GEOTECHNICAL INVESTIGATION
MIXED-INCOME TRANSIT-ORIENTED HOUSING DEVELOPMENT
LA MESA, CALIFORNIA

Prepared for

USA PROPERTIES FUND
3200 Douglas Boulevard, Suite 200
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Project No. SD634
October 14, 2019

GROUP DELTA

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USA Properties Fund
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Attention: Mr. Milo Terzich

SUBJECT: GEOTECHNICAL INVESTIGATION
Mixed-Income Transit-Oriented Housing Development
La Mesa, California

Mr. Terzich:

We are pleased to submit this geotechnical investigation for the proposed mixed-income transit-oriented housing development in the City of La Mesa, California. This report is based on our recent subsurface explorations at the site, the results of field and laboratory tests and geotechnical analyses conducted on samples collected from the borings, and our previous experience with similar materials in the site vicinity. Specific conclusions regarding the potential geologic constraints to site development, and geotechnical recommendations for remedial grading, shoring, foundation, slab, retaining wall, and pavement section design are provided in the following report. The results of our field infiltration tests are also provided.

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 presents our geotechnical investigation for the proposed apartment building development in the City of La Mesa, California. The approximate location of the site is shown in Figure 1A. The site vicinity is shown in more detail in Figure 1B. Selected photographs of the site vicinity are provided in Figures 1C to 1E. A rendering and plan view of the proposed development are provided in Figures 2A and 2B, respectively. The approximate locations of the five exploratory borings and two borehole percolation tests that we conducted at the site for this investigation are shown in Figures 3A and 3B. The approximate site topography is depicted in Figure 3C.

1.1 Scope of Services

This report was prepared in general accordance with the provisions of the referenced proposal (GDC, 2019). The purpose of this investigation was to characterize the general geotechnical constraints to site development and provide geotechnical recommendations for grading and the design of the proposed structure, pavements, utilities, retaining walls and surface improvements. The recommendations provided herein are based on the findings of the subsurface exploration, laboratory tests and engineering analyses, as well as our previous experience with similar geologic conditions in the site vicinity. In summary, we provided the following scope of services.

- A geologic reconnaissance of the surface characteristics of the site and surrounding areas, and a review of relevant reports referenced in Section 8.0.

- A subsurface exploration of the site including 5 exploratory borings in the area of the planned redevelopment. The approximate boring locations are shown on the Exploration Plan, Figure 3A. Boring logs are provided in Appendix A.

- Laboratory testing on selected soil samples collected from the exploratory borings. Laboratory testing included sieve analysis, Atterberg Limits, Expansion Index, moisture content, dry density, soil corrosion, maximum density, optimum moisture, remolded shear and R-Value. The test results are presented in Appendix B.

- Two field infiltration tests were conducted as part of this investigation at the approximate locations shown on the Exploration Plan, Figure 3A. The borehole percolation test results and infiltration assessment are presented in Appendix C.

- Engineering analysis of the field and laboratory data to help develop geotechnical recommendations for site preparation, remedial earthwork, shoring, foundations, pavements and retaining walls, soil reactivity, drainage and moisture protection.

- Preparation of this report summarizing our findings, conclusions and geotechnical recommendations for the proposed site development.

1.2 Site Description

The 1.3-acre site consists of Assessor's Parcel Number (APN) 470-572-22 located within the City of La Mesa, as shown on the Site Location Map, Figure 1A. The site is situated southeast of the intersection between Allison and Date Avenues, as shown on the Site Vicinity Plan, Figure 1B. Photographs depicting the existing condition of the site at the time of our field investigation are provided in Figures 1C and 1D. A photograph of a nearby outcrop showing the typical composition of the formational material underlying the property is provided in Figure 1E.

Much of the site is surfaced with asphalt concrete parking associated with the Police Headquarters building that previously occupied the property until it was demolished in 2012. Aerial photographs showing the site conditions in 2019, as well as the configuration of the Police Headquarters in 2010 are provided in Figures 3A and 3B, respectively. Note that the Police Headquarters building had a subterranean basement that was used as a shooting range. We understand that environmental remediation efforts were undertaken by others when the building was demolished in 2012 in order to remove any remnants of lead bullets that may have remained within the soil after demolition of was completed. The demolished basement excavation was backfilled using imported soil.

The site vicinity slopes gently down to the west as shown in Figure 3C. Existing surface elevations range from a low of about 525 feet above mean sea level (MSL) in the southwest portion of the property, to a high of about 535 feet MSL near the northeast corner of the site. The perimeter of the site is landscaped with a few trees and shrubs. A screen wall from the previous development still borders much of the southern and eastern property boundaries. Various subsurface utility remnants also exist on site from the previous Police Headquarters development, including water, sewer, storm drain, electrical and communication conduits. Note that these utilities appear not to have been completely removed from the site during the previous demolition operations in 2012, as evidenced by the pavement areas which appear to be in roughly the same condition as 2010.

1.3 Proposed Development

An architectural rendering and conceptual plan for the proposed high-density, transit-oriented four-story, 115-unit apartment building are shown in Figures 2A and 2B (USA Properties, 2019). We understand that the development may include a single-level subterranean parking garage beneath portions of the site. Due to the close proximity of the proposed building to the surrounding streets and other existing improvements, the basement excavation would likely be completed using a vertical soldier pile and lagging shoring system without tie-backs. We anticipate that the garage walls, slabs-on-grade and conventional foundations will be constructed using reinforced concrete.

We anticipate that site development will begin with the demolition of the existing pavements and surface improvements, and removal of landscaping vegetation. The existing subsurface utilities will also be removed or relocated. Remedial earthwork will then be conducted to prepare a new pad for the building slab-on-grade. Based on the depths of fill we encountered at the site, we anticipate that the new basement level garage foundations may bear entirely on dense formation.

2.0 FIELD AND LABORATORY INVESTIGATION

The field investigation included a geologic reconnaissance of the site, and the excavation of five exploratory borings between September 19^{th} and 20^{th}, 2019. The maximum depth of exploration was about 20 feet below existing grades. The approximate locations of the borings are shown on the Exploration Plan, Figure 3A. Logs of the borings are provided in the figures of Appendix A.

Soil samples were collected from the borings for laboratory testing. The testing program included gradation analyses and Atterberg Limits to aid in material classification according to the Unified Soil Classification System (USCS). Tests were conducted on relatively undisturbed ring samples to help estimate the in-situ dry density and moisture content of the soils we encountered. The maximum density and optimum moisture of the existing fill was also determined. Index tests were conducted on the bulk samples to help evaluate the soil expansion and corrosivity potential. Direct shear tests were conducted on remolded samples to aid in strength characterization. R-Value tests were conducted to aid in pavement section design. The laboratory test results are shown in Appendix B.

2.1 Infiltration Testing

Two field infiltration tests were conducted as part of this investigation at the approximate locations shown on the Exploration Plan, Figure 3A. The test results are presented in detail in Appendix C. The field infiltration tests indicated a factored vertical infiltration rate of less than 0.05 inches per hour at both test locations. The unfactored infiltration rates varied from 0.00 to 0.01 inches per hour. With a Safety Factor of 2, the average factored infiltration rate was less than 0.01 inches per hour, which is indicative of "No Infiltration" per the City of La Mesa BMP Design guidelines. The infiltration test results may be used by the project civil designer to help evaluate the storm water infiltration measures that may be proposed at the site. Worksheet C.4-1 from the City of La Mesa BMP Design Manual is provided in Appendix C for reference.

The field infiltration test results indicate that neither full nor partial infiltration is feasible at the site, since the equilibrium infiltration rate was measured at less than 0.05 inches per hour. Note that a minimum infiltration rate of 0.50 inches per hour is commonly considered the lower limit for effective implementation of full on-site infiltration measures. The entire site is underlain by very dense sandstone and conglomerate. Our previous experience with permeability testing per ASTM D5084 indicates that such soils typically have a hydraulic conductivity on the order of 10^{-7} cm/s or less. Such materials are essentially impermeable to groundwater infiltration.

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. As observed in our borings, the entire site is underlain by the Eocene-age Stadium Conglomerate (Map Symbol - Tst) to the maximum depth we explored. The conglomerate is covered with shallow fill soils throughout most of the site.

The geologic conditions in the site vicinity are depicted on the Local Geologic Map, Figure 4. The approximate locations of the five borings we conducted at the site are shown on the Exploration Plan, Figure 3A. Logs describing the subsurface conditions we encountered in the borings are provided in Appendix A. The various geologic materials observed at the site are described below.

3.1 Stadium Conglomerate

The Stadium Conglomerate underlies the entire site at depth. This formation primarily consists of massive beds of cobble conglomerate, with occasional interbeds of sandstone. A photograph of a nearby outcrop of the Stadium Conglomerate is provided in Figure 1E. Note that the photograph shows both the conglomerate and sandstone units within the formation. Well-rounded gravel and cobble typically comprise between 30 and 60 percent of the conglomerate by mass. The cobbles are typically 3 to 6 inches in maximum dimension but may include a few boulders up to 18 to 24 inches in diameter. The sandstone and matrix within the formation includes both silty and clayey sandstone (SM and SC), as well as poorly graded sandstone (SP).

The Stadium Conglomerate is dense to very dense in apparent density. Most of the Standard Penetration Test (SPT) blow counts that we collected at the site met with refusal on the gravel and cobble within the massive conglomerate. However, several of the SPT tests were conducted in the sandstone beds within the formation. These SPT tests indicated corrected blow counts (N_{60}) ranging from 69 to 113 and averaging 92 (indicating a very dense condition on average).

Laboratory tests and our previous experience indicate that the formational materials generally have a low expansion potential and negligible soluble sulfate content based on commonly accepted criteria. Our previous experience with remolded tests on samples of the matrix material from this formation suggest that the in-situ shear strength typically exceeds 39° with 200 lb/ft^2 cohesion.

3.2 Fill

Undocumented fill was encountered in all of the borings. A maximum fill depth of about 13½ feet was observed in Boring B-3 in the area where the Police Headquarters basement was previously demolished and backfilled. The fill appears to have been imported to the site, and most commonly consists of clayey sand (SC) with a variable amount of subrounded gravel. The sand was typically fine to medium grained, and the fines were predominantly low in plasticity. The Liquid Limit of the samples we tested varied from 31 to 36, and the Plasticity Index varied from 16 to 20.

The corrected SPT blow counts within the fill (N_{60}) varied from 32 to 47 and averaged 38 (excluding those SPT samples that encountered refusal on gravel or cobble). However, it should be noted that all of the SPT data throughout the site was likely inflated by the presence of gravel. One fill sample that was relatively undisturbed indicated an in-situ density of about 116 lb/ft^3. The rock corrected maximum density for this material is shown in Figure B-4. Based on the gravel content of 18.5 percent for the test specimen, the rock corrected maximum density was 134 lb/ft^3, which indicates a relative compaction for this fill sample of approximately 87 percent.

Borings B-1, B-4, B-5 and I-1 were all situated within paved parking areas. The pavement sections we encountered included 3 or 4 inches of asphalt concrete with no underlying aggregate base. The most common pavement section consisted of 3 inches of asphalt concrete with no base.

3.3 Groundwater

No seepage or groundwater was encountered in our exploratory borings. However, historic well data from the Chevron gasoline station located about 600 feet north of the site (at roughly the same ground surface elevation) indicates that the groundwater table in the site vicinity may be located at roughly 20 to 30 feet below existing grades (SWRCB, 2019). It should be noted that changes in rainfall, irrigation practices or site drainage may produce seepage or locally perched groundwater conditions at any location within the fill or formational units underlying the site. It has been our experience that light to moderate seepage is often encountered at or near the geologic contact between fill and the underlying Stadium Conglomerate. Accordingly, future excavations may encounter zones of wet soil and seepage. Due to the difficulty in predicting the location of perched groundwater, such conditions are typically mitigated if and where they occur.

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 our geotechnical investigation or literature review. The main geologic hazard at the site will be the potential for strong ground motion due to a seismic event on the Rose Canyon fault zone. The strong ground shaking hazard is typically mitigated through structural design of the building in general accordance with the applicable provisions of the current California Building Code. Each of the potential geologic hazards is described in more detail below.

4.1 Ground Rupture

Ground rupture is the result of movement on an active fault reaching the ground surface. The site is not located within an Alquist-Priolo Earthquake Fault Zone. No indications of Holocene-active faulting were found during our site investigation or literature review. The nearest known active faults are located within the Rose Canyon fault zone, roughly 13½ kilometers (km) west of the site, as shown on the Regional Fault Map, Figure 5. Ground rupture is not a geologic hazard at this site.

4.2 Seismicity

The site is located at latitude 32.7657° north and longitude 117.0218° west. The United States Geologic Survey has an interactive website that provides Next Generation Attenuation (NGA) probabilistic seismic analyses based on the site location and shear wave velocity (USGS, 2014). Based on an estimated average shear wave velocity of 360 meters per second, the peak ground accelerations (PGA) with a 2, 5 and 10 percent probability of being exceeded in a 50-year period at the site are estimated at about 0.40g, 0.29g and 0.22g, respectively. These risk levels reflect the Maximum Considered (MCE), Upper Bound (UBE) and Design Basis Earthquakes (DBE), respectively.

By comparison to the probabilistic PGA values noted above, the risk-targeted peak ground accelerations associated with the MCE and DBE risk levels from the 2016 California Building Code (CBC) are 0.369g and 0.246g, 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

The regional groundwater table is estimated at roughly 20 to 30 feet below the existing site grades, based on the available well data described in Section 3.3. However, the entire building will be underlain by very dense Stadium Conglomerate and compacted fill. Given the absence of shallow groundwater and the high density of the underlying materials, the potential for liquefaction and dynamic settlement to adversely affect the planned 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 site reconnaissance. The site slopes gently down to the west, matching the existing grades around the perimeter of the property (see Figure 3C). Provided that our recommendations are implemented during construction, and that shoring is used for vertical basement excavations, it is our opinion that slope instability will not adversely impact the proposed development.

4.5 Tsunamis, Seiches and Flooding

The site is located approximately 13½ miles east of the Pacific Ocean, at an elevation of more than 520 feet above mean sea level (MSL). Given the large distance between the subject site and the coast, and the elevation of the site above mean sea level, the potential for damage due to tsunamis (seismically induced waves) is considered negligible.

The site is not located below any lakes or confined bodies of water and is not located within a FEMA 100-year flood zone. Consequently, the potential for earthquake induced flooding at the site is also considered to be negligible.

5.0 CONCLUSIONS

The proposed development appears to be feasible from a geotechnical perspective, provided that appropriate measures are implemented during design development and earthwork construction. Several geotechnical conditions will need to be addressed.

- Most of the site is covered with a variable depth of potentially compressible undocumented fill soil. During fine grading of the site, all existing fill soils should be excavated and replaced as compacted fill under the observation and testing of the geotechnical consultant. If the new building slab-on-grade is located at or near existing street grades, the cut portion of the building pad may also need to be over-excavated to mitigate the transition. However, if the building is underlain by a basement garage that bears directly on the conglomerate, over-excavation may not be necessary. Therefore, the required remedial grading for the new building pad area will vary depending on the final building design, and the conditions observed by the geotechnical consultant during grading.

- Assuming that the entire building is underlain by a basement garage, we anticipate that the basement foundations will bear entirely on dense Stadium Conglomerate. These materials are very dense and strong, with a low expansion potential and low compressibility. The Stadium Conglomerate is considered suitable for the support of the planned foundations.

- The on-site soils are generally considered suitable for reuse in compacted fills, with the exception of any soils deemed to be contaminated by the environmental consultant. Any contaminated soils should be disposed at an off-site facility in general accordance with the existing Soil Management Plan prepared by the project environmental consultant.

- Laboratory tests indicate that the near surface soils at the site primarily consist of silty and clayey sand (SM and SC) with a low expansion potential. However, it should be noted that some moderately or highly expansive soils may also exist at the site. Additional laboratory testing should be conducted by the geotechnical consultant during fine grading of the site in order to confirm that all fill soil placed beneath the new structure consists of a granular (sandy or gravelly) material with an Expansion Index of less than 50 (EI<50).

- Laboratory tests indicate that the on-site soils likely present a *negligible* potential for sulfate attack. However, these soils still appear to be corrosive to buried metals. Typical corrosion control measures should also be incorporated into the project design. A corrosion consultant may be contacted for specific recommendations.

- The potential for active faults, seismic settlement or floods to adversely impact the planned development is considered remote. Other hazards that may impact site development include strong ground shaking from an earthquake on a nearby active fault. This hazard may be mitigated by structural design in accordance with the applicable building code.

6.0 RECOMMENDATIONS

The remainder of this report presents recommendations for earthwork construction and the design of 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, please contact our office for revisions or amendments.

6.1 Plan Review

We recommend that the demolition, grading and foundation plans be reviewed by Group Delta Consultants prior to construction. We anticipate that substantial changes in the development may occur from the preliminary design concepts used for this investigation. Such changes may require additional geotechnical evaluation, which may result in substantial modifications to the remedial grading and foundation recommendations provided in this report.

6.2 Excavation and Grading Observation

Foundation and grading excavations should be observed by the project geotechnical consultant. During grading, the geotechnical engineer's representative 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. The recommendations provided in this report 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 current California Building Code, the City of La Mesa, and the earthwork recommendations provided within this report. The following recommendations are provided regarding specific aspects of the proposed earthwork. These recommendations should be considered subject to revision based on the conditions observed by the geotechnical consultant during grading.

6.3.1 Site Preparation

General site preparation should begin with the removal of deleterious materials, including any existing structures, walls, foundations, concrete slabs, asphalt concrete pavements, landscaping vegetation, contaminated soil and demolition debris. Existing subsurface utilities that will be abandoned should be removed and the excavations backfilled and compacted as described in Section 6.3.4. Alternatively, abandoned pipes may be grouted with a two-sack sand-cement slurry under the observation of the project geotechnical consultant.

6.3.2 Improvement Areas

A minimum of two feet of material with an Expansion Index of 50 or less is recommended beneath all new sidewalks, courtyards, exterior flatwork areas and building slabs-on-grade. In order to accomplish this objective, the upper 12-inches of soil below the slab subgrade elevations may be excavated and stockpiled on site. The exposed subgrade should then be observed and tested by Group Delta. If formational material is exposed, no additional remediation excavation will be needed. If soil with an Expansion Index above 50 is encountered, the expansive soil should be excavated and replaced with low expansion material. Immediately prior to placing concrete or base within new improvement areas, the subgrade soil should be scarified 12 inches, brought to above optimum moisture content, and compacted as described in Section 6.3.4.

6.3.3 Building Areas

There are several potential geotechnical constraints within the proposed building area. The impact of these constraints will vary depending on the final elevation selected for the building slab-on-grade. The potential constraints include the presence of undocumented fill and expansive soil, and the presence of transitions between cut and fill beneath the new building slab. As a minimum, all existing undocumented fill beneath the new building perimeter should be excavated and replaced with compacted fill. At least two feet of low expansion material (with an Expansion Index of 50 or less) is recommended beneath the new building slab-on-grade, as described in Section 6.3.2.

We anticipate that the entire building may be underlain by a subterranean parking garage that will extend 12 or more feet below surrounding street grades. In this case, most of the backfill for the Police Headquarters basement will be removed by the planned basement excavation, and all of the new basement level foundations may bear directly on dense formation. However, if portions of the new building slab-on-grade are situated at or near street level, portions of the structure would be situated over undocumented fill, while others may bear directly on formation. In this case, the building pad should be over-excavated to a depth of H/2, where H is the maximum fill depth beneath the slab as determined by the geotechnical consultant during grading. The over-excavation should be at least 3 feet deep and need not extend more than 10 feet below slab subgrade elevation, as shown on the Transition Details, Figure 6. The stockpiled soil that is free of deleterious materials may then be replaced as uniformly compacted fill to the planned finish pad grades.

6.3.4 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 per ASTM D1557. All fill should be compacted at slightly above optimum moisture content per ASTM D1557. Sufficient observation and testing should be performed by the geotechnical consultant during grading so that an opinion can be rendered as to the compaction achieved. Rocks or concrete fragments greater than 6 inches in maximum dimension should not be used in structural compacted 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 import should be tested by the geotechnical consultant 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. The geotechnical consultant should be notified to evaluate the suitability of these soils.

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

6.3.5 Subgrade Stabilization

All excavation bottoms should be firm and unyielding prior to placing fill. In areas of saturated or "pumping" subgrade, a geogrid such as Tensar BX-1200 or Terragrid RX1200 may be placed directly on the excavation bottom, and then covered with at least 12 inches of minus ¾-inch aggregate base. Once the excavation is firm enough to attain the required compaction within the base, the remainder of the excavation may be backfilled using either compacted soil or aggregate base.

6.3.6 Surface Drainage

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

6.3.7 Storm Water Management

Various bioretention basins, swales or subterranean detention basins may be considered as part of the development in order to promote on-site infiltration for storm water Best Management Practice (BMP). Details of the planned storm water BMPs are not yet available. To help determine the feasibility of on-site infiltration, the on-site infiltration rates were estimated at the two locations shown on the Exploration Plan, Figure 3A. These tests showed an average infiltration rate of less than 0.05 inches per hour, which is indicative of a "No Infiltration" condition per the City of La Mesa BMP Design Manual. The infiltration test results are described in detail in Appendix C.

6.3.8 Temporary Excavations

Temporary excavations may be needed to construct the planned improvements. All excavations should conform to Cal-OSHA guidelines. In general, we recommended that temporary excavations be inclined no steeper than 1:1 for heights up to 20-feet. Vertical excavations should be shored.

The design, construction, maintenance and monitoring of all temporary slopes is the responsibility of the contractor. The contractor should have a competent person evaluate the geologic conditions encountered during excavation to determine permissible temporary slope inclinations and other measures as required by Cal-OSHA. Based on the findings of our subsurface investigation, the following OSHA Soil Types may be assumed for temporary slope design.

Geologic Unit	Cal/OSHA Soil Type
Undocumented Fill	Type B
Stadium Conglomerate	Type A[1]

1. Not subject to vibration, with no fracturing, fissuring or dip into the excavation.

6.3.9 Shored Excavations

We anticipate that shored vertical excavations will be used to construct the proposed subterranean parking garage. Cantilever shoring may be applicable for excavations up to about 15 feet deep, provided that about 1-inch of lateral deflection at the top of the shoring is acceptable to the design team. Existing improvements located within the retained zone behind the cantilever shoring system may be damaged by such lateral deformation and may need to be replaced once the project is completed. For deeper excavations, or where lateral movements must be limited to protect existing improvements, tie-backs or internal braces may be used.

The contractor should be responsible for the design of the shoring system. Cantilever shoring will likely include steel soldier piles and wood lagging (or shotcrete). Typically, steel I-beams are installed in pre-drilled 2 or 3-foot diameter holes spaced at 6 to 8-foot centers. The space between the hole and soldier beam would be filled with structural concrete, up to about 6-inches below the bottom of the planned basement foundations. A 1½ sack sand-cement slurry would then be used to backfill the remainder of the pile excavations to facilitate construction. Wood lagging or shotcrete would be placed between the I-beams as the excavation proceeds.

