Preliminary Geotechnical Evaluation

San Diego Fire-Rescue Air Operations Hangars
Montgomery-Gibbs Executive Airport
San Diego, California

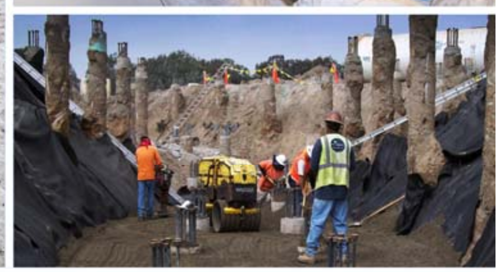
Platt/Whitelaw Architects, Inc.

4034 30th Street | San Diego, California 92104

September 6, 2018 | Project No. 108605001







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Ninyo & Moore

Geotechnical & Environmental Sciences Consultants

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1 INTRODUCTION

In accordance with your request, we have performed a preliminary geotechnical evaluation for the planned San Diego Fire-Rescue Air Operations Hangars project at the Montgomery-Gibbs Executive Airport located at 3750 John J. Montgomery Drive in San Diego, California (Figure 1). This report presents the results of our field explorations and laboratory testing as well as our conclusions regarding the geotechnical conditions at the site and our preliminary recommendations for use in project bridging documents and technical representation. We understand that design-build services, which will include additional subsurface evaluation, will be performed at a later date.

2 SCOPE OF SERVICES

Our scope of services for this evaluation included the following:

- Reviewing readily available published and in-house geotechnical literature including a previous geotechnical report for the adjacent Taxiway C (Ninyo & Moore, 2011a), topographic maps, geologic and geologic hazard maps, fault maps, flood zone maps, and stereoscopic aerial photographs.
- Performing a field reconnaissance to observe site conditions and to mark the locations of the exploratory borings.
- Notifying Underground Service Alert (USA) to clear excavation locations for the potential presence of underground utilities. In addition, a private utility locating company was used to clear the locations for the potential presence of underground utilities.
- Performing a subsurface exploration program consisting of the drilling, logging, and sampling of eight exploratory borings (B-1 through B-4 and IT-1 through IT-4). Relatively undisturbed drive and bulk soil samples of the materials encountered were collected at selected intervals from the borings and transported to our in-house geotechnical laboratory for testing.
- Performing infiltration tests in four of our borings to evaluate the infiltration rates of the underlying soils.
- Performing geotechnical laboratory testing of representative soil samples to evaluate soil characteristics and parameters for design purposes.
- Compiling and performing an engineering analysis of the information obtained from our background review, subsurface exploration, and laboratory testing.
- Preparing this geotechnical report presenting our preliminary findings, conclusions, and geotechnical recommendations for use in bridging documents for the eventual design and building of this project.

3 SITE AND PROJECT DESCRIPTION

The site is located within the Montgomery-Gibbs Executive Airport located at 3950 John J. Montgomery Drive in San Diego, California (Figure 1). The airport consists of three runways and various taxiways, buildings, and hangars. Other improvements include an air traffic control tower, a concrete helipad, and an operations building located in the northeast portion of the airport. An access road connects this area with Ponderosa Avenue to the northeast (Figure 2). The airport property is relatively level and elevations generally range from approximately 410 feet above mean sea level (MSL) in the southwestern portion of the site to approximately 425 feet above MSL in the eastern portion.

Based on our review of project information, including scoping documents and a project Feasibility Study (Atkins, 2017), as well as discussions with your office, we understand that the project will include the construction of new hangars and associated improvements in the vicinity of the existing operations building. Specifically, the project includes two new helicopter hangars, a concrete apron, a support building, a fueling station, parking areas, and a concrete helipad extension (Figure 2). In addition, the access road to Ponderosa Avenue will be improved and biofiltration basins may be constructed.

4 SUBSURFACE EXPLORATION

Our subsurface exploration was conducted on August 16 and August 17, 2018 and included the drilling, logging, and sampling of eight small-diameter borings (B-1 through B-4 and IT-1 through IT-4). Borings IT-1 through IT-4 were also used for infiltration testing. Prior to commencing the subsurface exploration, the locations were cleared of underground utilities of Underground Service Alert. In addition, a private utility locator was retained to locate existing utilities in the area of our exploratory borings. The purpose of the borings was to evaluate subsurface conditions and to collect soil samples for laboratory testing.

The borings were drilled to depths up to approximately 15 feet using manual equipment and a truck-mounted drill rig equipped with 8-inch diameter, continuous-flight, hollow-stem augers. Drilling refusal was encountered in three of our eight borings (B-1 through B-3). Ninyo & Moore personnel logged the borings in general accordance with the Unified Soil Classification System (USCS) by observing cuttings and drive samples. Representative bulk and in-place soil samples were collected at selected depths from within the exploratory borings and transported to our in-house geotechnical laboratory for analysis. The approximate locations of the borings are presented on Figure 2. The boring logs are presented in Appendix A.

Ninyo & Moore previously performed subsurface explorations within the Montgomery-Gibbs Executive Airport property for geotechnical evaluations associated with various runway and taxiway projects (Ninyo & Moore, 2004; 2008; 2011a; and 2011b). Information related to those evaluations are incorporated herein, as appropriate.

5 LABORATORY TESTING

Geotechnical laboratory testing was performed on representative soil samples collected during our subsurface exploration. This testing included an evaluation of in-situ moisture content, gradation, expansion index, soil corrosivity, and R-value. The results of the in-situ moisture content tests are presented at the corresponding depths on the boring logs in Appendix A. Descriptions of the geotechnical laboratory test methods and the results of the other geotechnical laboratory tests performed are presented in Appendix B.

6 INFILTRATION TESTING

Field infiltration testing was performed on August 16 and August 17, 2018 at locations selected by the project Civil Engineer. The infiltration test holes (IT-1 through IT-4) were excavated with a truck-mounted drill rig to depths of approximately 5 feet at the locations shown on Figure 2. The infiltration tests were performed in general accordance with the City of San Diego BMP Design Manual (2018). Approximately 2 inches of gravel was placed on the bottom of each prepared boring. A 2-inch diameter, perforated PVC pipe was installed in the boring and the annulus was then backfilled with pea gravel. As part of the test procedure, presoaking of each hole was performed on August 16, 2018 to represent adverse conditions for infiltration. The presoak consisted of maintaining approximately 1 foot of water in each boring for approximately 4 hours. The water level was then allowed to drop overnight. Infiltration testing was then performed in the presoaked test borings on August 17, 2018. Measurements of the water depth after infiltration were recorded approximately every thirty minutes. As necessary, the borings were refilled to maintain the water level until the infiltration rate stabilized.

Infiltration rates were calculated using the Porchet method. Based on the City of San Diego BMP Design Manual (2018), infiltration rates greater than 0.05 inches per hour and less than 0.5 inches per hour may be suitable for partial infiltration. Infiltration rates of 0.5 inches per hour or greater per hour may be considered suitable for full infiltration design. Infiltration rates less than 0.05 inches per hour are considered a no infiltration condition.

Our in-situ infiltration testing indicated that the water level within IT-1, IT-2, IT-3, and IT-4 generally remained constant over the 30 minute testing intervals and did not infiltrate. Accordingly, infiltration within the subsurface materials at IT-1, IT-2, IT-3, and IT-4 is not considered feasible. Based on the results of our infiltration testing, we recommend lining the sides of biofiltration basins with an impermeable liner or other hydraulic restricted layer. Infiltration test results and calculations are included in Appendix C. A completed Worksheet C.4-1: Categorization of Infiltration Feasibility Condition Based on Geotechnical Conditions with the appropriate geotechnical aspects is presented in Appendix C. Recommendations for placement, design, and construction of permanent stormwater BMPs are presented in Section 10.8 of this report.

