELTA

SOIL CLASSIFICATION

Document No. 16-0204R

Project No. SD508

FIGURE B-1.5

EXPANSION TEST RESULTS

(ASTM D4829)

Sample ID	Geologic Unit (Symbol)	Sample Description (USCS)	Expansion Index
B-1 @ 0' – 3'	Alluvium (Qya)	Dark brown sandy fat clay (CH)	149
B-2 @ 0' – 3'	Alluvium (Qya)	Dark brown sandy fat clay (CH)	128
B-3 @ 0' – 3'	Alluvium (Qya)	Dark brown sandy fat clay (CH)	120

NOTE: From supplemental geotechnical investigation (GDC, 2017).

B-1 @ 4½' – 6½'	Friars Formation (Tf)	Reddish brown sandy fat clay (CH)	116
B-2 @ 1' – 3'	Fill	Dark brown clayey sand (SC)	61
B-3 @ 3' – 5'	Colluvium (Qya)	Dark gray sandy lean clay (CL)	91
B-6 @ 2' – 4½'	Fill	Brown clayey sand (SC)	59

NOTE: From previous geotechnical investigation by others (GEI, 2016).

TP-2 @ 0' – 3'	Alluvium (Qya)	Dark gray fat clay (CH)	142
TP-6 @ 2' – 4'	Alluvium (Qya)	Dark yellow brown fat clay (CH)	134
TP-9 @ 3' – 5'	Alluvium (Qya)	Dark yellow brown sandy lean clay (CL)	91
TP-10 @ 1' – 3'	Alluvium (Qya)	Dark yellow brown clayey sand (SC)	43
TP-14 @ 0' – 2'	Alluvium (Qya)	Dark reddish brown clayey sand (SC)	64
TP-4 @ 3' – 5'	Weathered Granite (Kgr)	Yellow brown silty sand (SM)	6
TP-7 @ 3' – 5'	Weathered Granite (Kgr)	Yellow brown clayey sand (SC)	51
TP-10 @ 4' – 6'	Weathered Granite (Kgr)	Dark brown silty sand (SM)	0
TP-1 @ 6' – 8'	Friars Formation (Tf)	Light olive brown lean claystone (CL)	93
TP-3 @ 5' – 7'	Friars Formation (Tf)	Light yellow brown fat claystone (CH)	129

NOTE: From previous geotechnical investigation for Prospect Estates I Property to the east (GDC, 2016).

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

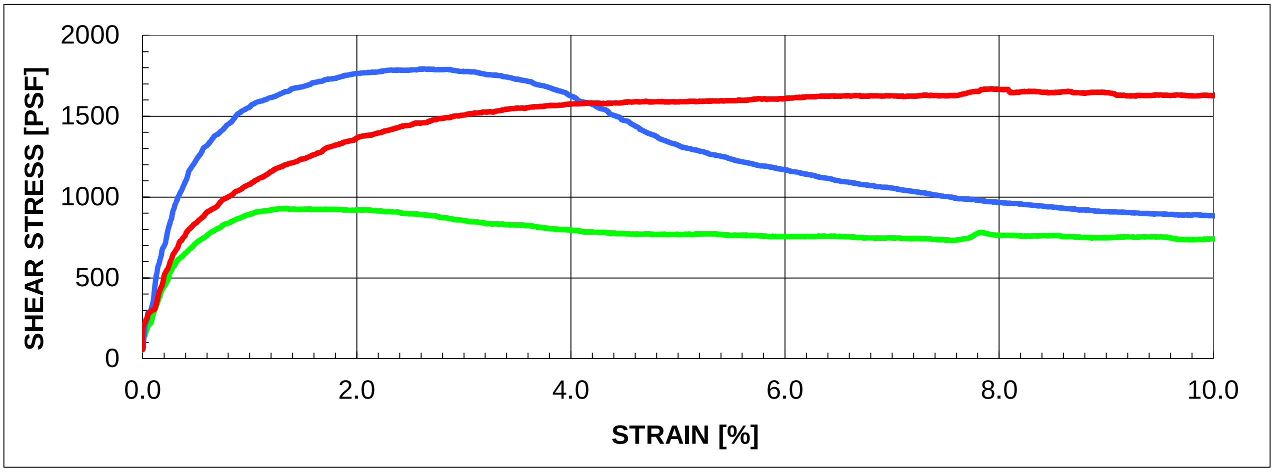
(ASTM D516, CTM 643)

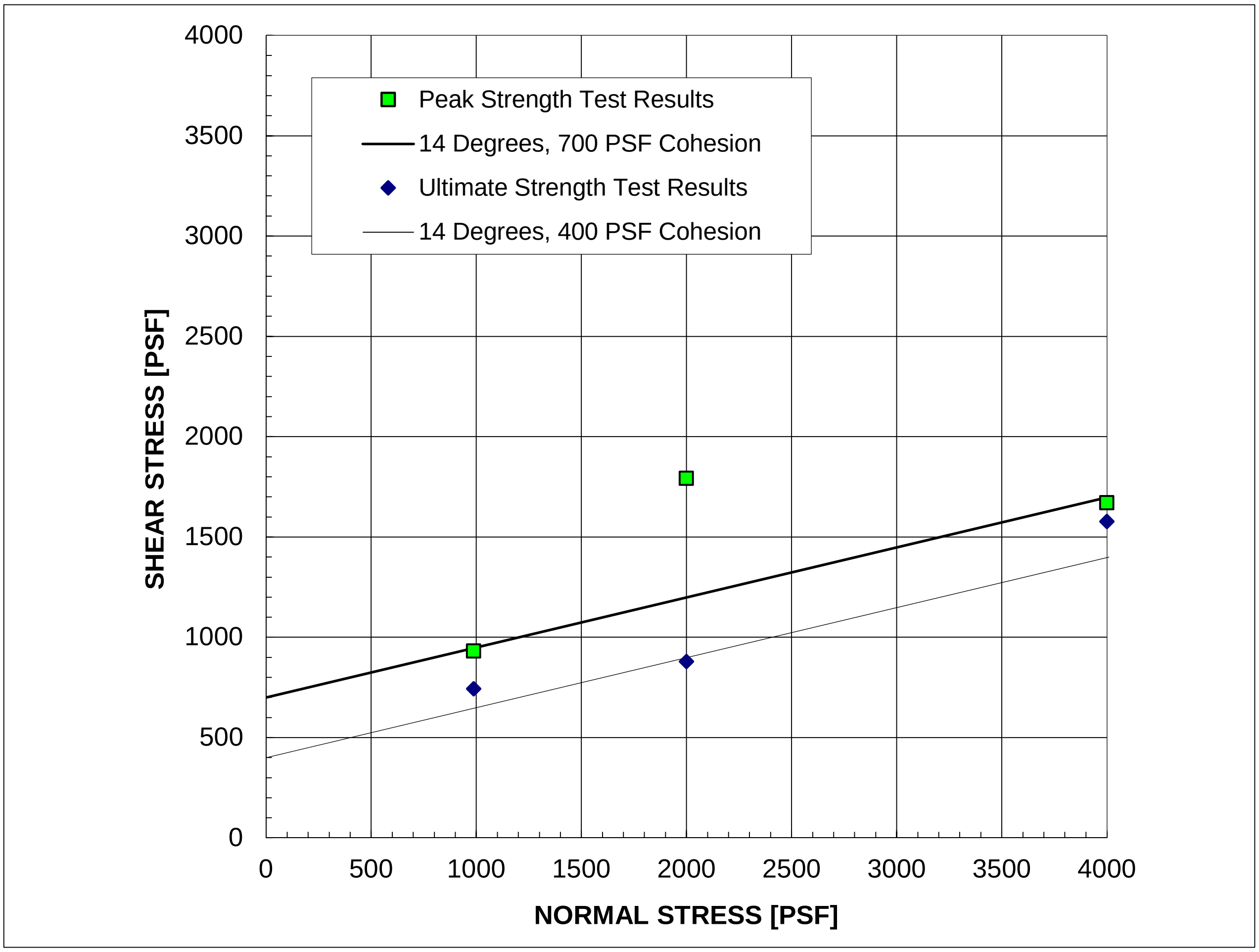
SAMPLE NO.	pH	RESISTIVITY [OHM-CM]	SULFATE CONTENT [%]	CHLORIDE CONTENT [%]
B-1 @ 0' – 3'	7.4	340	< 0.01	0.04
B-5 @ 0' – 3'	7.6	420	< 0.01	< 0.01

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





SAMPLE: B-5 @ 3'

Alluvium (Qal):
Dark brown sandy fat clay (CH)

	PEAK	ULTIMATE
ϕ'	14 °	14 °
C'	700 PSF	400 PSF

STRAIN RATE: 0.0002 IN/MIN
(Sample was consolidated and drained)

	IN-SITU	AS-TESTED
γ_d	96.6 PCF	96.6 PCF
w_c	24.5 %	28.9 %

GROUP DELTA

DIRECT SHEAR TEST RESULTS

Document No. 16-0204R
Project No. SD508
FIGURE B-4

BORING NO.: B-4
SAMPLE DATE: 5/16/17
BORING DEPTH: 0' - 3'
TEST DATE: 5/31/17
SAMPLE DESCRIPTION: Dark brown sandy fat clay (CH)

LABORATORY TEST DATA

	TEST SPECIMEN	**1**	**2**	**3**	**4**	**5**	
A	COMPACTOR PRESSURE	40					[PSI]
B	INITIAL MOISTURE						[%]
C	BATCH SOIL WEIGHT	1200					[G]
D	WATER ADDED	170					[ML]
E	WATER ADDED (D*(100+B)/C)						[%]
F	COMPACTION MOISTURE (B+E)						[%]
G	MOLD WEIGHT	2111.4					[G]
H	TOTAL BRIQUETTE WEIGHT						[G]
I	NET BRIQUETTE WEIGHT (H-G)						[G]
J	BRIQUETTE HEIGHT						[IN]
K	DRY DENSITY (30.3*I/((100+F)*J))						[PCF]
L	EXUDATION LOAD						[LB]
M	EXUDATION PRESSURE (L/12.54)						[PSI]
N	STABILOMETER AT 1000 LBS						[PSI]
O	STABILOMETER AT 2000 LBS						[PSI]
P	DISPLACEMENT FOR 100 PSI						[Turns]
Q	R VALUE BY STABILOMETER						
R	CORRECTED R-VALUE (See Fig. 14)						
S	EXPANSION DIAL READING						[IN]
T	EXPANSION PRESSURE (S*43,300)						[PSF]
U	COVER BY STABILOMETER						[FT]
V	COVER BY EXPANSION						[FT]

TRAFFIC INDEX:	5.0
GRAVEL FACTOR:	1.43
UNIT WEIGHT OF COVER [PCF]:	130
R-VALUE BY EXUDATION:	<5
R-VALUE BY EXPANSION:	<5
R-VALUE AT EQUILIBRIUM:	<5

*Note: Gravel factor estimated from pavement section using CTM 301, Section C, Part b.

REV. 2, DATED 1/31/15

GROUP DELTA

R-VALUE TEST RESULTS

BORING NO.: B-5
SAMPLE DATE: 5/16/17
BORING DEPTH: 0' - 3'
TEST DATE: 5/31/17
SAMPLE DESCRIPTION: Dark brown fat clay with sand (CH)

LABORATORY TEST DATA

	TEST SPECIMEN	**1**	**2**	**3**	**4**	**5**	
A	COMPACTOR PRESSURE	50					[PSI]
B	INITIAL MOISTURE						[%]
C	BATCH SOIL WEIGHT	1200					[G]
D	WATER ADDED	220					[ML]
E	WATER ADDED (D*(100+B)/C)						[%]
F	COMPACTION MOISTURE (B+E)						[%]
G	MOLD WEIGHT	2098.8					[G]
H	TOTAL BRIQUETTE WEIGHT						[G]
I	NET BRIQUETTE WEIGHT (H-G)						[G]
J	BRIQUETTE HEIGHT						[IN]
K	DRY DENSITY (30.3*I/((100+F)*J))						[PCF]
L	EXUDATION LOAD						[LB]
M	EXUDATION PRESSURE (L/12.54)						[PSI]
N	STABILOMETER AT 1000 LBS						[PSI]
O	STABILOMETER AT 2000 LBS						[PSI]
P	DISPLACEMENT FOR 100 PSI						[Turns]
Q	R VALUE BY STABILOMETER						
R	CORRECTED R-VALUE (See Fig. 14)						
S	EXPANSION DIAL READING						[IN]
T	EXPANSION PRESSURE (S*43,300)						[PSF]
U	COVER BY STABILOMETER						[FT]
V	COVER BY EXPANSION						[FT]

TRAFFIC INDEX:	5.0
GRAVEL FACTOR:	1.43
UNIT WEIGHT OF COVER [PCF]:	130
R-VALUE BY EXUDATION:	<5
R-VALUE BY EXPANSION:	<5
R-VALUE AT EQUILIBRIUM:	<5

*Note: Gravel factor estimated from pavement section using CTM 301, Section C, Part b.

REV. 2, DATED 1/31/15

GROUP DELTA

R-VALUE TEST RESULTS

APPENDIX C
INFILTRATION ASSESSMENT

Worksheet C.4-1: Categorization of Infiltration Feasibility Condition

Categorization of Infiltration Feasibility Condition	Worksheet C.4-1

Part 1 - Full Infiltration Feasibility Screening Criteria

Would infiltration of the full design volume be feasible from a physical perspective without any undesirable consequences that cannot be reasonably mitigated?

Criteria	Screening Question	Yes	No
1	**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.		No

Provide basis:

The on-site soils generally consist of sandy lean to fat clay (CL to CH). These fine grained soils have a very low permeability (roughly 10-7 cm/s or less), and would not permit infiltration at a rate of 0.5 inches per hour. Previous permeability tests conducted by Group Delta Consultants on similar fine grained sandy lean clay (CL) are provided at the end of Appendix C for reference.

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

Criteria	Screening Question	Yes	No
2	**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.		No

Provide basis:

See response to Item 1 above.

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

<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>No</td></tr>
<tr><td colspan="4">Provide basis:
See response to Item 1 above.
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>No</td></tr>
<tr><td colspan="4">Provide basis:
See response to Item 1 above.
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.		No

Provide basis:

See response to Item 1 above.

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.

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.		No

Provide basis:

See response to Item 1 above.

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.

<table>
<tr><th colspan="4">Worksheet C.4-1 Page 4 of 4</th></tr>
<tr><th>Criteria</th><th>Screening Question</th><th>Yes</th><th>No</th></tr>
<tr><td>7</td><td>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.</td><td></td><td>No</td></tr>
<tr><td colspan="4">Provide basis:
See response to Item 1 above.
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.</td></tr>
<tr><td>8</td><td>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.</td><td></td><td>No</td></tr>
<tr><td colspan="4">Provide basis:
See response to Item 1 above.
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.</td></tr>
<tr><td>Part 2 Result*</td><td colspan="2">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.</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 Agency/Jurisdictions to substantiate findings

GROUP DELTA

STANDARD TEST METHOD FOR MEASUREMENT OF HYDRAULIC CONDUCTIVITY OF SATURATED MATERIALS (ASTM D5084)

C:\ANALYSIS\LABPEARM2

PROJECT: Alberhill Clay and Aggregate Quarry TESTED BY: RHC SAMPLE: Olive #17 Document No. 16-0204
CLIENT: Pacific Aggregates CHECKED BY: MAF DATE: 06/21/10 Project No. SD508
DESCRIPTION: Remolded dark yellowish brown sandy lean clay (CL) with permeability of 2*(10^{-7}) cm/s **FIGURE C-1.1**

MOISTURE AND DENSITY

	INITIAL	FINAL	
A) WET WEIGHT OF SAMPLE	373.20	405.60	[G]
B) DRY WEIGHT OF SAMPLE	329.80	329.80	[G]
C) MOISTURE CONTENT [(A - B) / B]	13.2	23.0	
D) WET DENSITY (A / J * 62.4)	119.0	129.4	[PCF]
E) DRY DENSITY [D / (1 + C)]	105.2	105.2	[PCF]

TEST PARAMETERS

F) STANDPIPE AREAS	0.08	[CM^2]
G) SAMPLE DIAMETER	4.93	[CM]
H) SAMPLE AREA (π * G^2 / 4)	19.09	[CM^2]
I) INITIAL SAMPLE HEIGHT	10.25	[CM]
J) SAMPLE VOLUME (I * H)	195.66	[CM^3]

HYDRAULIC CONDUCTIVITY

	1	2	3	4	5	6	7	8	9	10	
K) CELL PRESSURE	1.500	1.500	1.500	1.500							[KG/CM^2]
L) DRIVING PRESSURE (LEFT)	1.300	1.300	1.300	1.300							[KG/CM^2]
M) BACK PRESSURE (RIGHT)	1.150	1.150	1.150	1.150							[KG/CM^2]
N) INITIAL WATER LEVEL (LEFT)	44.40	44.10	43.90	44.30							[CM]
O) INITIAL WATER LEVEL (RIGHT)	36.30	36.30	36.40	36.40							[CM]
P) FINAL WATER LEVEL (LEFT)	34.50	38.40	38.60	39.40							[CM]
Q) FINAL WATER LEVEL (RIGHT)	44.50	41.60	41.30	41.00							[CM]
R) FINAL SAMPLE HEIGHT	10.26	10.26	10.26	10.26							[CM]
S) TEST DURATION	11400	6660	6300	5700							[S]
T) PRESSURE HEAD [(L - M) * 1000 / 1.0]	150.00	150.00	150.00	150.00							[CM]
U) WATER DROP ON LEFT (N - P)	9.90	5.70	5.30	4.90							[CM]
V) WATER RISE ON RIGHT (O - Q)	-8.20	-5.30	-4.90	-4.60							[CM]
W) INITIAL WATER HEAD (N - O)	8.10	7.80	7.50	7.90							[CM]
X) FINAL WATER HEAD (P - Q)	-10.00	-3.20	-2.70	-1.60							[CM]
Y) INITIAL TOTAL HEAD (T + W)	158.10	157.80	157.50	157.90							[CM]
Z) FINAL TOTAL HEAD (T + X)	140.00	146.80	147.30	148.40							[CM]
α) OUTFLOW TO INFLOW RATIO (U / V)	1.21	1.08	1.08	1.07							
PERMEABILITY (F * R) / (2 * H *S) * LN (Y / Z)	2.3E-07	2.3E-07	2.3E-07	2.3E-07							[CM/S]

GROUP DELTA

STANDARD TEST METHOD FOR MEASUREMENT OF HYDRAULIC CONDUCTIVITY OF SATURATED MATERIALS (ASTM D5084)

C:\ANALYSIS\LABPEARM2

PROJECT: Vinje & Middleton
TESTED BY: RHC
SAMPLE: TP-4 @ 4'
Document No. 16-0204
CLIENT: 1382-001-00
CHECKED BY: MAF
DATE: 05/01/09
Project No. SD508
DESCRIPTION: Remolded reddish brown sandy clay (CL) with permeability of 2 * (10^{-8}) cm/s.