For design of cantilever shoring, we recommend assuming a triangular active pressure distribution approximated by a fluid with an equivalent unit weight of 35 lb/ft^3 (assuming level soil conditions behind the shoring). For the design of soldier piles spaced at least two pile diameters on center, the allowable passive pressure of the Stadium Conglomerate below the bottom of the excavation may be approximated by a fluid with an equivalent unit weight of 400 lb/ft^3. The allowable passive pressure incorporates a Safety Factor of 2 or more.

For existing settlement sensitive improvements located near the planned excavation, a survey and monitoring program may be needed to document deflections resulting from the excavations. The existing condition of the sensitive improvements would be documented prior to commencing with the excavations. The soldier piles would be surveyed periodically during the excavation process. The design team would review the survey data to verify that the displacements are tolerable. If lateral displacements exceed one inch, the excavations would be halted until further review.

6.4 Foundation Recommendations

The foundations for the new buildings should be designed by the project structural engineer using the following geotechnical parameters. These are only minimum criteria, and should not be considered a structural design, or to preclude more restrictive criteria of governing agencies or the structural engineer. The following recommendations should be considered preliminary, and subject to revision based on the conditions observed by the geotechnical consultant during grading.

6.4.1 Conventional Foundations

The following design parameters are considered appropriate for conventional shallow foundations that bear entirely on Stadium Conglomerate, or entirely on a relatively shallow depth of compacted fill prepared in accordance with the recommendations in Section 6.3.3.

Allowable Bearing: (Compacted Fill)	3,000 lbs/ft^2 (⅓ increase for short-term loads).
Allowable Bearing: (Conglomerate)	5,000 lbs/ft^2 (allow a ⅓ increase for short-term wind or seismic loads). The allowable bearing may be increased by 500 lbs/ft^2 per foot increase in width, and by 1,000 lbs/ft^2 for each additional foot of depth, up to a maximum value of 10,000 lbs/ft^2.
Minimum Footing Width:	18 inches
Minimum Footing Depth:	24 inches below lowest adjacent soil grade
Minimum Reinforcement:	Two No. 4 bars, top and bottom, continuous footings.

6.4.2 Post-Tensioned Slab Foundations

If the new building does not include a subterranean parking garage, extensive remedial grading will be used to excavate and compact the existing undocumented fill, over-excavate the cut portions of the site, and prepare a new building pad in accordance with the recommendations provided in Section 6.3.3. In that case, a post-tensioned slab foundation may be used for support of the new building. Note that the final post-tensioned slab foundation design parameters should be provided in the as-graded geotechnical report, after the recommended remedial grading is completed.

The following post-tension slab design parameters were developed in general accordance with the procedures described the referenced guidelines (PTI, 2007). These parameters are generally consistent with methods currently used in southern California. Other alternative design methods are available. The following design parameters are considered to be appropriate for a building underlain by a relatively uniform depth of compacted fill with a low expansion potential (EI<50). Preliminary post-tensioned slab foundation design may be performed by the project structural engineer using the following geotechnical parameters.

Preliminary Post-Tension Slab Design Parameters:

Moisture Variation, em:	*Center Lift:*	*5.3 feet*
	Edge Lift:	*2.6 feet*
Differential Swell, ym:	*Center Lift:*	*0.6 inches*
	Edge Lift:	*0.8 inches*
Allowable Bearing:	*2,000 psf at slab subgrade*	

6.4.3 Settlement

Provided that remedial grading is conducted as recommended in Section 6.3, total and differential settlement of the structure is not expected to exceed one inch and ¾-inch in 40 feet, respectively.

6.4.4 Lateral Resistance

Lateral loads against the structure may be resisted by friction between the bottoms of footings and slabs and the underlying soil, as well as passive pressure from the portion of vertical foundation members embedded into compacted fill or formational materials. A coefficient of friction of 0.35 and a passive pressure of 350 psf per foot of depth may be used.

6.4.5 Seismic Design

Structures should be designed in general accordance with the seismic provisions of the 2016 California Building Code (CBC) for Seismic Design Category D. Based on our understanding of the site geology and the findings of the subsurface explorations, it is our opinion that a 2016 CBC Site Class C would apply to the general site conditions.

The USGS mapped spectral ordinates S_S and S_1 equal 0.881 and 0.340, respectively. For Site Class C, the acceleration and velocity coefficients F_a and F_v equal 1.048 and 1.460, respectively. The spectral design parameters S_{DS} and S_{D1} equal 0.615 and 0.331, respectively. The peak ground acceleration from the 2016 CBC Design spectrum may be taken as 40 percent of S_{DS} or 0.246g. The MCE peak ground acceleration from the 2016 CBC is 0.369g. The 2016 CBC acceleration response spectra for a Site Class C are shown in Table 1.

6.5 On-Grade Slabs

Building slabs should be at least 5 inches thick and should be reinforced with at least No. 3 bars on 18-inch centers, each way. Slab thickness, control joints, and reinforcement should be designed by the structural engineer and should conform to the requirements of the current CBC. It should be reiterated that at least two feet of low expansion material (EI<50) is recommended beneath all new concrete sidewalks and building slabs on-grade.

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 to four 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, when required by the product manufacturer. The material should consist of a clean, fine graded sandy soil with roughly 10 to 30 percent passing the No. 100 sieve. The sand should not be contaminated with clay, silt, or organic material. The sand should be proof-rolled prior to placing the vapor membrane.

Based on current ACI recommendations, the concrete slab should be placed directly over the vapor membrane. The common practice of placing sand over the vapor membrane may increase moisture transmission through the slab, because it provides a reservoir for bleed water from the concrete to collect. The sand placed over the vapor membrane may also move during placement, resulting in an irregular slab thickness. When placing concrete directly on an impervious membrane, it should be noted that finishing delays may occur. Care should be taken to assure that a low water to cement ratio is used, and that the concrete is moist cured in accordance with ACI guidelines.

6.5.2 Exterior Slabs

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. The potential for differential movements across the control joints may be reduced by using steel reinforcement. Typical reinforcement for exterior slabs would consist of 6x6 W2.9/W2.9 welded wire fabric placed securely at mid-height of the slab.

6.5.3 Expansive Soils

The near surface soils we observed in the subsurface investigation primarily consisted of silty and clayey sand (SM and SC) with gravel. Laboratory tests and our previous experience suggests that these materials typically have a low expansion potential (EI<50), based on commonly accepted criteria. The Expansion Index test results are presented in Figure B-2.

6.5.4 Reactive Soils

In order 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. The test results indicate that the on-site soils typically have a *negligible* potential for sulfate attack based on commonly accepted criteria. The sulfate content of the finish grade soils should be confirmed during fine grading. In order to assess the reactivity of the site soils with buried metals, the pH, resistivity and chloride content were also determined (see Figure B-3). These tests suggest that the on-site soils may be *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 for buried metal structures. A corrosion consultant may be contacted for specific recommendations.

6.6 Earth-Retaining Structures

Backfilling retaining walls with expansive soil can increase lateral pressures well beyond normal active or at-rest pressures. Retaining walls should be backfilled with granular soil with an Expansion Index of 20 or less (EI<20). 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 retaining wall design, an allowable bearing capacity of 2,500 lbs/ft^2, a coefficient of friction of 0.35, and a passive pressure of 350 psf per foot of depth is recommended.

6.6.1 Cantilever Walls

Cantilever retaining walls with level granular 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. Subterranean walls (such as the garage basement walls) that are restrained so that such movement is not permitted should be designed for an at-rest equivalent fluid pressure of 60 lbs/ft^3. These pressures do not include groundwater forces. All retaining walls should contain backdrains to relieve hydrostatic pressures. Typical retaining wall drain details are shown in Figure 7A.

Any surcharges located within a 1:1 plane extending back and up from the base of the retaining wall should also be accounted for in the wall design. Retaining walls (and temporary shoring) situated adjacent to vehicular traffic areas may be designed to resist a uniform lateral surcharge pressure of 100 lb/ft^2, resulting from a typical 300 lb/ft^2 traffic surcharge acting behind the wall.

6.6.2 Seismic Wall Loads

Per the provisions of the 2016 California Building Code (CBC), seismic design is required for all earth retaining structures over 6 feet in height. Basement walls may also require seismic design. We recommend that seismic wall design be conducted using the Mononabe-Okabe solution which incorporates a pseudo-static horizontal load. According to the provisions of the 2016 CBC, the design level peak ground acceleration (PGA) for the site may be taken as 40 percent of the short period spectral ordinate (S_{DS} ~ 0.615) or 0.246g, as shown in Table 1. One-half of the design level peak ground acceleration is typically used for pseudo-static seismic wall design. Consequently, we have provided seismic retaining wall design parameters for a pseudo-static load of 0.12g.

The seismic load increment may be idealized as an inverted triangular pressure distribution with the resultant acting at 0.6H above the base of the wall (see Figure 7B). The Mononabe-Okabe solution is based on active earth pressures and requires that the retaining walls are free to yield. For restrained basement walls, we recommend that the static wall design be based on the at-rest earth pressure, which we have approximated by a fluid with an equivalent unit weight of 60 lb/ft^3 (assuming level backfill). We recommend that the equivalent seismic pressure increment shown in Figure 7B (γ_e ~10 PCF) be added to the at-rest earth pressure for seismic design of the restrained basement walls at the site which are not free to yield at least ½ percent of the wall height.

6.7 Pavement Design

For all pavement areas, upper 12 inches of subgrade soil should be scarified immediately prior to constructing the pavements, brought to optimum moisture, and compacted to at least 95 percent of the maximum density per ASTM D1557. Aggregate base should also be compacted to 95 percent relative compaction. 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 91 and 97 percent of the Rice density per ASTM D2041.

6.7.1 Asphalt Concrete

In order to aid in preliminary pavement section design, several R-Value tests were conducted on soil samples collected during the field investigation. The testing was conducted in general accordance with CTM 301. The test results are presented in Figures B-6.1 and B-6.2. The R-Values of the samples we tested varied from 11 to 12. The final pavement section designs should be based on R-Value testing of the actual pavement subgrade soils collected during fine grading.

Asphalt concrete pavement design was conducted in general accordance with the Caltrans Design Method. We anticipate that a Traffic Index ranging from 5.0 to 7.0 may apply to any new pavement areas. The project civil engineer should review the assumed Traffic Indices to determine if and where they apply to the various new pavements proposed on site. Based on the minimum R-Value of 11, and an assumed range of Traffic Indices, the following pavement sections would apply.

PAVEMENT TYPE	TRAFFIC INDEX	ASPHALT SECTION	BASE SECTION
Passenger Car Parking	5.0	3 Inches	9 Inches
Light Truck Traffic Areas	6.0	4 Inches	11 Inches
Heavy Truck Traffic Areas	7.0	4 Inches	14 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 concrete was assumed to have a minimum flexural strength of 600 psi. The flexural strength of the pavement concrete should be confirmed during construction using ASTM C78.

For concrete pavement design, the subgrade materials were assumed to provide “low” support, based on the results of the R-Value tests. Using these assumptions and 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.

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 planned development may include various 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 below.

6.8.1 Thrust Blocks

Lateral resistance for thrust blocks may be determined by a passive pressure value of 350 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 the 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 2,000 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.

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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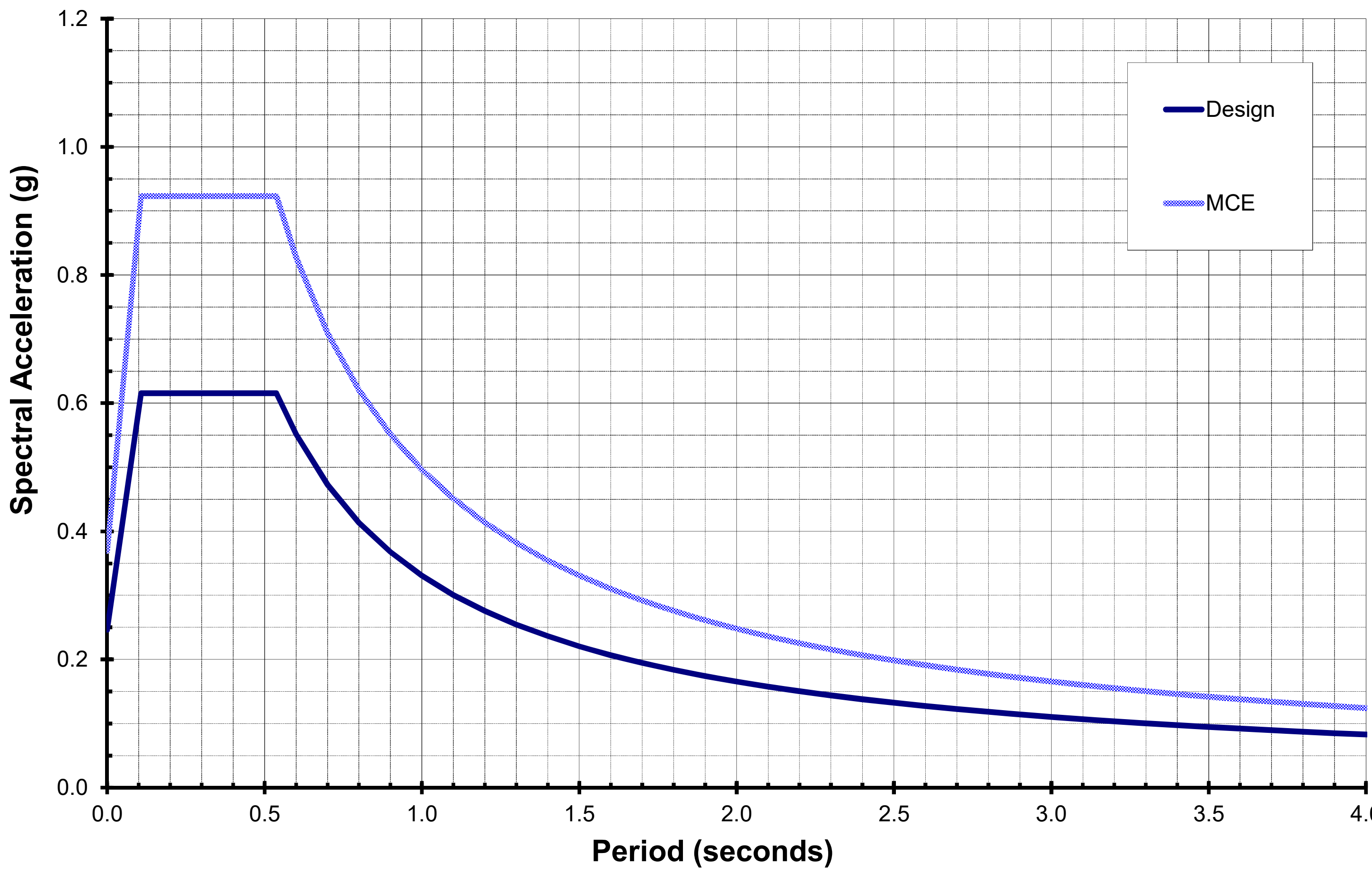
TABLES

TABLE 1 - 2016 CBC ACCELERATION RESPONSE SPECTRA

INPUT	S_S=	0.881	g = short period (0.2 sec) mapped spectral response acceleration MCE Site Class B (**CBC 2013 Fig. 1613.5(3) or USGS Ground Motion Calculator**)	**Site Latitude:** 32.7657
	S_1=	0.340	g = 1.0 sec period mapped spectral response acceleration MCE Site Class B (**CBC 2013 Fig. 1613.5(4) or USGS Ground Motion Calculator**)	**Site Longitude:** -117.0218
	Site Class=	C	= Site Class definition based on **CBC 2013 Table 1613.5.2**	**Seismic Design Category:** D
	F_a=	1.048	= Site Coefficient applied to S_s to account for soil type (**CBC 2013 Table 1613.5.3(1)**)	**Site Modified Peak Ground Acceleration (PGA_M):** 0.361
	F_v=	1.460	= Site Coefficient applied to S_1 to account for soil type (**CBC 2013 Table 1613.5.3(2)**)	
	T_L=	8.00	sec = Long Period Transition Period (**ASCE 7-05 Figure 22-16**)	
OUTPUT	S_{MS}=	0.923	= site class modified short period (0.2 sec) MCE spectral response acceleration = F_a x S_s (**CBC 2013 Eqn. 16-36**)	
	S_{M1}=	0.496	= site class modified 1.0 sec period MCE spectral response acceleration = F_v x S_1 (**CBC 2013 Eqn. 16-37**)	
	S_{DS}=	0.615	= site class modified short period (0.2 sec) Design spectral response acceleration = 2/3 x S_{MS} (**CBC 2013 Eqn. 16-38**)	
	S_{D1}=	0.331	= site class modified 1.0 sec period Design spectral response acceleration = 2/3 x S_{M1} (**CBC 2013 Eqn. 16-39**)	
	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 7-05)	
	T_S=	0.538	sec = S_{D1}/S_{DS} = Control Period (right end of peak) for ARS Curve (Section 11.4.5 ASCE 7-05)	

SPECTRUM CALCULATION

T (seconds)	Design Sa (g)	MCE Sa (g)
0.000	0.246	0.369
0.108	0.615	0.923
0.538	0.615	0.923
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
5.000	0.066	0.099



FIGURES

N

SCALE (FT):

0' 150' 300'

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PROJECT NAME	PROJECT NUMBER SD634
La Mesa Apartments USA Properties Fund	DOCUMENT NUMBER 19-0141
	FIGURE NUMBER 1A

SITE LOCATION MAP

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PROJECT NUMBER	SD634
DOCUMENT NUMBER	19-0141
PROJECT NAME	La Mesa Apartments USA Properties Fund
FIGURE NUMBER	1B

SITE VICINITY PLAN

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USA Properties Fund

PROJECT NUMBER
SD634

DOCUMENT NUMBER
19-0141

FIGURE NUMBER
1C

SITE PHOTOGRAPHS

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PROJECT NAME
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USA Properties Fund

PROJECT NUMBER
SD634

DOCUMENT NUMBER
19-0141

FIGURE NUMBER
1D

SITE PHOTOGRAPHS

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PROJECT NUMBER	SD634
DOCUMENT NUMBER	19-0141
FIGURE NUMBER	1E

PROJECT NAME
La Mesa Apartments
USA Properties Fund

STADIUM CONGLOMERATE

REFERENCE: USA Properties Fund (2019). *Mixed Income TOD Housing, La Mesa, CA*, April 8.

GROUP DELTA	GROUP DELTA CONSULTANTS, INC. ENGINEERS AND GEOLOGISTS 9245 ACTIVITY ROAD, SUITE 103 SAN DIEGO, CA 92126 (858) 536-1000	PROJECT NUMBER SD634 DOCUMENT NUMBER 19-0141
	PROJECT NAME La Mesa Apartments USA Properties Fund	FIGURE NUMBER 2A
	PROPOSED DEVELOPMENT	

EXPLANATION:

B-5 — Approximate locations of the 5 exploratory borings (Group Delta, 2019).

I-2 — Approximate locations of the 2 borehole percolation infiltration tests (Group Delta, 2019).

REFERENCE: USA Properties Fund (2019). *Mixed Income TOD Housing, La Mesa, CA*, April 8.

N

SCALE (FT):
0' 50' 100'

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PROJECT NAME
La Mesa Apartments
USA Properties Fund

PROJECT NUMBER
SD634

DOCUMENT NUMBER
19-0141

FIGURE NUMBER
2B

PROPOSED DEVELOPMENT

EXPLANATION:

B-5 Approximate locations of the 5 exploratory borings (Group Delta, 2019).

I-2 Approximate locations of the 2 borehole percolation infiltration tests (Group Delta, 2019).

N

SCALE (FT):
0' 50' 100'

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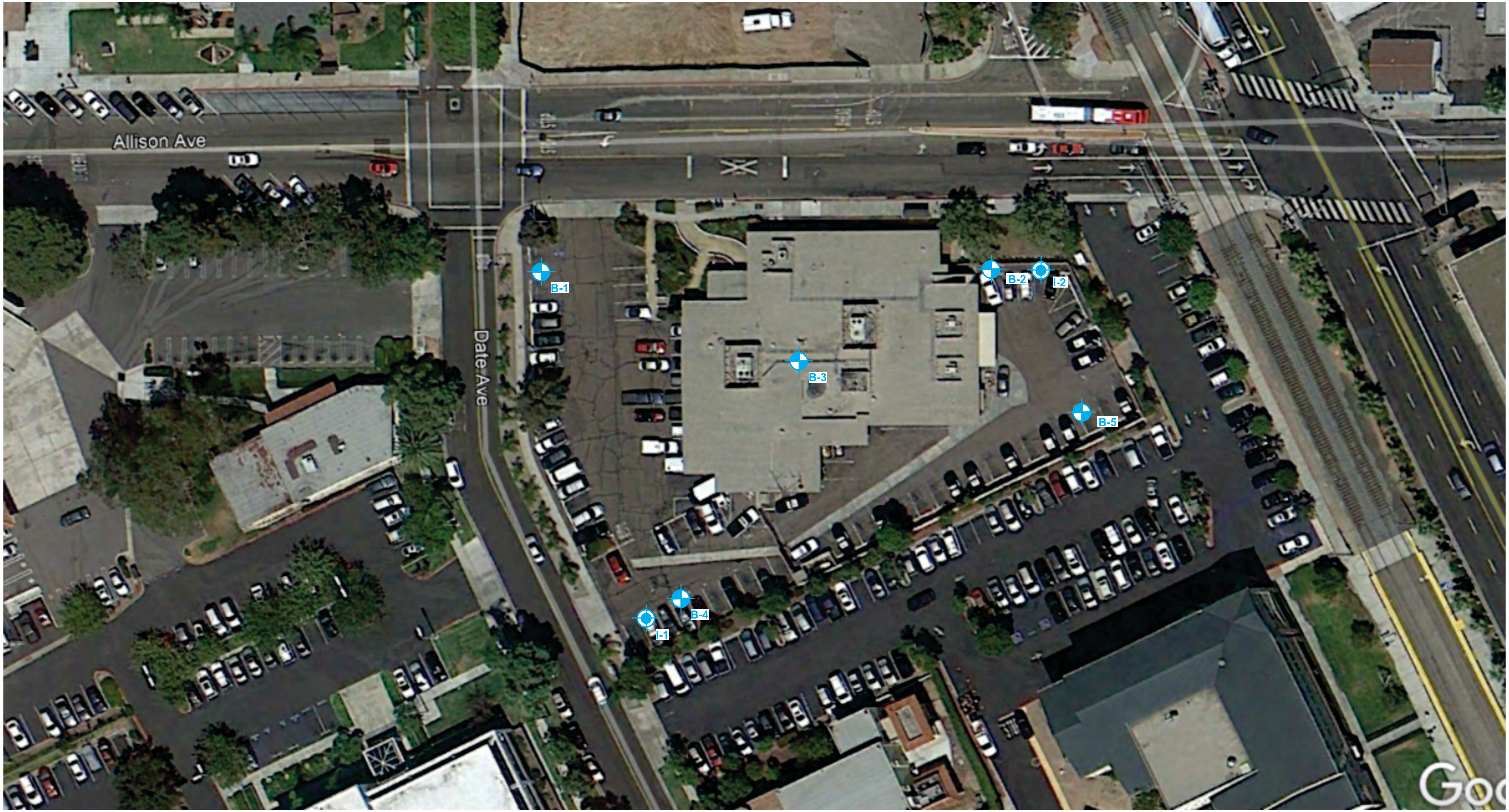
SAN DIEGO, CA 92126 (858) 536-1000

PROJECT NUMBER	SD634
DOCUMENT NUMBER	19-0141
PROJECT NAME: La Mesa Apartments USA Properties Fund	FIGURE NUMBER: 3A

EXPLORATION PLAN (2019)



EXPLANATION:

B-5 Approximate locations of the 5 exploratory borings (Group Delta, 2019).