Other areas of the site not specifically tested may or may not accommodate partial infiltration of storm water. Additional infiltration testing would be needed in these other areas to evaluate whether infiltration in these areas/depths are feasible. It is noted that the soils underlying the site are mapped by the Natural Resources Conservation Service (NCRS, 2018) as belonging to Hydrologic Soil Group D, which typically exhibits very slow infiltration rates. In addition, seasonal vernal pools, which are ephemeral pools of standing water, are present in the site vicinity. Based on these conditions, we anticipate that other areas of the site will also possess poor infiltration characteristics.

7 GEOLOGY AND SUBSURFACE CONDITIONS

Our findings regarding regional and site geology at the project location are provided in the following sections.

7.1 Regional Geologic Setting

The project area is situated in the coastal foothill section of the Peninsular Ranges Geomorphic Province. This geomorphic province encompasses an area that extends approximately 900 miles from the Transverse Ranges and the Los Angeles Basin south to the southern tip of Baja California (Norris and Webb, 1990; Harden, 2004). The province varies in width from approximately 30 to 100 miles. In general, the province consists of rugged mountains underlain by Jurassic metavolcanic and metasedimentary rocks, and Cretaceous igneous rocks of the southern California batholith.

The Peninsular Ranges Province is traversed by a group of sub-parallel faults and fault zones trending roughly northwest. Several of these faults, which are shown on Figure 3, are considered active faults. The Elsinore, San Jacinto, and San Andreas faults are active fault systems located northeast of the project area and the Rose Canyon, Coronado Bank, San Diego Trough, and San Clemente faults are active faults located west of the project area. The Rose Canyon Fault Zone, the nearest active fault system, has been mapped approximately 4½ miles west of the project site. Major tectonic activity associated with these and other faults within this regional tectonic framework consists primarily of right-lateral, strike-slip movement. Further discussion of faulting relative to the site is provided in the Faulting and Seismicity section of this report.

7.2 Site Geology

Geologic units encountered during our field reconnaissance and subsurface exploration included fill, topsoils, and very old paralic deposits. Generalized descriptions of the earth units encountered during our subsurface exploration are provided below. The geology of the site vicinity is shown on Figure 4. Additional descriptions are provided on the boring logs in Appendix A.

7.2.1 Pavement Sections

Our exploratory borings B-1, IT-3, and IT-4 encountered pavement sections that consisted of asphalt concrete (AC) and aggregate base material underlain by fill materials and very old paralic deposits. Table 1 below summarizes the pavement sections as encountered in our borings.

Table 1 – Encountered Pavement Sections

Boring	AC thickness (inches)	Base Thickness (inches)
B-1	3½	3
IT-3	2½	3½
IT-4	2½	9½

7.2.2 Fill

Fill materials were encountered at the ground surface or underlying the pavement sections in borings B-1, B-4, and IT-3 to depths of up to 4 feet. Refusal was encountered in the fill material within B-1. As encountered, the fill soils generally consisted of brown and reddish brown, moist, loose to medium dense, clayey sand, and stiff, sandy clay. Gravel and cobbles were encountered within the fill materials. Documentation regarding placement of these fills was not available for review.

7.2.3 Topsoil

Topsoil was encountered at the ground surface in borings B-2, B-3, IT-1, and IT-2. In our borings, the topsoil was relatively thin and generally one-foot in thickness or less. As encountered, the topsoil materials generally consisted of brown, dry to moist, loose to medium dense, silty sand with roots.

7.2.4 Very Old Paralic Deposits

Materials of the middle to early Pleistocene-aged very old paralic deposits are mapped at the site (Figure 4; Kennedy and Tan, 2008), previously designated as the Lindavista Formation (Kennedy, 1975), and were encountered in borings B-2 through B-4 and IT-1 through IT-4 underlying the pavements, fill, and topsoil and extending to the total depths explored. As encountered, these materials generally consisted of reddish brown, olive brown, grayish brown, and gray, dry to moist, moderately to strongly cemented, silty and clayey sandstone. Cobbles were also encountered in the very old paralic deposits and drilling refusal within the very old paralic deposits occurred in three of our borings (B-1, B-2, and B-3).

7.3 Groundwater

Groundwater was not encountered in our exploratory borings. According to our review of readily available data from the Geotracker (2018) website, groundwater is anticipated at depths greater than 50 feet. Six borings were drilled to depths ranging from approximately 20 to 50 feet below the ground surface as part of an assessment by SCS Engineers (2008) of a former underground storage tank located approximately 15 feet west of the existing air traffic control tower. The assessment report by SCS (2008) indicated that the borings, which were drilled at roughly the same elevation as those performed in our evaluation, did not encounter groundwater. Existing utility trench lines may act as conduits for perched water conditions and seepage may be anticipated. Fluctuations in the groundwater level and perched conditions may occur due to variations in ground surface topography, subsurface geologic conditions and structure, rainfall, irrigation, and other factors. While surface water was not observed at the site during our exploration activities, seasonal vernal pools, which are ephemeral pools of standing water, are present in the site vicinity.

8 GEOLOGIC HAZARDS

In general, hazards associated with seismic activity include strong ground motion, ground surface rupture, and liquefaction. These considerations and other geologic hazards, such as landsliding and flooding, are discussed in the following section.

8.1 Faulting and Seismicity

Based on our review of the referenced geologic maps and stereoscopic aerial photographs, as well as on our geologic field mapping, the subject site is not underlain by known active or potentially active faults (i.e., faults that exhibit evidence of ground displacement in the last 11,000 years and 2,000,000 years, respectively). However, like the majority of southern California, the site is located in a seismically active area and the potential for strong ground motion is considered significant during the design life of the proposed structures. The nearest known active fault is the Rose Canyon fault, located approximately 4½ miles west of the site. Table 2 lists selected principal known active faults that may affect the subject site, including the approximate fault-to-site distances, and the maximum moment magnitudes (Mmax) as published by the USGS (2018a).

Table 2 – Principal Active Faults

Fault	Approximate Fault-to-Site Distance miles (kilometers)	Maximum Moment Magnitude (Mmax)
Rose Canyon	4.5 (7.3)	6.9
Coronado Bank	18 (29)	7.4
Newport-Inglewood (Offshore)	29 (47)	7.0
Elsinore (Julian Segment)	36 (57)	7.4
Elsinore (Temecula Segment)	37 (59)	7.1
Earthquake Valley	40 (65)	6.8
Elsinore (Coyote Mountain)	48 (77)	6.9

In general, hazards associated with seismic activity include surface ground rupture, strong ground motion, and liquefaction. A brief description of these hazards and the potential for their occurrences on site are discussed below.

8.2 Surface Ground Rupture

Based on our review of the referenced literature and our field evaluation, no active faults are known to cross the project vicinity. Therefore, the potential for ground rupture due to faulting at the project site is considered low. However, lurching or cracking of the ground surface as a result of nearby seismic events is possible.

8.3 Strong Ground Motion

The 2016 California Building Code (CBC) specifies that the Risk-Targeted, Maximum Considered Earthquake (MCE_R) ground motion response accelerations be used to evaluate seismic loads for design of buildings and other structures. The MCE_R ground motion response accelerations are based on the spectral response accelerations for 5 percent damping in the direction of maximum horizontal response and incorporate a target risk for structural collapse equivalent to 1 percent in 50 years with deterministic limits for near-source effects. The horizontal peak ground acceleration (PGA) that corresponds to the MCE_R for the segments was calculated as 0.44g using the United States Geological Survey (USGS, 2018b) seismic design tool (web-based).

The 2016 CBC specifies that the potential for liquefaction and soil strength loss be evaluated, where applicable, for the Maximum Considered Earthquake Geometric Mean (MCE_G) peak ground acceleration with adjustment for site class effects in accordance with the American Society of Civil Engineers (ASCE) 7-10 Standard. The MCE_G peak ground acceleration is based on the geometric mean peak ground acceleration with a 2 percent probability of exceedance in 50 years. The MCE_G peak ground acceleration with adjustment for site class effects (PGA_M) was calculated as 0.45g using the USGS (USGS, 2018b) seismic design tool that yielded a mapped MCE_G peak ground acceleration of 0.414g for the site and a site coefficient (F_{PGA}) of 1.086 for Site Class D.