FIGURE C-1.2

MOISTURE AND DENSITY

MOISTURE AND DENSITY	INITIAL	FINAL	
A) WET WEIGHT OF SAMPLE	403.56	406.54	[G]
B) DRY WEIGHT OF SAMPLE	358.80	358.80	[G]
C) MOISTURE CONTENT [(A - B) / B]	12.5	13.3	
D) WET DENSITY (A / J * 62.4)	128.8	129.8	[PCF]
E) DRY DENSITY [D / (1 + C)]	114.5	114.5	[PCF]

TEST PARAMETERS

F) STANDPIPE AREAS	0.08	[CM^2]
G) SAMPLE DIAMETER	4.93	[CM]
H) SAMPLE AREA (π * G^2 / 4)	19.09	[CM^2]
I) INITIAL SAMPLE HEIGHT	10.24	[CM]
J) SAMPLE VOLUME (I * H)	195.47	[CM^3]

HYDRAULIC CONDUCTIVITY

HYDRAULIC CONDUCTIVITY	1	2	3	4	5	6	7	8	9	10	
K) CELL PRESSURE	3.000	3.500	3.500	3.500							[KG/CM^2]
L) DRIVING PRESSURE (LEFT)	2.800	3.300	3.300	3.300							[KG/CM^2]
M) BACK PRESSURE (RIGHT)	2.500	3.000	3.000	3.000							[KG/CM^2]
N) INITIAL WATER LEVEL (LEFT)	36.70	36.20	36.20	36.60							[CM]
O) INITIAL WATER LEVEL (RIGHT)	34.50	34.80	34.90	34.90							[CM]
P) FINAL WATER LEVEL (LEFT)	32.40	34.50	26.70	32.60							[CM]
Q) FINAL WATER LEVEL (RIGHT)	35.70	36.20	43.80	38.90							[CM]
R) FINAL SAMPLE HEIGHT	10.22	10.22	10.22	10.22							[CM]
S) TEST DURATION	12780	11880	61980	20760							[S]
T) PRESSURE HEAD [(L - M) * 1000 / 1.0]	300.00	300.00	300.00	300.00							[CM]
U) WATER DROP ON LEFT (N - P)	4.30	1.70	9.50	4.00							[CM]
V) WATER RISE ON RIGHT (O - Q)	-1.20	-1.40	-8.90	-4.00							[CM]
W) INITIAL WATER HEAD (N - O)	2.20	1.40	1.30	1.70							[CM]
X) FINAL WATER HEAD (P - Q)	-3.30	-1.70	-17.10	-6.30							[CM]
Y) INITIAL TOTAL HEAD (T + W)	302.20	301.40	301.30	301.70							[CM]
Z) FINAL TOTAL HEAD (T + X)	296.70	298.30	282.90	293.70							[CM]
α) OUTFLOW TO INFLOW RATIO (U / V)	3.58	1.21	1.07	1.00							
PERMEABILITY (F * R) / (2 * H *S) * LN (Y / Z)	3.0E-08	1.8E-08	2.2E-08	2.7E-08							[CM/S]

GROUP DELTA

STANDARD TEST METHOD FOR MEASUREMENT OF HYDRAULIC CONDUCTIVITY OF SATURATED MATERIALS (ASTM D5084)

C:\ANALYSIS\LABPEARM2

PROJECT: Vinje & Middleton
TESTED BY: RHC
SAMPLE: TP-8 @ 3'
Document No. 16-0204
CLIENT: 1382-001-00
CHECKED BY: MAF
DATE: 05/07/09
Project No. SD508
DESCRIPTION: Remolded dark gray sandy clay (CL) with permeability of 4 * (10^{-7}) cm/s.
FIGURE C-1.3

MOISTURE AND DENSITY

	INITIAL	FINAL	
A) WET WEIGHT OF SAMPLE	347.89	374.10	[G]
B) DRY WEIGHT OF SAMPLE	280.33	280.33	[G]
C) MOISTURE CONTENT [(A - B) / B]	24.1	33.4	
D) WET DENSITY (A / J * 62.4)	111.5	119.9	[PCF]
E) DRY DENSITY [D / (1 + C)]	89.8	89.8	[PCF]

TEST PARAMETERS

F) STANDPIPE AREAS	0.08	[CM^2]
G) SAMPLE DIAMETER	4.93	[CM]
H) SAMPLE AREA (π * G^2 / 4)	19.09	[CM^2]
I) INITIAL SAMPLE HEIGHT	10.20	[CM]
J) SAMPLE VOLUME (I * H)	194.71	[CM^3]

HYDRAULIC CONDUCTIVITY

	1	2	3	4	5	6	7	8	9	10	
K) CELL PRESSURE	3.000	3.000	3.000	3.000							[KG/CM^2]
L) DRIVING PRESSURE (LEFT)	2.805	2.810	2.808	2.808							[KG/CM^2]
M) BACK PRESSURE (RIGHT)	2.607	2.612	2.608	2.608							[KG/CM^2]
N) INITIAL WATER LEVEL (LEFT)	61.50	61.30	61.30	61.40							[CM]
O) INITIAL WATER LEVEL (RIGHT)	49.80	49.90	50.30	50.40							[CM]
P) FINAL WATER LEVEL (LEFT)	52.70	52.00	53.30	46.80							[CM]
Q) FINAL WATER LEVEL (RIGHT)	57.80	59.30	58.60	64.30							[CM]
R) FINAL SAMPLE HEIGHT	10.39	10.43	10.43	10.43							[CM]
S) TEST DURATION	3720	4473	4320	7920							[S]
T) PRESSURE HEAD [(L - M) * 1000 / 1.0]	198.00	198.00	200.00	200.00							[CM]
U) WATER DROP ON LEFT (N - P)	8.80	9.30	8.00	14.60							[CM]
V) WATER RISE ON RIGHT (O - Q)	-8.00	-9.40	-8.30	-13.90							[CM]
W) INITIAL WATER HEAD (N - O)	11.70	11.40	11.00	11.00							[CM]
X) FINAL WATER HEAD (P - Q)	-5.10	-7.30	-5.30	-17.50							[CM]
Y) INITIAL TOTAL HEAD (T + W)	209.70	209.40	211.00	211.00							[CM]
Z) FINAL TOTAL HEAD (T + X)	192.90	190.70	194.70	182.50							[CM]
α) OUTFLOW TO INFLOW RATIO (U / V)	1.10	0.99	0.96	1.05							
PERMEABILITY (F * R) / (2 * H *S) * LN (Y / Z)	4.8E-07	4.5E-07	4.0E-07	4.0E-07							[CM/S]

APPENDIX D

CORRESPONDENCE

GROUP DELTA

July 24, 2017

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

Attention: Michael Grant, President

SUBJECT: **Response to Comments – Update Geotechnical Report**
Prospect Estates II
Santee, California

Reference: *Updated Geotechnical Investigation, Prospect Estates II Development, Santee, California, Group Delta Consultants, dated May 31, 2017 (Project No. SD508).*

Dear Mr. Grant:

In accordance with your request, Group Delta is providing responses to review comments submitted by Geocon, Inc. in their letter dated June 20, 2017. Provided below is their comment (in italics) followed by our response.

1. Section 6.3.2 indicates the existing fill and alluvium should be removed to competent formational materials and replaced with compacted fill. Based on Borings B-5 and B-6 performed by GEI, these excavations may extend up to 15 feet deep. The project geotechnical states remedial grading should extend off-site. The project developer should obtain written permission from the adjacent property owners to allow the off-site grading. If permission is not obtained, the geotechnical consultant should update their recommendations to achieve the planned removals. It appears structures are present near the property lines on the southwest portion of the property that would inhibit off-site grading.

If the project developer cannot obtain written permission from the adjacent property owners to allow for the off-site grading, the following measures could be adopted based on conditions encountered during construction:

1. Allow partial removal of the alluvium (slopewash in GEI borings). Explorations in this area indicate the consistency of these materials are very stiff.
2. Conduct full removal in slots within the existing site boundaries where there are nearby offsite improvements.
3. Establish a structure setback zone along the perimeter that considers the extent of the removal that is ultimately achieved onsite.
4. Locally deepen foundations to place the bottom outward edge of the footing behind a line projected downward at a 1:1 inclination to formational or other competent materials.

We propose to add these items to Section 6.3.2 of the geotechnical report to present a strategy for mitigating restricted removal depths if the project developer cannot obtain written permission from the adjacent property owners to allow for the off-site grading.

2. Section 6.3.4 states the over-excavation should extend at least 10 feet outside of the planned building envelope. This lateral removal distance may be difficult to achieve on lots adjacent to the property lines. The geotechnical consultant should evaluate if additional recommendations will be required if the lateral removal distance is less than 10 feet.

This recommendation can be reduced to five feet outside of the planned building envelope. Except for one lot, the minimum horizontal distance to the property line is 10 feet. We will revise the recommendation accordingly.

3. Section 6.4.1 provides post-tensioned foundation recommendations. The geotechnical consultant should reference the source of the recommendations (e.g., PTI DC 10.5 in accordance with the 2016 CBC).

The preliminary post-tension design slab design parameters were developed using guidelines in the Standard Requirements for Analysis of Shallow Concrete Foundations on Expansive Soils, Post-Tensioning Institute, May 2008. The final as-graded post-tension design slab design parameters should be developed using the latest guidelines and code at the completion of grading. This information and recommendation will be added to Section 6.4.1.

4. It appears the geotechnical consultant has provided permeability results for other projects on remolded samples based on the designations. Has the geotechnical consultant performed in-place infiltration testing in the area of the planned storm water management basin? Can the basin be extended into the existing granitic rock and allow infiltration? The granitic rock is located about 5 feet below existing grade based on Boring B-1 performed Group Delta.

Based on the Tentative Map and Preliminary Grading Plan (Polaris dated June 7, 2017) the base of the infiltration basin, which is at an elevation of approximately 336 feet above mean sea level (MSL) should extend into the existing granitic rock. On July 20, 2017, Group Delta drilled two additional borings in the area of the proposed infiltration basin. The borings extended into the granitic rock materials and were terminated approximately 2 feet below the bottom of the planned infiltration basin. Following drilling, the borings were converted to infiltration test holes and presoaked for about 24 hours. Infiltration testing was conducted in the two test holes on July 21, 2017 using the Borehole Percolation Test method (Riverside County Percolation Test, 2011) referenced in the City of Santee BMP Design Manual (2016). The average design infiltration rate was approximately 0.02 inches per hour, assuming a factor of safety of 2. The field test data sheets are attached to this letter.

5. Worksheet C.4-l states the existing soil will possess an infiltration rate of less than 0.5 inches/hour within Criteria l. Does the geotechnical consultant have test data on in-situ samples to evaluate the infiltration rate of less than 0.5 to 0.05 inches/hour to answer Criteria 5? Based on the elevation of the planned basin, will remolded compacted fill be present at the base of the basin?

Partial infiltration is not likely to be possible if the bottom of the basin extends to the decomposed granite. Percent fines tests conducted on soils samples obtained at five and 10 feet have fines contents of about 20 percent. Hough (1957) and Hoek and Bray (1977), as reproduced in Hunt (1986)[1], provide a correlation of permeability to soil and rock type respectively (attached). The correlation for "silty sand" estimates a permeability of 0.16 inches per hour. The correlation for "weathered granite" estimates a permeability of 0.14 inches per hour. A factor of safety of 2.0 and 3.0 would reduce the average estimated permeability to 0.07 to 0.05 inches per hour respectively, which is the lower bound of the range of infiltration stated in the comment above. We understand the City of Santee Stormwater Design Manual (Manual) recommends a maximum factor of safety of 2.0 for infiltration feasibility screening, but allows selection of a higher factor at the discretion of engineer. We recommend using the higher factor of safety because a potentially "impervious layer" is about 10 feet from the bottom of the basin. The Manual considers a depth to an impervious layer of 5 to 15 feet below the bottom of the basin to be a "Medium Concern". Very dense (SPT blows/foot of 50 for six inches) decomposed granite was logged at a depth of about 12 feet (elevation of 335 feet, or 11 feet below invert level of infiltration basin).

In addition, recent field infiltration testing conducted in the vicinity of the proposed infiltration basin resulted in an average design infiltration rate of about 0.02 inches per hour, assuming a factor of safety of 2. We propose to add this information and conclusion to the geotechnical report.

There should not be remolded compacted fill be present at the base of the basin.

6. Are there geotechnical constraints that would preclude partial infiltration within the planned basin at the northwest corner of the Please include a response within Criteria 6 of Worksheet C.4-l.

Site to the north is undeveloped. Residential development is planned (see attached Tentative Map). Partial infiltration could negatively impact the foundations of perimeter retaining walls or other improvements close to the proposed basin. It could also create an undesirable long term liability exposure to the developers/owners of the Prospect Estates II project. We propose to add this information and conclusion to the geotechnical report.

7. The geotechnical engineering consultant should provide an answer to each criterion (1 through 8) on Worksheet C.4-1 and submit the updated Worksheet for review.

Revised Worksheet C.4-1 attached.

8. The design team should submit a plan that shows existing topography, proposed topography, planned development and details regarding the storm water management devices.

To be provided by Civil Engineer.

---

[1] Hunt, Roy E. 1986. Geotechnical Engineering Techniques and Practices, McGraw Hill Book Company, First Edition.