I-2 Approximate locations of the 2 borehole percolation infiltration tests (Group Delta, 2019).

N

SCALE (FT):
0' 50' 100'

GROUP DELTA

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ENGINEERS AND GEOLOGISTS
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SAN DIEGO, CA 92126 (858) 536-1000

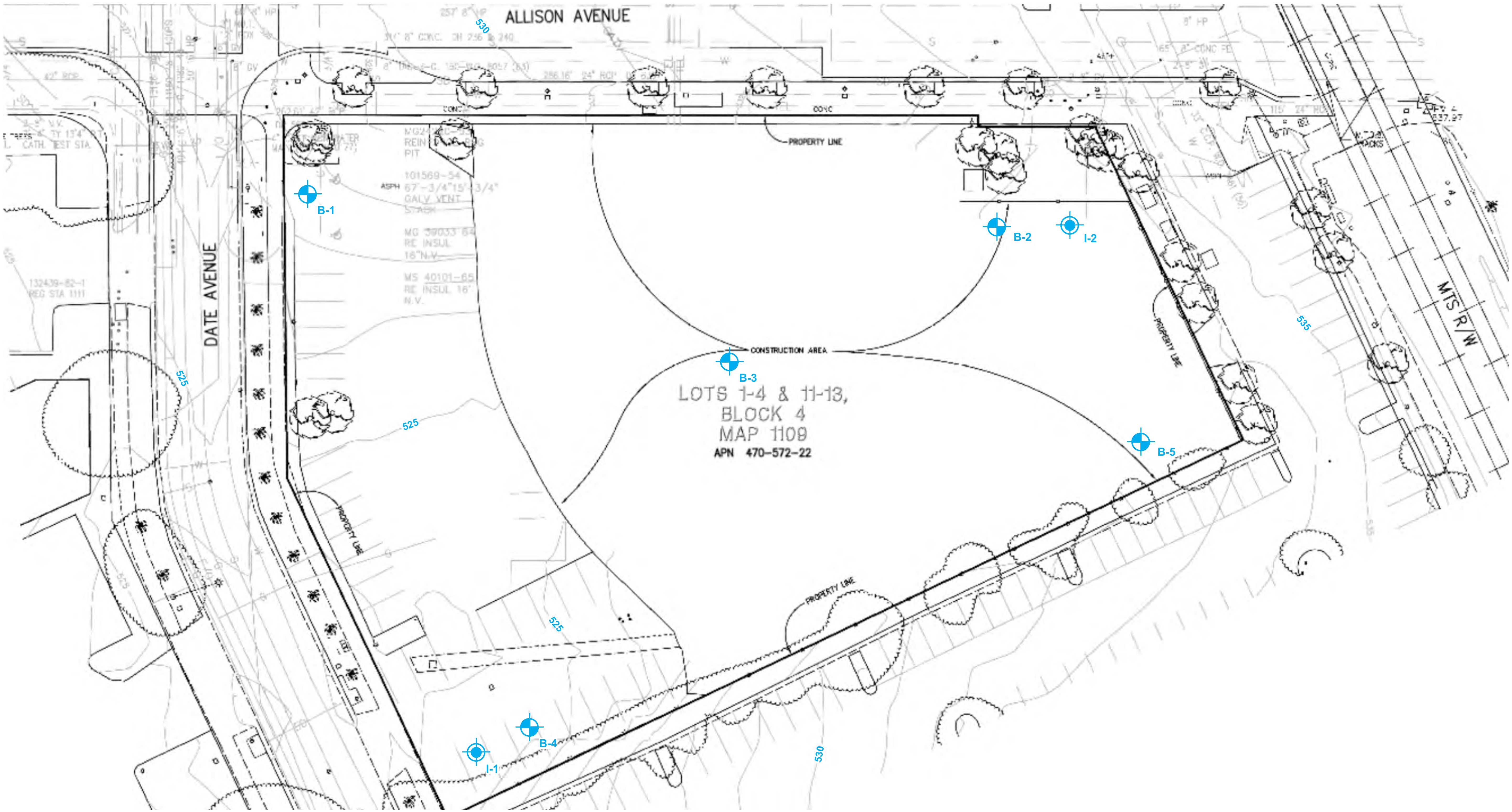
PROJECT NAME
La Mesa Apartments
USA Properties Fund

PROJECT NUMBER
SD634

DOCUMENT NUMBER
19-0141

FIGURE NUMBER
3B

SITE CONDITIONS (2010)



EXPLANATION:

B-5 — Approximate locations of the 5 exploratory borings (Group Delta, 2019).

525 — Approximate elevation of surrounding grade (feet).

REFERENCE: Aguirre & Associates (2015). *Preliminary Parcel Map, Lots 1-4 & 11-13, Block 4, Map 1109, APN 470-572-22*, July 24

N

SCALE (FT):
0' 20' 40'

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PROJECT NUMBER	SD634
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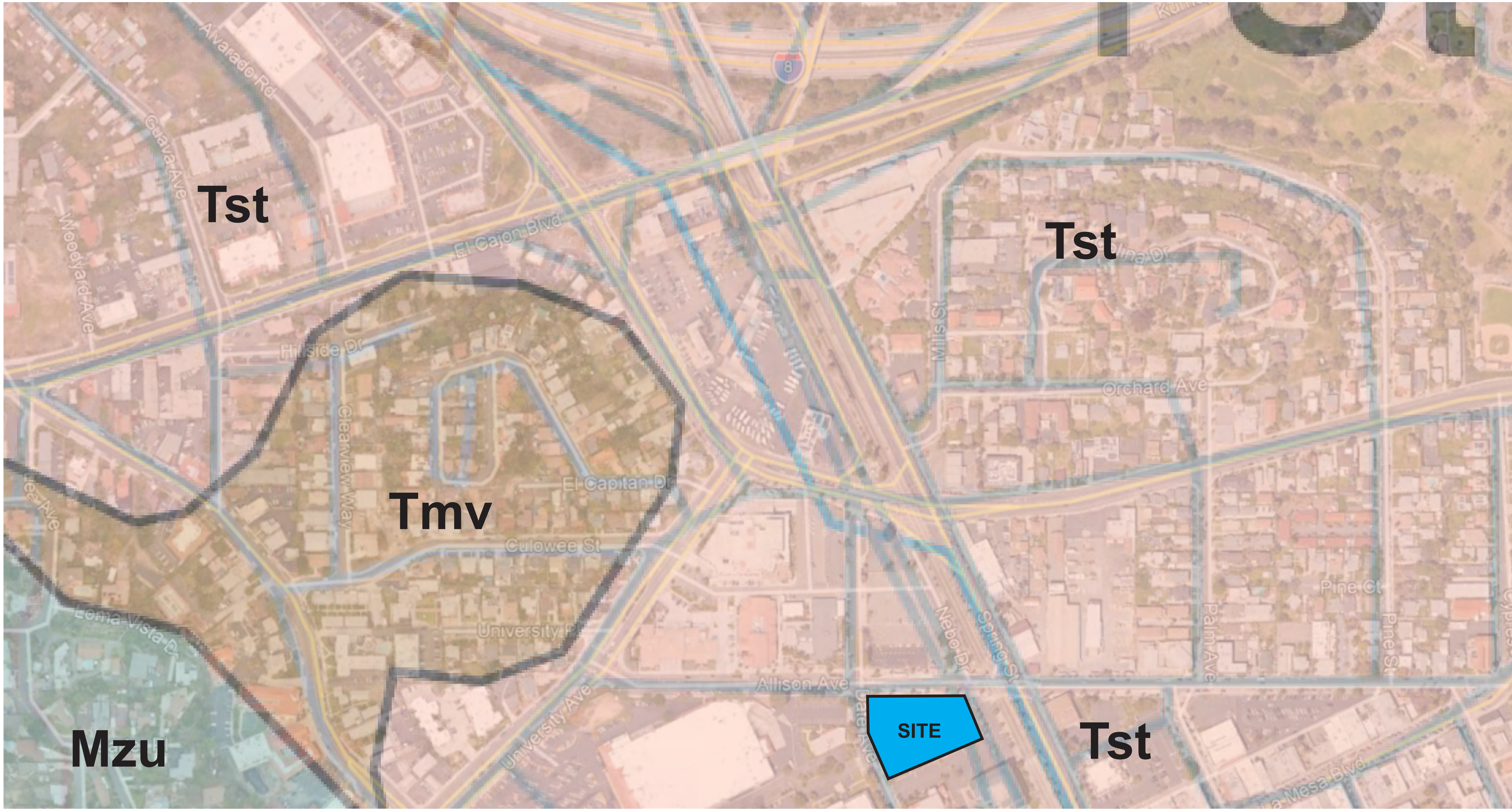
PROJECT NAME	La Mesa Apartments USA Properties Fund
FIGURE NUMBER	3C

SITE TOPOGRAPHY



EXPLANATION:

Tst	Stadium Conglomerate
Tmv	Mission Valley Formation
Mzu	Metavolcanic Rock

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

N

SCALE (FT):
0' 150' 300'

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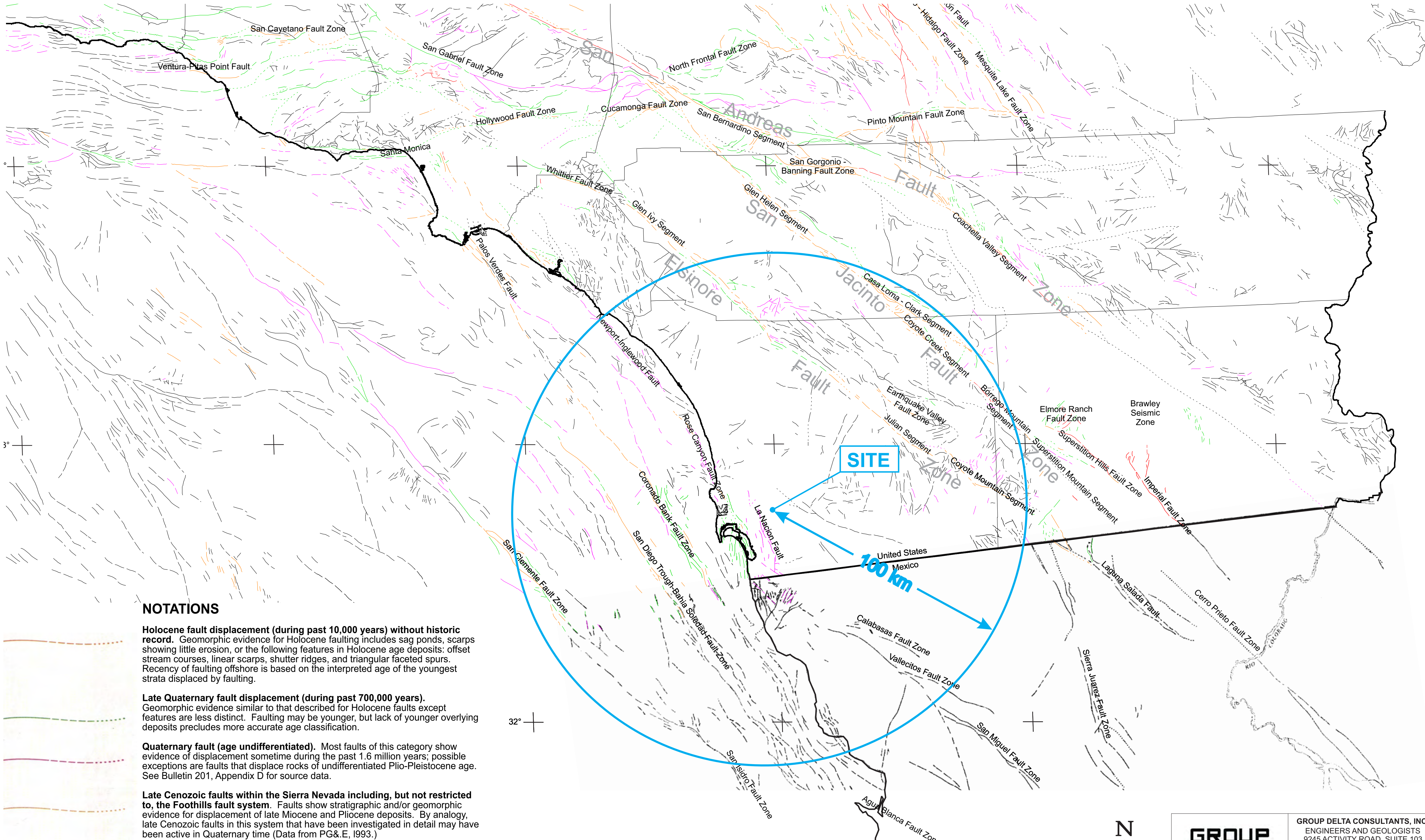
PROJECT NUMBER
SD634

DOCUMENT NUMBER
19-0141

PROJECT NAME
La Mesa Apartments
USA Properties Fund

FIGURE NUMBER
4

LOCAL 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.

REFERENCE: Jennings, C.W. (1994). *Fault Activity Map of California and Adjacenet Areas*, CDMG Geologic Data Map Series, Map No. 6.

N

SCALE (KM):
0 15 30

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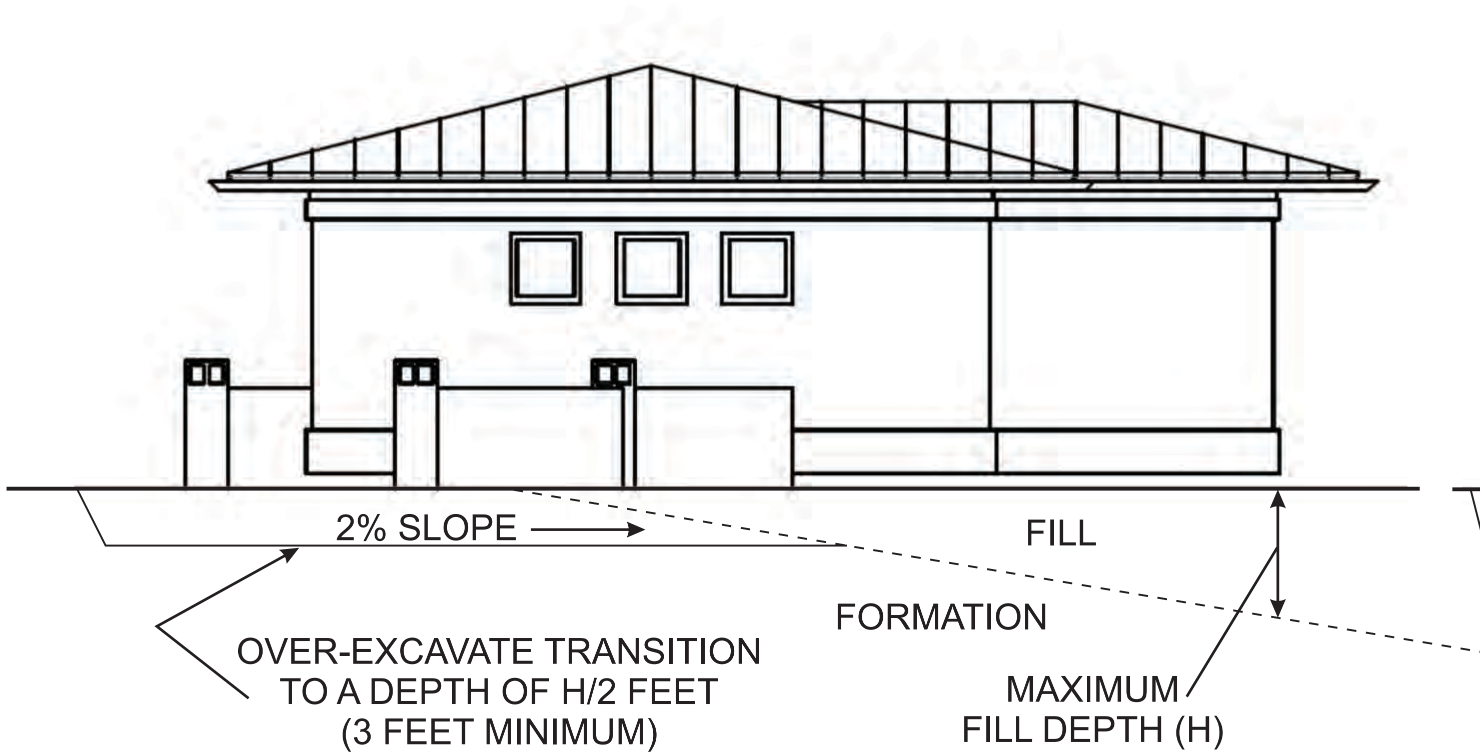
DOCUMENT NUMBER
19-0141