8.4 Liquefaction

Liquefaction of cohesionless soils can be caused by strong vibratory motion due to earthquakes. Research and historical data indicate that loose granular soils and non-plastic silts that are saturated by a relatively shallow groundwater table are susceptible to liquefaction. Based on the relatively dense nature of the very old paralic deposits encountered in our borings, it is our opinion that the potential for liquefaction to occur at the site is not a design consideration.

8.5 Geologic Hazard Map

Per the City of San Diego's Seismic Safety Study (2008), the project site is located within an area designated as Category 51, which is described as "Level mesas, underlain by terrace deposits and bedrock, nominal risk." A portion of the Seismic Safety Study map that includes the site and vicinity is presented in Figure 5.

8.6 Landslides

Our review of referenced geologic maps, literature, topographic maps, and stereoscopic aerial photographs, no landslides or indications of deep-seated landsliding underlie the subject site (Kennedy and Tan, 2008; Tan, 1995). In addition, no indications of landsliding were observed during our site reconnaissance or subsurface exploration. As such, the potential for significant large-scale slope instability at the site is not a design consideration.

8.7 Flood Hazards

Based on review of the Federal Emergency Management Agency Flood Insurance Rate Maps (FIRM), flood hazard mapping has not been published at the project site. Based on our review of maps indicating the presence of vernal pools on the site (Atkins, 2017), seasonal flooding may be anticipated.

9 CONCLUSIONS

Based on our review of the referenced background data, the subsurface exploration, and geotechnical laboratory testing, it is our opinion that construction of the proposed project is feasible from a geotechnical standpoint provided the recommendations presented in this report are incorporated into subsequent evaluations for the design and construction of the project. In general, the following conclusions were made:

- The project site is generally underlain by fill, topsoil, and very old paralic deposits. The existing fill and topsoil are not considered suitable for structural support in their current condition. The very old paralic deposits encountered at the site are considered suitable for structural support.

- Groundwater was not encountered during our subsurface exploration that included borings that extended to a depth of approximately 15 feet. Perched conditions and fluctuations in groundwater may occur due to variations in ground surface topography, subsurface geologic structure, rainfall, irrigation, and other factors.

- Gravel and cobble were encountered in the very old paralic deposits and drilling refusal within the very old paralic deposits occurred in two of our borings (B-2 and B-3). Accordingly, the contractor for site development should anticipate encountering difficult excavation conditions that may require additional efforts including heavy ripping and/or coring for drilling operations.

- Soils derived from on-site excavations are anticipated to generate gravel, cobbles, and oversize pieces of cemented sandstone. On-site soils may be suitable for reuse as engineered fill, provided they are processed in accordance with the following recommendations. Additional processing and handling of materials including screening and/or crushing should be anticipated.

- The closest known active fault, the Rose Canyon fault, has been mapped approximately 4½ miles west of the site. No active faults are reported underlying the subject site. Therefore, potential for ground rupture due to faulting at the site is considered low.
- Field infiltration testing indicated that infiltration within the subsurface materials is not feasible. Recommendations for placement, design, and construction of permanent stormwater BMPs are presented herein.
- Results of our geotechnical laboratory testing indicate that the upper soils at the site possess a very low expansion potential. However, variability of onsite soils should be anticipated as soils possessing medium and high expansion potential were encountered in a previous evaluation for Taxiway C, located northwest of the project site (Ninyo & Moore, 2011a).
- Based on the results of our limited geotechnical laboratory testing presented in Appendix B, as compared to the Caltrans (2018) corrosion guidelines, the on-site soils would be classified as corrosive
- Additional evaluation should be performed by the design-build team.

10 PRELIMINARY RECOMMENDATIONS

The following preliminary recommendations are provided for the design and construction of the proposed project. These preliminary recommendations are based on our evaluation of the site geotechnical conditions and our assumptions regarding the planned development. Subsequent evaluations and the proposed construction should be performed in accordance with the requirements of applicable governing agencies including the Federal Aviation Administration (FAA) and the San Diego County Regional Airport Authority. As noted previously, our preliminary recommendations are intended for use in project bridging documents and technical representation. We understand that design-build services, which will include additional subsurface evaluation, will be performed at a later date.

10.1 Earthwork

In general, earthwork should be performed in accordance with the preliminary recommendations presented in this report.

10.1.1 Site Preparation

Site preparation should begin with the removal of existing improvements, vegetation, utility lines, asphalt, concrete, and other deleterious debris from areas to be graded. Tree stumps and roots should be removed to such a depth that organic material is generally not present. Clearing and grubbing should extend to the outside of the proposed excavation and fill areas. The debris and unsuitable material generated during clearing and grubbing should be removed from areas to be graded and disposed of at a legal dumpsite away from the project area, unless noted otherwise in the following sections.

10.1.2 Excavation Characteristics

The results of our background review and field exploration program indicate that the project site is underlain by fill, topsoils, and very old paralic deposits. Excavation of the on-site materials should be should be generally achievable with heavy-duty earth moving equipment in good working condition. However, as noted, drilling refusal was encountered in three of our borings. Due to the presence of cobbles and possible strongly cemented zones within the very old paralic deposits, some areas may require heavy ripping or mechanical rock breaking equipment. Excavations may generate oversized material and additional processing and handling of these materials, including screening and/or crushing, should be anticipated.

10.1.3 Remedial Grading for Structures

In order to provide suitable support for proposed settlement-sensitive structures, including the proposed hangars and building, we recommend that the existing undocumented fill soils within the limits of the structures be removed to competent very old paralic deposits. Based on the subsurface information in our exploratory borings within the building areas, the existing fill is anticipated to extend to depths of up to 4 feet within the project limits. However, the depth of removals may be deeper and should be evaluated in the field to confirm that existing fills have been removed. The removed materials may be processed and replaced as compacted fill. The lateral extent of these removals should be approximately 5 feet outside the limits of proposed settlement-sensitive structures, including foundations for attached overhangs, canopies, and other building appurtenances.

Subsequent to removal, the resulting surface should be scarified to a depth of approximately 6 inches, moisture conditioned, and recompacted to a relative compaction of 90 percent as evaluated by the ASTM D 1557 prior to placing new fill. Once the resulting removal surface has been recompacted, the overexcavation should be backfilled with generally granular soils that possess a very low to low expansion potential (i.e., an expansion index [EI] less than 50).

10.1.4 Temporary Excavations

For temporary excavations, we recommend that the following Occupational Safety and Health Administration (OSHA) soil classifications be used:

Fill and Topsoil	*Type C*
Very Old Paralic Deposits	*Type B*

Upon making the excavations, the soil classifications and excavation performance should be evaluated in the field by the geotechnical consultant in accordance with the OSHA regulations. Temporary excavations should be constructed in accordance with OSHA recommendations. For trenches or other excavations, OSHA requirements regarding personnel safety should be met using appropriate shoring (including trench boxes) or by laying back the slopes to no steeper than 1.5:1 (horizontal to vertical) in fill and topsoil and 1:1 for very old paralic deposits. Temporary excavations that encounter seepage may be shored or stabilized by placing sandbags or gravel along the base of the seepage zone. Excavations encountering seepage should be evaluated on a case-by-case basis. On-site safety of personnel is the responsibility of the contractor.

10.1.5 Materials For Fill

Soils derived from on-site excavations are anticipated to generate gravel, cobbles, and oversize pieces of cemented sandstone. On-site soils may be suitable for reuse as engineered fill, provided they are processed in accordance with the following recommendations. Additional processing and handling of materials including screening and/or crushing should be anticipated. Engineered fill soils should possess an organic content of less than approximately 3 percent by volume (or 1 percent by weight). In general, engineered fill material should not contain rocks or lumps over approximately 3 inches in diameter, and not more than approximately 30 percent larger than ¾ inch. Oversize materials should be separated from material to be used for fill and removed from the site.