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

GROUP DELTA CONSULTANTS

Charles Robin (Rob) Stroop, G.E. 2298
Associate Geotechnical Engineer

REGISTERED PROFESSIONAL ENGINEER
CHARLES ROBIN STROOP
GE2298
Exp. 03-31-18
GEOTECHNICAL
STATE OF CALIFORNIA

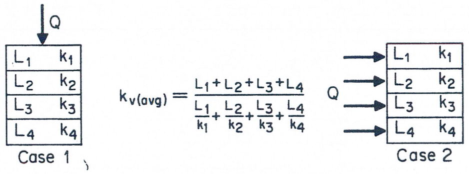
Attachments: Extracts from Geotechnical Engineering Techniques and Practices, (Hunt 1986)
Marrokal Lane Tentative Map (June 7, 2005)
Borehole Percolation Test Data Sheets
Worksheet C.4-1

Distribution: (1) Addressee, Michael Grant (grant.michael@sbcglobal.net)

TABLE 2.8
TYPICAL PERMEABILITY COEFFICIENTS FOR VARIOUS MATERIALS*

	Particle-size range				"Effective" size		Permeability coefficient k		
	Inches		Millimeters						
	D_{max}	D_{min}	D_{max}	D_{min}	D_{20}, in	D_{10}, mm	ft/year	ft/month	cm/s
				TURBULENT FLOW					
Derrick stone	120	36			48		100×10^6	100×10^5	100
One-man stone	12	4			6		30×10^6	30×15^5	30
Clean, fine to coarse gravel	3	¼	80	10	½		10×10^6	10×10^5	10
Fine, uniform gravel	⅜	1/16	8	1.5	⅛		5×10^6	5×10^5	5
Very coarse, clean, uniform sand	⅛	1/32	3	0.8	1/16		3×10^6	3×10^5	3
				LAMINAR FLOW					
Uniform, coarse sand	⅛	1/64	2	0.5		0.6	0.4×10^6	0.4×10^5	0.4
Uniform, medium sand			0.5	0.25		0.3	0.1×10^6	0.1×10^5	0.1
Clean, well-graded sand and gravel			10	0.05		0.1	0.01×10^6	0.01×10^5	0.01
Uniform, fine sand			0.25	0.05		0.06	4000	400	40×10^{-4}
Well-graded, silty sand and gravel			5	0.01		0.02	400	40	4×10^{-4}
Silty sand			2	0.005		0.01	100	10	10^{-4}
Uniform silt			0.05	0.005		0.006	50	5	0.5×10^{-4}
Sandy clay			1.0	0.001		0.002	5	0.5	0.05×10^{-4}
Silty clay			0.05	0.001		0.0015	1	0.1	0.01×10^{-4}
Clay (30 to 50% clay sizes)			0.05	0.0005		0.0008	0.1	0.01	0.001×10^{-4}
Colloidal clay ($-2\mu \leq$ 50%)			0.01	10 Å		40 Å	0.001	10^{-4}	10^{-9}

*From Hough (1957).[10] Reprinted with permission of John Wiley & Sons, Inc.



$$k_{v(avg)} = \frac{L_1+L_2+L_3+L_4}{\frac{L_1}{k_1}+\frac{L_2}{k_2}+\frac{L_3}{k_3}+\frac{L_4}{k_4}}$$

$$k_{h(avg)} = \frac{k_1\ L_1+k_2\ L_2+k_3\ L_3+k_4\ L_4}{L_1+L_2+L_3+L_4}$$

where Q = quantity of flow
L = flow path length
k = coefficient of permeability
NOTE: The electrical analogy:
If $L_1 = L_2 = L_3 = L_4 = 1$, and $k_1 = 1$, $k_2 = 2$, $k_3 = 3$, $k_4 = 4$, then in case 1, $k_{v(avg)} = 1.9$ and in case 2, $k_{h(avg)} = 2.5$

FIG. 2.7 An evaluation of the effect of stratification on permeability. [From Salzman (1974).[11]]

TABLE 2.9
TYPICAL PERMEABILITY COEFFICIENTS FOR ROCK AND SOIL FORMATIONS*

	k, cm/s	Intact rock	Porosity n, %	Fractured rock	Soil
Practically impermeable	10^{-10}	Massive low-porosity rocks	0.1–0.5		Homogeneous clay below zone of weathering
	10^{-9}		0.5–5.0		
	10^{-8}				
	10^{-7}				
Low discharge, poor drainage	10^{-6}	Sandstone Weathered granite Schist	5.0–30.0	Clay-filled joints	Very fine sands, organic and inorganic silts, mixtures of sand and clay, glacial till, stratified clay deposits
	10^{-5}				
	10^{-4}				
	10^{-3}				
High discharge, free draining	10^{-2}			Jointed rock	Clean sand, clean sand and gravel mixtures
	10^{-1}				
	1.0			Open-jointed rock	
	10^{1}				
	10^{2}			Heavily fractured rock	Clean gravel

*After Hoek and Bray (1977).[20]

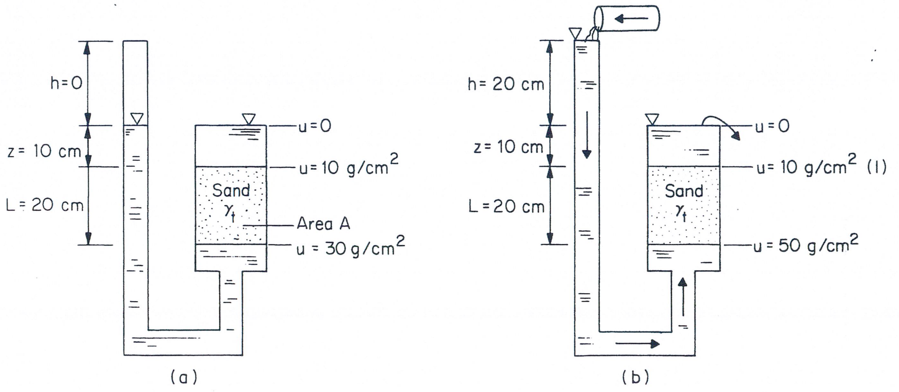


FIG. 2.8 Pore-water pressures. (a) No-flow condition. Bouyancy pressures act on each end of the soil specimen. (b) Upward flow condition. Boundary pressures act on each end of the specimen. At (1) there is 20 g/cm² lost in seepage.

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Pressures

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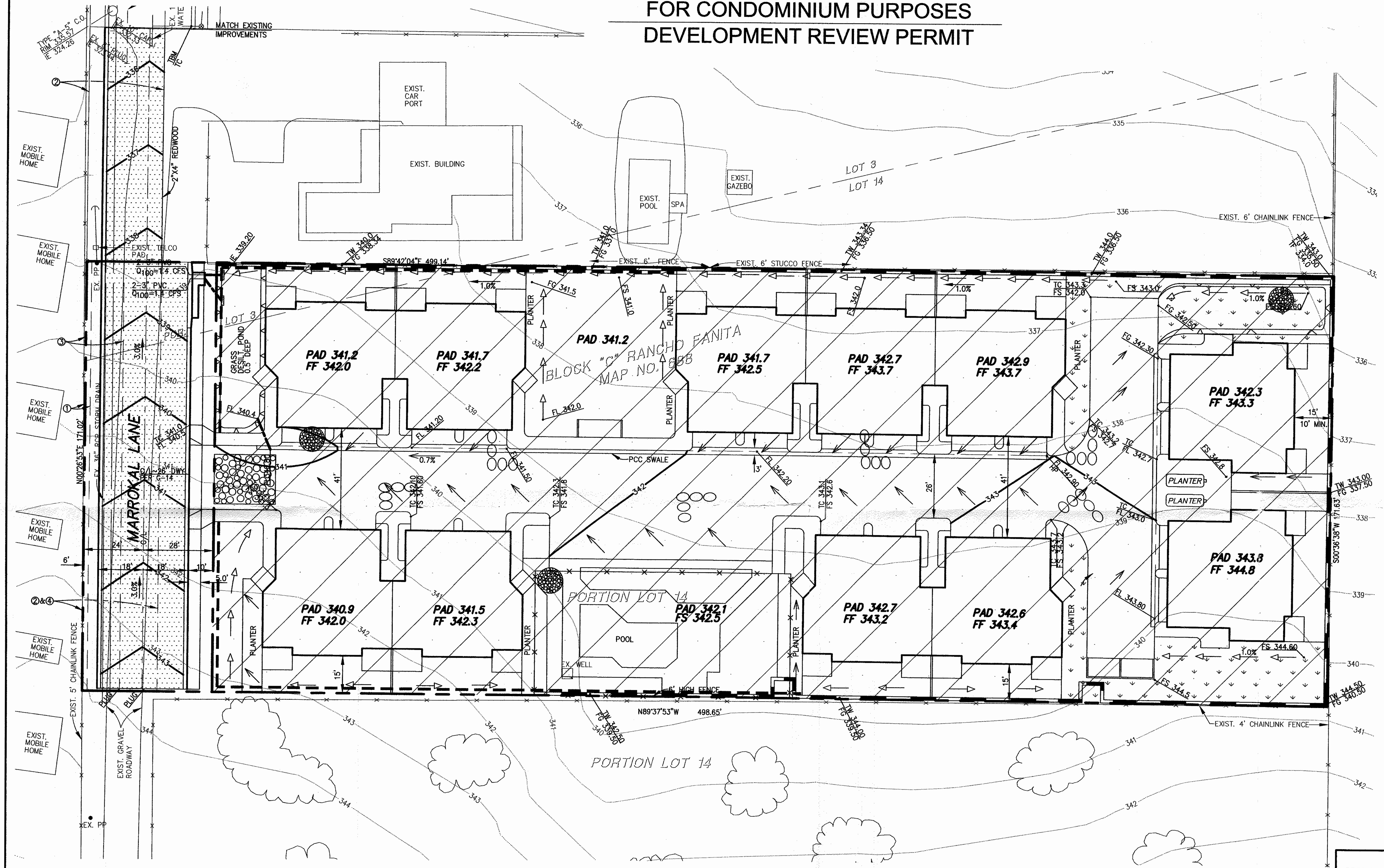
by the fl

TENTATIVE TRACT MAP
FOR CONDOMINIUM PURPOSES
DEVELOPMENT REVIEW PERMIT



LEGEND
TEMPORARY BMP's

3"-6" DIA. ROCK RIP-RAP PAD	▨
TEMPORARY 10' DIA. DESILTING POND	●
GRAVEL BAGS	○○○
STRAW ROLL	▬ ▬ ▬
SLOPE BINDER, FIBER MULCH OR STRAW MAT	⁘
HYDROSEED OR SHREDDED MULCH	▨

IF HYDROSEED IS NOT ESTABLISHED PRIOR TO RAIN EVENTS AND THERE IS EROSION, INSTALL STRAW MAT ON ALL GRADED SLOPES.

NOTE

ALL AREAS WILL BE COVERED OR RE-APPLIED WITH STRAW MAT OR JUTE MATTING UNTIL LANDSCAPING HAS BEEN ESTABLISHED AND COVERED 90 PERCENT OF EXPOSED DIRT SURFACES.

CITY OF SANTEE
APPROVED

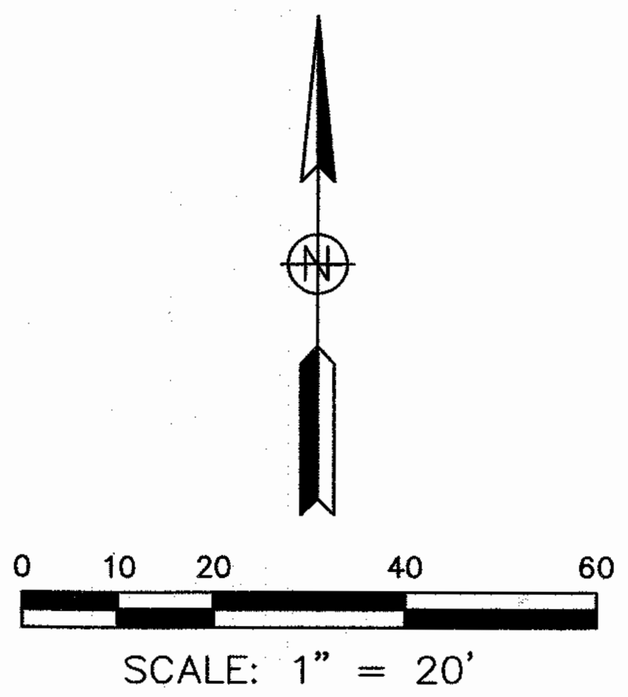
DATE APPROVED 3/14/07

APPROVED BY DEPARTMENT OF DEVELOPMENT SERVICES TM05-05/DR05-07

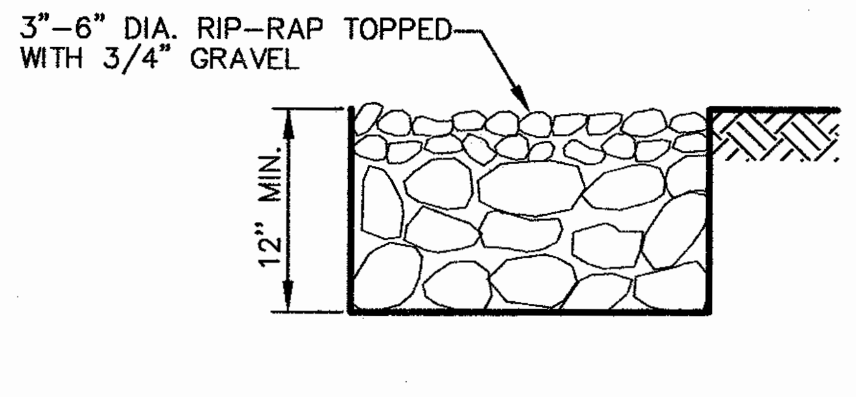
SIGNATURE Mark Bulle

PG. 1 OF 10

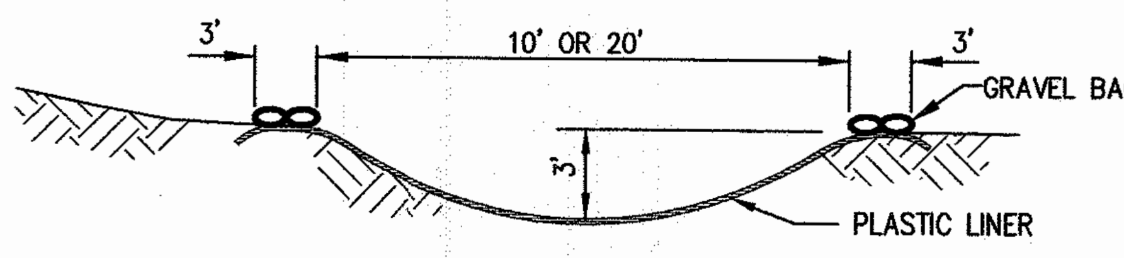
RECEIVED
TM05-05/DR05-07
JUN 16 2005
AEIS 05-14
Dept. of Development Services
City of Santee



SCALE: 1" = 20'



TEMPORARY 20' W X 20' L
ROCK PAD AT ENTRANCE FOR SILT REMOVAL ON TIRES (TYP.)
NO SCALE



10' OR 20' TEMPORARY DESILT POND
NO SCALE

CEA
COOPER ENGINEERING ASSOCIATES
CIVIL ENGINEERING PLANNING LAND SURVEYING
8369 VICKERS STREET, SUITE C, SAN DIEGO, CALIFORNIA 92111
PHONE:(858) 277-0441

PREPARED BY:
NAME: **COOPER ENGINEERING ASSOCIATES**
ADDRESS: 8369 VICKERS ST., STE. C
SAN DIEGO, CA. 92111
PHONE NO.: (858) 277-0441
PROJECT ADDRESS:
MARROKAL LANE
SANTEE, CA
PROJECT NAME:
MARROKAL LANE
SHEET TITLE:
EROSION CONTROL PLAN

REVISION 1: 06-07-05
REVISION 2: ______
REVISION 3: ______
REVISION 4: ______
REVISION 5: ______
REVISION 6: ______
REVISION 7: ______
REVISION 8: ______
REVISION 9: ______
REVISION 10: ______
ORIG. DATE: 05-24-04
SHEET C-3 OF 5
P.T.S. NO. ______
PERMIT NO. ______