PROJECT NAME
La Mesa Apartments
USA Properties Fund

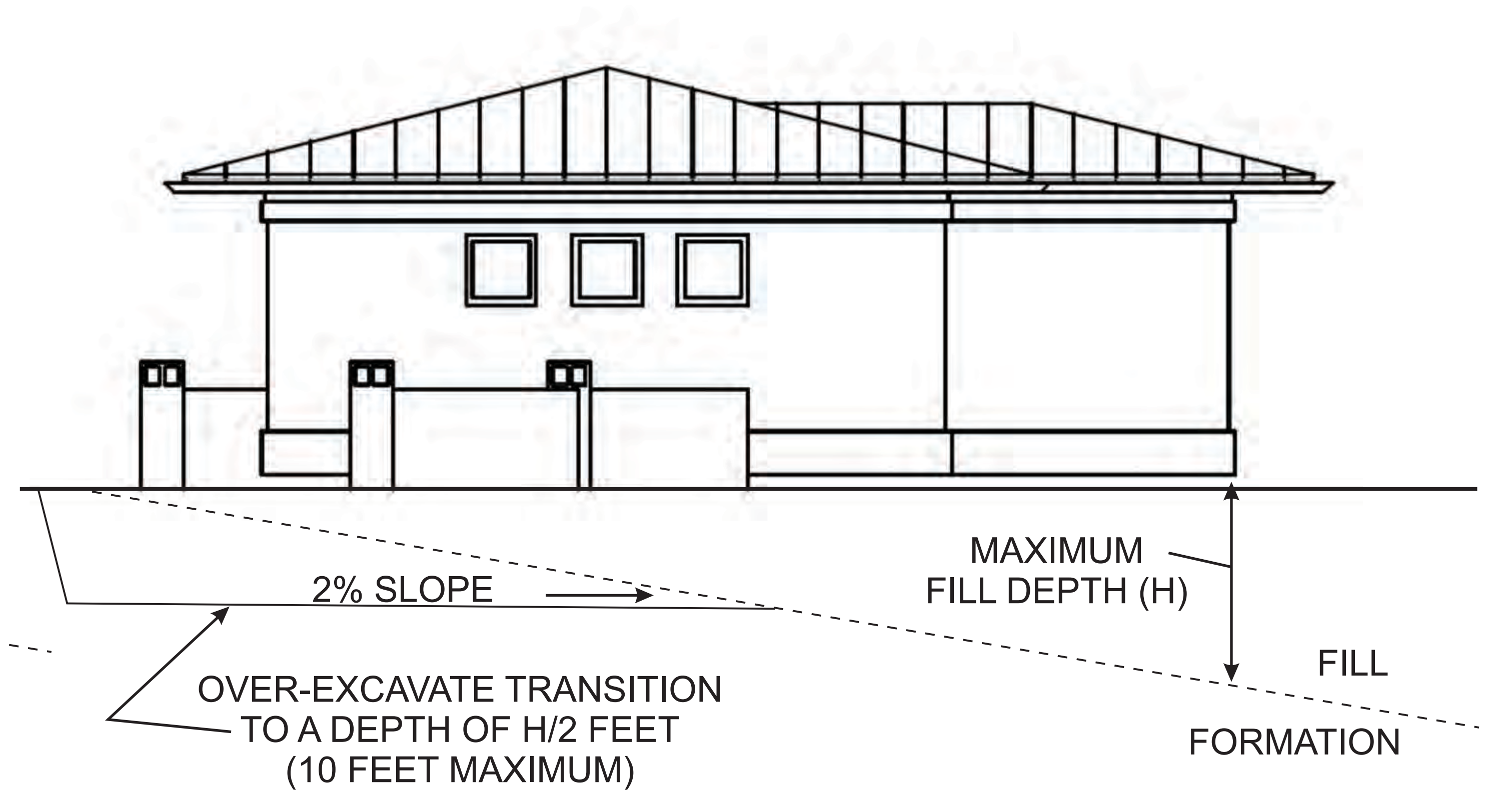
FIGURE NUMBER
5

REGIONAL FAULT 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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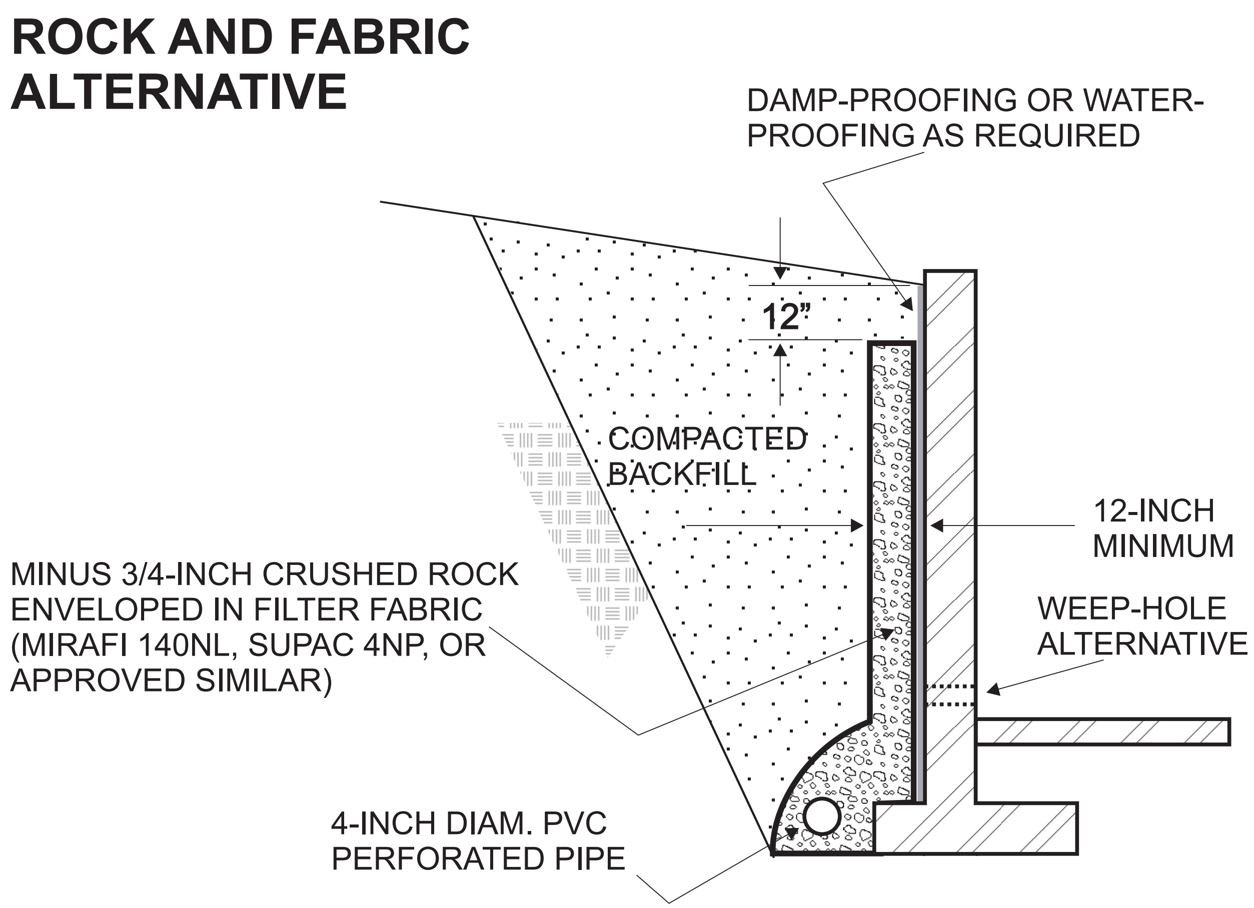
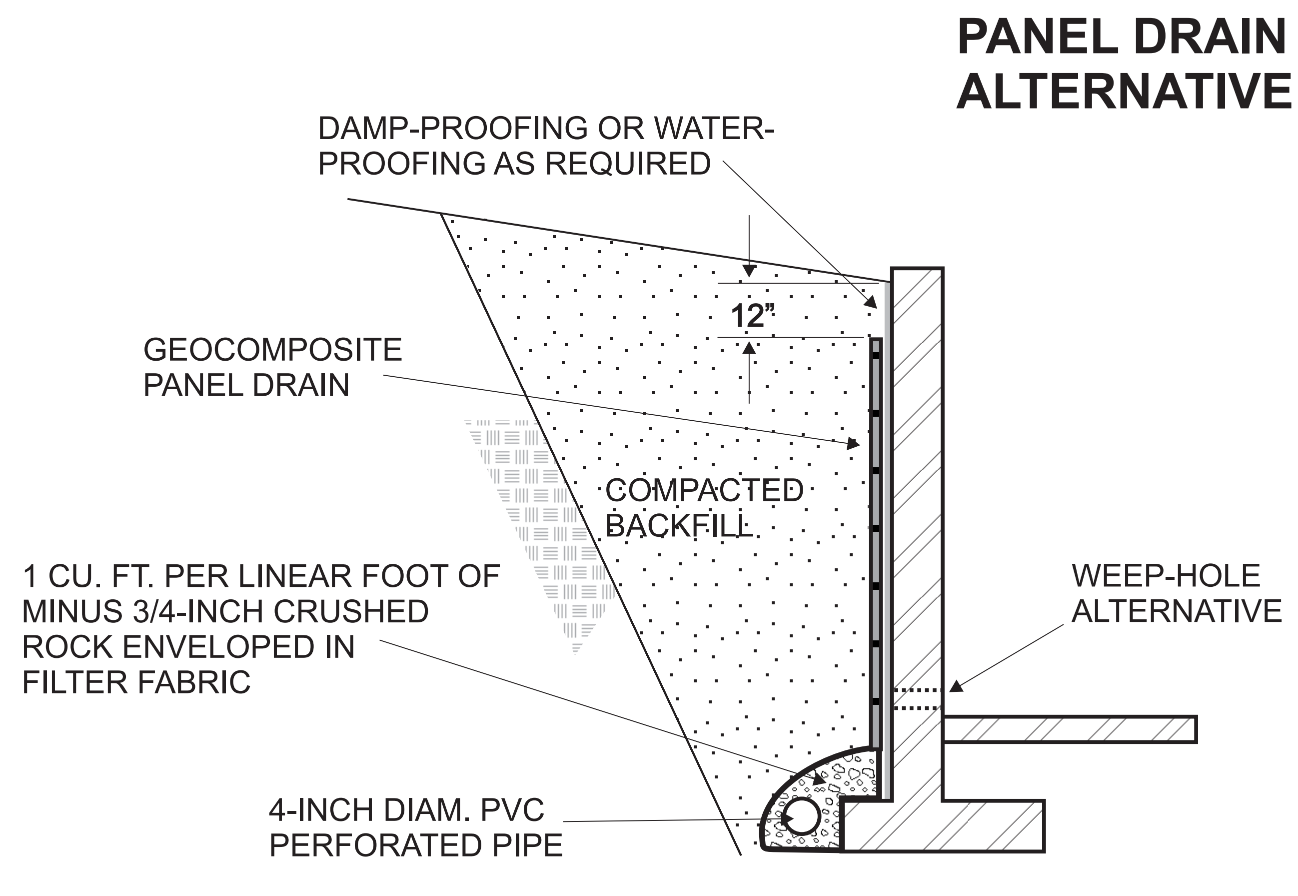
SAN DIEGO, CA 92126 (858) 536-1000

PROJECT NAME La Mesa Apartments USA Properties Fund	PROJECT NUMBER SD634
	DOCUMENT NUMBER 19-0141
	FIGURE NUMBER 6

TRANSITION DETAILS





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.

GROUP DELTA	GROUP DELTA CONSULTANTS, INC. ENGINEERS AND GEOLOGISTS 9245 ACTIVITY ROAD, SUITE 103 SAN DIEGO, CA 92126 (858) 536-1000	PROJECT NUMBER SD634 DOCUMENT NUMBER 19-0141
	PROJECT NAME La Mesa Apartments USA Properties Fund	FIGURE NUMBER 7A
	WALL DRAIN DETAILS	

INPUT PARAMETERS

Unit Weight of Soil [PCF]	130
Backfill Soil Friction Angle (ϕ) [°]:	34
Wall Friction Angle (δ) [°]:	23
Soil Backfill Angle (α) [°]:	0
Wall Batter Angle (β) [°]:	90
Horizontal Acceleration (K_h) [g's]:	0.12
Vertical Acceleration (K_v) [g's]:	0.00

CALCULATED PARAMETERS

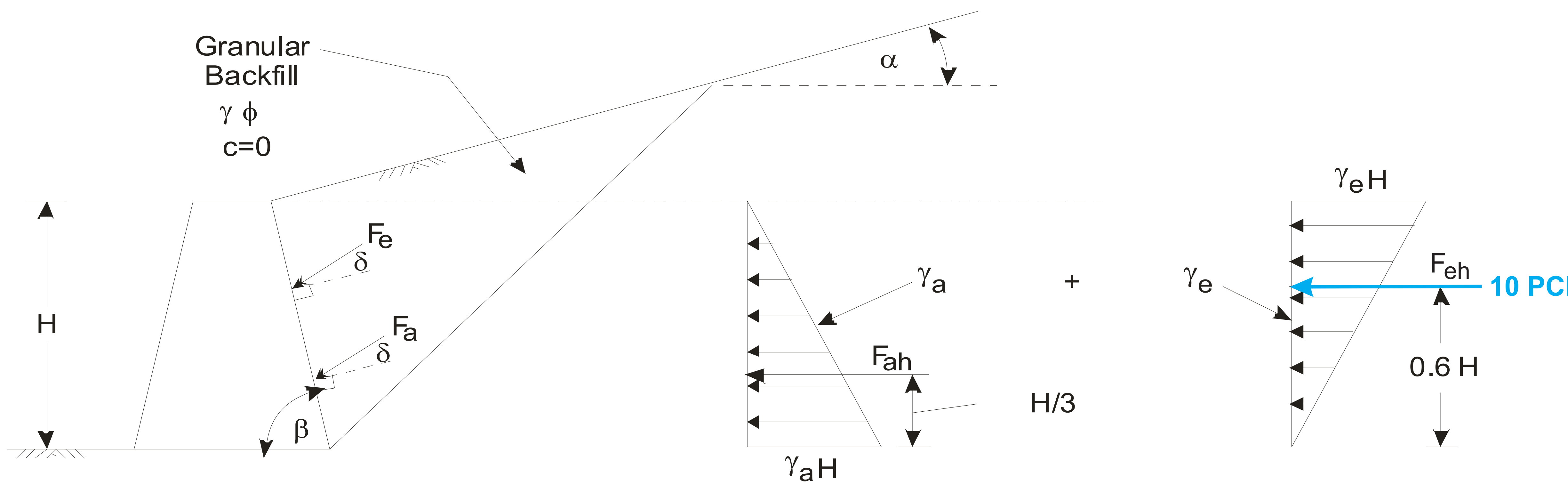
Active Pressure Coefficient (K_a):	0.254
Equivalent Fluid Pressure (γ_a):	33.1
Seismic Pressure Coefficient (K_{ae}):	0.332
Equivalent Fluid Pressure (γ_{ae}):	43.1
Equivalent Seismic Pressure (γ_e):	**10**

Active Pressure Resultant: $F_a = 1/2\ \gamma_a\ H^2$

Earthquake Pressure Resultant: $F_e = 1/2\ \gamma_e\ H^2$

Horizontal Component of Active Pressure Resultant $F_{ah} = F_a \cos(\delta+90-\beta)$

Horizontal Component of Seismic Pressure Resultant $F_{eh} = F_e \cos(\delta+90-\beta)$



GROUP DELTA	GROUP DELTA CONSULTANTS, INC. ENGINEERS AND GEOLOGISTS 9245 ACTIVITY ROAD, SUITE 103 SAN DIEGO, CA 92126 (858) 536-1000	PROJECT NUMBER SD634 DOCUMENT NUMBER 19-0141
	PROJECT NAME La Mesa Apartments USA Properties Fund	FIGURE NUMBER 7B
	SEISMIC WALL LOADS	

APPENDIX A
FIELD EXPLORATION

APPENDIX A

FIELD EXPLORATION

Field exploration included a visual reconnaissance of the site and the excavation of five exploratory borings, and the completion of two infiltration test between September 19th and 20th, 2019. The exploratory borings and infiltration test holes were drilled by Pacific Drilling Company using their Marl M10 (Grizzly) truck mounted drill rig with a 6-inch diameter hollow stem flight auger. The maximum depth of exploration was about 20 feet below surrounding grades. The approximate boring locations are shown on the Exploration Plan, Figure 3A. The boring logs are provided in Figures A-1 through A-7, immediately following the Boring Record Legends.

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. The drive samples were collected from the borings using an automatic hammer with a calibrated Energy Transfer Ratios (ETR) of 92 percent. 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 corrected to reflect a standard 60 percent ETR (N_{60}), as shown on the logs. Bulk soil samples were also collected from the borings.

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

SOIL IDENTIFICATION AND DESCRIPTION SEQUENCE

Sequence	Identification Components	Refer to Section: Field	Refer to Section: Lab	Required	Optional
1	Group Name	2.5.2	3.2.2	●	
2	Group Symbol	2.5.2	3.2.2	●	
	Description Components				
3	Consistency of Cohesive Soil	2.5.3	3.2.3	●	
4	Apparent Density of Cohesionless Soil	2.5.4		●	
5	Color	2.5.5		●	
6	Moisture	2.5.6		●	
7	Percent or Proportion of Soil	2.5.7	3.2.4	●	● (grey)
	Particle Size	2.5.8	2.5.8	●	● (grey)
	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).

● (grey) = 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. SD634

La Mesa Apartments
USA Properties Fund

BORING RECORD LEGEND #1

GROUP SYMBOLS AND NAMES

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

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. SD634

La Mesa Apartments
USA Properties Fund

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. SD634

La Mesa Apartments
USA Properties Fund

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. SD634

La Mesa Apartments
USA Properties Fund

BORING RECORD LEGEND #4

BORING RECORD

PROJECT NAME	PROJECT NUMBER	BORING
La Mesa Apartment Development	SD634	**B-1**

SITE LOCATION	START	FINISH	SHEET NO.
Southeast of Allison Avenue and Date Avenue	9/19/2019	9/19/2019	1 of 1

DRILLING COMPANY	DRILLING METHOD	LOGGED BY	CHECKED BY
Pacific Drilling Company	Hollow Stem Auger	SRN	MAF

DRILLING EQUIPMENT	BORING DIA. (in)	TOTAL DEPTH (ft)	GROUND ELEV (ft)	DEPTH/*ELEV.* GROUND WATER (ft)
Marl M10 Truck Mounted Rig (Grizzly)	6	7	528	▼ N/A / *na*

SAMPLING METHOD	NOTES
Hammer: 140 lbs., Drop: 30 in. (Automatic)	ETR ~ 92%, N_{60} ~ 92/60 * N ~ 1.53 * N

DEPTH (feet)	ELEVATION (feet)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS / 6 IN)	BLOW/FT "N"	N_{60}	MOISTURE (%)	DRY DENSITY (pcf)	OTHER TESTS	DEPTH (feet)	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
												PAVEMENT: Asphalt Concrete (3"), no Base (0").
			B-1									**FILL:** SILTY SAND (SM); brown; moist; nonplastic.
			S-2	50 (1")	(REF)	(REF)						**STADIUM CONGLOMERATE:** Poorly indurated COBBLE CONGLOMERATE; medium to coarse grained; moderately weathered; soft; unfractured (POORLY GRADED GRAVEL WITH SAND (GP); very dense; brown; dry to moist; nonplastic; mostly GRAVEL and COBBLE; little SAND; trace fines).
	525											
5										5		
			S-3	5 50 (4")	80	122						
	520											Total Depth: 7 feet Refusal on gravel and cobble No groundwater encountered
10										10		Refusal was encountered at a depth of 1½ feet at the initial boring location. The boring was subsequently moved approximately 12 feet south and redrilled to refusal at 7 feet. Extremely difficult drilling conditions and extensive rig chatter were encountered.
	515											
15										15		
	510											
20										20		
	505											

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FIGURE A-1

BORING RECORD

PROJECT NAME	PROJECT NUMBER	BORING
La Mesa Apartment Development	SD634	B-2

SITE LOCATION	START	FINISH	SHEET NO.
Southeast of Allison Avenue and Date Avenue	9/19/2019	9/19/2019	1 of 1

DRILLING COMPANY	DRILLING METHOD	LOGGED BY	CHECKED BY
Pacific Drilling Company	Hollow Stem Auger	SRN	MAF

DRILLING EQUIPMENT	BORING DIA. (in)	TOTAL DEPTH (ft)	GROUND ELEV (ft)	DEPTH/*ELEV.* GROUND WATER (ft)
Marl M10 Truck Mounted Rig (Grizzly)	6	20.5	534	▼ N/A / *na*

SAMPLING METHOD	NOTES
Hammer: 140 lbs., Drop: 30 in. (Automatic)	ETR ~ 92%, N_{60} ~ 92/60 * N ~ 1.53 * N

DEPTH (feet)	ELEVATION (feet)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS / 6 IN)	BLOW/FT "N"	N_{60}	MOISTURE (%)	DRY DENSITY (pcf)	OTHER TESTS	DEPTH (feet)	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
0–5	534–530		B-1						PA PI CR EI~8			**FILL:** CLAYEY SAND WITH GRAVEL (SC); medium dense to dense; brown; moist; mostly fine to coarse SAND; some fines; little GRAVEL; low plasticity. (19% Gravel; 53% Sand; 28% Fines) (LL=35; PL=17; PI=18)
5			S-2	8 8 14	22	34				5		Grayish brown in color.
	525											**STADIUM CONGLOMERATE:** Poorly indurated SANDSTONE; fine grained; moderately weathered; very soft; unfractured (CLAYEY SAND (SC); very dense; grayish brown; moist; mostly fine SAND; little fines; few GRAVEL; low plasticity; weakly cemented).
10			S-3	12 18 27	45	69			PA	10		(10% Gravel; 64% Sand; 26% Fines) Difficult drilling with rig chatter from 10' to 18'. Increased GRAVEL and COBBLE content with depth.
	520											Poorly indurated COBBLE CONGLOMERATE; medium to coarse grained; moderately weathered; soft; unfractured (CLAYEY GRAVEL WITH SAND (GC); very dense; brown; moist; low plasticity; mostly GRAVEL and COBBLE; little SAND; little fines; low plasticity).
15			R-4	50 (6")	100	102	---	---		15		No sample recovery.
	515											
20			S-5	50 (5")	120	184				20		
	510											Total Depth: 20½ feet No groundwater encountered

GDC_LOG_BORING_MMX_SOIL_SD SD634 LOGS.GPJ GDCLOG.GDT 10/9/19

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FIGURE
A-2

BORING RECORD

PROJECT NAME: La Mesa Apartment Development	PROJECT NUMBER: SD634	BORING: **B-3**	
SITE LOCATION: Southeast of Allison Avenue and Date Avenue	START: 9/19/2019	FINISH: 9/19/2019	SHEET NO.: 1 of 1
DRILLING COMPANY: Pacific Drilling Company	DRILLING METHOD: Hollow Stem Auger	LOGGED BY: SRN	CHECKED BY: MAF
DRILLING EQUIPMENT: Marl M10 Truck Mounted Rig (Grizzly)	BORING DIA. (in): 6	TOTAL DEPTH (ft): 21.5	GROUND ELEV (ft): 530
DEPTH/*ELEV.* GROUND WATER (ft): ▼ N/A / *na*			
SAMPLING METHOD: Hammer: 140 lbs., Drop: 30 in. (Automatic)	NOTES: ETR ~ 92%, N_{60} ~ 92/60 * N ~ 1.53 * N		

DEPTH (feet)	ELEVATION (feet)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS / 6 IN)	BLOW/FT "N"	N_{60}	MOISTURE (%)	DRY DENSITY (pcf)	OTHER TESTS	DEPTH (feet)	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
												FILL: CLAYEY SAND (SC); medium dense to dense; brown; moist; mostly fine to medium SAND; some fines; little GRAVEL; low to medium plasticity.
			B-1						PA PI CR EI~28			(13% Gravel; 49% Sand; 38% Fines) (LL=36; PL=16; PI=20)
5	525		S-2	8 15 16	31	47				5		Grayish brown in color.
10	520		R-3	43 50 (5")	103	105	5.2	---		10		CLAYEY SAND WITH GRAVEL (SC); very dense; grayish brown; moist; mostly fine to medium SAND; little fines; little GRAVEL; low plasticity.
15	515		S-4	50 (2")	(REF)	(REF)				15		**STADIUM CONGLOMERATE:** Poorly indurated COBBLE CONGLOMERATE; medium to coarse grained; moderately weathered; soft; unfractured (POORLY GRADED GRAVEL WITH SAND (GP); very dense; brown; moist; nonplastic; mostly GRAVEL and COBBLE; little SAND; trace fines).
20	510		S-5	8 24 50	74	113			PA	20		Poorly indurated SANDSTONE; fine to medium grained; moderately weathered; very soft; unfractured (SILTY SAND (SM); very dense; grayish brown; moist; mostly fine to medium SAND; little fines; nonplastic to low plasticity). (0% Gravel; 78% Sand; 22% Fines)
												Total Depth: 21½ feet No groundwater encountered

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FIGURE A-3

BORING RECORD

PROJECT NAME	La Mesa Apartment Development
PROJECT NUMBER	SD634
BORING	**B-4**
SITE LOCATION	Southeast of Allison Avenue and Date Avenue
START	9/19/2019
FINISH	9/19/2019
SHEET NO.	1 of 1
DRILLING COMPANY	Pacific Drilling Company
DRILLING METHOD	Hollow Stem Auger
LOGGED BY	SRN
CHECKED BY	MAF
DRILLING EQUIPMENT	Marl M10 Truck Mounted Rig (Grizzly)
BORING DIA. (in)	6
TOTAL DEPTH (ft)	21
GROUND ELEV (ft)	525
DEPTH/*ELEV.* GROUND WATER (ft)	▼ N/A / *na*
SAMPLING METHOD	Hammer: 140 lbs., Drop: 30 in. (Automatic)
NOTES	ETR ~ 92%, N_{60} ~ 92/60 * N ~ 1.53 * N

DEPTH (feet)	ELEVATION (feet)	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS / 6 IN)	BLOW/FT "N"	N_{60}	MOISTURE (%)	DRY DENSITY (pcf)	OTHER TESTS
~2.5		B-1						PA PI CR R~12
5	520	S-2	5 22 42	64	98			
~8.5		B-3						
10	515	S-4	50 (2")	(REF)	(REF)			
15	510	S-5	11 50 (2")	(REF)	(REF)			
20	505	R-6	33 50 (3")	133	136	5.1	---	

DESCRIPTION AND CLASSIFICATION

PAVEMENT: Asphalt Concrete (3"), no Base (0").

FILL: CLAYEY SAND WITH GRAVEL (SC); medium dense; dark yellow brown; moist; mostly SAND; little fines; little GRAVEL; low plasticity.

STADIUM CONGLOMERATE: Poorly indurated COBBLE CONGLOMERATE; medium to coarse grained; moderately weathered; soft; unfractured (CLAYEY SAND WITH GRAVEL (SC); very dense; brown; moist; low plasticity; mostly SAND; some GRAVEL and COBBLE; little fines).

(37% Gravel; 41% Sand; 22% Fines)

(LL=31; PL=15; PI=16)

Very difficult drilling conditions.

Extensive rig chatter from 6' to 7' and 9' to 14'.

Poorly indurated COBBLE CONGLOMERATE; medium to coarse grained; moderately weathered; soft; unfractured (CLAYEY GRAVEL WITH SAND (GC); very dense; yellowish brown; moist; mostly GRAVEL and COBBLE; little fines; little SAND; low to medium plasticity; weakly cemented).

Very difficult drilling with rig chatter from 16' to 20'.