Imported fill material, if needed, should generally be granular soils with a very low to low expansion potential (i.e., an expansion index [EI] of 50 or less). Import material should also be non-corrosive in accordance with the Caltrans (2018) corrosion guidelines. Based on the Caltrans (2018) criteria, soil is classified as corrosive if one or more of the following conditions exist: chloride concentration of 500 ppm or greater, soluble sulfate concentration of 1,500 ppm or greater, an electrical resistivity of 1,100 ohm-centimeters or less, and a pH 5.5 or less. Materials for use as fill should be evaluated prior to filling or importing.

10.1.6 Compacted Fill

Prior to placement of compacted fill, the contractor should request an evaluation of the exposed ground surface by Ninyo & Moore. Unless otherwise recommended, the exposed ground surface should then be scarified to a depth of approximately 6 inches and watered or dried, as needed, to achieve moisture contents generally at or slightly above the optimum moisture content. The scarified materials should then be compacted to a relative compaction of 90 percent as evaluated in accordance with the ASTM D 1557. The evaluation of compaction by the geotechnical consultant should not be considered to preclude any requirements for observation or approval by governing agencies. It is the contractor's responsibility to notify this office and the appropriate governing agency when project areas are ready for observation, and to provide reasonable time for that review.

Fill materials should be moisture conditioned to generally at or slightly above the laboratory optimum moisture content prior to placement. The optimum moisture content will vary with material type and other factors. Moisture conditioning of fill soils should be generally consistent within the soil mass.

Prior to placement of additional compacted fill material following a delay in the grading operations, the exposed surface of previously compacted fill should be prepared to receive fill. Preparation may include scarification, moisture conditioning, and recompaction.

Compacted fill should be placed in horizontal lifts of approximately 8 inches in loose thickness. Prior to compaction, each lift should be watered or dried as needed to achieve a moisture content generally at or slightly above the laboratory optimum, mixed, and then compacted by mechanical methods to a relative compaction of 90 percent as evaluated by ASTM D 1557. The upper 12 inches of the subgrade materials beneath vehicular pavements should be compacted to a relative compaction of 95 percent relative density as evaluated by ASTM D 1557. Successive lifts should be treated in a like manner until the desired finished grades are achieved. Where planned under airport pavements, fill should be placed per FAA guidelines.

10.1.7 Drainage

Roof, pad, and slope drainage should be conveyed such that runoff water is diverted away from slopes and structures to suitable discharge areas by nonerodible devices (e.g., gutters, downspouts, concrete swales, etc.). Positive drainage adjacent to structures should be established and maintained. Positive drainage may be accomplished by providing drainage away from the foundations of the structure at a gradient of 2 percent or steeper for a distance of 5 feet or more outside building perimeters, and further maintained by a graded swale leading to an appropriate outlet, in accordance with the recommendations of the project civil engineer and/or landscape architect.

Surface drainage on the site should be provided so that water is not permitted to pond. A gradient of 2 percent or steeper should be maintained over the pad area and drainage patterns should be established to divert and remove water from the site to appropriate outlets.

Care should be taken by the contractor during grading to preserve any berms, drainage terraces, interceptor swales or other drainage devices of a permanent nature on or adjacent to the property. Drainage patterns established at the time of grading should be maintained for the life of the project. The property owner and the maintenance personnel should be made aware that altering drainage patterns might be detrimental to foundation performance.

10.2 Seismic Design Parameters

Design of the proposed improvements should be performed in accordance with the requirements of governing jurisdictions and applicable building codes. Table 3 presents the seismic design parameters for the site in accordance with the CBC (2016) guidelines and adjusted MCE spectral response acceleration parameters (USGS, 2018b).

Table 3 – 2016 California Building Code Seismic Design Criteria

Seismic Design Factors	Values
Site Class	D
Site Coefficient, F_a	1.098
Site Coefficient, F_v	1.631
Mapped Spectral Acceleration at 0.2-second Period, S_s	1.004g
Mapped Spectral Acceleration at 1.0-second Period, S_1	0.385g
Spectral Acceleration at 0.2-second Period Adjusted for Site Class, S_{MS}	1.103g
Spectral Acceleration at 1.0-second Period Adjusted for Site Class, S_{M1}	0.627g
Design Spectral Response Acceleration at 0.2-second Period, S_{DS}	0.735g
Design Spectral Response Acceleration at 1.0-second Period, S_{D1}	0.418g

10.3 Foundations

Based on our understanding of the proposed structures, we are providing the following recommendations. The proposed hangars and building may be supported on shallow, continuous and/or spread footings bearing on compacted fill or very old paralic deposits. Foundations should be designed in accordance with structural considerations and the following recommendations. In addition, requirements of the appropriate governing jurisdictions and applicable building codes should be considered in the design of the structures.

10.3.1 Bearing Capacity

Shallow, spread or continuous footings supported on compacted fill or competent very old paralic deposits may be designed using an allowable bearing capacity of 3,000 pounds per square foot (psf). These allowable bearing capacities may be increased by one-third when considering loads of short duration such as wind or seismic forces. Footings should be designed and reinforced in accordance with the recommendations of the project structural engineer.

10.3.2 Lateral Resistance

For resistance to lateral loads when footings are supported in compacted fill or competent very old paralic deposits, we recommend an allowable passive pressure of 350 pounds per cubic foot (pcf) be used with an upper bound value of up to 3,500 psf. This value assumes that the ground is horizontal for a distance of 10 feet, or three times the height generating the passive pressure, whichever is more. We recommend that the upper 1 foot of soil not protected by pavement or a concrete slab be neglected when calculating passive resistance.

For frictional resistance to lateral loads, we recommend a coefficient of friction of 0.35 be used between soil and concrete. The lateral resistance values presented above may be increased by one-third when considering loads of short duration such as wind or seismic forces.

10.4 Pavements

Based on the results of our previous evaluations at Montgomery-Gibbs Executive Airport (Ninyo & Moore, 2004, 2008, 2011a, and 2011b), site soils have been classified as “cohesive” based on FAA guidelines. Laboratory testing performed as part of these previous evaluations indicated California Bearing Ratio (CBR) values at the site generally range from 3 to 14 for pavement subgrade with a relative compaction of 95 percent. CBR values were not assessed within the project limits during this evaluation. CBR values should be evaluated during design-build services in accordance with applicable FAA specifications.

10.5 Preliminary Access Road Pavement Design

Our laboratory testing indicated the site soils along the access road to Ponderosa Avenue possess an R-value of 13. Accordingly, we have used a design R-value of 13 and Traffic Indices (TI) of 6 and 7 for the basis of preliminary design of flexible pavements for the access road. However, actual pavement recommendations should be based on R-value tests performed on bulk samples of the soils exposed at the finished subgrade elevations following grading operations. We recommend that the geotechnical consultant re-evaluate the pavement design at the time of construction. The recommended preliminary flexible pavement sections for the access road are presented in the table below.

Table 4 – Recommended Preliminary Flexible Pavement Sections			
Traffic Index (Pavement Usage)	Design R-Value	Asphalt Concrete (in)	Class 2 Aggregate Base (in)
6 (Drive Aisles)	13	4	10
7 (Fire Lanes and Delivery Routes	13	5	12

These values assume traffic indices of seven or less for site pavements. In addition, we recommend that the upper 12 inches of the subgrade and aggregate base materials be compacted to a relative compaction of 95 percent relative density as evaluated by the current version of ASTM D 1557. The AC materials should be compacted to a relative compaction of 95 percent as evaluated by the materials Hveem density. If traffic loads are different from those assumed, the pavement design should be re-evaluated.