BOREHOLE PERCOLATION TEST DATA SHEET

Storm Water Infiltration

Project Name: Prospect Estates II

Job Number: SD508

Tested By: C. Vonk

Test Hole No: I-1

Date Drilled: 7/20/2017

Date Tested: 7/21/2017

Drilling Method: Hollow Stem Auger

Borehole Radius: 3 inches

Depth of Hole as Drilled: 10 ft

Casing Stick-up: 0.0 ft

Test Depth: 8 - 10 ft

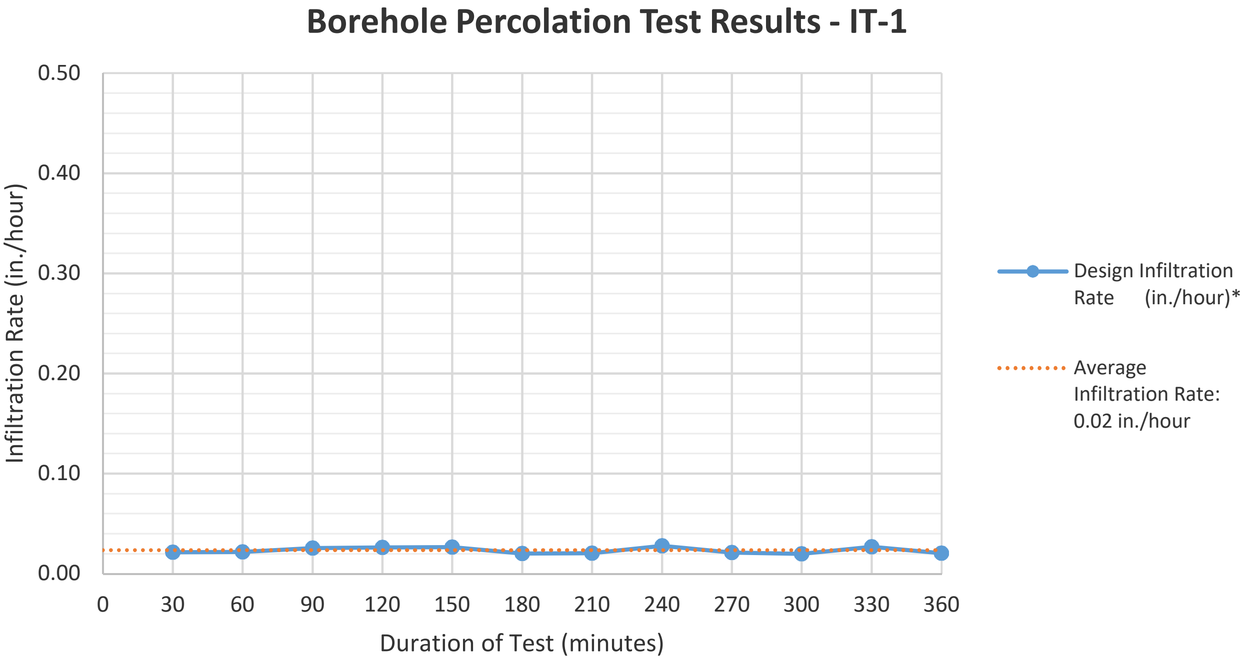
Reading Number	Time	Time Interval (min.)	Total Depth of Hole (ft.)	Initial Depth of Water (ft.)	Final Depth of Water (ft.)	Change in Water Level (in.)	Field Infiltration Rate (min./in.)	Design Infiltration Rate (in./hour)*
1	8:49 9:19	0:30	10.00	8.01	8.04	0.36	0.04	**0.02**
2	9:19 9:49	0:30	10.00	8.04	8.07	0.36	0.04	**0.02**
3	9:49 10:19	0:30	10.00	7.77	7.81	0.48	0.05	**0.03**
4	10:19 10:49	0:30	10.00	7.81	7.85	0.48	0.05	**0.03**
5	10:49 11:19	0:30	10.00	7.85	7.89	0.48	0.05	**0.03**
6	11:19 11:49	0:30	10.00	7.89	7.92	0.36	0.04	**0.02**
7	11:49 12:19	0:30	10.00	7.92	7.95	0.36	0.04	**0.02**
8	12:19 12:49	0:30	10.00	7.95	7.99	0.48	0.06	**0.03**
9	12:49 13:19	0:30	10.00	7.99	8.02	0.36	0.04	**0.02**
10	13:19 13:49	0:30	10.00	7.85	7.88	0.36	0.04	**0.02**
11	13:49 14:19	0:30	10.00	7.88	7.92	0.48	0.05	**0.03**
12	14:19 14:49	0:30	10.00	7.92	7.95	0.36	0.04	**0.02**

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

GROUP DELTA

BOREHOLE
PERCOLATION
TEST I-1

Project No. SD527
Document No. 17-0061
FIGURE C-I-1.1



GROUP DELTA

BOREHOLE PERCOLATION TEST I-1

Project No. SD527
Document No. 17-0061
FIGURE C-I-1.2

BOREHOLE PERCOLATION TEST DATA SHEET

Storm Water Infiltration

Project Name: Prospect Estates II
Job Number: SD508
Tested By: C. Vonk
Test Hole No: I-2
Date Drilled: 7/20/2017
Date Tested: 7/21/2017
Drilling Method: Hollow Stem Auger
Borehole Radius: 3 inches
Depth of Hole as Drilled: 10 ft
Casing Stick-up: 0.3 ft
Test Depth: 8 - 10 ft

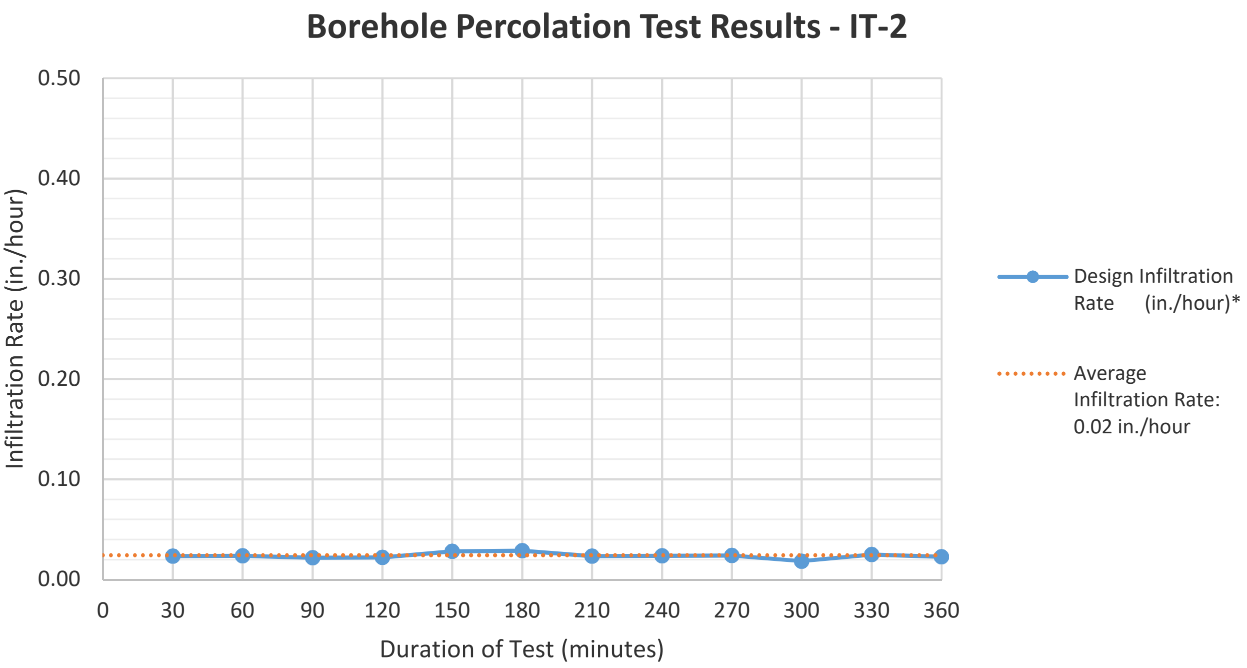
Reading Number	Time	Time Interval (min.)	Total Depth of Hole (ft.)	Initial Depth of Water (ft.)	Final Depth of Water (ft.)	Change in Water Level (in.)	Field Infiltration Rate (min./in.)	Design Infiltration Rate (in./hour)*
1	8:51 9:21	0:30	9.67	7.21	7.25	0.48	0.05	**0.02**
2	9:21 9:51	0:30	9.67	7.25	7.29	0.48	0.05	**0.02**
3	9:51 10:21	0:30	9.67	7.03	7.07	0.48	0.04	**0.02**
4	10:21 10:51	0:30	9.67	7.07	7.11	0.48	0.04	**0.02**
5	10:51 11:21	0:30	9.67	7.11	7.16	0.60	0.06	**0.03**
6	11:21 11:51	0:30	9.67	7.16	7.21	0.60	0.06	**0.03**
7	11:51 12:21	0:30	9.67	7.21	7.25	0.48	0.05	**0.02**
8	12:21 12:51	0:30	9.67	7.25	7.29	0.48	0.05	**0.02**
9	12:51 13:21	0:30	9.67	7.29	7.33	0.48	0.05	**0.02**
10	13:21 13:51	0:30	9.67	7.33	7.36	0.36	0.04	**0.02**
11	13:51 14:21	0:30	9.67	7.36	7.40	0.48	0.05	**0.02**
12	14:21 14:51	0:30	9.67	7.15	7.19	0.48	0.05	**0.02**

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

GROUP DELTA

BOREHOLE
PERCOLATION
TEST I-2

Project No. SD527
Document No. 17-0061
FIGURE C-I-2.1



GROUP DELTA

BOREHOLE PERCOLATION TEST I-2

Project No. SD527
Document No. 17-0061
FIGURE C-I-2.2

Worksheet C.4-1: Categorization of Infiltration Feasibility Condition

Categorization of Infiltration Feasibility Condition	Worksheet C.4-1

Part 1 - Full Infiltration Feasibility Screening Criteria

Would infiltration of the full design volume be feasible from a physical perspective without any undesirable consequences that cannot be reasonably mitigated?

Criteria	Screening Question	Yes	No
1	**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.		No

Provide basis:

The on-site soils generally consist of sandy lean to fat clay (CL to CH). These fine grained soils have a very low permeability (roughly 10-7 cm/s or less), and would not permit infiltration at a rate of 0.5 inches per hour.
In addition, recent infiltration testing performed at the siteresulted in an average design infiltration rate of approximately 0.02 inches per hour.

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

Criteria	Screening Question	Yes	No
2	**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.		No

Provide basis:

Site to the north is undeveloped. Residential development is planned (see attached Tentative Map).
Partial infiltration could negatively impact the foundations of perimeter retaining walls or other improvements close to the proposed basin. It could also create an undesirable long term liability exposure to the developers/owners of the Prospect Estates II project.

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

Worksheet C.4-1 Page 2 of 4

Criteria	Screening Question	**Yes**	**No**
3	**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.	Yes	

Provide basis:

Shallow groundwater is not present.

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

4	**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.	Yes	

Provide basis:

Sources of surface waters are not nearby.

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

Part 1 Result*	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	NO

*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.		No

Provide basis:

Partial infiltration is not likely to be possible if the bottom of the basin extends to the decomposed granite. Percent fines tests conducted on soils samples obtained at five and 10 feet have fines contents of about 20 percent. Hough (1957) and Hoek and Bray (1977), as reproduced in Hunt (1986) , provide a correlation of permeability to soil and rock type respectively (attached). The correlation for “silty sand” estimates a permeability of 0.16 inches per hour. The correlation for “weathered granite” estimates a permeability of 0.14 inches per hour. A factor of safety of 2.0 and 3.0 would reduce the average estimated permeability to 0.07 to 0.05 inches per hour respectively, which is the lower bound of the range of infiltration stated in the comment above. We understand the City of Santee Stormwater Design Manual (Manual) recommends a maximum factor of safety of 2.0 for infiltration feasibility screening, but allows selection of a higher factor at the discretion of engineer. We recommend using the higher factor of safety because a potentially “impervious layer” is about 10 feet from the bottom of the basin. The Manual considers a depth to an impervious layer of 5 to 15 feet below the bottom of the basin to be a “Medium Concern”. Very dense (SPT blows/foot of 50 for six inches) decomposed granite was logged at a depth of about 12 feet (elevation of 335 feet, or 11 feet below invert level of infiltration basin). Recent infiltration testing resulted in an average design infiltration rate of approximately 0.02 inches per hour.

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.		No

Provide basis:

Site to the north is undeveloped. Residential development is planned (see attached Tentative Map).
Partial infiltration could negatively impact the foundations of perimeter retaining walls or other improvements close to the proposed basin. It could also create an undesirable long term liability exposure to the developers/owners of the Prospect Estates II project.

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.

<table>
<tr><th colspan="4">Worksheet C.4-1 Page 4 of 4</th></tr>
<tr><th>Criteria</th><th>Screening Question</th><th>Yes</th><th>No</th></tr>
<tr><td>7</td><td>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.</td><td>Yes</td><td></td></tr>
<tr><td colspan="4">Provide basis:
Shallow groundwater is not present.

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.</td></tr>
<tr><td>8</td><td>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.</td><td>Yes</td><td></td></tr>
<tr><td colspan="4">Provide basis:
Sources of surface waters are not nearby.

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.</td></tr>
<tr><td>Part 2 Result*</td><td colspan="2">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.</td><td>NO</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 Agency/Jurisdictions to substantiate findings

GROUP DELTA

STANDARD TEST METHOD FOR MEASUREMENT OF HYDRAULIC CONDUCTIVITY OF SATURATED MATERIALS (ASTM D5084)

C:\ANALYSIS\LABPEARM2

PROJECT: Alberhill Clay and Aggregate Quarry TESTED BY: RHC SAMPLE: Olive #17 Document No. 16-0204
CLIENT: Pacific Aggregates CHECKED BY: MAF DATE: 06/21/10 Project No. SD508
DESCRIPTION: Remolded dark yellowish brown sandy lean clay (CL) with permeability of 2*(10^{-7}) cm/s **FIGURE C-1.1**

MOISTURE AND DENSITY

	INITIAL	FINAL	
A) WET WEIGHT OF SAMPLE	373.20	405.60	[G]
B) DRY WEIGHT OF SAMPLE	329.80	329.80	[G]
C) MOISTURE CONTENT [(A - B) / B]	13.2	23.0	
D) WET DENSITY (A / J * 62.4)	119.0	129.4	[PCF]
E) DRY DENSITY [D / (1 + C)]	105.2	105.2	[PCF]

TEST PARAMETERS

F) STANDPIPE AREAS	0.08	[CM^2]
G) SAMPLE DIAMETER	4.93	[CM]
H) SAMPLE AREA (π * G^2 / 4)	19.09	[CM^2]
I) INITIAL SAMPLE HEIGHT	10.25	[CM]
J) SAMPLE VOLUME (I * H)	195.66	[CM^3]

HYDRAULIC CONDUCTIVITY

	1	2	3	4	5	6	7	8	9	10	
K) CELL PRESSURE	1.500	1.500	1.500	1.500							[KG/CM^2]
L) DRIVING PRESSURE (LEFT)	1.300	1.300	1.300	1.300							[KG/CM^2]
M) BACK PRESSURE (RIGHT)	1.150	1.150	1.150	1.150							[KG/CM^2]
N) INITIAL WATER LEVEL (LEFT)	44.40	44.10	43.90	44.30							[CM]
O) INITIAL WATER LEVEL (RIGHT)	36.30	36.30	36.40	36.40							[CM]
P) FINAL WATER LEVEL (LEFT)	34.50	38.40	38.60	39.40							[CM]
Q) FINAL WATER LEVEL (RIGHT)	44.50	41.60	41.30	41.00							[CM]
R) FINAL SAMPLE HEIGHT	10.26	10.26	10.26	10.26							[CM]
S) TEST DURATION	11400	6660	6300	5700							[S]
T) PRESSURE HEAD [(L - M) * 1000 / 1.0]	150.00	150.00	150.00	150.00							[CM]
U) WATER DROP ON LEFT (N - P)	9.90	5.70	5.30	4.90							[CM]
V) WATER RISE ON RIGHT (O - Q)	-8.20	-5.30	-4.90	-4.60							[CM]
W) INITIAL WATER HEAD (N - O)	8.10	7.80	7.50	7.90							[CM]
X) FINAL WATER HEAD (P - Q)	-10.00	-3.20	-2.70	-1.60							[CM]
Y) INITIAL TOTAL HEAD (T + W)	158.10	157.80	157.50	157.90							[CM]
Z) FINAL TOTAL HEAD (T + X)	140.00	146.80	147.30	148.40							[CM]
α) OUTFLOW TO INFLOW RATIO (U / V)	1.21	1.08	1.08	1.07							
PERMEABILITY (F * R) / (2 * H *S) * LN (Y / Z)	2.3E-07	2.3E-07	2.3E-07	2.3E-07							[CM/S]

GROUP DELTA

STANDARD TEST METHOD FOR MEASUREMENT OF HYDRAULIC CONDUCTIVITY OF SATURATED MATERIALS (ASTM D5084)

C:\ANALYSIS\LABPEARM2

PROJECT: Vinje & Middleton
TESTED BY: RHC
SAMPLE: TP-4 @ 4'
Document No. 16-0204

CLIENT: 1382-001-00
CHECKED BY: MAF
DATE: 05/01/09
Project No. SD508

DESCRIPTION: Remolded reddish brown sandy clay (CL) with permeability of 2 * (10^{-8}) cm/s.