Total Depth: 21 feet
No groundwater encountered

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FIGURE A-4

BORING RECORD

PROJECT NAME	PROJECT NUMBER	BORING
La Mesa Apartment Development	SD634	**B-5**

SITE LOCATION	START	FINISH	SHEET NO.
Southeast of Allison Avenue and Date Avenue	9/19/2019	9/19/2019	1 of 1

DRILLING COMPANY	DRILLING METHOD	LOGGED BY	CHECKED BY
Pacific Drilling Company	Hollow Stem Auger	SRN	MAF

DRILLING EQUIPMENT	BORING DIA. (in)	TOTAL DEPTH (ft)	GROUND ELEV (ft)	DEPTH/*ELEV.* GROUND WATER (ft)
Marl M10 Truck Mounted Rig (Grizzly)	6	12	533	▼ N/A / *na*

SAMPLING METHOD	NOTES
Hammer: 140 lbs., Drop: 30 in. (Automatic)	ETR ~ 92%, N_{60} ~ 92/60 * N ~ 1.53 * N

DEPTH (feet)	ELEVATION (feet)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS / 6 IN)	BLOW/FT "N"	N_{60}	MOISTURE (%)	DRY DENSITY (pcf)	OTHER TESTS	DEPTH (feet)	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
												PAVEMENT: Asphalt Concrete (4"), no Base (0").
	530		B-1						PA EI~5 R~11			**FILL:** CLAYEY SAND WITH GRAVEL (SC); medium dense; yellowish brown; moist; mostly fine to medium SAND; some fines; little GRAVEL; low plasticity. (20% Gravel; 52% Sand; 28% Fines)
5			R-2	18 50 (4")	93	95	5.4	---		5		**STADIUM CONGLOMERATE:** Poorly indurated SANDSTONE; fine to medium grained; moderately weathered; very soft; unfractured (POORLY GRADED SAND (SP); very dense; yellowish brown; moist; mostly fine to medium SAND; few to little fines; few GRAVEL; nonplastic to low plasticity).
	525											
10			S-3	12 50 (5")	72	110				10		Poorly indurated COBBLE CONGLOMERATE; medium to coarse grained; moderately weathered; soft; unfractured (POORLY GRADED GRAVEL (GP); very dense; brown; moist; nonplastic; mostly GRAVEL and COBBLE; few SAND; trace fines). Very difficult drilling with rig chatter from 11' to 12'.
	520											Total Depth: 12 Feet Refusal on gravel and cobble No groundwater encountered
15										15		
	515											
20										20		
	510											

GDC_LOG_BORING_MMX_SOIL_SD SD634 LOGS.GPJ GDCLOG.GDT 10/9/19

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FIGURE

A-5

BORING RECORD

PROJECT NAME	La Mesa Apartment Development
PROJECT NUMBER	SD634
BORING	I-1
SITE LOCATION	Southeast of Allison Avenue and Date Avenue
START	9/19/2019
FINISH	9/20/2019
SHEET NO.	1 of 1
DRILLING COMPANY	Pacific Drilling Company
DRILLING METHOD	Hollow Stem Auger
LOGGED BY	SRN
CHECKED BY	MAF
DRILLING EQUIPMENT	Marl M10 Truck Mounted Rig (Grizzly)
BORING DIA. (in)	6
TOTAL DEPTH (ft)	5
GROUND ELEV (ft)	525
DEPTH/*ELEV.* GROUND WATER (ft)	▼ N/A / *na*
SAMPLING METHOD	Hammer: 140 lbs., Drop: 30 in. (Automatic)
NOTES	ETR ~ 92%, N_{60} ~ 92/60 * N ~ 1.53 * N

DEPTH (feet)	ELEVATION (feet)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS / 6 IN)	BLOW/FT "N"	N_{60}	MOISTURE (%)	DRY DENSITY (pcf)	OTHER TESTS	DEPTH (feet)	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
												PAVEMENT: Asphalt Concrete (3"), no Base (0").
			B-1									**FILL:** SILTY SAND (SM); medium dense; reddish brown; moist; mostly fine SAND; some fines; nonplastic.
			R-2	14 46 50 (2")	(REF)	(REF)	---	---				**STADIUM CONGOMERATE:** Poorly indurated COBBLE CONGLOMERATE; medium to coarse grained; moderately weathered; soft; unfractured (CLAYEY GRAVEL (GC); very dense; yellowish brown; moist; mostly GRAVEL and COBBLE; little SAND; little fines; low plasticity).
5	520									5		
												Total Depth: 5 feet No groundwater encountered Converted to borehole percolation test
10	515									10		
15	510									15		
20	505									20		

GDC_LOG_BORING_MMX_SOIL_SD SD634 LOGS.GPJ GDCLOG.GDT 10/9/19

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FIGURE A-6

BORING RECORD

PROJECT NAME	PROJECT NUMBER	BORING
La Mesa Apartment Development	SD634	**I-2**

SITE LOCATION	START	FINISH	SHEET NO.
Southeast of Allison Avenue and Date Avenue	9/19/2019	9/20/2019	1 of 1

DRILLING COMPANY	DRILLING METHOD	LOGGED BY	CHECKED BY
Pacific Drilling Company	Hollow Stem Auger	SRN	MAF

DRILLING EQUIPMENT	BORING DIA. (in)	TOTAL DEPTH (ft)	GROUND ELEV (ft)	DEPTH/*ELEV.* GROUND WATER (ft)
Marl M10 Truck Mounted Rig (Grizzly)	6	5	534	▼ N/A / *na*

SAMPLING METHOD	NOTES
Hammer: 140 lbs., Drop: 30 in. (Automatic)	ETR ~ 92%, N_{60} ~ 92/60 * N ~ 1.53 * N

DEPTH (feet)	ELEVATION (feet)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS / 6 IN)	BLOW/FT "N"	N_{60}	MOISTURE (%)	DRY DENSITY (pcf)	OTHER TESTS	DEPTH (feet)	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
			B-1									**FILL:** CLAYEY SAND (SC); medium dense to dense; reddish brown; moist; mostly fine to coarse SAND; some fines; few GRAVEL; low plasticity.
			R-2	4 14 18	32	32	10.7	115.8	CP DS			
	530											
5										5		Total Depth: 5 feet No groundwater encountered Converted to borehole percolation test
	525											
10										10		
	520											
15										15		
	515											
20										20		
	510											

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FIGURE

A-7

APPENDIX B
LABORATORY TESTING

APPENDIX B

LABORATORY TESTING

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

Classification: Soils were visually classified 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 general accordance with ASTM D422, and were used to supplement visual soil classifications. The test results are summarized in Figures B-1.1 through B-1.6.

Atterberg Limits: ASTM D4318 was used to determine the liquid and plastic limits, and plasticity index of selected samples. The results are shown in selected Figures B-1.1 through B-1.6.

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

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

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

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

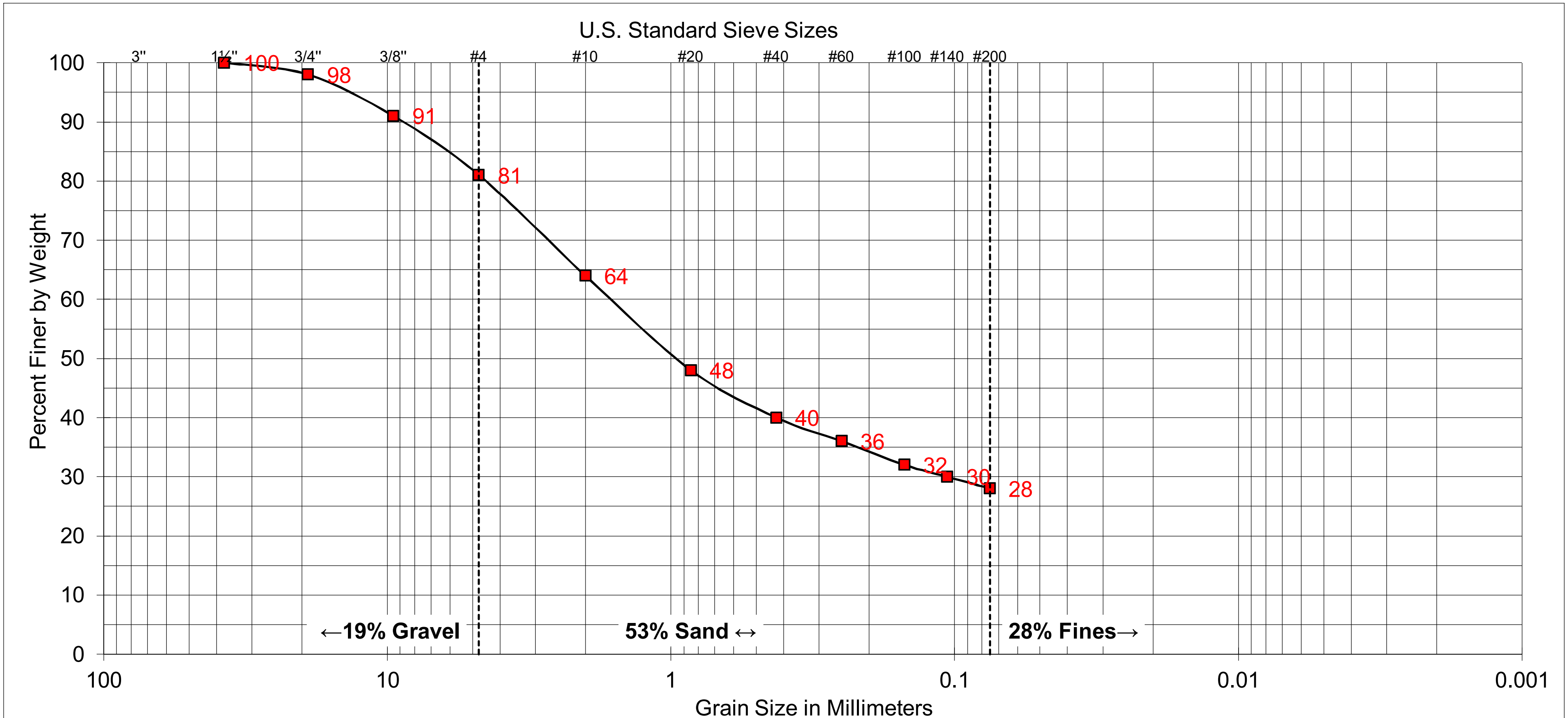
APPENDIX B

LABORATORY TESTING (Continued)

Maximum Density/Optimum Moisture: The maximum density and optimum moisture content of a selected soil sample were determined using ASTM D1557 (modified Proctor). The results were corrected for over-size material using ASTM D4718. The test results are summarized in Figure B-4.

Direct Shear: The shear strength of a selected sample of the on-site soil was assessed using direct shear testing performed in general accordance with ASTM D3080. The gravel was first removed from the sample, and the remaining matrix material was then remolded to approximate 90 percent relative compaction prior to shear testing. The test results are shown in Figure B-5.1. Similar direct shear tests that we have previously conducted on remolded samples of the matrix material from the Stadium Conglomerate are presented in Figures B-5.2 and B-5.3.

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-6.1 and B-6.2.



COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND
GRAVEL		SAND			CLAY

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

UNIFIED SOIL CLASSIFICATION: SC

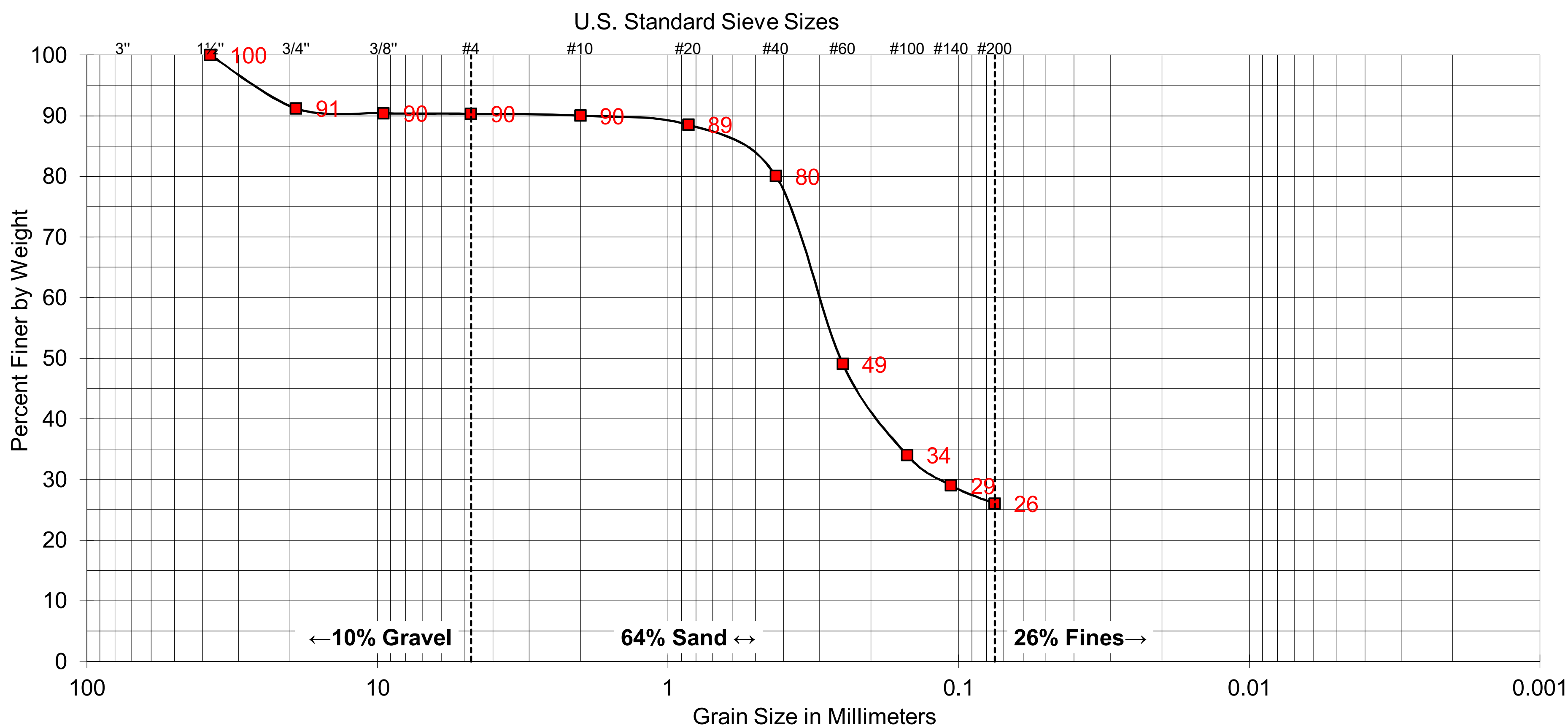
DESCRIPTION: CLAYEY SAND WITH GRAVEL

ATTERBERG LIMITS	
LIQUID LIMIT:	35
PLASTIC LIMIT:	17
PLASTICITY INDEX:	18

GROUP DELTA

SOIL CLASSIFICATION

Document No. 19-0141
Project No. SD634
FIGURE B-1.1



COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND
GRAVEL		SAND			CLAY

SAMPLE	
BORING NUMBER:	B-2
SAMPLE DEPTH:	10' - 11½'

UNIFIED SOIL CLASSIFICATION: SC

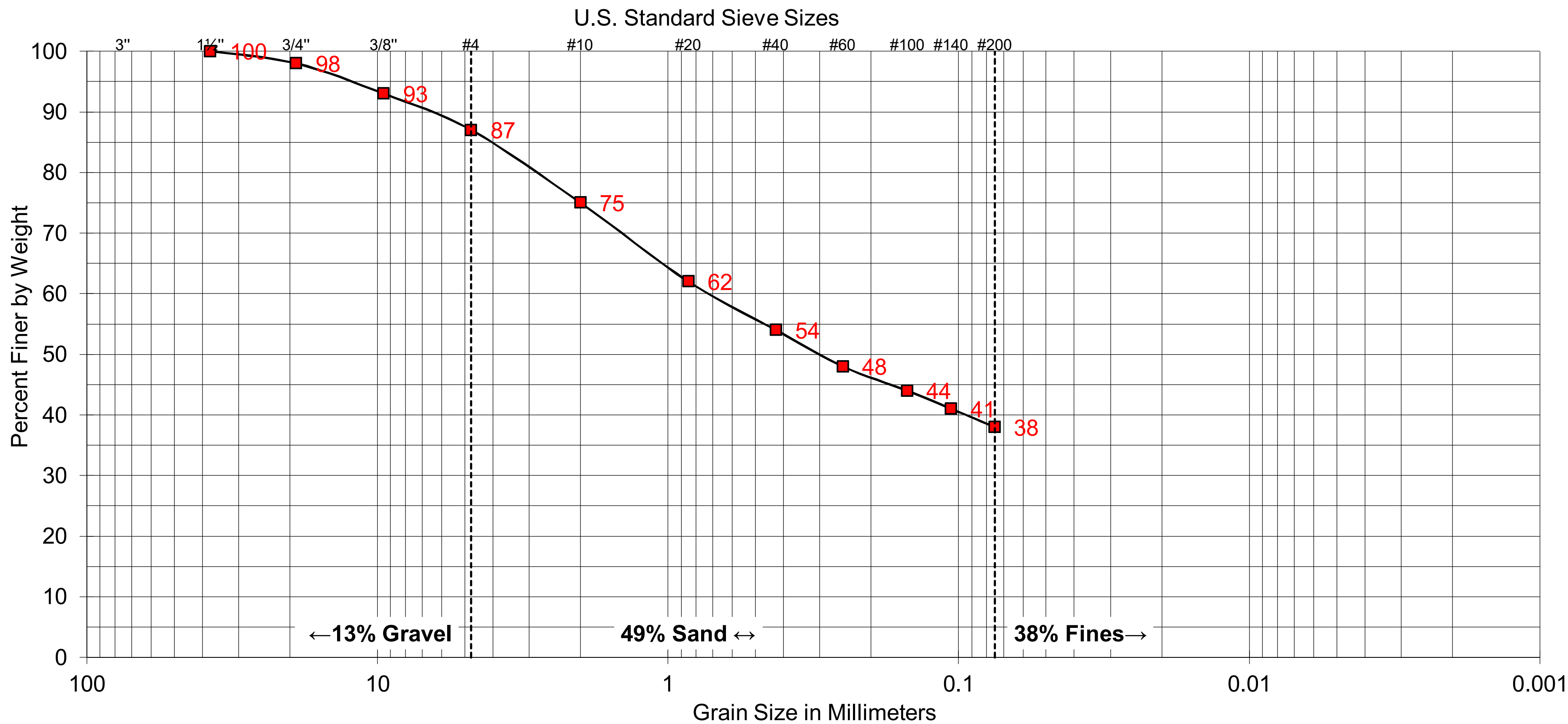
DESCRIPTION: CLAYEY SAND

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

GROUP DELTA

SOIL CLASSIFICATION

Document No. 19-0141
Project No. SD634
FIGURE B-1.2



COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND
GRAVEL		SAND			CLAY

SAMPLE	
BORING NUMBER:	B-3
SAMPLE DEPTH:	0' - 5'

UNIFIED SOIL CLASSIFICATION: SC

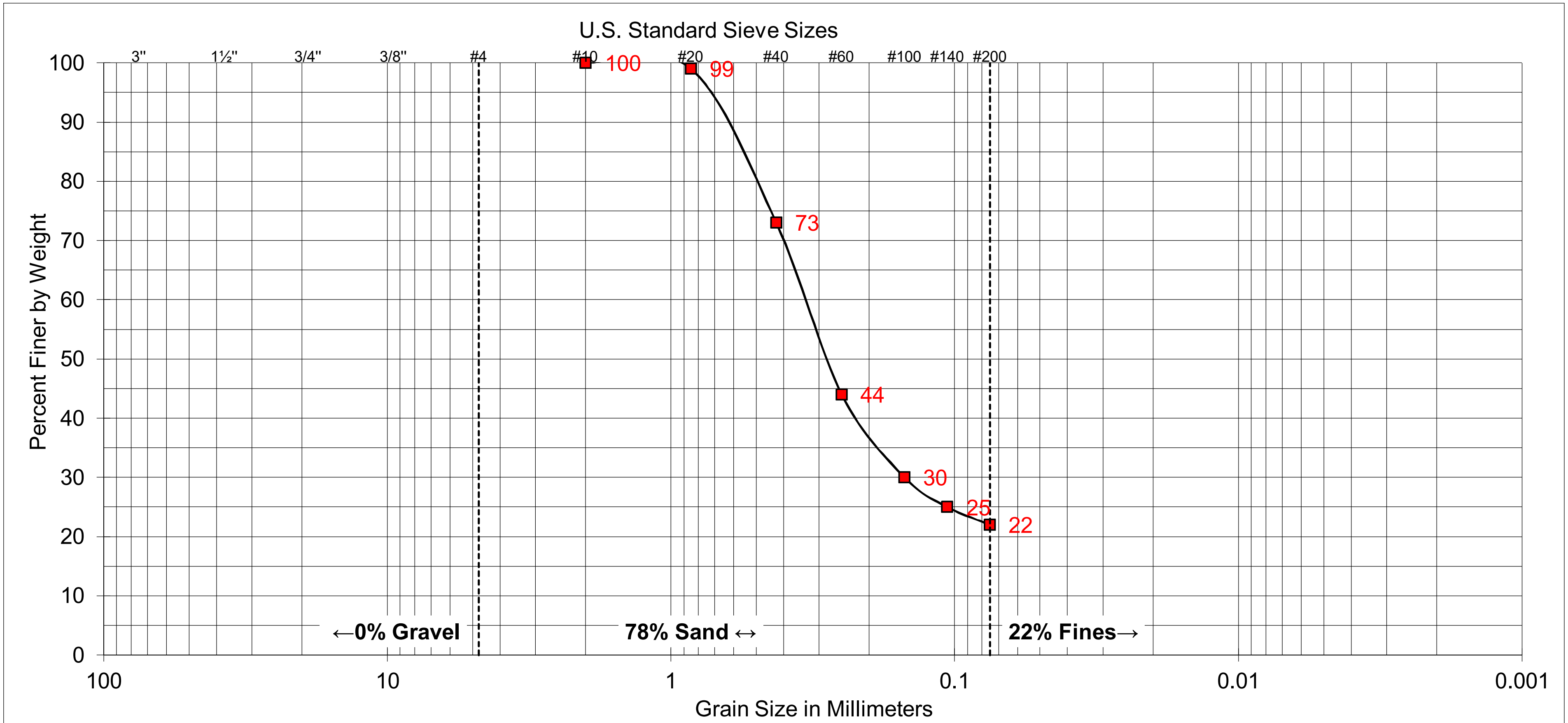
DESCRIPTION: CLAYEY SAND

ATTERBERG LIMITS	
LIQUID LIMIT:	36
PLASTIC LIMIT:	16
PLASTICITY INDEX:	20

GROUP DELTA

SOIL CLASSIFICATION

Document No. 19-0141
Project No. SD634
FIGURE B-1.3



COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND
GRAVEL		SAND			CLAY

SAMPLE	
BORING NUMBER:	B-3
SAMPLE DEPTH:	20' - 21½'