10.5.1 Subgrade Stabilization

Due to the relatively impermeable nature of the very old paralic deposits, we anticipate that perched groundwater may be present in some areas. Due to the potential presence of perched groundwater or wet subgrade soils, excavations may encounter yielding subgrade conditions. Mitigation measures may include the removal and replacement of the wet soils or stabilization through a combination of aggregate base material reinforced with geogrid or geotextiles. Specific recommendations should be based on conditions exposed in the field during construction and evaluated on a case-by-case basis.

10.6 Soil Corrosivity

Laboratory testing was performed on a representative sample of the near-surface soil to evaluate soil pH, electrical resistivity, water-soluble chloride content, and water-soluble sulfate content. The soil pH and electrical resistivity tests were performed in general accordance with California Test Method (CT) 643. The chloride content test was performed in general accordance with CT 422. Sulfate testing was performed in general accordance with CT 417.

The results of the corrosivity testing indicated an electrical resistivity of 880 ohm-centimeters (ohm-cm), a soil pH of 8.6, a chloride content of 400 parts per million (ppm), and a sulfate content of 0.011 percent (i.e., 110 ppm). A comparison with the Caltrans corrosion (2018) criteria indicates that the on-site soils would be classified as corrosive. Based on the Caltrans (2018) criteria, a project site is classified as corrosive if one or more of the following conditions exist for the representative soil samples retrieved from the site: chloride concentration of 500 ppm or greater, soluble sulfate concentration of 1,500 ppm or greater, an electrical resistivity of 1,100 ohm-centimeters or less, and a pH 5.5 or less.

10.7 Concrete

Concrete in contact with soil or water that contains high concentrations of water-soluble sulfates can be subject to premature chemical and/or physical deterioration. A soil samples tested during this evaluation indicated a water-soluble sulfate content of 0.011 percent (i.e., 110 ppm). Based on the ACI 318 criteria, the potential for sulfate attack is considered negligible for water-soluble sulfate contents in soil ranging from 0 to 0.10 percent by weight (0 to 1,000 ppm), indicating that soils underlying the site may be considered to have a negligible potential for sulfate attack. However, due to the potential for variability of on-site soils, we recommend that Type II, II/V, or V cement be used for concrete in contact with soil.

10.8 Permanent Stormwater BMPs

We understand that the project will include construction of BMP devices to satisfy the City of San Diego Stormwater requirements. As presented in Section 6, the results of in-situ testing of the underlying materials indicate that infiltration within the subsurface soils at IT-1, IT-2, IT-3, and IT-4 is not feasible. Based on the relatively impermeable nature of the very old paralic deposits, it is anticipated that lateral movement of infiltrating water will affect surrounding improvements including underground utility trenches, pavement subgrades, and foundation elements. Therefore, we recommend that permanent biofiltration basins be lined with an impermeable liner to restrict the movement of water to nearby improvements. The permanent biofiltration basins should be equipped with a drain to an appropriate outlet.

11 LIMITATIONS

The field evaluation, laboratory testing, and geotechnical analyses presented in this geotechnical report have been conducted in general accordance with current practice and the standard of care exercised by geotechnical consultants performing similar tasks in the project area. No warranty, expressed or implied, is made regarding the conclusions, recommendations, and opinions presented in this report. There is no evaluation detailed enough to reveal every subsurface condition. Variations may exist and conditions not observed or described in this report may be encountered during construction. Uncertainties relative to subsurface conditions can be reduced through additional subsurface exploration. Additional subsurface evaluation will be performed upon request. Please also note that our evaluation was limited to assessment of the geotechnical aspects of the project, and did not include evaluation of structural issues, environmental concerns, or the presence of hazardous materials.

This document is intended to be used only in its entirety. No portion of the document, by itself, is designed to completely represent any aspect of the project described herein. Ninyo & Moore should be contacted if the reader requires additional information or has questions regarding the content, interpretations presented, or completeness of this document.

This report is intended for design purposes only. It does not provide sufficient data to prepare an accurate bid by contractors. It is suggested that the bidders and their geotechnical consultant perform an independent evaluation of the subsurface conditions in the project areas. The independent evaluations may include, but not be limited to, review of other geotechnical reports prepared for the adjacent areas, site reconnaissance, and additional exploration and laboratory testing.

Our conclusions, recommendations, and opinions are based on an analysis of the observed site conditions. If geotechnical conditions different from those described in this report are encountered, our office should be notified and additional recommendations, if warranted, will be provided upon request. It should be understood that the conditions of a site could change with time as a result of natural processes or the activities of man at the subject site or nearby sites. In addition, changes to the applicable laws, regulations, codes, and standards of practice may occur due to government action or the broadening of knowledge. The findings of this report may, therefore, be invalidated over time, in part or in whole, by changes over which Ninyo & Moore has no control.

This report is intended exclusively for use by the client. Any use or reuse of the findings, conclusions, and/or recommendations of this report by parties other than the client is undertaken at said parties' sole risk.

12 REFERENCES

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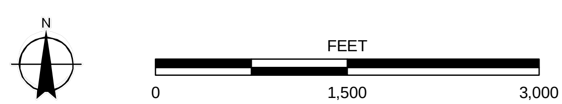
FIGURES



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NOTE: DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE. | SOURCE: ESRI WORLD TOPO, 2017

FIGURE 1

SITE LOCATION

SAN DIEGO FIRE-RESCUE AIR OPERATIONS HANGARS
MONTGOMERY-GIBBS EXECUTIVE AIRPORT, SAN DIEGO, CALIFORNIA

108605001 | 9/18

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SOURCE: ATKINS, 2017.
GOOGLE EARTH, 2017

LEGEND

B-4 TD=15.2 — BORING, TD=TOTAL DEPTH IN FEET

IT-4 TD=5.0 — INFILTRATION TEST, TD=TOTAL DEPTH IN FEET

N

FEET
0 120 240

NOTE: DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE.