FIGURE C-1.2

MOISTURE AND DENSITY

MOISTURE AND DENSITY	INITIAL	FINAL	
A) WET WEIGHT OF SAMPLE	403.56	406.54	[G]
B) DRY WEIGHT OF SAMPLE	358.80	358.80	[G]
C) MOISTURE CONTENT [(A - B) / B]	12.5	13.3	
D) WET DENSITY (A / J * 62.4)	128.8	129.8	[PCF]
E) DRY DENSITY [D / (1 + C)]	114.5	114.5	[PCF]

TEST PARAMETERS

F) STANDPIPE AREAS	0.08	[CM^2]
G) SAMPLE DIAMETER	4.93	[CM]
H) SAMPLE AREA (π * G^2 / 4)	19.09	[CM^2]
I) INITIAL SAMPLE HEIGHT	10.24	[CM]
J) SAMPLE VOLUME (I * H)	195.47	[CM^3]

HYDRAULIC CONDUCTIVITY

HYDRAULIC CONDUCTIVITY	1	2	3	4	5	6	7	8	9	10	
K) CELL PRESSURE	3.000	3.500	3.500	3.500							[KG/CM^2]
L) DRIVING PRESSURE (LEFT)	2.800	3.300	3.300	3.300							[KG/CM^2]
M) BACK PRESSURE (RIGHT)	2.500	3.000	3.000	3.000							[KG/CM^2]
N) INITIAL WATER LEVEL (LEFT)	36.70	36.20	36.20	36.60							[CM]
O) INITIAL WATER LEVEL (RIGHT)	34.50	34.80	34.90	34.90							[CM]
P) FINAL WATER LEVEL (LEFT)	32.40	34.50	26.70	32.60							[CM]
Q) FINAL WATER LEVEL (RIGHT)	35.70	36.20	43.80	38.90							[CM]
R) FINAL SAMPLE HEIGHT	10.22	10.22	10.22	10.22							[CM]
S) TEST DURATION	12780	11880	61980	20760							[S]
T) PRESSURE HEAD [(L - M) * 1000 / 1.0]	300.00	300.00	300.00	300.00							[CM]
U) WATER DROP ON LEFT (N - P)	4.30	1.70	9.50	4.00							[CM]
V) WATER RISE ON RIGHT (O - Q)	-1.20	-1.40	-8.90	-4.00							[CM]
W) INITIAL WATER HEAD (N - O)	2.20	1.40	1.30	1.70							[CM]
X) FINAL WATER HEAD (P - Q)	-3.30	-1.70	-17.10	-6.30							[CM]
Y) INITIAL TOTAL HEAD (T + W)	302.20	301.40	301.30	301.70							[CM]
Z) FINAL TOTAL HEAD (T + X)	296.70	298.30	282.90	293.70							[CM]
α) OUTFLOW TO INFLOW RATIO (U / V)	3.58	1.21	1.07	1.00							
PERMEABILITY (F * R) / (2 * H *S) * LN (Y / Z)	3.0E-08	1.8E-08	2.2E-08	2.7E-08							[CM/S]

GROUP DELTA

STANDARD TEST METHOD FOR MEASUREMENT OF HYDRAULIC CONDUCTIVITY OF SATURATED MATERIALS (ASTM D5084)

C:\ANALYSIS\LABPEARM2

PROJECT: Vinje & Middleton
TESTED BY: RHC
SAMPLE: TP-8 @ 3'
Document No. 16-0204
CLIENT: 1382-001-00
CHECKED BY: MAF
DATE: 05/07/09
Project No. SD508
DESCRIPTION: Remolded dark gray sandy clay (CL) with permeability of 4 * (10^{-7}) cm/s.

FIGURE C-1.3

MOISTURE AND DENSITY

	INITIAL	FINAL	
A) WET WEIGHT OF SAMPLE	347.89	374.10	[G]
B) DRY WEIGHT OF SAMPLE	280.33	280.33	[G]
C) MOISTURE CONTENT [(A - B) / B]	24.1	33.4	
D) WET DENSITY (A / J * 62.4)	111.5	119.9	[PCF]
E) DRY DENSITY [D / (1 + C)]	89.8	89.8	[PCF]

TEST PARAMETERS

F) STANDPIPE AREAS	0.08	[CM^{2}]
G) SAMPLE DIAMETER	4.93	[CM]
H) SAMPLE AREA (π * G^{2} / 4)	19.09	[CM^{2}]
I) INITIAL SAMPLE HEIGHT	10.20	[CM]
J) SAMPLE VOLUME (I * H)	194.71	[CM^{3}]

HYDRAULIC CONDUCTIVITY

	1	2	3	4	5	6	7	8	9	10	
K) CELL PRESSURE	3.000	3.000	3.000	3.000							[KG/CM^{2}]
L) DRIVING PRESSURE (LEFT)	2.805	2.810	2.808	2.808							[KG/CM^{2}]
M) BACK PRESSURE (RIGHT)	2.607	2.612	2.608	2.608							[KG/CM^{2}]
N) INITIAL WATER LEVEL (LEFT)	61.50	61.30	61.30	61.40							[CM]
O) INITIAL WATER LEVEL (RIGHT)	49.80	49.90	50.30	50.40							[CM]
P) FINAL WATER LEVEL (LEFT)	52.70	52.00	53.30	46.80							[CM]
Q) FINAL WATER LEVEL (RIGHT)	57.80	59.30	58.60	64.30							[CM]
R) FINAL SAMPLE HEIGHT	10.39	10.43	10.43	10.43							[CM]
S) TEST DURATION	3720	4473	4320	7920							[S]
T) PRESSURE HEAD [(L - M) * 1000 / 1.0]	198.00	198.00	200.00	200.00							[CM]
U) WATER DROP ON LEFT (N - P)	8.80	9.30	8.00	14.60							[CM]
V) WATER RISE ON RIGHT (O - Q)	-8.00	-9.40	-8.30	-13.90							[CM]
W) INITIAL WATER HEAD (N - O)	11.70	11.40	11.00	11.00							[CM]
X) FINAL WATER HEAD (P - Q)	-5.10	-7.30	-5.30	-17.50							[CM]
Y) INITIAL TOTAL HEAD (T + W)	209.70	209.40	211.00	211.00							[CM]
Z) FINAL TOTAL HEAD (T + X)	192.90	190.70	194.70	182.50							[CM]
α) OUTFLOW TO INFLOW RATIO (U / V)	1.10	0.99	0.96	1.05							
PERMEABILITY (F * R) / (2 * H *S) * LN (Y / Z)	4.8E-07	4.5E-07	4.0E-07	4.0E-07							[CM/S]

GROUP DELTA

December 19, 2017

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

Attention: Michael Grant, President

SUBJECT: **Response to City of Santee Comments – 5th Review**
Prospect Estates II
Santee, California

References: *Updated Geotechnical Investigation, Prospect Estates II Development, Santee, California, Group Delta Consultants, dated May 31, 2017 (Project No. SD508).*

Response to Comments – Update Geotechnical Report, Prospect Estates II, Santee, California, Group Delta Consultants, dated July 24, 2017 (Project No. SD508).

Dear Mr. Grant:

In accordance with your request, Group Delta is providing a response to a review comment prepared by Cecila Tipton, Storm Water Program Manager, City of Santee and submitted in memorandum dated September 29, 2017. Provided below is their comment (in italics) followed by our response.

1. *The follow up investigation conducted by Group Delta and dated July 14, 2017 does not provide the back up documentation required for the test method selected (Borehole Percolation Tests (Riverside (2011)). Please see* ***City of Santee BMP Manual Appendix C.4*** *for the minimum requirements for the geotechnical report, and* ***Appendix D.3*** *for the testing methods. This method is to be used when 'other tests are not possible'. When this method is used under the proper circumstances, the percolation rate obtained must be converted to an infiltration rate using the Porchet method (this calculation is not provided). Additionally, the borehole depth should have been 15 feet; the test was conducted at 10 feet. In addition, tests should be repeated until consistent results are obtained. Only one test was performed for each of two boreholes. See D.3.3.2 Based on existing data, partial infiltration is feasible.*
    a. *Worksheet C.4-1: insufficient documentation to justify outcomes.*

The borehole percolation test was selected based on Appendix D.3 of the City of Santee BMP Manual. The borehole percolation test is suitable at BMP Design Phase investigations when "in areas of proposed cut where other tests are not possible". The bottom of the infiltration basin is proposed approximately ten feet below the existing grade. Therefore, surface tests are not possible at the infiltration elevation. Based on the Tentative Map and Preliminary Grading Plan (Polaris dated June 7, 2017) the base of the infiltration basin is at an elevation of approximately 336 feet above mean sea level (MSL). The existing grade in this area is approximately 346 to 347 feet MSL.

On July 20, 2017, Group Delta drilled two additional borings in the area of the proposed infiltration basin to assess infiltration rates. Previous borings (shown in the referenced report) were extended below the bottom of the proposed basin, to a maximum depth of 16½ feet below existing grade. The additional test borings extended into the granitic rock materials and were terminated approximately near the bottom of the planned infiltration basin, as recommended in the BMP Manual. Following drilling, the borings were converted to test holes and presoaked for about 24 hours. On July 21, 2017, per the City of Santee BMP Manual, Appendix D.4.5, percolation testing was conducted in two test holes within 50 feet of the proposed basin. The Borehole Percolation Test method (Riverside County Percolation Test, 2011) referenced in the City of Santee BMP Design Manual (2016) was used to conduct tests in two boreholes over a six-hour period, while readings were taken every half an hour, as suggested in the Riverside County – Deep Percolation Test method. Repeated measurements showed results were consistent over time.

During the test, water percolated into the surrounding ground both horizontally through the side walls of the hole and vertically through the bottom of the hole. To more accurately approximate the desired vertical infiltration rate, the measured percolation rate was then modified mathematically. The City of Santee BMP Design Manual recommends using a formula called the simplified Porchet method, shown below in Equation 1. The average design infiltration rate was then calculated to be approximately 0.02 inches per hour, assuming a factor of safety of 2.75. The field test data sheets showing the raw field data, as well as all of the input parameters for the Porchet method, and the calculated infiltration rates obtained using the Porchet method, are attached to this letter.

Our conclusion regarding infiltration remains the same as stated in the referenced July 24, 2017 Response to Comment letter with attached Worksheet C.4-1 – Partial Infiltration is Not Feasible. Salient information from this letter is repeated below.

- Partial infiltration is not likely to be possible if the bottom of the basin extends to the decomposed granite. As discussed above, recent field infiltration testing conducted in the vicinity of the proposed infiltration basin resulted in an average design infiltration rate of about 0.02 inches per hour, assuming a factor of safety of 2.75. *We understand the range of acceptable partial infiltration is 0.5 to 0.05 inches/hour (as indicated by Geocon in a comment provided in a letter dated June 20, 2017).*
- The percent fines tests conducted on soils samples obtained at five and 10 feet have fines contents of about 20 percent. Hough (1957) and Hoek and Bray (1977), as reproduced in Hunt (1986)[1], provide a correlation of permeability to soil and rock type respectively (attached). The correlation for "silty sand" estimates a permeability of 0.16 inches per hour. The correlation for "weathered granite" estimates a permeability of 0.14 inches per hour. A factor of safety of 2.0 and 3.0 would reduce the average estimated permeability to 0.07 to 0.05 inches per hour respectively, which is the lower bound of the range of infiltration stated in the comment above.
- We understand the City of Santee Stormwater Design Manual (Manual) recommends a maximum factor of safety of 2.0 for infiltration feasibility screening, but allows selection of a higher factor at the discretion of engineer. We recommend using the higher factor of safety because a potentially "impervious layer" is less than 5 feet from the bottom of the basin. The Manual considers a depth to an impervious layer of less than 5 feet below the bottom of the basin to be a "High Concern". Very dense (SPT blows/foot of 50 for six inches) decomposed granite was sampled at a depth of about 15 feet (elevation of 332 feet, or 4 feet below invert level of infiltration basin).

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[1] Hunt, Roy E. 1986. Geotechnical Engineering Techniques and Practices, McGraw Hill Book Company, First Edition.

Equation 1 (simplified Porchet method):

$$I_t = \frac{\Delta H\, \pi r^2\, 60}{\Delta t(\pi r^2 + 2\pi r H_{avg})} = \frac{\Delta H\, 60\, r}{\Delta t(r + 2H_{avg})}$$

Where:

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

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

GROUP DELTA CONSULTANTS

Charles Robin (Rob) Stroop, G.E. 2298
Associate Geotechnical Engineer

REGISTERED PROFESSIONAL ENGINEER
CHARLES R. STROOP
2298
Exp. 3/31/18
GEOTECHNICAL
STATE OF CALIFORNIA

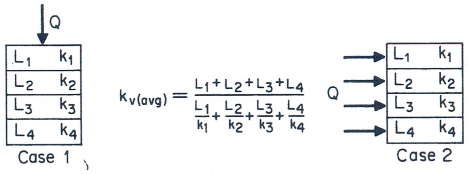
Attachments: Extracts from Geotechnical Engineering Techniques and Practices, (Hunt 1986)
Marrokal Lane Tentative Map (June 7, 2005)
Borehole Percolation Test Data Sheets
Worksheet C.4-1

Distribution: (1) Addressee, Michael Grant (grant.michael@sbcglobal.net)
(2) Joel Waymire, Polaris Development Consultants, Inc. (joel@polarisdc.com)

TABLE 2.8
TYPICAL PERMEABILITY COEFFICIENTS FOR VARIOUS MATERIALS*

	Particle-size range				"Effective" size		Permeability coefficient k		
	Inches		Millimeters						
	D_{max}	D_{min}	D_{max}	D_{min}	D_{20}, in	D_{10}, mm	ft/year	ft/month	cm/s
				TURBULENT FLOW					
Derrick stone	120	36			48		100×10^6	100×10^5	100
One-man stone	12	4			6		30×10^6	30×15^5	30
Clean, fine to coarse gravel	3	¼	80	10	½		10×10^6	10×10^5	10
Fine, uniform gravel	⅜	1/16	8	1.5	⅛		5×10^6	5×10^5	5
Very coarse, clean, uniform sand	⅛	1/32	3	0.8	1/16		3×10^6	3×10^5	3
				LAMINAR FLOW					
Uniform, coarse sand	⅛	1/64	2	0.5		0.6	0.4×10^6	0.4×10^5	0.4
Uniform, medium sand			0.5	0.25		0.3	0.1×10^6	0.1×10^5	0.1
Clean, well-graded sand and gravel			10	0.05		0.1	0.01×10^6	0.01×10^5	0.01
Uniform, fine sand			0.25	0.05		0.06	4000	400	40×10^{-4}
Well-graded, silty sand and gravel			5	0.01		0.02	400	40	4×10^{-4}
Silty sand			2	0.005		0.01	100	10	10^{-4}
Uniform silt			0.05	0.005		0.006	50	5	0.5×10^{-4}
Sandy clay			1.0	0.001		0.002	5	0.5	0.05×10^{-4}
Silty clay			0.05	0.001		0.0015	1	0.1	0.01×10^{-4}
Clay (30 to 50% clay sizes)			0.05	0.0005		0.0008	0.1	0.01	0.001×10^{-4}
Colloidal clay ($-2\mu \leq$ 50%)			0.01	10 Å		40 Å	0.001	10^{-4}	10^{-9}

*From Hough (1957).[10] Reprinted with permission of John Wiley & Sons, Inc.



$$k_{v(avg)} = \frac{L_1+L_2+L_3+L_4}{\frac{L_1}{k_1}+\frac{L_2}{k_2}+\frac{L_3}{k_3}+\frac{L_4}{k_4}}$$

$$k_{h(avg)} = \frac{k_1\ L_1+k_2\ L_2+k_3\ L_3+k_4\ L_4}{L_1+L_2+L_3+L_4}$$

where Q = quantity of flow
L = flow path length
k = coefficient of permeability
NOTE: The electrical analogy:
If $L_1 = L_2 = L_3 = L_4 = 1$, and $k_1 = 1$, $k_2 = 2$, $k_3 = 3$, $k_4 = 4$, then in case 1, $k_{v(avg)} = 1.9$ and in case 2, $k_{h(avg)} = 2.5$

FIG. 2.7 An evaluation of the effect of stratification on permeability. [From Salzman (1974).[11]]

TABLE 2.9
TYPICAL PERMEABILITY COEFFICIENTS FOR ROCK AND SOIL FORMATIONS*

	k, cm/s	Intact rock	Porosity n, %	Fractured rock	Soil
Practically impermeable	10^{-10} 10^{-9} 10^{-8} 10^{-7}	Massive low-porosity rocks	0.1–0.5 0.5–5.0		Homogeneous clay below zone of weathering
Low discharge, poor drainage	10^{-6} 10^{-5} 10^{-4} 10^{-3}	Sandstone Weathered granite Schist	5.0–30.0	Clay-filled joints	Very fine sands, organic and inorganic silts, mixtures of sand and clay, glacial till, stratified clay deposits
High discharge, free draining	10^{-2} 10^{-1} 1.0 10^{1} 10^{2}			Jointed rock Open-jointed rock Heavily fractured rock	Clean sand, clean sand and gravel mixtures Clean gravel

*After Hoek and Bray (1977).[20]

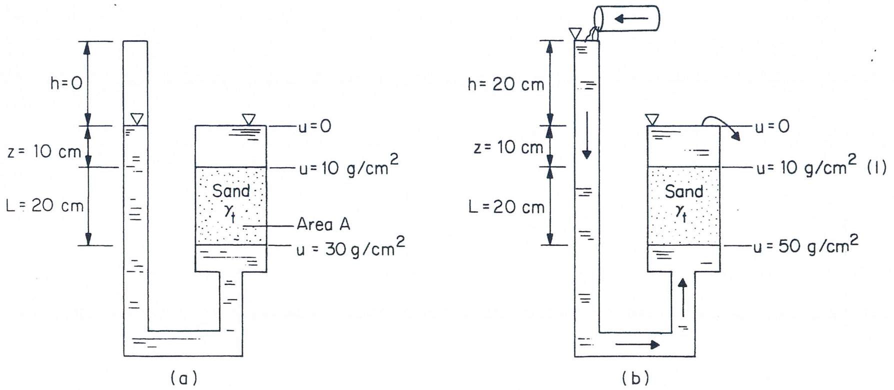


FIG. 2.8 Pore-water pressures. (*a*) No-flow condition. Bouyancy pressures act on each end of the soil specimen. (*b*) Upward flow condition. Boundary pressures act on each end of the specimen. At (1) there is 20 g/cm² lost in seepage.