UNIFIED SOIL CLASSIFICATION: SM

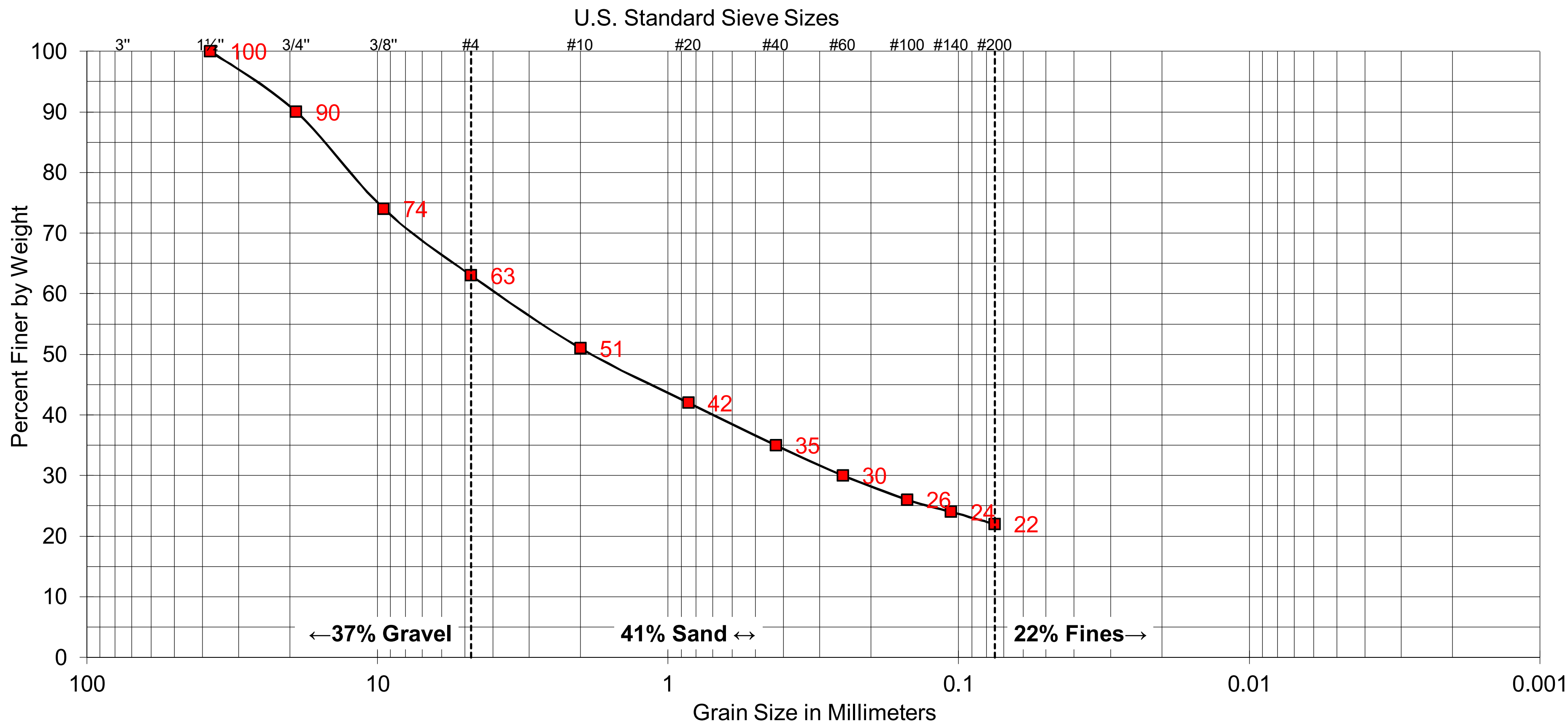
DESCRIPTION: SILTY SAND

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

GROUP DELTA

SOIL CLASSIFICATION

Document No. 19-0141
Project No. SD634
FIGURE B-1.4



COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND
GRAVEL		SAND			CLAY

SAMPLE	
BORING NUMBER:	B-4
SAMPLE DEPTH:	0' - 5'

UNIFIED SOIL CLASSIFICATION: SC

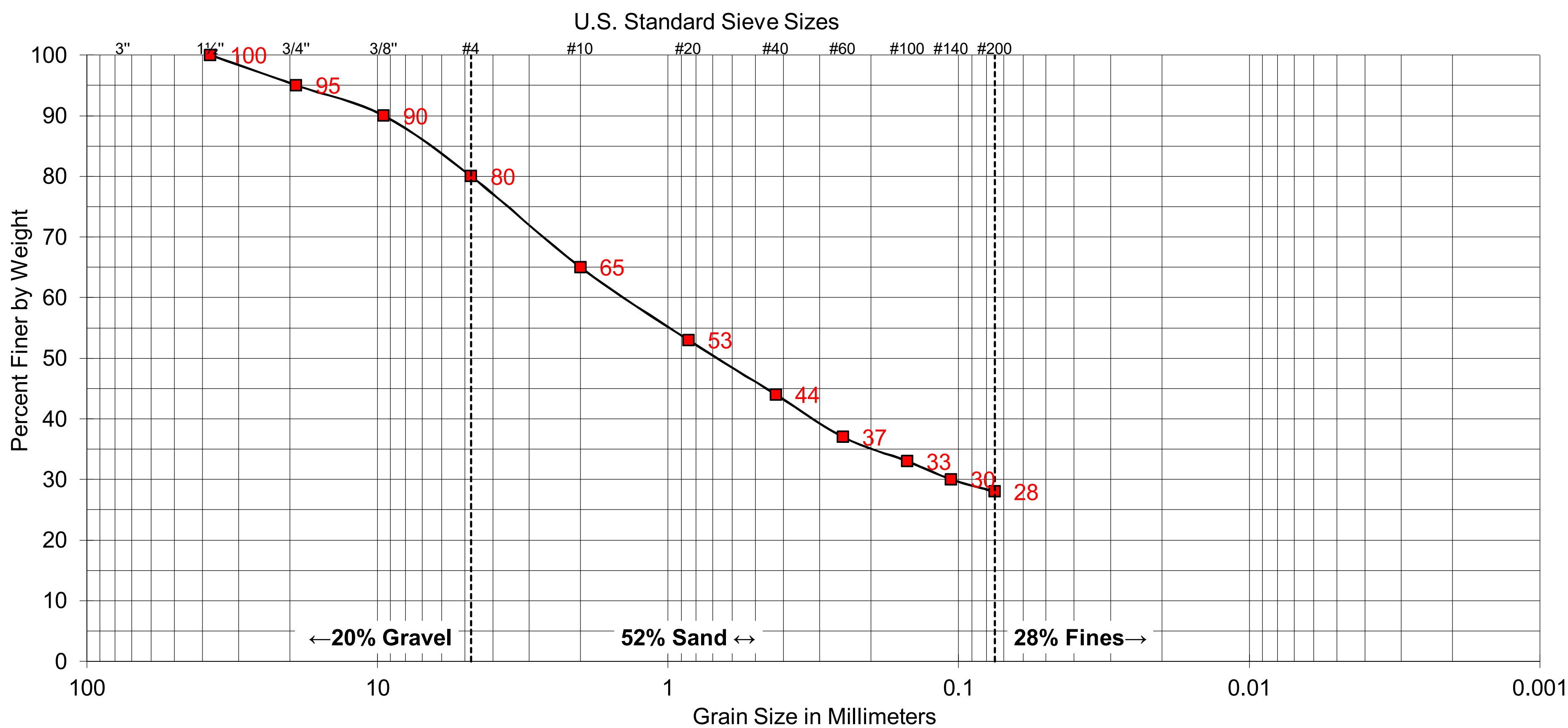
DESCRIPTION: CLAYEY SAND WITH GRAVEL

ATTERBERG LIMITS	
LIQUID LIMIT:	31
PLASTIC LIMIT:	15
PLASTICITY INDEX:	16

GROUP DELTA

SOIL CLASSIFICATION

Document No. 19-0141
Project No. SD634
FIGURE B-1.5



COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND
GRAVEL		SAND			CLAY

SAMPLE	
BORING NUMBER:	B-5
SAMPLE DEPTH:	0' - 5'

UNIFIED SOIL CLASSIFICATION: SC

DESCRIPTION: CLAYEY SAND WITH GRAVEL

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

GROUP DELTA

SOIL CLASSIFICATION

Document No. 19-0141
Project No. SD634
FIGURE B-1.6

EXPANSION TEST RESULTS
(ASTM D4829)

SAMPLE	DESCRIPTION	EXPANSION INDEX
B-2 @ 0' – 5'	**Fill**: Brown clayey sand with gravel (SC).	8
B-3 @ 0' – 5'	**Fill**: Brown clayey sand (SC).	28
B-5 @ 0' – 5'	**Fill**: Yellow brown clayey sand with gravel (SC).	5

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	pH	RESISTIVITY [OHM-CM]	SULFATE CONTENT [%]	CHLORIDE CONTENT [%]
B-2 @ 0' – 5'	6.4	3,650	0.01	< 0.01
B-3 @ 0' – 5'	6.0	1.790	0.02	0.01
B-4 @ 0' – 5'	6.6	2,170	0.03	< 0.01

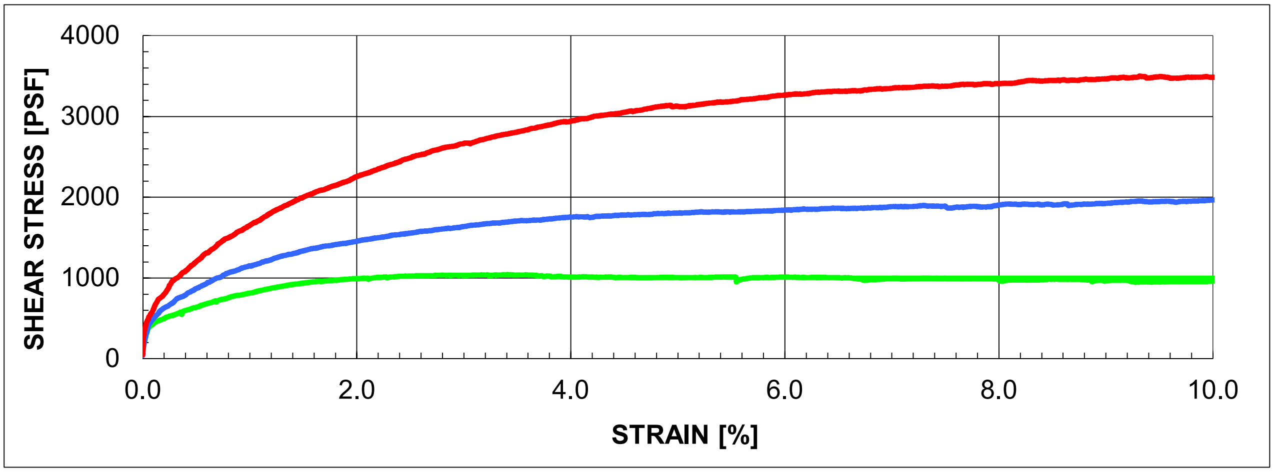
SULFATE CONTENT [%]	SULFATE EXPOSURE	CEMENT TYPE
0.00 to 0.10	Negligible	-
0.10 to 0.20	Moderate	II, IP(MS), IS(MS)
0.20 to 2.00	Severe	V
Above 2.00	Very Severe	V plus pozzolan

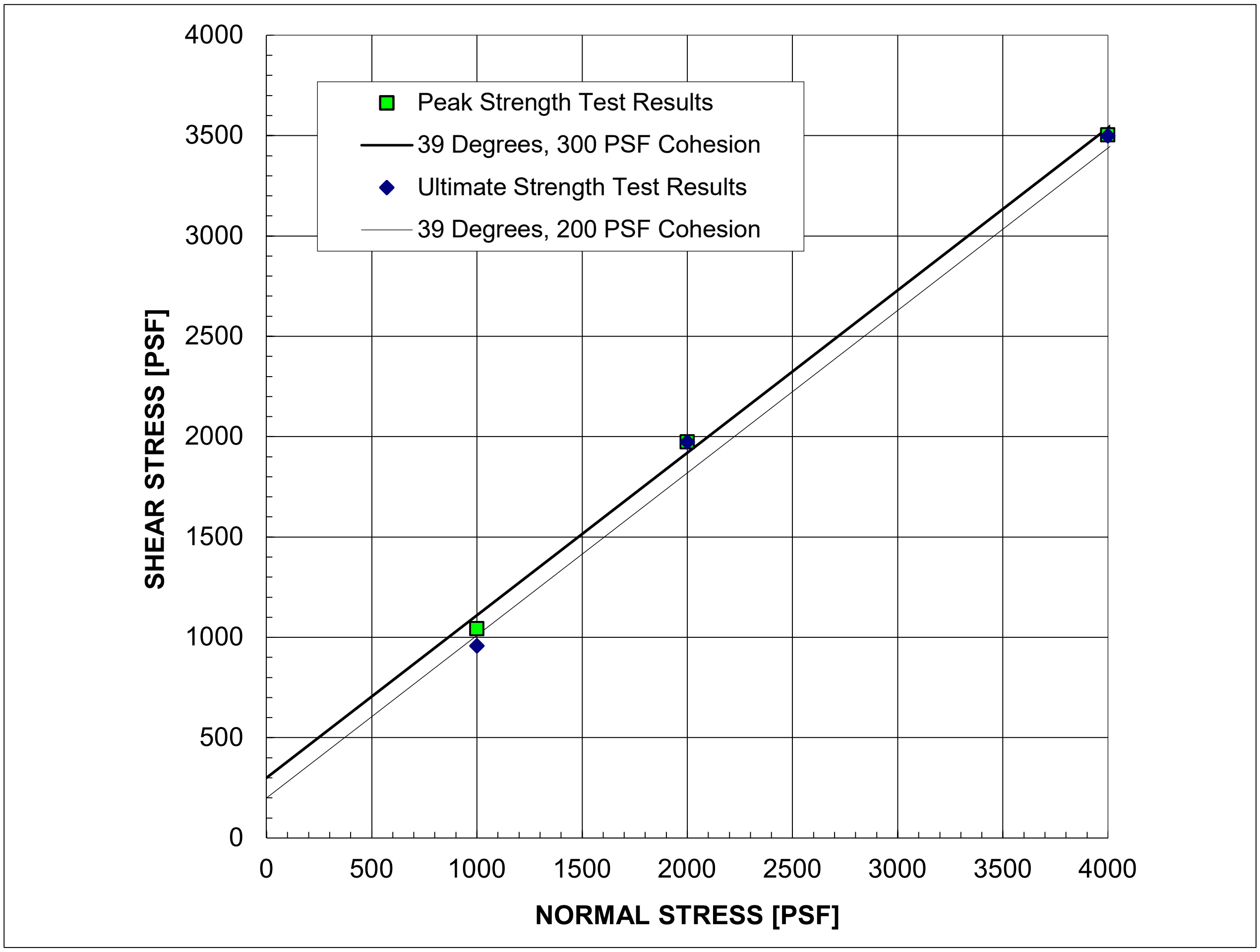
SOIL RESISTIVITY	GENERAL DEGREE OF CORROSIVITY TO FERROUS
0 to 1,000	Very Corrosive
1,000 to 2,000	Corrosive
2,000 to 5,000	Moderately Corrosive
5,000 to 10,000	Mildly Corrosive
Above 10,000	Slightly Corrosive

CHLORIDE (Cl) CONTENT	GENERAL DEGREE OF
0.00 to 0.03	Negligible
0.03 to 0.15	Corrosive
Above 0.15	Severely Corrosive

MAXIMUM DENSITY & OPTIMUM MOISTURE
(ASTM D1557 & D4718)

SAMPLE ID (I-2 @ 0' – 5')	**DESCRIPTION**	**MAXIMUM DENSITY** [lb/ft^3]	**OPTIMUM MOISTURE** [%]
Max #1A	**FILL**: Reddish brown clayey sand with 0% gravel (SC).	129.0	8.6
Max #1B	**FILL**: Reddish brown clayey sand with 5% gravel (SC).	130.3	8.2
Max #1C	**FILL**: Reddish brown clayey sand with 10% gravel (SC).	131.7	7.8
Max #1D	**FILL**: Reddish brown clayey sand with 15% gravel (SC).	133.1	7.5
Max #1E	**FILL**: Reddish brown clayey sand with 20% gravel (SC).	134.5	7.1
Max #1F	**FILL**: Reddish brown clayey sand with 25% gravel (SC).	136.0	6.7
Max #1G	**FILL**: Reddish brown clayey sand with 30% gravel (SC).	137.4	6.3





SAMPLE: I-2 @ 0' - 5'

Fill: Reddish brown clayey sand (SC).
(Remolded to ~90% Maximum @ Optimum)

	PEAK	ULTIMATE
ϕ'	39 °	39 °
C'	300 PSF	200 PSF

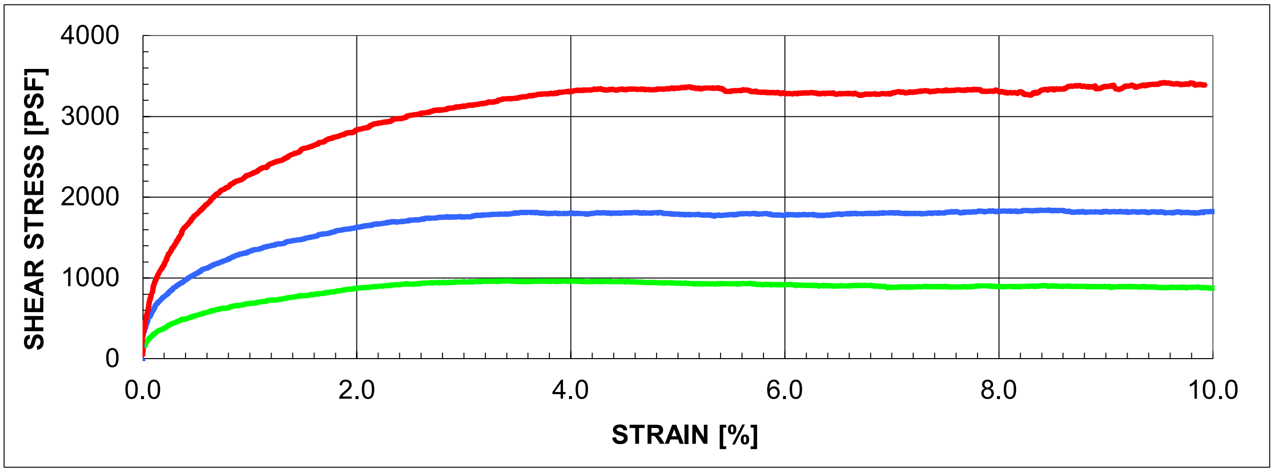
	IN-SITU	AS-TESTED
γ_d	120.4 PCF	120.4 PCF
w_c	8.9 %	14.8 %

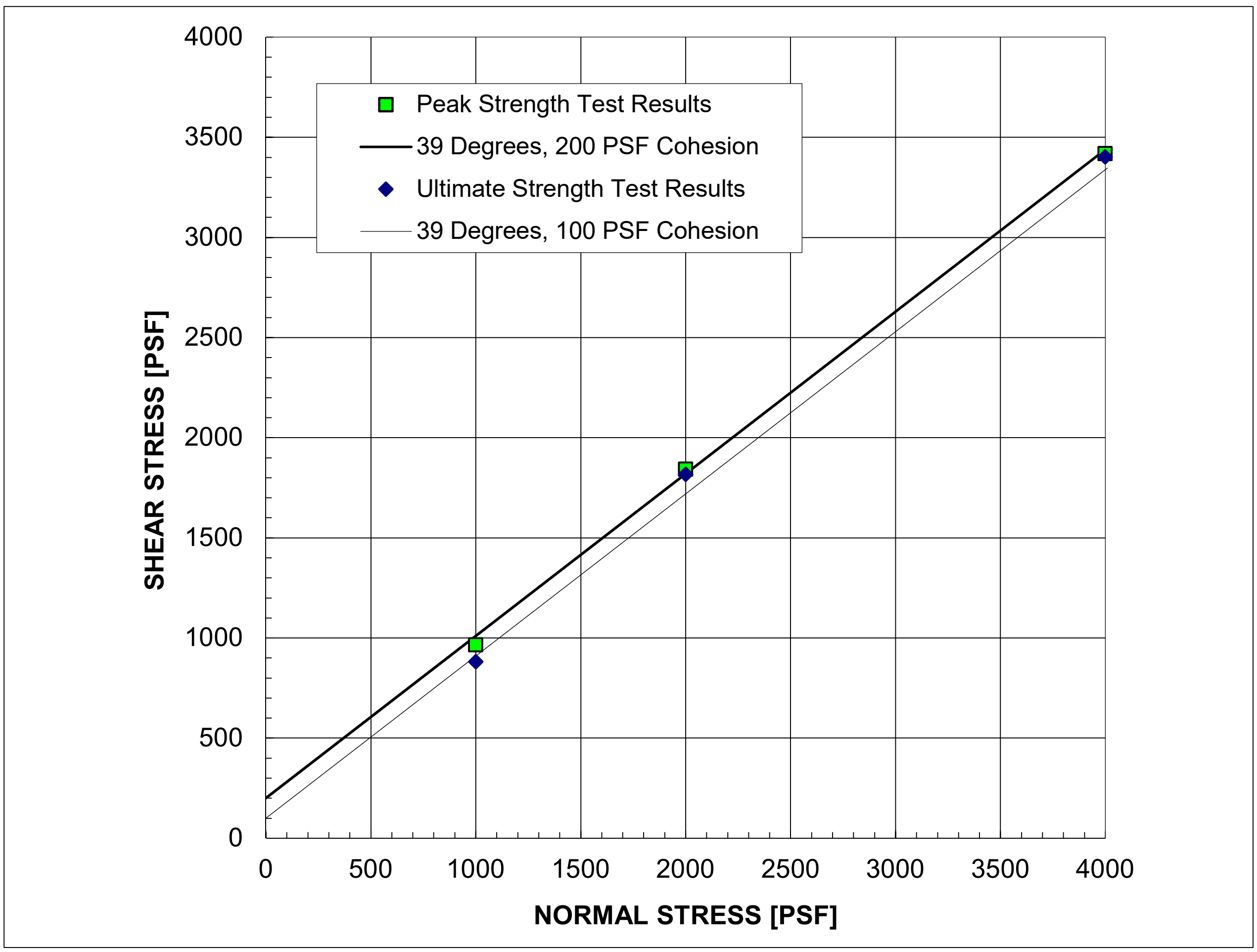
STRAIN RATE: 0.0030 IN/MIN
(Sample was consolidated and drained)