FIGURE 2

BORING LOCATIONS

SAN DIEGO FIRE-RESCUE AIR OPERATIONS HANGARS
MONTGOMERY-GIBBS EXECUTIVE AIRPORT, SAN DIEGO, CALIFORNIA

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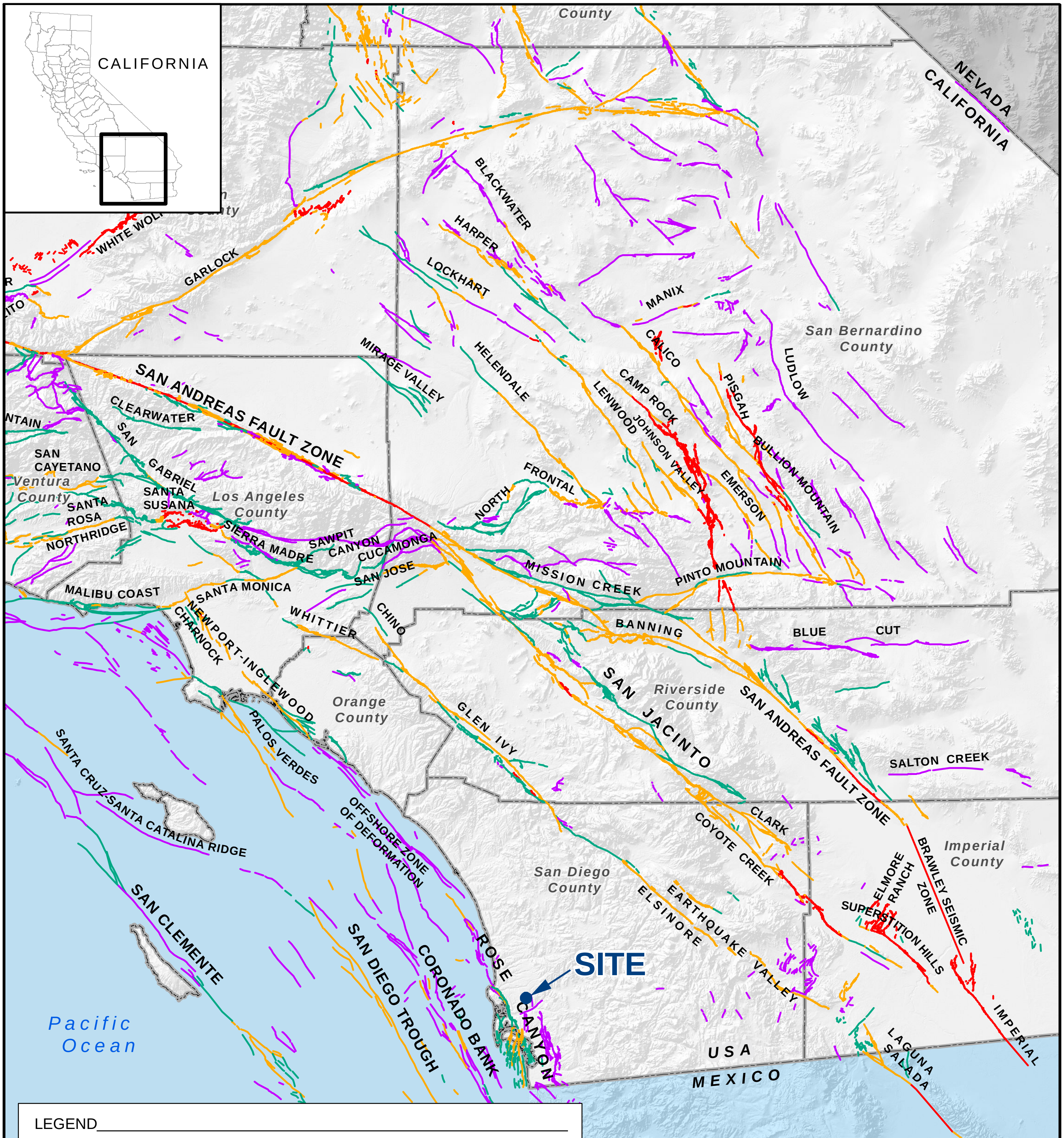

LEGEND

CALIFORNIA FAULT ACTIVITY

- HISTORICALLY ACTIVE
- HOLOCENE ACTIVE
- LATE QUATERNARY (POTENTIALLY ACTIVE)
- QUATERNARY (POTENTIALLY ACTIVE)
- STATE/COUNTY BOUNDARY

SOURCE: U.S. GEOLOGICAL SURVEY AND CALIFORNIA GEOLOGICAL SURVEY, 2006, QUATERNARY FAULT AND FOLD DATABASE FOR THE UNITED STATES.

N

MILES
0 30 60

NOTE: DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE.

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FIGURE 3

FAULT LOCATIONS

SAN DIEGO FIRE-RESCUE AIR OPERATIONS HANGARS
MONTGOMERY-GIBBS EXECUTIVE AIRPORT, SAN DIEGO, CALIFORNIA

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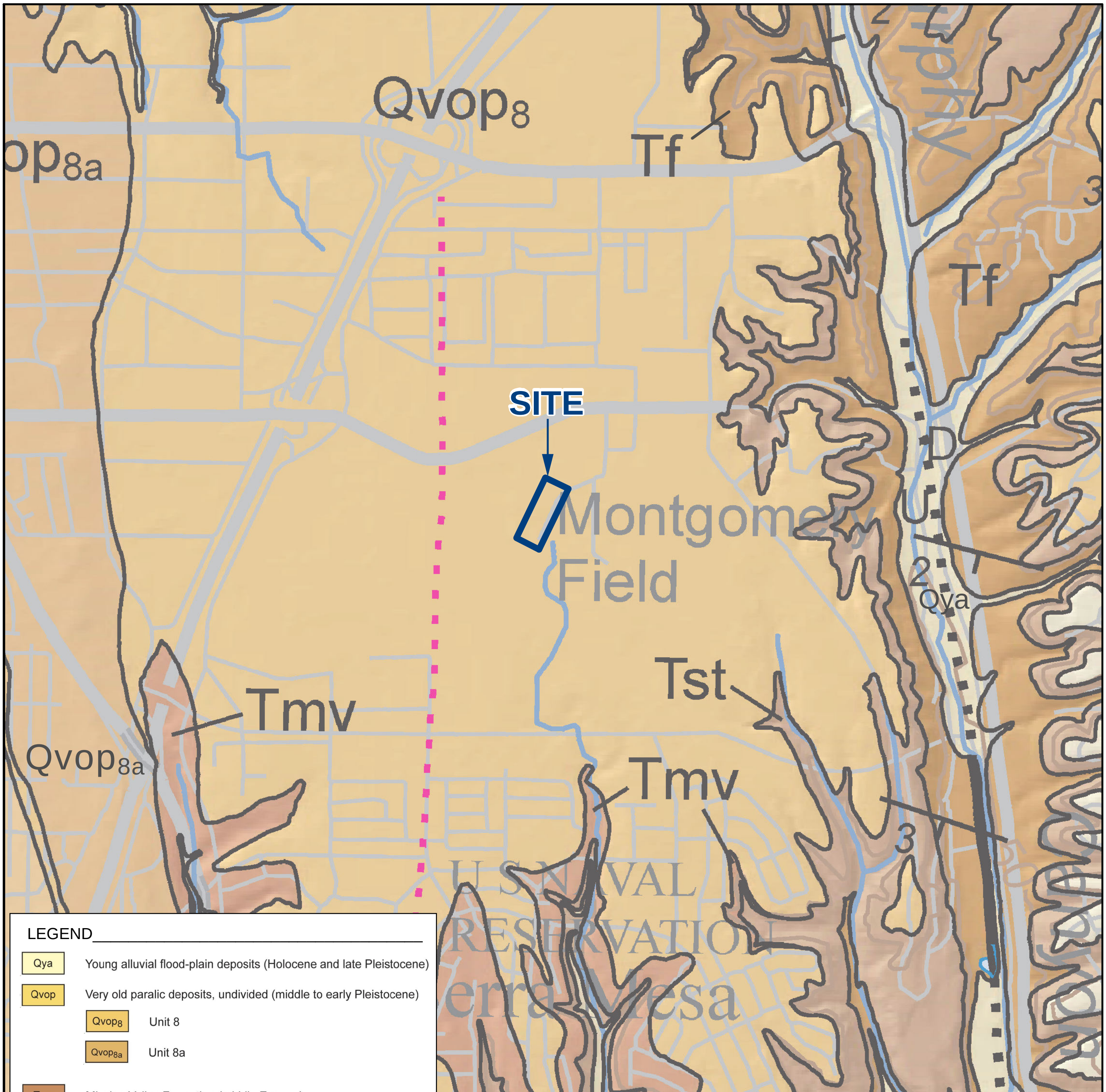

LEGEND

Qya	Young alluvial flood-plain deposits (Holocene and late Pleistocene)
Qvop	Very old paralic deposits, undivided (middle to early Pleistocene)
$Qvop_8$	Unit 8
$Qvop_{8a}$	Unit 8a
Tmv	Mission Valley Formation (middle Eocene)
Tst	Stadium Conglomerate (middle Eocene)
Tf	Friars Formation (middle Eocene)

70 U D — Fault - Solid where accurately located; dashed where approximately located; dotted where concealed. U = upthrown block, D = downthrown block. Arrow and number indicate direction and angle of dip of fault plane.

70 — Strike and dip of beds

REFERENCE: KENNEDY, M.P., TAN, S.S., 2008, GEOLOGIC MAP OF THE SAN DIEGO 30 X 60-MINUTE QUADRANGLE, CALIFORNIA

N

FEET

0 2,000 4,000

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NOTE: DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE.