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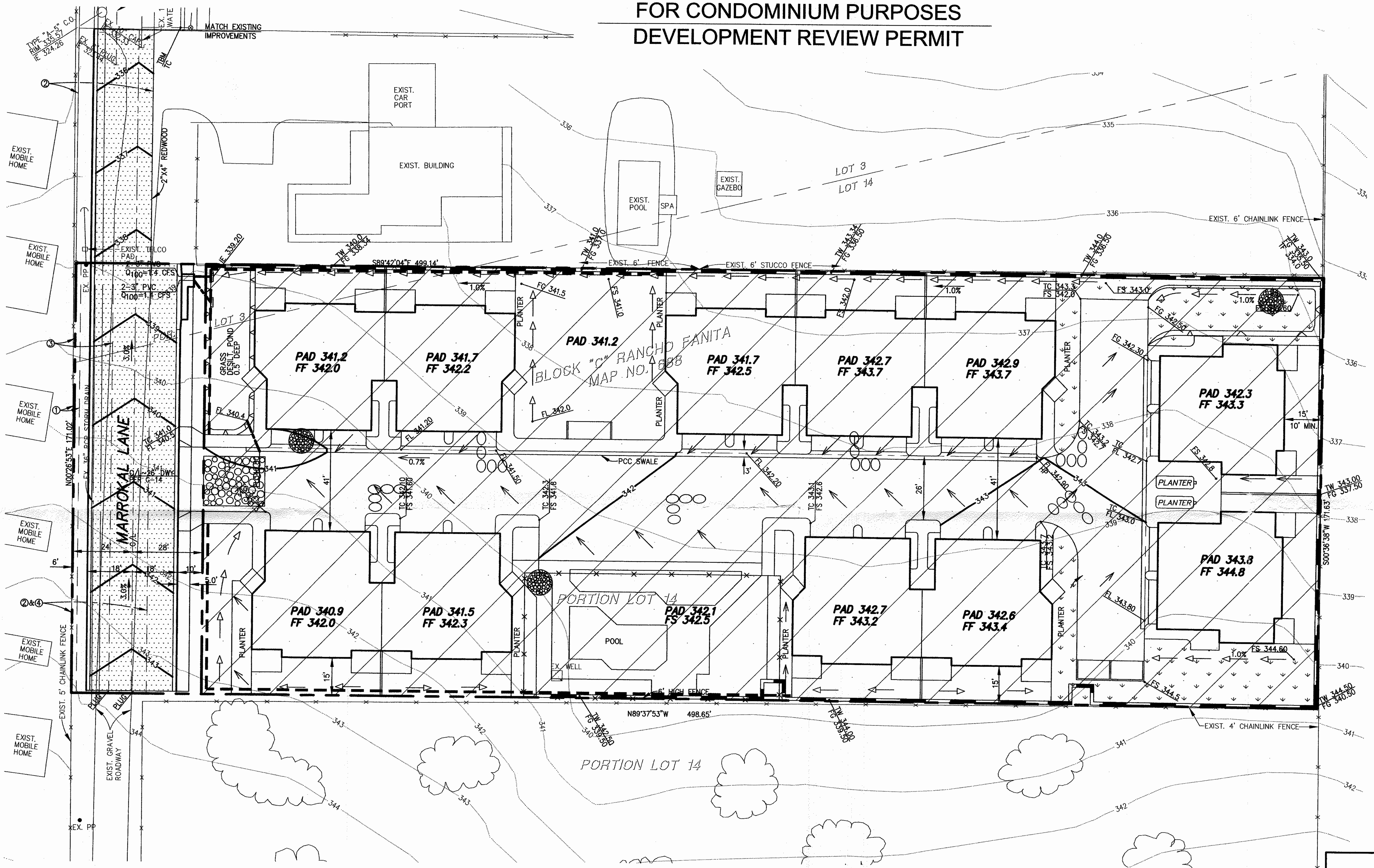
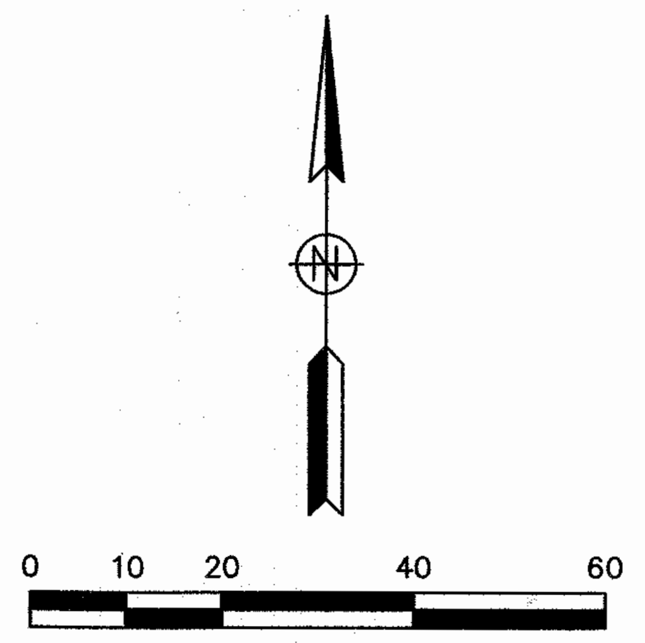
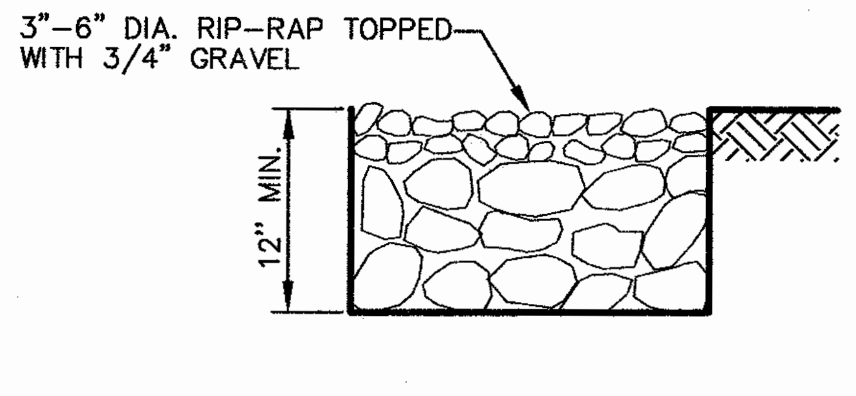
TENTATIVE TRACT MAP
FOR CONDOMINIUM PURPOSES
DEVELOPMENT REVIEW PERMIT



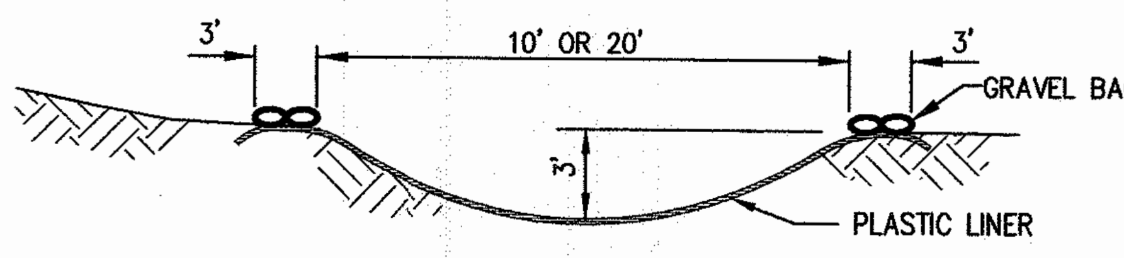


SCALE: 1" = 20'



TEMPORARY 20' W X 20' L ROCK PAD AT ENTRANCE FOR SILT REMOVAL ON TIRES (TYP.)

NO SCALE



10' OR 20' TEMPORARY DESILT POND

NO SCALE

LEGEND

TEMPORARY BMP's

3"-6" DIA. ROCK RIP-RAP PAD	▨
TEMPORARY 10' DIA. DESILTING POND	●
GRAVEL BAGS	○○○
STRAW ROLL	▬ ▬ ▬
SLOPE BINDER, FIBER MULCH OR STRAW MAT	⁘
HYDROSEED OR SHREDDED MULCH	▨

IF HYDROSEED IS NOT ESTABLISHED PRIOR TO RAIN EVENTS AND THERE IS EROSION, INSTALL STRAW MAT ON ALL GRADED SLOPES.

NOTE

ALL AREAS WILL BE COVERED OR RE-APPLIED WITH STRAW MAT OR JUTE MATTING UNTIL LANDSCAPING HAS BEEN ESTABLISHED AND COVERED 90 PERCENT OF EXPOSED DIRT SURFACES.

CITY OF SANTEE
APPROVED

DATE APPROVED 3/14/07

APPROVED BY DEPARTMENT OF DEVELOPMENT SERVICES TM05-05/DR05-07

SIGNATURE Mark Bubb

PG. 1 OF 10

RECEIVED
TM05-05/DR05-07
JUN 16 2005
AEIS 05-14
Dept. of Development Services
City of Santee

PREPARED BY:
NAME: **COOPER ENGINEERING ASSOCIATES**
ADDRESS: 8369 VICKERS ST., STE. C
SAN DIEGO, CA. 92111
PHONE NO.: (858) 277-0441
PROJECT ADDRESS:
MARROKAL LANE
SANTEE, CA
PROJECT NAME:
MARROKAL LANE

REVISION 1: 06-07-05
REVISION 2: ____
REVISION 3: ____
REVISION 4: ____
REVISION 5: ____
REVISION 6: ____
REVISION 7: ____
REVISION 8: ____
REVISION 9: ____
REVISION 10: ____
ORIG. DATE: 05-24-04

CEA
COOPER ENGINEERING ASSOCIATES
CIVIL ENGINEERING PLANNING LAND SURVEYING
8369 VICKERS STREET, SUITE C, SAN DIEGO, CALIFORNIA 92111
PHONE:(858) 277-0441

SHEET TITLE:
EROSION CONTROL PLAN

SHEET C-3 OF 5
P.T.S. NO. ____
PERMIT NO. ____

BOREHOLE PERCOLATION TEST DATA SHEET

Storm Water Infilitration

Project Name: Prospect Estates II
Date Drilled: 7/20/2017
Borehole Radius (*r): 3 in.

Project Number: SD508
Logged By: C. Vonk
Depth of Hole as Drilled: 10.0 ft

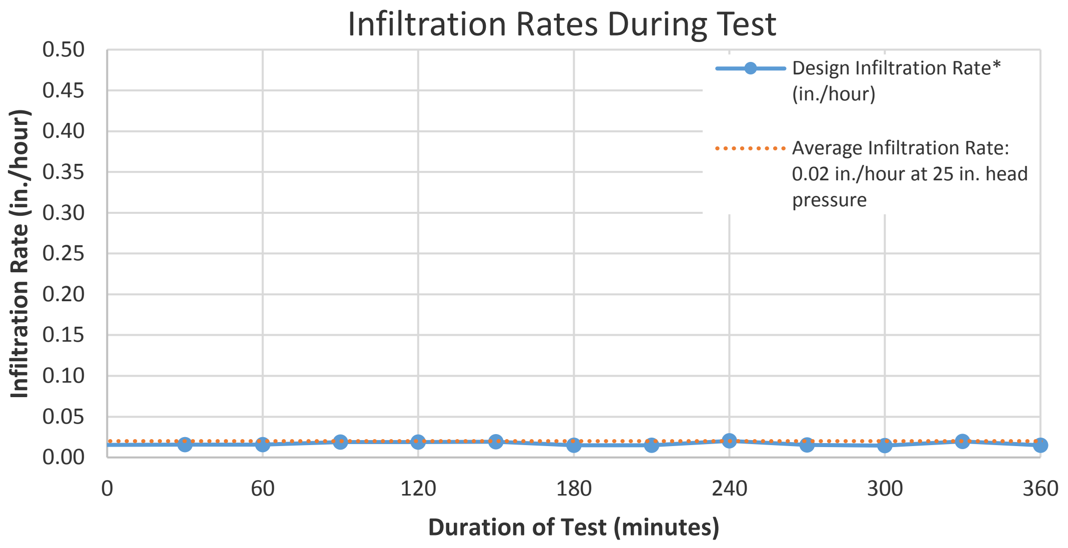
Test Hole No: I-1
Date Tested: 7/21/2017
Casing Stick-up: 0.0 ft

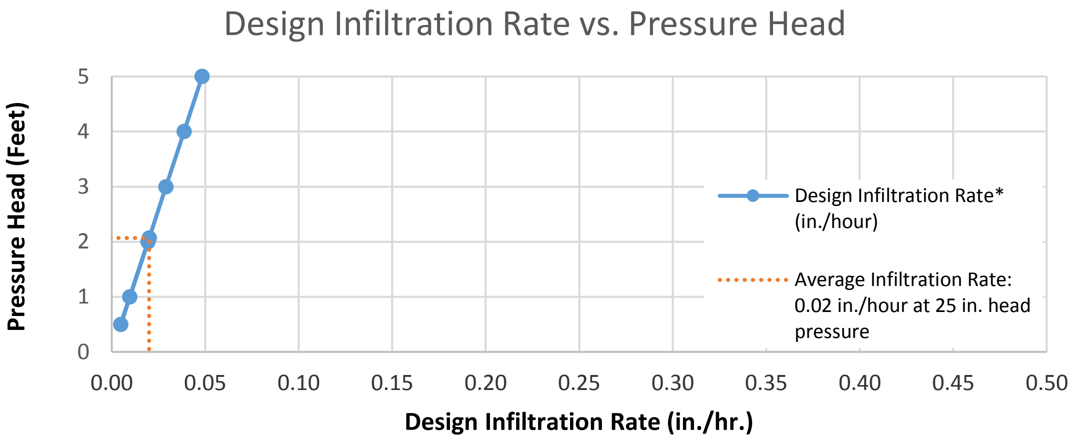
Drilling Method: Hollow Stem Auger
Tested By: C. Vonk
Test Depth: 7.9' - 10'

Reading Number	Time Interval (min.)	Total Depth of Hole (ft.)	Initial Depth of Water (ft.)	Final Depth of Water (ft.)	Change in Water Level (in.)	Average Head of Water (in.)	Unfactored Percolation Rate (in./min.)	Design Infiltration Rate* (in./hour)
	Δt				ΔH	H_{avg}	$\Delta H/\Delta t$	I_t / F.S.*
Pre-Soak	1440	10.00	--	--	--	--	--	--
1	30	10.00	8.01	8.04	0.36	23.70	0.01	0.02
2	30	10.00	8.04	8.07	0.36	23.34	0.01	0.02
3	30	10.00	7.77	7.81	0.48	26.52	0.02	0.02
4	30	10.00	7.81	7.85	0.48	26.04	0.02	0.02
5	30	10.00	7.85	7.89	0.48	25.56	0.02	0.02
6	30	10.00	7.89	7.92	0.36	25.14	0.01	0.01
7	30	10.00	7.92	7.95	0.36	24.78	0.01	0.01
8	30	10.00	7.95	7.99	0.48	24.36	0.02	0.02
9	30	10.00	7.99	8.02	0.36	23.94	0.01	0.02
10	30	10.00	7.85	7.88	0.36	25.62	0.01	0.01
11	30	10.00	7.88	7.92	0.48	25.20	0.02	0.02
12	30	10.00	7.92	7.95	0.36	24.78	0.01	0.01

*Results for 25 in. of head pressure. Factor of Safety of 2.75 was used to calculate final values.

Stabilized Infiltration Rate*: 0.02 inch/hour





Factor of Safety and Design Infiltration Rate Worksheet			**Worksheet D.5-1***		
Factor Category		**Factor Description**	**Assigned Weight (w)**	**Factor Value (v)**	**Product (p) p = w * v**
A	**Suitability Assessment**	*Soil Assessment Methods*	0.25	2	0.5
		Predominant soil texture	0.25	1	0.25
		Site soil variability	0.25	1	0.25
		Depth to groundwater/impervious layer	0.25	3	0.75
		Suitability Assessment Safety Factor, $S_A = \Sigma p$			1.75
B	**Design**	*Level of pretreatment / expected loads*	0.5	1	0.5
		Redundancy / resiliency	0.25	1	0.25
		Compaction during construction	0.25	1	0.25
		Design Safety Factor, $S_D = \Sigma p$			1
		Combined Safety Factor, $S_{tot} = S_A * S_D$			**2.75**

*Reference: Model BMP Design Manual, San Diego Region (2016).