GROUP DELTA

DIRECT SHEAR TEST RESULTS

Document No. 19-0141
Project No. SD634
FIGURE B-5.1



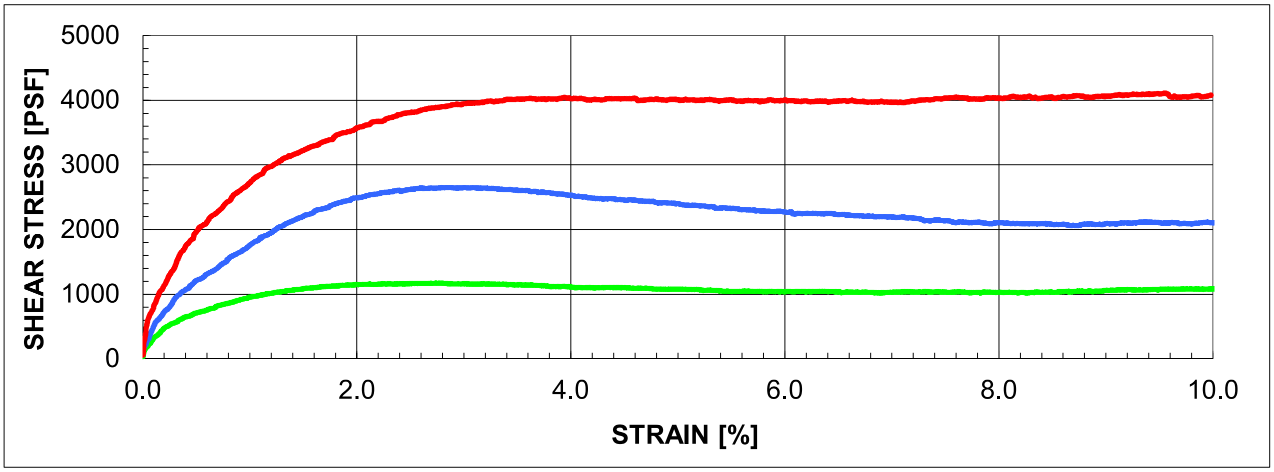


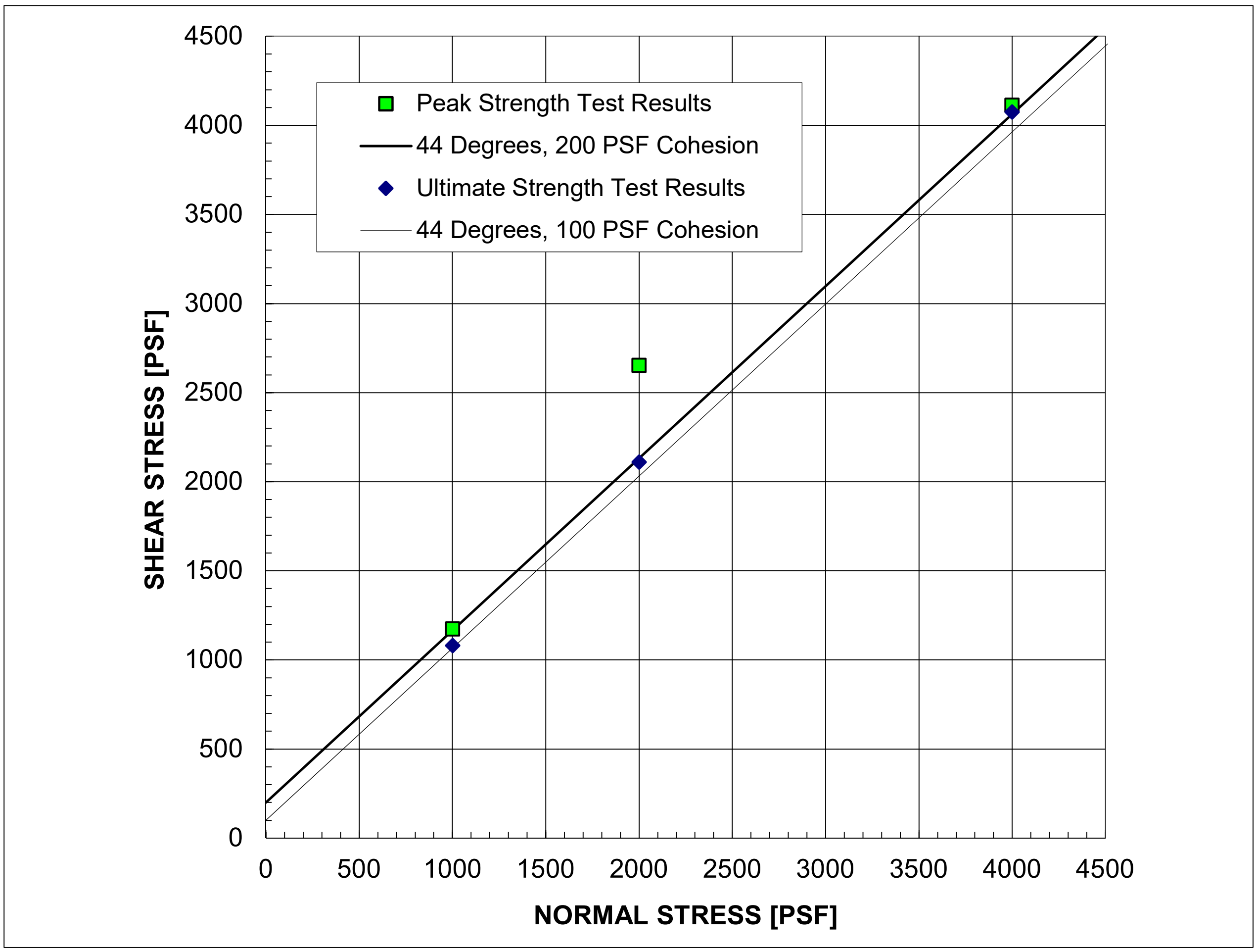
Stadium Conglomerate:
Remolded brown silty sand matrix (SM).

	PEAK	ULTIMATE
ϕ'	39 °	39 °
C'	200 PSF	100 PSF

STRAIN RATE: 0.0030 IN/MIN
(Sample was consolidated and drained)

	IN-SITU	AS-TESTED
γ_d	114.7 PCF	114.7 PCF
w_c	8.8 %	17.1 %





Stadium Conglomerate:
Remolded brown silty sand matrix (SM).

	PEAK	ULTIMATE
φ'	44 °	44 °
C'	200 PSF	100 PSF

STRAIN RATE: 0.0050 IN/MIN

(Sample was consolidated and drained)

	IN-SITU	AS-TESTED
γ_d	113.2 PCF	113.2 PCF
w_c	9.2 %	17.4 %

GROUP DELTA

DIRECT SHEAR TEST RESULTS

Document No. 19-0141
Project No. SD634
FIGURE B-5.3

SAMPLE NO.: B-4 **SAMPLE DATE:** 9/19/19

SAMPLE LOCATION: 0' - 5' **TEST DATE:** 10/7/19

SAMPLE DESCRIPTION: Dark brown clayey sand with gravel (SC)

LABORATORY TEST DATA

	TEST SPECIMEN	**1**	**2**	**3**	**4**	**5**	
A	COMPACTOR PRESSURE	150	120	190			[PSI]
B	INITIAL MOISTURE	1.1	1.1	1.1			[%]
C	BATCH SOIL WEIGHT	1200	1200	1200			[G]
D	WATER ADDED	85	94	78			[ML]
E	WATER ADDED (D*(100+B)/C)	7.2	7.9	6.6			[%]
F	COMPACTION MOISTURE (B+E)	8.3	9.0	7.7			[%]
G	MOLD WEIGHT	2019.7	2013.7	2091.8			[G]
H	TOTAL BRIQUETTE WEIGHT	3219.6	3162.2	3256.9			[G]
I	NET BRIQUETTE WEIGHT (H-G)	1199.9	1148.5	1165.1			[G]
J	BRIQUETTE HEIGHT	2.53	2.46	2.45			[IN]
K	DRY DENSITY (30.3*I/((100+F)*J))	132.7	129.8	133.8			[PCF]
L	EXUDATION LOAD	4305	3359	5120			[LB]
M	EXUDATION PRESSURE (L/12.54)	343	268	408			[PSI]
N	STABILOMETER AT 1000 LBS	48	55	47			[PSI]
O	STABILOMETER AT 2000 LBS	118	128	107			[PSI]
P	DISPLACEMENT FOR 100 PSI	5.02	5.75	4.57			[Turns]
Q	R VALUE BY STABILOMETER	15	10	21			
R	CORRECTED R-VALUE (See Fig. 14)	15	10	21			
S	EXPANSION DIAL READING	0.0000	0.0000	0.0000			[IN]
T	EXPANSION PRESSURE (S*43,300)	0	0	0			[PSF]
U	COVER BY STABILOMETER	0.77	0.82	0.72			[FT]
V	COVER BY EXPANSION	0.00	0.00	0.00			[FT]

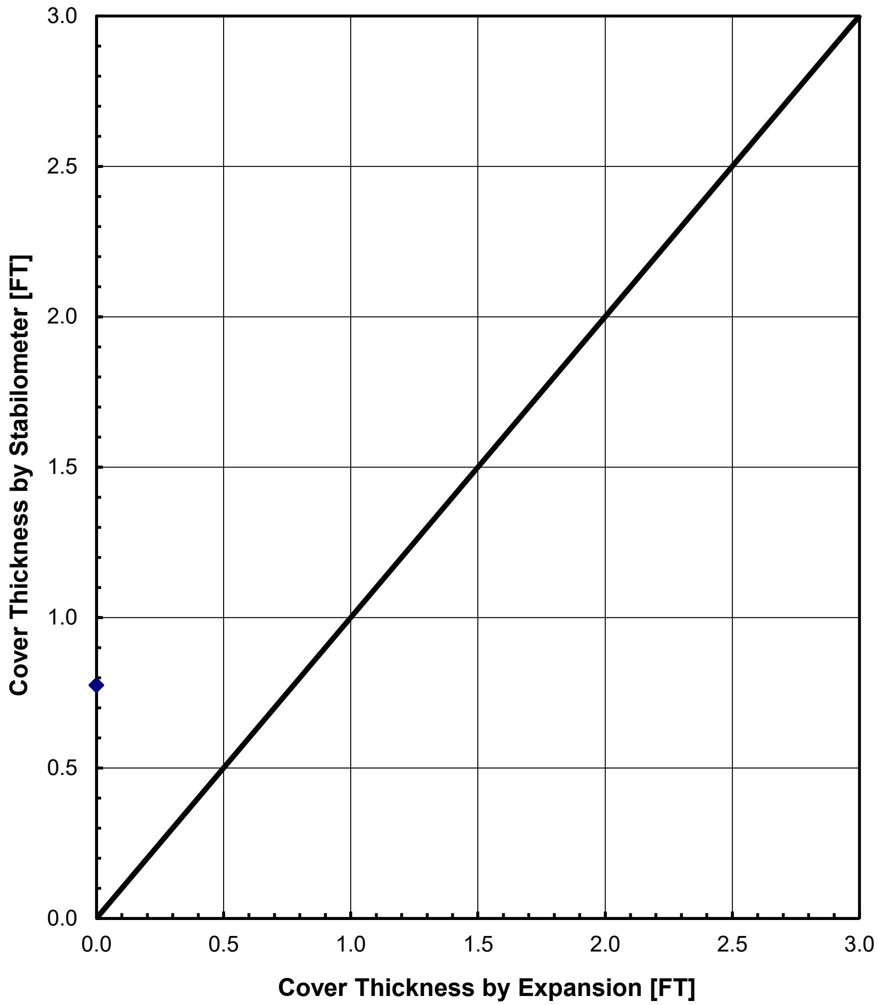
TRAFFIC INDEX:	5.0
GRAVEL FACTOR:	1.46
UNIT WEIGHT OF COVER [PCF]:	130
R-VALUE BY EXUDATION:	12
R-VALUE BY EXPANSION:	30
R-VALUE AT EQUILIBRIUM:	12

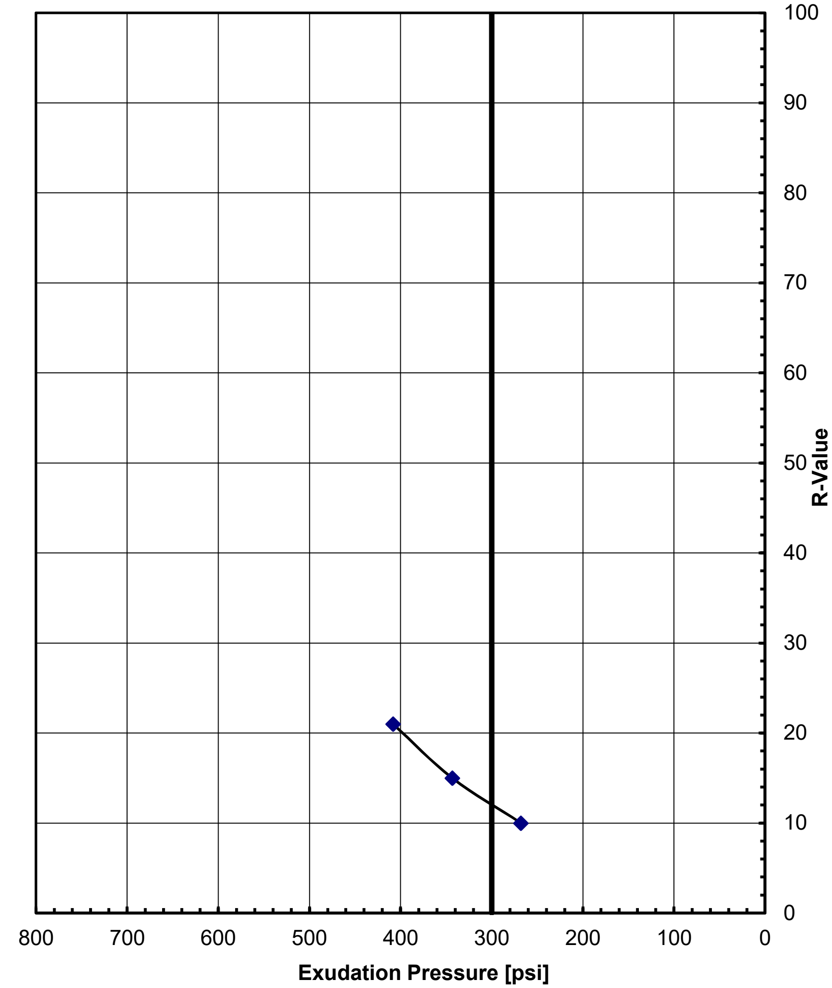
*Note: Gravel factor estimated from pavement section using CTM 301, Section C, Part b.

REV. 2, DATED 1/31/15

Sample: B-4 @ 0' - 5'

R-Value at Equilibrium: 12





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COVER AND EXUDATION CHARTS

Document No. 19-0141
Project No. SD634
FIGURE B-6.1b

SAMPLE NO.: B-5 **SAMPLE DATE:** 9/19/19

SAMPLE LOCATION: 0' - 5' **TEST DATE:** 10/7/19

SAMPLE DESCRIPTION: Yellowish brown clayey sand (SC)

LABORATORY TEST DATA

	TEST SPECIMEN	1	2	3	4	5	
A	COMPACTOR PRESSURE	100	50	130			[PSI]
B	INITIAL MOISTURE	3.6	3.6	3.6			[%]
C	BATCH SOIL WEIGHT	1200	1200	1200			[G]
D	WATER ADDED	85	95	75			[ML]
E	WATER ADDED (D*(100+B)/C)	7.3	8.2	6.5			[%]
F	COMPACTION MOISTURE (B+E)	10.9	11.8	10.1			[%]
G	MOLD WEIGHT	2013.0	2010.2	2010.2			[G]
H	TOTAL BRIQUETTE WEIGHT	3148.3	3178.8	3181.1			[G]
I	NET BRIQUETTE WEIGHT (H-G)	1135.3	1168.6	1170.9			[G]
J	BRIQUETTE HEIGHT	2.51	2.53	2.52			[IN]
K	DRY DENSITY (30.3*I/((100+F)*J))	123.5	125.2	127.9			[PCF]
L	EXUDATION LOAD	3301	2944	4446			[LB]
M	EXUDATION PRESSURE (L/12.54)	263	235	355			[PSI]
N	STABILOMETER AT 1000 LBS	59	65	51			[PSI]
O	STABILOMETER AT 2000 LBS	134	150	120			[PSI]
P	DISPLACEMENT FOR 100 PSI	5.43	6.35	5.10			[Turns]
Q	R VALUE BY STABILOMETER	8	3	14			
R	CORRECTED R-VALUE (See Fig. 14)	8	3	14			
S	EXPANSION DIAL READING	0.0000	0.0000	0.0000			[IN]
T	EXPANSION PRESSURE (S*43,300)	0	0	0			[PSF]
U	COVER BY STABILOMETER	0.84	0.88	0.78			[FT]
V	COVER BY EXPANSION	0.00	0.00	0.00			[FT]

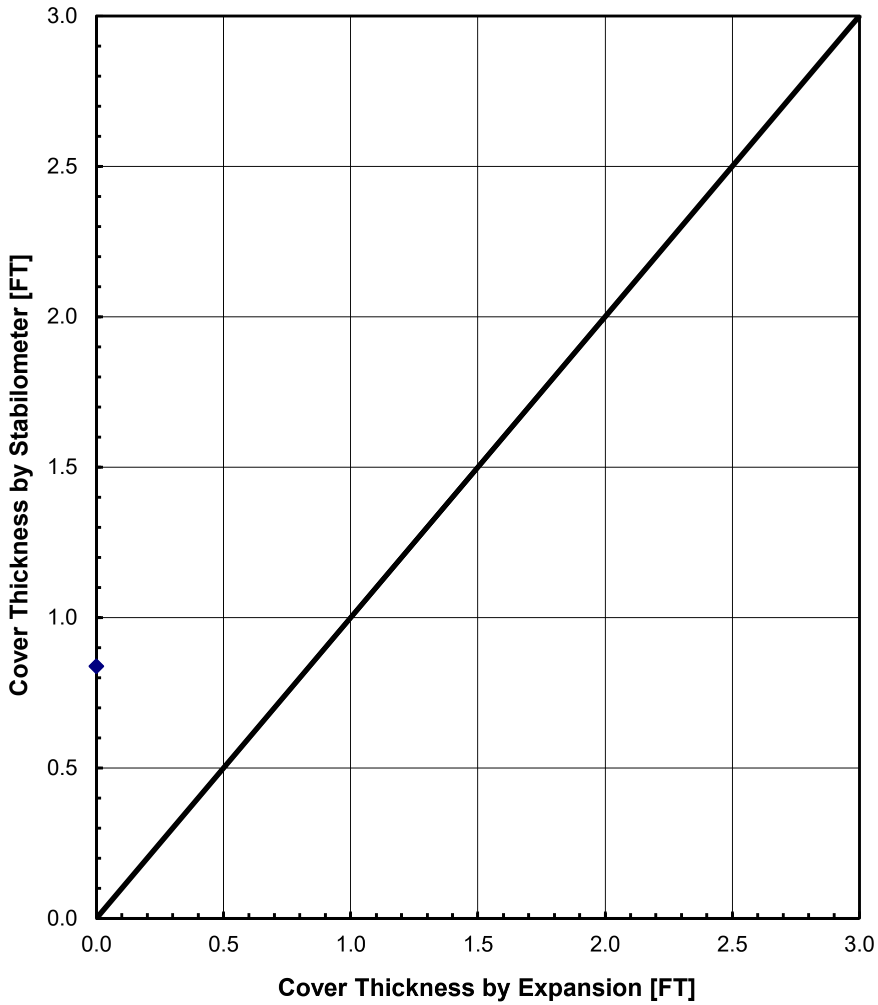
TRAFFIC INDEX:	5.0
GRAVEL FACTOR:	1.46
UNIT WEIGHT OF COVER [PCF]:	130
R-VALUE BY EXUDATION:	11
R-VALUE BY EXPANSION:	23
R-VALUE AT EQUILIBRIUM:	11

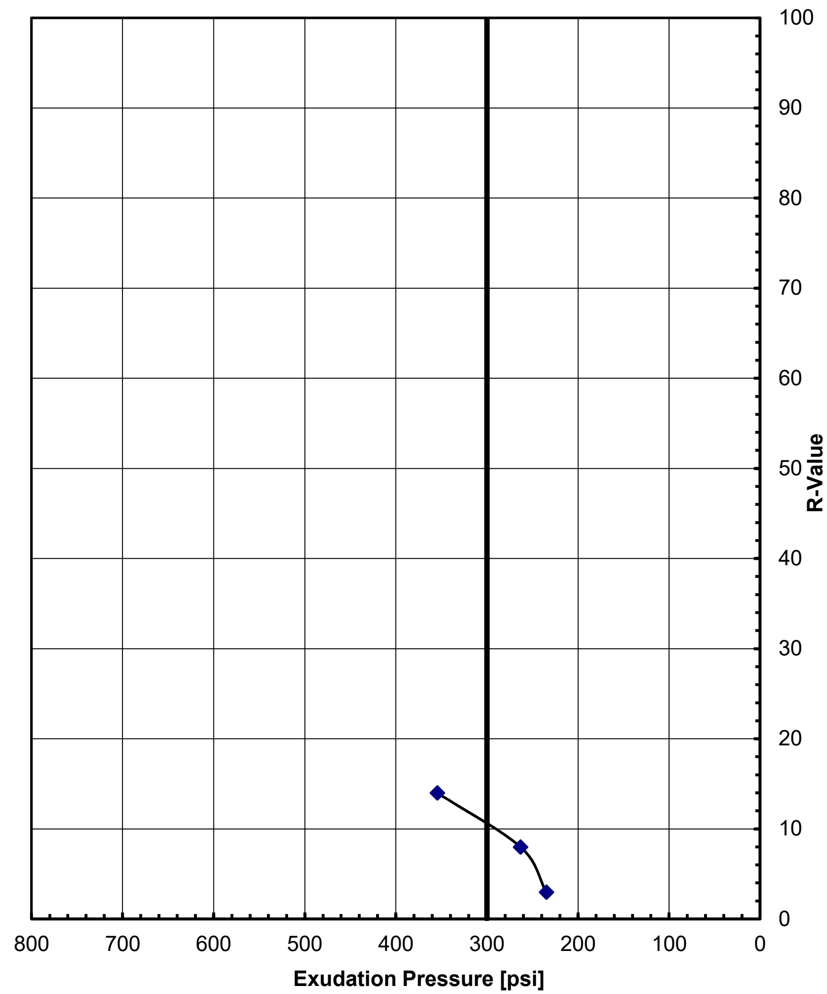
*Note: Gravel factor estimated from pavement section using CTM 301, Section C, Part b.

REV. 2, DATED 1/31/15

Sample: B-5, 0' - 5'

R-Value at Equilibrium: 11





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COVER AND EXUDATION CHARTS

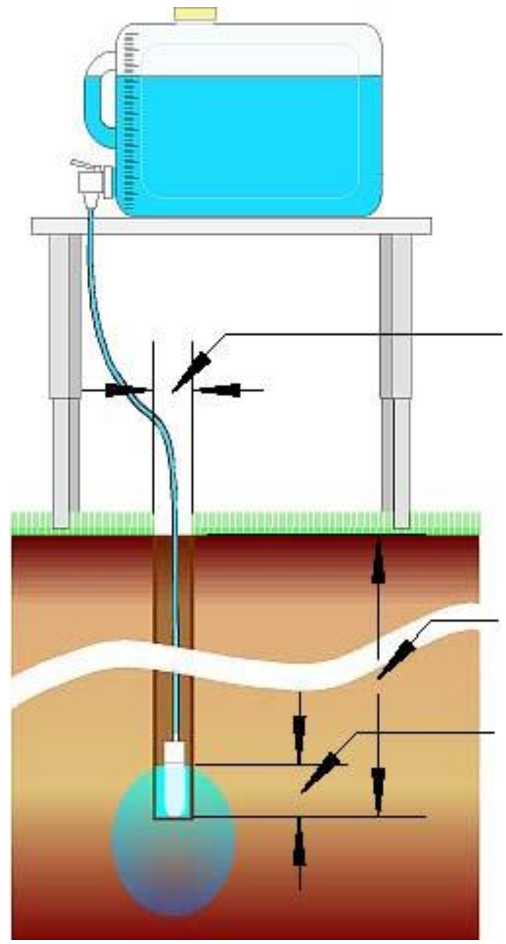
Document No. 19-0141
Project No. SD634
FIGURE B-6.2b

APPENDIX C
INFILTRATION ASSESSMENT

APPENDIX C

INFILTRATION ASSESSMENT

We understand that bioretention basins, detention basins or swales may be incorporated into the site development. In order to aid in BMP design, the vertical infiltration rates were estimated at two locations using the borehole percolation method. The general configuration of the borehole test is depicted schematically in the figure below. The field infiltration tests were conducted between September 19th and 20th, 2019. The approximate infiltration test locations are shown on the Exploration Plan, Figure 3A. The infiltration test results are presented in Figures C-1.1 to C-2.2.