FIGURE 4

GEOLOGY

SAN DIEGO FIRE-RESCUE AIR OPERATIONS HANGARS
MONTGOMERY-GIBBS EXECUTIVE AIRPORT, SAN DIEGO, CALIFORNIA

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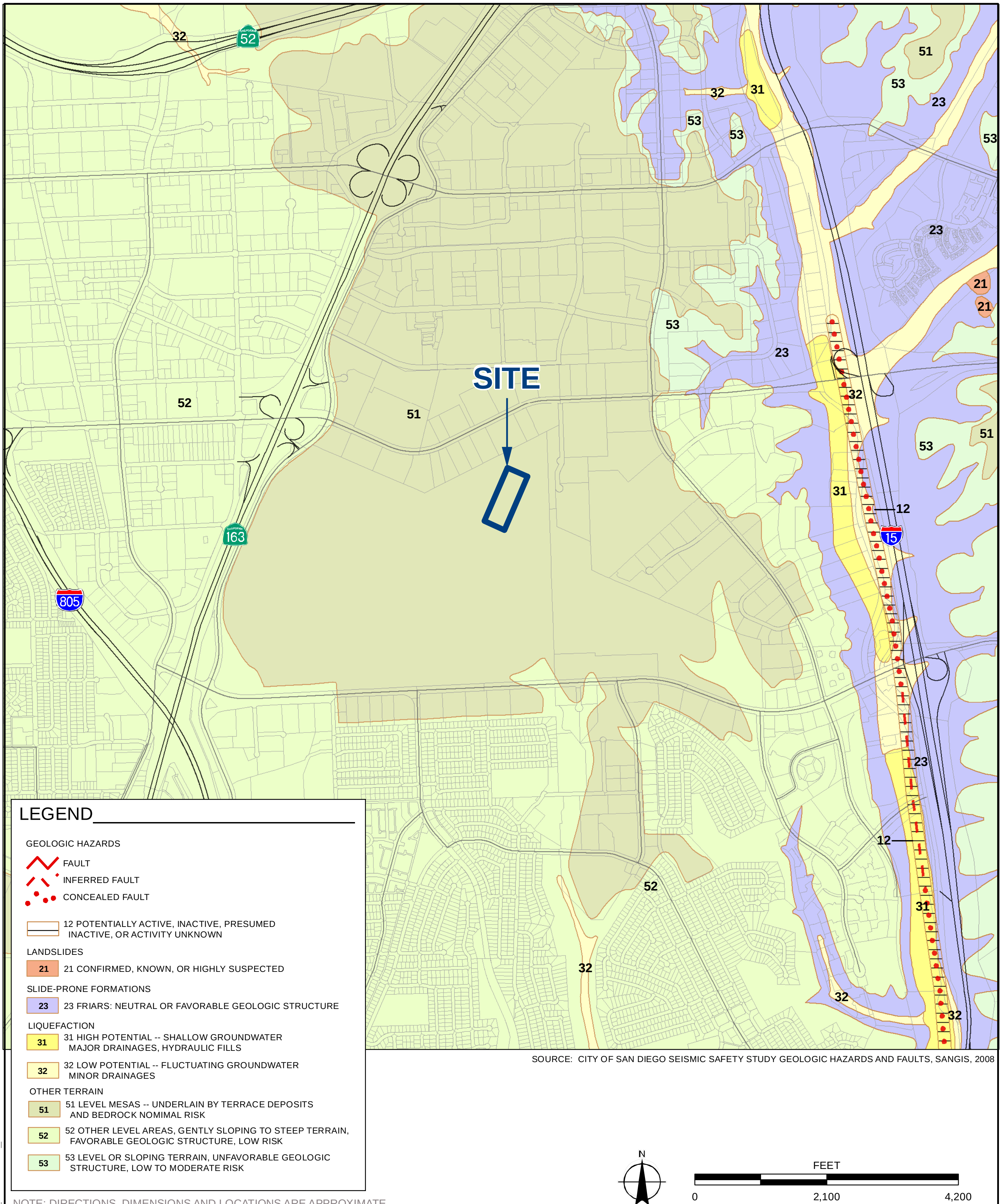

SOURCE: CITY OF SAN DIEGO SEISMIC SAFETY STUDY GEOLOGIC HAZARDS AND FAULTS, SANGIS, 2008

NOTE: DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE.

FIGURE 5

GEOLOGIC HAZARDS

SAN DIEGO FIRE-RESCUE AIR OPERATIONS HANGARS
MONTGOMERY-GIBBS EXECUTIVE AIRPORT, SAN DIEGO, CALIFORNIA

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APPENDIX A

Boring Logs

APPENDIX A

BORING LOGS

Field Procedure for the Collection of Disturbed Samples

Disturbed soil samples were obtained in the field using the following methods.

Bulk Samples

Bulk samples of representative earth materials were obtained from the exploratory borings. The samples were bagged and transported to the laboratory for testing.

The Standard Penetration Test (SPT) Sampler

Disturbed drive samples of earth materials were obtained by means of a Standard Penetration Test sampler. The sampler is composed of a split barrel with an external diameter of 2 inches and an unlined internal diameter of 1⅜ inches. The sampler was driven into the ground with a 140-pound hammer free-falling from a height of 30 inches in general accordance with ASTM D 1586. The blow counts were recorded for every 6 inches of penetration; the blow counts reported on the logs are those for the last 12 inches of penetration. Soil samples were observed and removed from the sampler, bagged, sealed and transported to the laboratory for testing.

Field Procedure for the Collection of Relatively Undisturbed Samples

Relatively undisturbed soil samples were obtained in the field using the following method.

The Modified Split-Barrel Drive Sampler

The sampler, with an external diameter of 3 inches, was lined with 1-inch-long, thin brass rings with inside diameters of approximately 2.4 inches. The sample barrel was driven into the ground with the weight of a hammer in general accordance with ASTM D 3550. The driving weight was permitted to fall freely. The approximate length of the fall, the weight of the hammer, and the number of blows per foot of driving are presented on the boring logs as an index to the relative resistance of the materials sampled. The samples were removed from the sample barrel in the brass rings, sealed, and transported to the laboratory for testing.

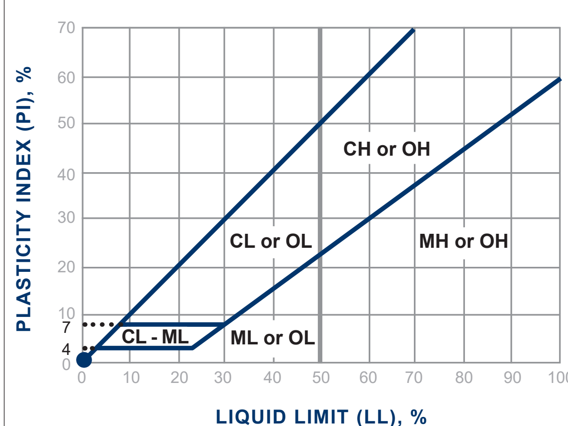
Soil Classification Chart Per ASTM D 2488

Primary Divisions			Secondary Divisions: Group Symbol	Secondary Divisions: Group Name
COARSE-GRAINED SOILS more than 50% retained on No. 200 sieve	**GRAVEL** more than 50% of coarse fraction retained on No. 4 sieve	CLEAN GRAVEL less than 5% fines	GW	well-graded GRAVEL
			GP	poorly graded GRAVEL
		GRAVEL with DUAL CLASSIFICATIONS 5% to 12% fines	GW-GM	well-graded GRAVEL with silt
			GP-GM	poorly graded GRAVEL with silt
			GW-GC	well-graded GRAVEL with clay
			GP-GC	poorly graded GRAVEL with
		GRAVEL with FINES more than 12% fines	GM	silty GRAVEL
			GC	clayey GRAVEL
			GC-GM	silty, clayey GRAVEL
	SAND 50% or more of coarse fraction passes No. 4 sieve	CLEAN SAND less than 5% fines	SW	well-graded SAND
			SP	poorly graded SAND
		SAND with DUAL CLASSIFICATIONS 5% to 12% fines	SW-SM	well-graded SAND with silt
			SP-SM	poorly graded SAND with silt
			SW-SC	well-graded SAND with clay
			SP-SC	poorly graded SAND with clay
		SAND with FINES more than 12% fines	SM	silty SAND
			SC	clayey SAND
			SC-SM	silty, clayey SAND
FINE-GRAINED SOILS 50% or more passes No. 200 sieve	**SILT and CLAY** liquid limit less than 50%	INORGANIC	CL	lean CLAY
			ML	SILT
			CL-ML	silty CLAY
		ORGANIC	OL (PI > 4)	organic CLAY
			OL (PI < 4)	organic SILT
	SILT and CLAY liquid limit 50% or more	INORGANIC	CH	fat CLAY
			MH	elastic SILT
		ORGANIC	OH (plots on or above "A"-line)	organic CLAY
			OH (plots below "A"-line)	organic SILT
	Highly Organic Soils		PT	Peat