BOREHOLE PERCOLATION TEST DATA SHEET

Storm Water Infiltration

Project Name: Prospect Estates II
Date Drilled: 7/20/2017
Borehole Radius (*r): 3 in.
Project Number: SD508
Logged By: C. Vonk
Depth of Hole as Drilled: 10.0 ft
Test Hole No: I-2
Date Tested: 7/21/2017
Casing Stick-up: 0.3 ft
Drilling Method: Hollow Stem Auger
Tested By: C. Vonk
Test Depth: 7.2' - 9.7'

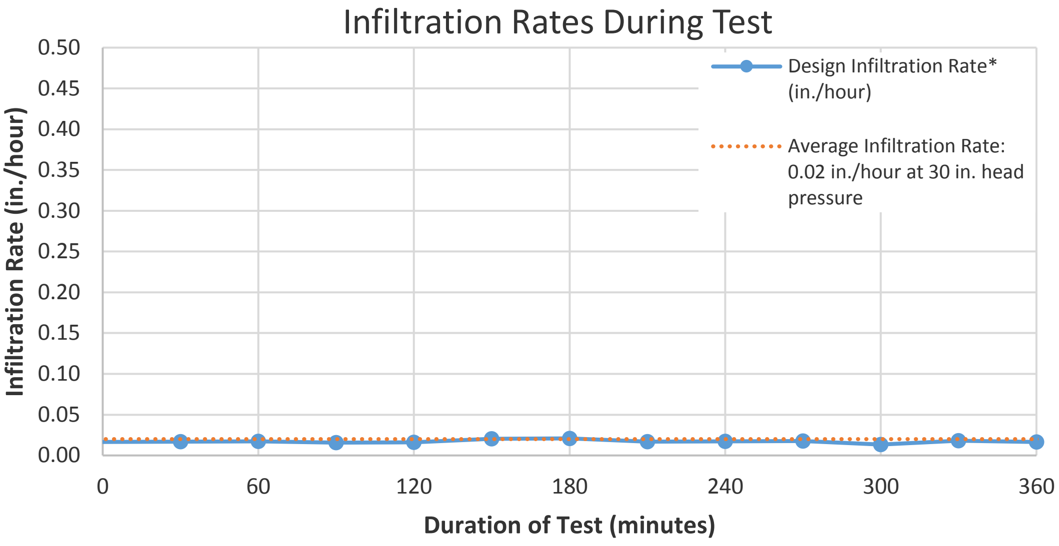
Reading Number	Time Interval (min.)	Total Depth of Hole (ft.)	Initial Depth of Water (ft.)	Final Depth of Water (ft.)	Change in Water Level (in.)	Average Head of Water (in.)	Unfactored Percolation Rate (in./min.)	Design Infiltration Rate* (in./hour)
	Δt				ΔH	H_{avg}	$\Delta H/\Delta t$	I_t / F.S.*
Pre-Soak	1440	9.67	--	--	--	--	--	--
1	30	9.67	7.21	7.25	0.48	29.28	0.02	0.02
2	30	9.67	7.25	7.29	0.48	28.80	0.02	0.02
3	30	9.67	7.03	7.07	0.48	31.44	0.02	0.02
4	30	9.67	7.07	7.11	0.48	30.96	0.02	0.02
5	30	9.67	7.11	7.16	0.60	30.42	0.02	0.02
6	30	9.67	7.16	7.21	0.60	29.82	0.02	0.02
7	30	9.67	7.21	7.25	0.48	29.28	0.02	0.02
8	30	9.67	7.25	7.29	0.48	28.80	0.02	0.02
9	30	9.67	7.29	7.33	0.48	28.32	0.02	0.02
10	30	9.67	7.33	7.36	0.36	27.90	0.01	0.01
11	30	9.67	7.36	7.40	0.48	27.48	0.02	0.02
12	30	9.67	7.15	7.19	0.48	30.00	0.02	0.02

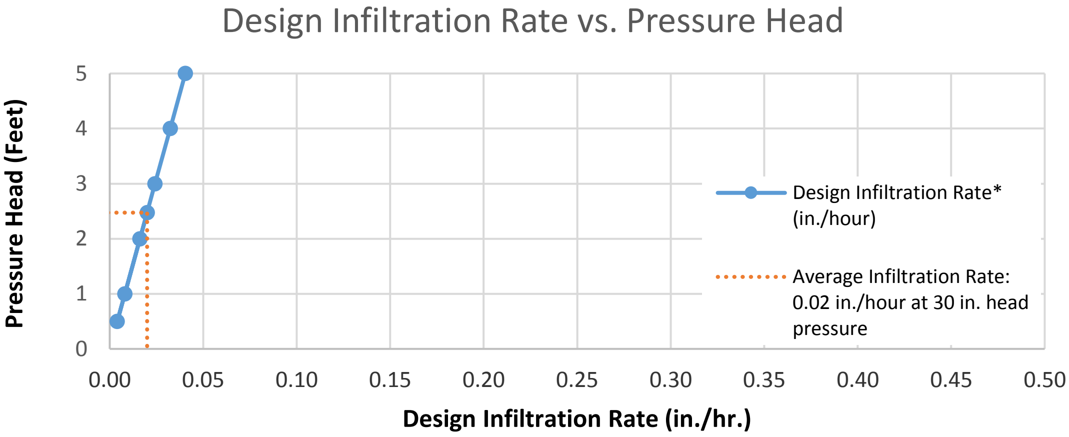
*Results for 30 in. of head pressure. Factor of Safety of 2.75 was used to calculate final values.

Stabilized Infiltration Rate*: 0.02 inch/hour

GROUP DELTA

I-2





Factor of Safety and Design Infiltration Rate Worksheet			**Worksheet D.5-1***		
Factor Category		**Factor Description**	**Assigned Weight (w)**	**Factor Value (v)**	**Product (p) p = w * v**
A	**Suitability Assessment**	*Soil Assessment Methods*	0.25	2	0.5
		Predominant soil texture	0.25	1	0.25
		Site soil variability	0.25	1	0.25
		Depth to groundwater/impervious layer	0.25	3	0.75
		Suitability Assessment Safety Factor, $S_A = \Sigma p$			1.75
B	**Design**	*Level of pretreatment / expected loads*	0.5	1	0.5
		Redundancy / resiliency	0.25	1	0.25
		Compaction during construction	0.25	1	0.25
		Design Safety Factor, $S_D = \Sigma p$			1
		Combined Safety Factor, $S_{tot} = S_A * S_D$			**2.75**

*Reference: Model BMP Design Manual, San Diego Region (2016).

Worksheet C.4-1: Categorization of Infiltration Feasibility Condition

Categorization of Infiltration Feasibility Condition			Worksheet C.4-1
Part 1 - Full Infiltration Feasibility Screening Criteria **Would infiltration of the full design volume be feasible from a physical perspective without any undesirable consequences that cannot be reasonably mitigated?**			
Criteria	Screening Question	**Yes**	**No**
1	**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.		No
Provide basis: The on-site soils generally consist of sandy lean to fat clay (CL to CH). These fine grained soils have a very low permeability (roughly 10-7 cm/s or less), and would not permit infiltration at a rate of 0.5 inches per hour. In addition, recent infiltration testing performed at the siteresulted in an average design infiltration rate of approximately 0.02 inches per hour. Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.			
2	**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.		No
Provide basis: Site to the north is undeveloped. Residential development is planned (see attached Tentative Map). Partial infiltration could negatively impact the foundations of perimeter retaining walls or other improvements close to the proposed basin. It could also create an undesirable long term liability exposure to the developers/owners of the Prospect Estates II project. Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.			

Worksheet C.4-1 Page 2 of 4

Criteria	Screening Question	Yes	No
3	**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.	Yes	

Provide basis:

Shallow groundwater is not present.

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

Criteria	Screening Question	Yes	No
4	**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.	Yes	

Provide basis:

Sources of surface waters are not nearby.

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

Part 1 Result*	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	NO

*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.		No

Provide basis:

Partial infiltration is not likely to be possible if the bottom of the basin extends to the decomposed granite. Percent fines tests conducted on soils samples obtained at five and 10 feet have fines contents of about 20 percent. Hough (1957) and Hoek and Bray (1977), as reproduced in Hunt (1986) , provide a correlation of permeability to soil and rock type respectively (attached). The correlation for “silty sand” estimates a permeability of 0.16 inches per hour. The correlation for “weathered granite” estimates a permeability of 0.14 inches per hour. A factor of safety of 2.0 and 3.0 would reduce the average estimated permeability to 0.07 to 0.05 inches per hour respectively, which is the lower bound of the range of infiltration stated in the comment above. We understand the City of Santee Stormwater Design Manual (Manual) recommends a maximum factor of safety of 2.0 for infiltration feasibility screening, but allows selection of a higher factor at the discretion of engineer. We recommend using the higher factor of safety because a potentially “impervious layer” is less than 5 feet from the bottom of the basin. The Manual considers a depth to an impervious layer of <5 feet below the bottom of the basin to be a “High Concern”. Very dense (SPT blows/foot of 50 for six inches) decomposed granite was logged at a depth of about 15 feet (elevation of 332 feet, or 4 feet below invert level of infiltration basin). Recent infiltration testing resulted in an average design infiltration rate of approximately 0.02 inches per hour.

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.		No

Provide basis:

Site to the north is undeveloped. Residential development is planned (see attached Tentative Map).
Partial infiltration could negatively impact the foundations of perimeter retaining walls or other improvements close to the proposed basin. It could also create an undesirable long term liability exposure to the developers/owners of the Prospect Estates II project.

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.

<table>
<tr><th colspan="4">Worksheet C.4-1 Page 4 of 4</th></tr>
<tr><th>Criteria</th><th>Screening Question</th><th>Yes</th><th>No</th></tr>
<tr><td>7</td><td>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.</td><td>Yes</td><td></td></tr>
<tr><td colspan="4">Provide basis:
Shallow groundwater is not present.

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.</td></tr>
<tr><td>8</td><td>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.</td><td>Yes</td><td></td></tr>
<tr><td colspan="4">Provide basis:
Sources of surface waters are not nearby.

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.</td></tr>
<tr><td>Part 2 Result*</td><td>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.</td><td colspan="2">NO</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 Agency/Jurisdictions to substantiate findings

GROUP DELTA

STANDARD TEST METHOD FOR MEASUREMENT OF HYDRAULIC CONDUCTIVITY OF SATURATED MATERIALS (ASTM D5084)

C:\ANALYSIS\LABPEARM2

PROJECT: Alberhill Clay and Aggregate Quarry TESTED BY: RHC SAMPLE: Olive #17 Document No. 16-0204
CLIENT: Pacific Aggregates CHECKED BY: MAF DATE: 06/21/10 Project No. SD508
DESCRIPTION: Remolded dark yellowish brown sandy lean clay (CL) with permeability of $2*(10^{-7})$ cm/s **FIGURE C-1.1**

MOISTURE AND DENSITY

	INITIAL	FINAL	
A) WET WEIGHT OF SAMPLE	373.20	405.60	[G]
B) DRY WEIGHT OF SAMPLE	329.80	329.80	[G]
C) MOISTURE CONTENT [(A - B) / B]	13.2	23.0	
D) WET DENSITY (A / J * 62.4)	119.0	129.4	[PCF]
E) DRY DENSITY [D / (1 + C)]	105.2	105.2	[PCF]

TEST PARAMETERS

F) STANDPIPE AREAS	0.08	[CM^2]
G) SAMPLE DIAMETER	4.93	[CM]
H) SAMPLE AREA $(\pi * G^2 / 4)$	19.09	[CM^2]
I) INITIAL SAMPLE HEIGHT	10.25	[CM]
J) SAMPLE VOLUME (I * H)	195.66	[CM^3]

HYDRAULIC CONDUCTIVITY

	1	2	3	4	5	6	7	8	9	10	
K) CELL PRESSURE	1.500	1.500	1.500	1.500							[KG/CM^2]
L) DRIVING PRESSURE (LEFT)	1.300	1.300	1.300	1.300							[KG/CM^2]
M) BACK PRESSURE (RIGHT)	1.150	1.150	1.150	1.150							[KG/CM^2]
N) INITIAL WATER LEVEL (LEFT)	44.40	44.10	43.90	44.30							[CM]
O) INITIAL WATER LEVEL (RIGHT)	36.30	36.30	36.40	36.40							[CM]
P) FINAL WATER LEVEL (LEFT)	34.50	38.40	38.60	39.40							[CM]
Q) FINAL WATER LEVEL (RIGHT)	44.50	41.60	41.30	41.00							[CM]
R) FINAL SAMPLE HEIGHT	10.26	10.26	10.26	10.26							[CM]
S) TEST DURATION	11400	6660	6300	5700							[S]
T) PRESSURE HEAD [(L - M) * 1000 / 1.0]	150.00	150.00	150.00	150.00							[CM]
U) WATER DROP ON LEFT (N - P)	9.90	5.70	5.30	4.90							[CM]
V) WATER RISE ON RIGHT (O - Q)	-8.20	-5.30	-4.90	-4.60							[CM]
W) INITIAL WATER HEAD (N - O)	8.10	7.80	7.50	7.90							[CM]
X) FINAL WATER HEAD (P - Q)	-10.00	-3.20	-2.70	-1.60							[CM]
Y) INITIAL TOTAL HEAD (T + W)	158.10	157.80	157.50	157.90							[CM]
Z) FINAL TOTAL HEAD (T + X)	140.00	146.80	147.30	148.40							[CM]
α) OUTFLOW TO INFLOW RATIO (U / V)	1.21	1.08	1.08	1.07							
PERMEABILITY (F * R) / (2 * H *S) * LN (Y / Z)	2.3E-07	2.3E-07	2.3E-07	2.3E-07							[CM/S]

GROUP DELTA

STANDARD TEST METHOD FOR MEASUREMENT OF HYDRAULIC CONDUCTIVITY OF SATURATED MATERIALS (ASTM D5084)

C:\ANALYSIS\LABPEARM2

PROJECT: Vinje & Middleton TESTED BY: RHC SAMPLE: TP-4 @ 4' Document No. 16-0204

CLIENT: 1382-001-00 CHECKED BY: MAF DATE: 05/01/09 Project No. SD508

DESCRIPTION: Remolded reddish brown sandy clay (CL) with permeability of 2 * (10^{-8}) cm/s.

FIGURE C-1.2

MOISTURE AND DENSITY

MOISTURE AND DENSITY	INITIAL	FINAL	
A) WET WEIGHT OF SAMPLE	403.56	406.54	[G]
B) DRY WEIGHT OF SAMPLE	358.80	358.80	[G]
C) MOISTURE CONTENT [(A - B) / B]	12.5	13.3	
D) WET DENSITY (A / J * 62.4)	128.8	129.8	[PCF]
E) DRY DENSITY [D / (1 + C)]	114.5	114.5	[PCF]

TEST PARAMETERS

F) STANDPIPE AREAS	0.08	[CM^2]
G) SAMPLE DIAMETER	4.93	[CM]
H) SAMPLE AREA (π * G^2 / 4)	19.09	[CM^2]
I) INITIAL SAMPLE HEIGHT	10.24	[CM]
J) SAMPLE VOLUME (I * H)	195.47	[CM^3]

HYDRAULIC CONDUCTIVITY

HYDRAULIC CONDUCTIVITY	1	2	3	4	5	6	7	8	9	10	
K) CELL PRESSURE	3.000	3.500	3.500	3.500							[KG/CM^2]
L) DRIVING PRESSURE (LEFT)	2.800	3.300	3.300	3.300							[KG/CM^2]
M) BACK PRESSURE (RIGHT)	2.500	3.000	3.000	3.000							[KG/CM^2]
N) INITIAL WATER LEVEL (LEFT)	36.70	36.20	36.20	36.60							[CM]
O) INITIAL WATER LEVEL (RIGHT)	34.50	34.80	34.90	34.90							[CM]
P) FINAL WATER LEVEL (LEFT)	32.40	34.50	26.70	32.60							[CM]
Q) FINAL WATER LEVEL (RIGHT)	35.70	36.20	43.80	38.90							[CM]
R) FINAL SAMPLE HEIGHT	10.22	10.22	10.22	10.22							[CM]
S) TEST DURATION	12780	11880	61980	20760							[S]
T) PRESSURE HEAD [(L - M) * 1000 / 1.0]	300.00	300.00	300.00	300.00							[CM]
U) WATER DROP ON LEFT (N - P)	4.30	1.70	9.50	4.00							[CM]
V) WATER RISE ON RIGHT (O - Q)	-1.20	-1.40	-8.90	-4.00							[CM]
W) INITIAL WATER HEAD (N - O)	2.20	1.40	1.30	1.70							[CM]
X) FINAL WATER HEAD (P - Q)	-3.30	-1.70	-17.10	-6.30							[CM]
Y) INITIAL TOTAL HEAD (T + W)	302.20	301.40	301.30	301.70							[CM]
Z) FINAL TOTAL HEAD (T + X)	296.70	298.30	282.90	293.70							[CM]
α) OUTFLOW TO INFLOW RATIO (U / V)	3.58	1.21	1.07	1.00							
PERMEABILITY (F * R) / (2 * H *S) * LN (Y / Z)	3.0E-08	1.8E-08	2.2E-08	2.7E-08							[CM/S]

GROUP DELTA

STANDARD TEST METHOD FOR MEASUREMENT OF HYDRAULIC CONDUCTIVITY OF SATURATED MATERIALS (ASTM D5084)

C:\ANALYSIS\LABPEARM2

PROJECT: Vinje & Middleton
TESTED BY: RHC
SAMPLE: TP-8 @ 3'
Document No. 16-0204

CLIENT: 1382-001-00
CHECKED BY: MAF
DATE: 05/07/09
Project No. SD508

DESCRIPTION: Remolded dark gray sandy clay (CL) with permeability of 4 * (10^{-7}) cm/s.