Worksheet C.4-1 of the 2016 City of La Mesa BMP Design Manual is shown in the following figures. Per Table D.3-1 of the BMP Design manual, the borehole percolation test may be used for both planning level screening and BMP design purposes. Per Section D.4.5 of the BMP Manual, the testing "…shall be conducted at approximately the same depth and the same material as the base of the proposed storm water BMP." The Storm Water Manual requires that two infiltration tests be conducted within 50-feet of each proposed BMP. The BMP locations and configurations have yet to be determined. Consequently, we conducted two infiltration tests at the locations and depths indicated by the project civil design engineer.

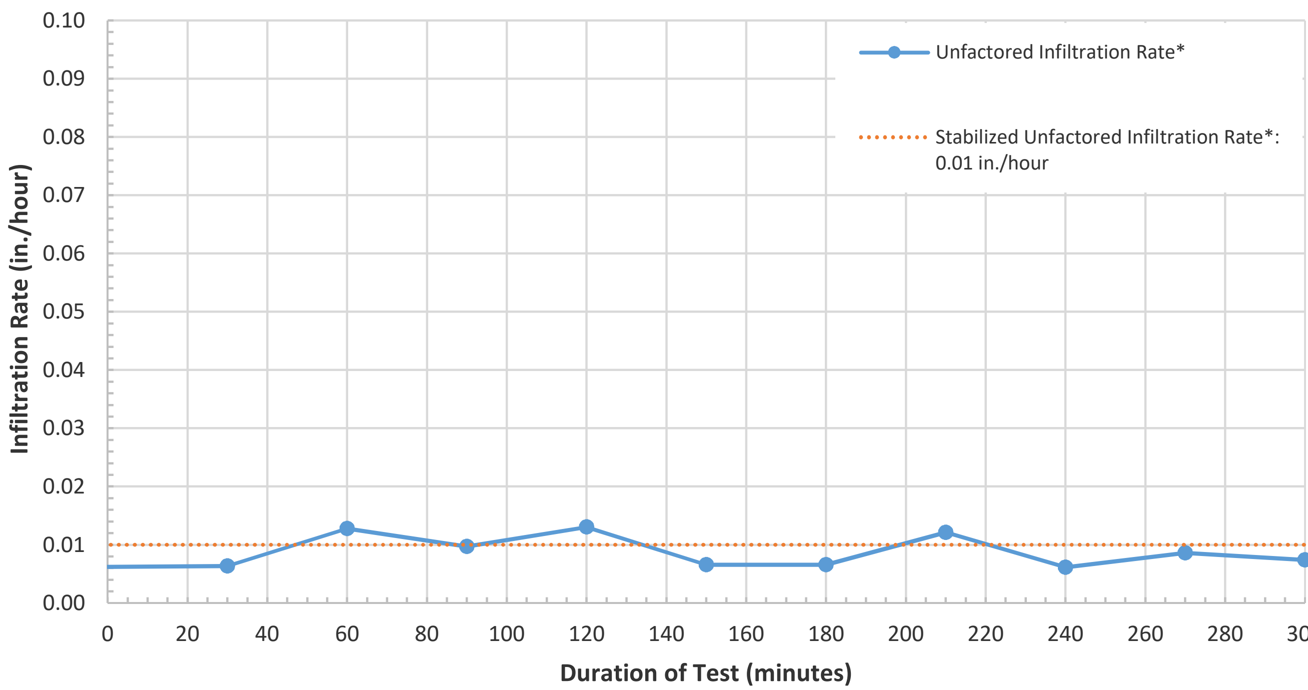
The field infiltration tests were conducted in general accordance with the requirements of the City of La Mesa BMP Design Manual. The two borehole percolation test locations were each drilled to a depth of about 5 feet. Prior to testing, each well was cleared of loose soil and presoaked with water overnight. Additional water was then allowed to infiltrate into the soil under constant head with flow measurements taken at 30-minute time intervals. The tests were continued within each well until a relatively constant infiltration rate was attained.

The field testing indicates stabilized (unfactored) infiltration rates ranging from about 0.00 to 0.01 inches per hour and averaging less than 0.01 inches per hour (see Figures C-1.1 to C-2.2). A Factor of Safety of 2.0 is recommended for BMP design. A threshold of 0.50 inches per hour is commonly considered the minimum rate for effective on-site infiltration measures. Our previous experience indicates that clayey soils such as those at the subject site typically have a hydraulic conductivity less than 10^{-7} cm/s, which is essentially impermeable (see Figures C-3.1 to C-3.3).

BOREHOLE PERCOLATION TEST

Project Name:	La Mesa Apartments	**Date Drilled:**	9/19/2019	**Borehole Radius (*r):**	3 in.
Project Number:	SD634	**Date Tested:**	9/20/2019	**Casing Diameter:**	4 in.
Test Hole Number:	I-1 (Near B-4)	**Tested By:**	SRN	**Depth of Hole:**	5.0 ft
Drilling Method:	Hollow Stem Auger	**Average Water Temperature:**	72 F	**Average Test Depth:**	2.5' - 5'

UNFACTORED INFILTRATION RATES* DURING TEST



Preliminary Factored Infiltration Rate[1]: 0.00 in./hr.

Feasibility Screening Factor of Safety, F.S.[2]: 2

Temperature Correction Factor[2,3]: 0.85

Factored Infiltration Rate[2]	Design Condition[2]
Below 0.05	No Infiltration
0.05 to 0.50	Partial Infiltration
Above 0.50	Full Infiltration

*Porchet method used to convert percolation rate to infiltration rate. See text of Appendix C.

1: Rate Factored by Factor of Safety and Temperature Correction Factor.

2: Reference: The City of La Mesa, BMP Design Manual (2016).

3: Factor based on as-tested water temperature of 72 F and rainfall temperature of 60 F.

LA MESA APARTMENT DEVELOPMENT ALLISON AVENUE AND DATE AVENUE USA PROPOERTIES FUND	BOREHOLE PERCOLATION TEST I-1 INFILTRATION RATE	GROUP DELTA PROJECT NUMBER: SD634 FIGURE NUMBER: C-1.1

BOREHOLE PERCOLATION TEST

Project Name: La Mesa Apartments
Project Number: SD634
Test Hole Number: I-1 (Near B-4)
Drilling Method: Hollow Stem Auger

Date Drilled: 9/19/2019
Date Tested: 9/20/2019
Tested By: SRN
Average Water Temperature: 72 F

Borehole Radius (*r): 4 in.
Casing Diameter: 4 in.
Depth of Hole: 5.0 ft
Gravel Base Thickness: 4 in.

DATA SHEET

Reading Number	Time Interval (min.)	Cumulative Time (min.)	Initial Depth to Water (ft.)	Final Depth to Water (ft.)	Avg. Height of Water above Gravel Base (in.)		Measured Drop in Water Level (in.)	Corrected Drop in Water Level[1] (in.)	Corrected Percolation Rate[1] (in./hour)	Unfactored Infiltration Rate* (in./hour)
	Δt	T	[from ground surface]		H_{avg}	X radius	ΔH	ΔH_c	$\Delta H_c/\Delta t$	I_t
Pre-soak	(1,320)	(1,320)	--	--	--	--	--	--	--	--
1	30	30	2.47	2.48	26.30	6.6*r	0.12	0.05	0.09	0.01
2	30	60	2.48	2.50	26.12	6.5*r	0.24	0.09	0.18	0.01
3	30	90	2.50	2.52	25.91	6.5*r	0.18	0.07	0.14	0.01
4	30	120	2.52	2.54	25.64	6.4*r	0.24	0.09	0.18	0.01
5	30	150	2.54	2.55	25.46	6.4*r	0.12	0.04	0.09	0.01
6	30	180	2.55	2.56	25.34	6.3*r	0.12	0.05	0.09	0.01
7	30	210	2.35	2.37	27.68	6.9*r	0.24	0.09	0.18	0.01
8	30	240	2.37	2.38	27.50	6.9*r	0.12	0.04	0.09	0.01
9	30	270	2.38	2.39	27.36	6.8*r	0.17	0.06	0.13	0.01
10	30	300	2.39	2.40	27.25	6.8*r	0.14	0.05	0.11	0.01

1: Porosity of gravel assumed to be 0.4 to correct drop in water. See text of Appendix C for details.
*Porchet method used to convert percolation rate to infiltration rate. See text of Appendix C.

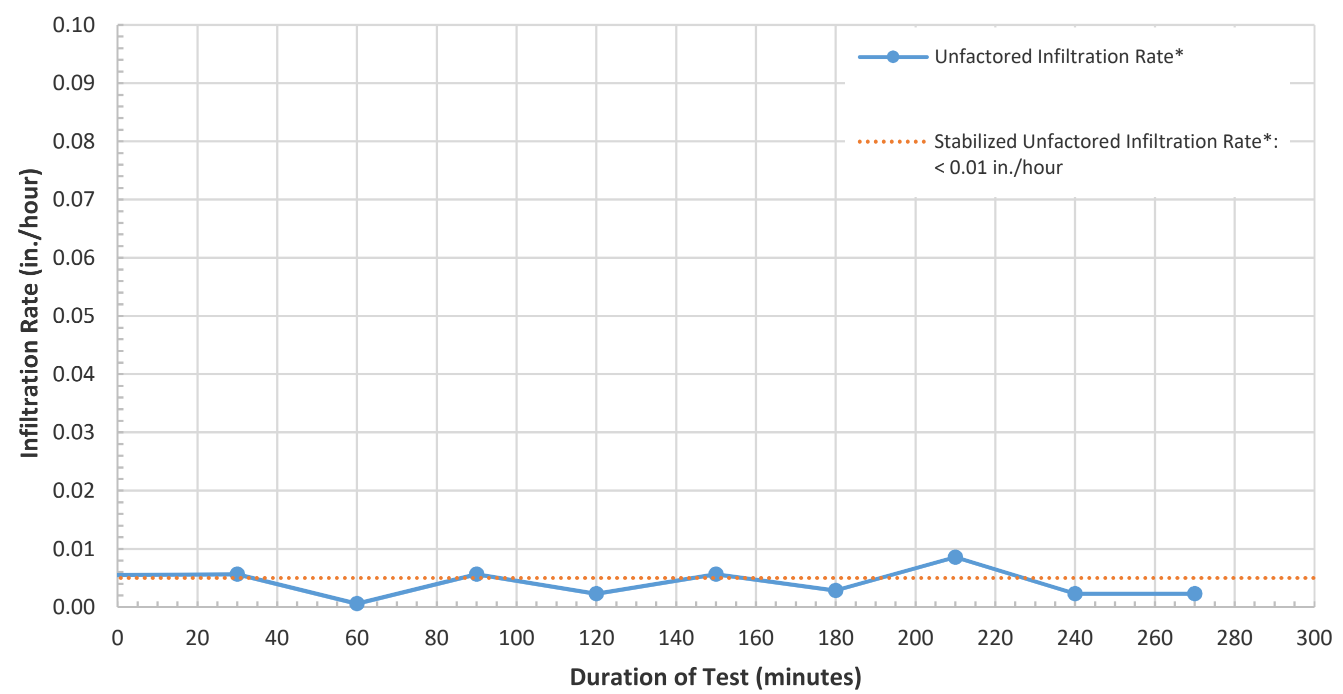
Stabilized, Unfactored Infiltration Rate*: 0.01 inch/hour

LA MESA APARTMENT DEVELOPMENT ALLISON AVENUE AND DATE AVENUE USA PROPOERTIES FUND	BORHOLE PERCOLATION TEST I-1 INFILTRATION RATE	GROUP DELTA PROJECT NUMBER SD634	FIGURE NUMBER C-1.2

BOREHOLE PERCOLATION TEST

Project Name:	La Mesa Apartments	**Date Drilled:**	9/19/2019	**Borehole Radius (*r):**	3 in.
Project Number:	SD634	**Date Tested:**	9/20/2019	**Casing Diameter:**	4 in.
Test Hole Number:	I-2 (Near B-2)	**Tested By:**	SRN	**Depth of Hole:**	5.0 ft
Drilling Method:	Hollow Stem Auger	**Average Water Temperature:**	72 F	**Average Test Depth:**	2.3' - 5'

UNFACTORED INFILTRATION RATES* DURING TEST



Preliminary Factored Infiltration Rate[1]: 0.00 in./hr.

Feasibility Screening Factor of Safety, F.S.[2]: 2

Temperature Correction Factor[2,3]: 0.85

Factored Infiltration Rate[2]	Design Condition[2]
Below 0.05	No Infiltration
0.05 to 0.50	Partial Infiltration
Above 0.50	Full Infiltration

*Porchet method used to convert percolation rate to infiltration rate. See text of Appendix C.

1: Rate Factored by Factor of Safety and Temperature Correction Factor.

2: Reference: The City of La Mesa, BMP Design Manual (2016).

3: Factor based on as-tested water temperature of 72 F and rainfall temperature of 60 F.

LA MESA APARTMENT DEVELOPMENT ALLISON AVENUE AND DATE AVENUE USA PROPOERTIES FUND	BOREHOLE PERCOLATION TEST I-2 INFILTRATION RATE	GROUP DELTA	
		PROJECT NUMBER SD634	FIGURE NUMBER C-2.1

BOREHOLE PERCOLATION TEST

Project Name: La Mesa Apartments
Date Drilled: 9/19/2019
Borehole Radius (*r): 4 in.
Project Number: SD634
Date Tested: 9/20/2019
Casing Diameter: 4 in.
Test Hole Number: I-2 (Near B-2)
Tested By: SRN
Depth of Hole: 5.0 ft
Drilling Method: Hollow Stem Auger
Average Water Temperature: 72 F
Gravel Base Thickness: 2 in.

DATA SHEET

Reading Number	Time Interval (min.)	Cumulative Time (min.)	Initial Depth to Water (ft.)	Final Depth to Water (ft.)	Avg. Height of Water above Gravel Base (in.)		Measured Drop in Water Level (in.)	Corrected Drop in Water Level[1] (in.)	Corrected Percolation Rate[1] (in./hour)	Unfactored Infiltration Rate* (in./hour)
	Δt	T	[from ground surface]		H_{avg}	X radius	ΔH	ΔH_c	$\Delta H_c/\Delta t$	I_t
Pre-soak	(1,230)	(1,230)	--	--	--	--	--	--	--	--
1	30	30	2.33	2.34	29.98	7.5*r	0.12	0.04	0.09	0.01
2	30	60	2.33	2.33	30.03	7.5*r	0.01	0.00	0.01	0.00
3	30	90	2.33	2.34	29.98	7.5*r	0.12	0.04	0.09	0.01
4	30	120	2.34	2.34	29.90	7.5*r	0.05	0.02	0.04	0.00
5	30	150	2.34	2.35	29.86	7.5*r	0.12	0.05	0.09	0.01
6	30	180	2.35	2.36	29.77	7.4*r	0.06	0.02	0.04	0.00
7	30	210	2.36	2.38	29.59	7.4*r	0.18	0.07	0.14	0.01
8	30	240	2.38	2.38	29.42	7.4*r	0.05	0.02	0.04	0.00
9	30	270	2.38	2.38	29.42	7.4*r	0.05	0.02	0.04	0.00

1: Porosity of gravel assumed to be 0.4 to correct drop in water. See text of Appendix C for details.
*Porchet method used to convert percolation rate to infiltration rate. See text of Appendix C.

Stabilized, Unfactored Infiltration Rate*: 0.00 inch/hour

LA MESA APARTMENT DEVELOPMENT ALLISON AVENUE AND DATE AVENUE USA PROPOERTIES FUND	BOREHOLE PERCOLATION TEST I-2 INFILTRATION RATE	GROUP DELTA PROJECT NUMBER SD634	FIGURE NUMBER C-2.2

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

GROUP DELTA

C:\ANALYSIS\LABPEARM2

PROJECT: Alberhill Clay and Aggregate Quarry TESTED BY: RHC SAMPLE: Olive #17 Document No. 19-0141

CLIENT: Pacific Aggregates CHECKED BY: MAF DATE: 06/21/10 Project No. SD634

DESCRIPTION: Remolded dark yellowish brown sandy lean clay (CL) with permeability of 2*(10^{-7}) cm/s **FIGURE C-3.1**

MOISTURE AND DENSITY	INITIAL	FINAL		TEST PARAMETERS		
A) WET WEIGHT OF SAMPLE	373.20	405.60	[G]	F) STANDPIPE AREAS	0.08	[CM^2]
B) DRY WEIGHT OF SAMPLE	329.80	329.80	[G]	G) SAMPLE DIAMETER	4.93	[CM]
C) MOISTURE CONTENT [(A - B) / B]	13.2	23.0		H) SAMPLE AREA (π * G^2 / 4)	19.09	[CM^2]
D) WET DENSITY (A / J * 62.4)	119.0	129.4	[PCF]	I) INITIAL SAMPLE HEIGHT	10.25	[CM]
E) DRY DENSITY [D / (1 + C)]	105.2	105.2	[PCF]	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. 19-0141

CLIENT: 1382-001-00 | CHECKED BY: MAF | DATE: 05/01/09 | Project No. SD634

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

FIGURE C-3.2

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

	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. 19-0141

CLIENT: 1382-001-00 | CHECKED BY: MAF | DATE: 05/07/09 | Project No. SD634

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

FIGURE C-3.3

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]

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.		X
Provide basis: The preliminary factored infiltration rate was measured at less than 0.05 inches per hour at the two borehole percolation test locations (see Figures C-1.1 through C-2.2). This rate corresponds to a "No Infiltration" condition, per the City of La Mesa 2016 BMP Design Manual. The on-site soils are essentially impermeable. 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.		X
Provide basis: See answer 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.			

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.		X
Provide basis: See answer 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.			
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.		X
Provide basis: See answer 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.			
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 INFILTRATION

*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.		X
Provide basis: The preliminary factored infiltration rate was measured at less than 0.05 inches per hour at the two borehole percolation test locations (see Figures C-1.1 through C-2.2). This rate corresponds to a "No Infiltration" condition, per the City of La Mesa 2016 BMP Design Manual. The on-site soils are essentially impermeable. 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.		X
Provide basis: See answer to Item 5 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.			

Worksheet C.4-1 Page 4 of 4			
Criteria	Screening Question	**Yes**	**No**
7	**Can Infiltration in any appreciable quantity be allowed without posing significant risk for groundwater related concerns (shallow water table, storm water pollutants or other factors)?** The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.		X
Provide basis: See answer to Item 5 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.			
8	**Can infiltration be allowed without violating downstream water rights**? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.		X
Provide basis: See answer to Item 5 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.			
Part 2 Result*	If all answers from row 5-8 are yes then partial infiltration design is potentially feasible. The feasibility screening category is **Partial Infiltration.** If any answer from row 5-8 is no, then infiltration of any volume is considered to be **infeasible** within the drainage area. The feasibility screening category is **No Infiltration.**	NO INFILTRATION	

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

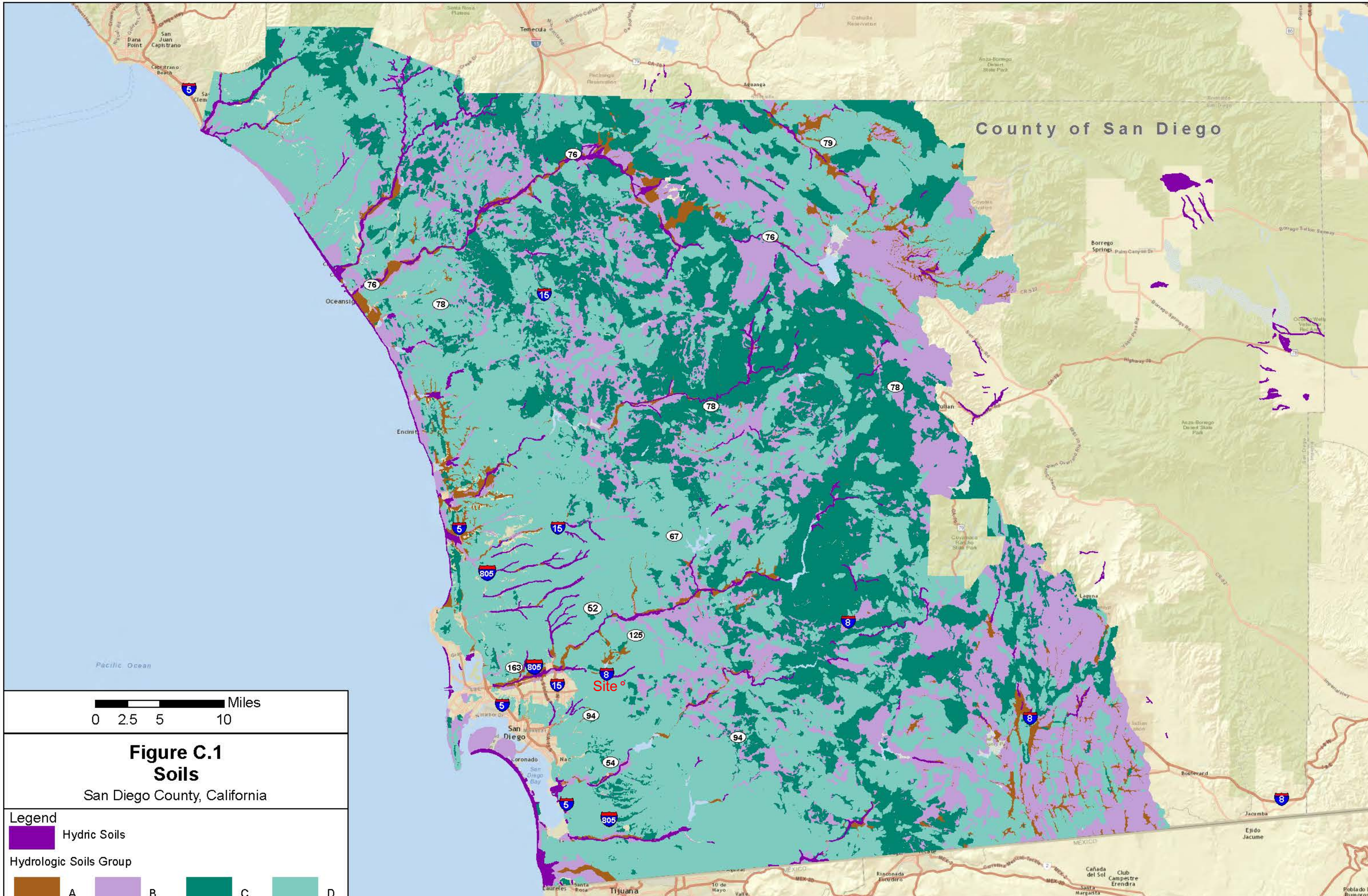


Figure C.1
Soils
San Diego County, California