FIGURE C-1.3

MOISTURE AND DENSITY

	INITIAL	FINAL	
A) WET WEIGHT OF SAMPLE	347.89	374.10	[G]
B) DRY WEIGHT OF SAMPLE	280.33	280.33	[G]
C) MOISTURE CONTENT [(A - B) / B]	24.1	33.4	
D) WET DENSITY (A / J * 62.4)	111.5	119.9	[PCF]
E) DRY DENSITY [D / (1 + C)]	89.8	89.8	[PCF]

TEST PARAMETERS

F) STANDPIPE AREAS	0.08	[CM^2]
G) SAMPLE DIAMETER	4.93	[CM]
H) SAMPLE AREA (π * G^2 / 4)	19.09	[CM^2]
I) INITIAL SAMPLE HEIGHT	10.20	[CM]
J) SAMPLE VOLUME (I * H)	194.71	[CM^3]

HYDRAULIC CONDUCTIVITY

	1	2	3	4	5	6	7	8	9	10	
K) CELL PRESSURE	3.000	3.000	3.000	3.000							[KG/CM^2]
L) DRIVING PRESSURE (LEFT)	2.805	2.810	2.808	2.808							[KG/CM^2]
M) BACK PRESSURE (RIGHT)	2.607	2.612	2.608	2.608							[KG/CM^2]
N) INITIAL WATER LEVEL (LEFT)	61.50	61.30	61.30	61.40							[CM]
O) INITIAL WATER LEVEL (RIGHT)	49.80	49.90	50.30	50.40							[CM]
P) FINAL WATER LEVEL (LEFT)	52.70	52.00	53.30	46.80							[CM]
Q) FINAL WATER LEVEL (RIGHT)	57.80	59.30	58.60	64.30							[CM]
R) FINAL SAMPLE HEIGHT	10.39	10.43	10.43	10.43							[CM]
S) TEST DURATION	3720	4473	4320	7920							[S]
T) PRESSURE HEAD [(L - M) * 1000 / 1.0]	198.00	198.00	200.00	200.00							[CM]
U) WATER DROP ON LEFT (N - P)	8.80	9.30	8.00	14.60							[CM]
V) WATER RISE ON RIGHT (O - Q)	-8.00	-9.40	-8.30	-13.90							[CM]
W) INITIAL WATER HEAD (N - O)	11.70	11.40	11.00	11.00							[CM]
X) FINAL WATER HEAD (P - Q)	-5.10	-7.30	-5.30	-17.50							[CM]
Y) INITIAL TOTAL HEAD (T + W)	209.70	209.40	211.00	211.00							[CM]
Z) FINAL TOTAL HEAD (T + X)	192.90	190.70	194.70	182.50							[CM]
α) OUTFLOW TO INFLOW RATIO (U / V)	1.10	0.99	0.96	1.05							
PERMEABILITY (F * R) / (2 * H *S) * LN (Y / Z)	4.8E-07	4.5E-07	4.0E-07	4.0E-07							[CM/S]

GROUP DELTA

June 5, 2018

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

Attention: Michael Grant, President

SUBJECT: **Expansion Soil Mitigation with On-Site Soils**
Prospect Estates II
Santee, California

Reference: *Updated Geotechnical Investigation, Prospect Estates II Development, Santee, California, Group Delta Consultants, dated May 31, 2017 (Project No. SD508)*

Dear Mr. Grant:

In accordance with your request, Group Delta Consultants, Inc. (Group Delta) is submitting this letter describing the use of on-site soils for mitigating expansive soil heave.

SITE EXPANSIVE SOIL CONDITIONS

Subsurface exploration completed for the referenced geotechnical investigation indicated that variable depth colluvium and undocumented fill[1] cover the site, which is underlain at depth by claystone belonging to the Friars Formation. Laboratory testing of soil samples indicate the colluvium should possess a “High” expansion potential and the undocumented fill should possess a “Medium” expansion potential when these soils are reused as compacted fill. Laboratory testing of the soils derived from the claystone belonging to the Friars Formation indicate it should possess a High expansion potential if excavated and reused as compacted fill. However, large excavations in the Friars Formation are not expected.

EXPANSIVE SOIL MITIGATION USING ON-SITE SOILS

The referenced geotechnical report recommended mitigating expansive soil heave by selectively grading the site so that soils with a lower potential for expansion are used within the upper three feet of subgrade below all single family residential buildings, as well as the surrounding concrete sidewalks and driveways. For preliminary design, the report recommended targeting an Expansion Index (EI) of 70 or less (EI<70), which is the median of the range specified for Medium potential expansion soils (EI of 51 to 90). This process combined with post-tensioned slab foundations can accommodate an increased potential expansion since the foundation design uses the specific as-graded expansion profile. The current Post-Tensioning Institute (PTI) design method estimates differential swell based on comprehensive laboratory testing of soil samples obtained from the upper nine feet of the as-grade soil profile.

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[1] Soil placed and compacted in an uncontrolled manner with no documentation of observation and compaction testing by a Geotechnical Engineer.

The soil used for the three-foot cap could be derived from cut excavations within the sandy portions of the undocumented fill and colluvium, where the potential expansion is lower. The sandy soil could also be mixed and blended with some of the higher potential expansion derived from cut excavation to meet the target EI. Depending on the actual quantity of sandy soils available, an alternative method would be to use the sandy materials to create a cap of low expansive soils below the lightly loaded exterior surface improvements only (i.e. the concrete sidewalks and driveways), and to then design the foundations for the residences using the PTI design method considering the higher differential swell estimated in the upper nine feet of the as-graded soil profile. The latter method would require more comprehensive planning of mining, stockpiling and processing of the on-site soils during earthwork, and it could result in tighter spacing of the post-tensioning tendons within the slab foundations. It may also require more careful detailing of the exterior surface improvements.

CLOSURE

As summarized in this letter, it is our opinion the detrimental effects of expansive soil heave below building locations should be effectively mitigated using soil mixing/blending along with post tensioned foundations to provide satisfactory long-term performance. We appreciate this opportunity to be of professional service. Please feel free to contact the office with any questions or comments, or if you need anything else.

GROUP DELTA CONSULTANTS

Charles Robin (Rob) Stroop, G.E. 2298
Associate Geotechnical Engineer

REGISTERED PROFESSIONAL ENGINEER
CHARLES ROBIN STROOP
GE2298
Exp. 03-31-20
GEOTECHNICAL
STATE OF CALIFORNIA

Distribution: (1) Addressee, Michael Grant (grant.michael@sbcglobal.net)

GROUP DELTA

October 9, 2018

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

Attention: Michael Grant, President

SUBJECT: **Addendum No. 1, Updated Geotechnical Investigation**
New Tentative Map, Prospect Estates II
Santee, California

Reference: *Updated Geotechnical Investigation, Prospect Estates II Development, Santee, California, Group Delta Consultants, dated May 31, 2017 (Project No. SD508)*

Tentative Map & Preliminary Grading Plan 2016-13, Prospect Estates II, Polaris Development Consultants, dated September 18, 2018.

Mr. Grant:

Group Delta Consultants is submitting this Addendum to incorporate the above referenced Tentative Map and Preliminary Grading Plan into the geotechnical investigation report. The recommendations in the geotechnical investigation report are valid for the new Tentative Map.

This addendum should be read and bound with the referenced geotechnical report. We appreciate this opportunity to be of continued professional service. Please feel free to contact the office with any questions or comments, or if you need anything else.

GROUP DELTA CONSULTANTS

Charles Robin (Rob) Stroop, G.E. 2298
Associate Geotechnical Engineer

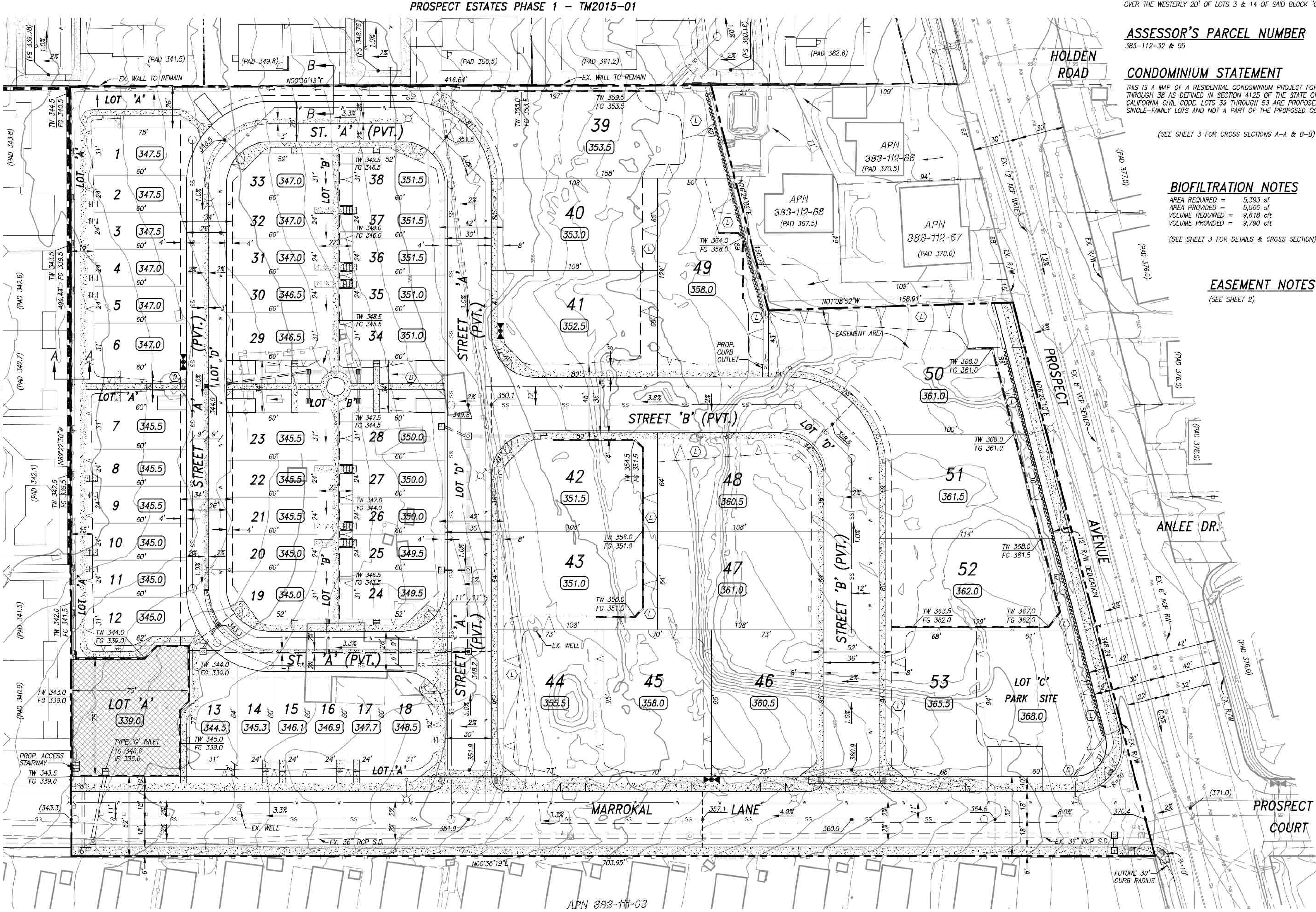
REGISTERED PROFESSIONAL ENGINEER
CHARLES ROBIN STROOP
GE2298
Exp. 03-31-20
GEOTECHNICAL
STATE OF CALIFORNIA

Attachment: Tentative Map & Preliminary Grading Plan 2016-13, Prospect Estates II,
Polaris Development Consultants, dated September 18, 2018

Distribution: (1) Addressee, Michael Grant (grant.michael@sbcglobal.net)

SHEET 1 OF 3

TENTATIVE MAP & PRELIMINARY GRADING PLAN
FOR PROSPECT ESTATES II



LEGAL DESCRIPTION

A PORTION OF LOT 14 IN BLOCK "C" OF FANITA RANCHO, IN THE CITY OF SANTEE, COUNTY OF SAN DIEGO, STATE OF CALIFORNIA, ACCORDING TO MAP THEREOF NO. 688, FILED IN THE OFFICE OF THE COUNTY RECORDER OF SAN DIEGO COUNTY, OCTOBER 22, 1891.

TOGETHER WITH AN EASEMENT FOR RIGHT-OF-WAY AND ROAD PURPOSES OVER THE WESTERLY 20' OF LOTS 3 & 14 OF SAID BLOCK 'C'.

ASSESSOR'S PARCEL NUMBER

383-112-32 & 55

CONDOMINIUM STATEMENT

THIS IS A MAP OF A RESIDENTIAL CONDOMINIUM PROJECT FOR UNITS 1 THROUGH 38 AS DEFINED IN SECTION 4125 OF THE STATE OF CALIFORNIA CIVIL CODE. LOTS 39 THROUGH 53 ARE PROPOSED AS SINGLE-FAMILY LOTS AND NOT A PART OF THE PROPOSED CONDO MAP.

(SEE SHEET 3 FOR CROSS SECTIONS A-A & B-B)

BIOFILTRATION NOTES

AREA REQUIRED =	5,393 sf
AREA PROVIDED =	5,500 sf
VOLUME REQUIRED =	9,618 cft
VOLUME PROVIDED =	9,790 cft

(SEE SHEET 3 FOR DETAILS & CROSS SECTION)

EASEMENT NOTES

(SEE SHEET 2)

LEGEND

SYMBOL	DESCRIPTION
━━━	EXISTING RIGHT OF WAY
– – –	EXISTING LOT LINE
—— ——	EXISTING EASEMENT
———	EXISTING CURB
—330—	EXISTING MAJOR CONTOUR
—333—	EXISTING MINOR CONTOUR
—W—	EXISTING WATER MAIN
—RW—RW—	EXISTING RECYCLED WATER MAIN
—SS—	EXISTING SEWER MAIN
═ ═ ═	EXISTING STORM DRAIN
	EXISTING BUILDING
	EXISTING FIRE HYDRANT
●	EXISTING POWER POLE
—□—□—	EXISTING FENCE
	EXISTING STREET LIGHT
⊗	EX. WATER WELL TO BE ABANDONED PER DEH REQUIREMENTS
━━ ━ ━ ━━	SUBDIVISION BOUNDARY
———	PROPOSED UNIT LINE
———	PROPOSED CURB
—— ——	PROPOSED EASEMENT LINE
—W—	PROPOSED WATER LINE
—SS—	PROPOSED SEWER LINE
═ ═ ═	PROPOSED STORM DRAIN
	PROPOSED MASONRY RETAINING WALL
	PROP. 6' HIGH MASONRY SCREEN WALL
	PROPOSED SLOPE (2:1 U.O.N.)
	PROPOSED DAYLIGHT LINE
▨	PROPOSED CONCRETE PAVING
▨	PROPOSED BIOFILTRATION AREA
	PROPOSED STREET LIGHT
	PROPOSED FIRE HYDRANT
53	PROPOSED UNIT/LOT NUMBER
360.0	PROPOSED PAD ELEVATION
L	PROPOSED SLOPE OVER 3' IN HEIGHT TO BE LANDSCAPED & IRRIGATED
D	PROPOSED DOG WASTE STATION

OWNER/APPLICANT

M. GRANT REAL ESTATE, INC.
NAME: MICHAEL GRANT
ADDRESS: 8520 RAILROAD AVENUE
SANTEE, CA 92071
PHONE: (619) 449-0249

MICHAEL GRANT DATE

ENGINEER OF WORK

POLARIS DEVELOPMENT CONSULTANTS, INC.
2514 JAMACHA ROAD, SUITE 502-31
EL CAJON, CA 92019
(619) 444-2923

NO. 56258
EXP. 12-31-2018

JOEL A. WAYMIRE DATE
R.C.E. 56258 EXP. 12-31-2018

SCALE: 1" = 30'
0 15 30 60 90

Planning ✦ Engineering ✦ Mapping
POLARIS
Development Consultants, Inc.
2514 Jamacha Road, Suite 502-31 • El Cajon, CA 92019 • 619-444-2923

SEPTEMBER 18, 2018

CITY OF SANTEE
TENTATIVE MAP AND PRELIMINARY
GRADING PLAN 2016-03
PROSPECT ESTATES II