Grain Size

Description		Sieve Size	Grain Size	Approximate Size
Boulders		> 12"	> 12"	Larger than basketball-sized
Cobbles		3 - 12"	3 - 12"	Fist-sized to basketball-sized
Gravel	Coarse	3/4 - 3"	3/4 - 3"	Thumb-sized to fist-sized
	Fine	#4 - 3/4"	0.19 - 0.75"	Pea-sized to thumb-sized
Sand	Coarse	#10 - #4	0.079 - 0.19"	Rock-salt-sized to pea-sized
	Medium	#40 - #10	0.017 - 0.079"	Sugar-sized to rock-salt-sized
	Fine	#200 - #40	0.0029 - 0.017"	Flour-sized to sugar-sized
Fines		Passing #200	< 0.0029"	Flour-sized and smaller

Plasticity Chart



Apparent Density - Coarse-Grained Soil

Apparent Density	Spooling Cable or Cathead: SPT (blows/foot)	Spooling Cable or Cathead: Modified Split Barrel (blows/foot)	Automatic Trip Hammer: SPT (blows/foot)	Automatic Trip Hammer: Modified Split Barrel (blows/foot)
Very Loose	≤ 4	≤ 8	≤ 3	≤ 5
Loose	5 - 10	9 - 21	4 - 7	6 - 14
Medium Dense	11 - 30	22 - 63	8 - 20	15 - 42
Dense	31 - 50	64 - 105	21 - 33	43 - 70
Very Dense	> 50	> 105	> 33	> 70

Consistency - Fine-Grained Soil

Consistency	Spooling Cable or Cathead: SPT (blows/foot)	Spooling Cable or Cathead: Modified Split Barrel (blows/foot)	Automatic Trip Hammer: SPT (blows/foot)	Automatic Trip Hammer: Modified Split Barrel (blows/foot)
Very Soft	< 2	< 3	< 1	< 2
Soft	2 - 4	3 - 5	1 - 3	2 - 3
Firm	5 - 8	6 - 10	4 - 5	4 - 6
Stiff	9 - 15	11 - 20	6 - 10	7 - 13
Very Stiff	16 - 30	21 - 39	11 - 20	14 - 26
Hard	> 30	> 39	> 20	> 26

BORING LOG EXPLANATION SHEET

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	
0								Bulk sample.
								Modified split-barrel drive sampler.
								No recovery with modified split-barrel drive sampler.
								Sample retained by others.
								Standard Penetration Test (SPT).
5								No recovery with a SPT.
			XX/XX					Shelby tube sample. Distance pushed in inches/length of sample recovered in inches.
								No recovery with Shelby tube sampler.
								Continuous Push Sample.
								Seepage.
10								Groundwater encountered during drilling.
								Groundwater measured after drilling.
							SM	MAJOR MATERIAL TYPE (SOIL): Solid line denotes unit change.
							CL	Dashed line denotes material change.
15								Attitudes: Strike/Dip b: Bedding c: Contact j: Joint f: Fracture F: Fault cs: Clay Seam s: Shear bss: Basal Slide Surface sf: Shear Fracture sz: Shear Zone sbs: Shear Bedding Surface
20								The total depth line is a solid line that is drawn at the bottom of the boring.

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BORING LOG

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0								ASPHALT CONCRETE: Approximately 3-1/2 inches thick.
							GC	AGGREGATE BASE: Brown, moist, medium dense, clayey GRAVEL; approximately 3 inches thick.
							CL	FILL: Reddish brown to olive, moist, stiff, sandy CLAY; scattered gravel and cobbles.
								Total Depth = 2.5 feet. (Refusal) Groundwater not encountered during. Backfilled and patched shortly after drilling on 8/16/18.
								Note: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report.
5								The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.
10								
15								
20								

DATE DRILLED 8/16/18 BORING NO. B-1

GROUND ELEVATION 420' ± (MSL) SHEET 1 OF 1

METHOD OF DRILLING 8" Diameter Core/Manual

DRIVE WEIGHT N/A DROP N/A

SAMPLED BY GSW LOGGED BY GSW REVIEWED BY NMM

DATE DRILLED 8/16/18 BORING NO. B-2

GROUND ELEVATION 420' ± (MSL) SHEET 1 OF 1

METHOD OF DRILLING 8" Diameter Hollow Stem Auger (CME-95) (Baja)

DRIVE WEIGHT 140 lbs. (Auto-Trip) DROP 30"

SAMPLED BY GSW LOGGED BY GSW REVIEWED BY NMM

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							SM	TOPSOIL: Brown, dry to moist, medium dense, silty SAND; scattered roots.
								VERY OLD PARALIC DEPOSITS: Reddish brown, moist, strongly cemented, silty fine- to medium-grained SANDSTONE; few gravel and cobbles.
				24.4				
5			50/3"	5.4				Dry to moist.
								@ 7': Some gravel.
10			50/2"					
								Cobbles; difficult drilling.
								Total Depth = 12 feet. (Refusal) Groundwater not encountered during drilling. Backfilled shortly after drilling on 8/16/18. Note: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.
15								
20								

DATE DRILLED 8/16/18 BORING NO. B-3

GROUND ELEVATION 420' ± (MSL) SHEET 1 OF 1

METHOD OF DRILLING 8" Diameter Hollow Stem Auger (CME-95) (Baja)

DRIVE WEIGHT 140 lbs. (Auto-Trip) DROP 30"

SAMPLED BY GSW LOGGED BY GSW REVIEWED BY NMM

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							SM	TOPSOIL: Brown, moist, medium dense, silty SAND; scattered roots.
	Bulk (1–5 ft)			10.5				VERY OLD PARALIC DEPOSITS: Reddish brown to gray, moist, moderately cemented, clayey fine- to medium-grained SANDSTONE; few gravel and cobbles.
5		Driven	66/11"	15.2				
10		Driven	50/6"					Grayish brown.
								Cobbles; difficult drilling.
13								Total Depth = 13 feet. (Refusal) Groundwater not encountered during drilling. Backfilled shortly after drilling on 8/16/18.
15								Note: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.
20								

DATE DRILLED	8/16/18
BORING NO.	B-4
GROUND ELEVATION	420' ± (MSL)
SHEET	1 OF 1
METHOD OF DRILLING	8" Diameter Hollow Stem Auger (CME-95) (Baja)
DRIVE WEIGHT	140 lbs. (Auto-Trip)
DROP	30"
SAMPLED BY	GSW
LOGGED BY	GSW
REVIEWED BY	NMM

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0	X						SC	FILL: Brown to reddish brown, moist, medium dense, clayey SAND; scattered gravel and roots.
	X			9.9				
								VERY OLD PARALIC DEPOSITS: Reddish brown, moist, strongly cemented, silty fine- to medium-grained SANDSTONE; few gravel and cobbles.
5		X	50/3"	7.8				
10		X	50/2"					Cobbles; difficult drilling.
15		X	50/2"					
								Total Depth = 15.2 feet. Groundwater not encountered during drilling. Backfilled shortly after drilling on 8/16/18. Note: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.
20								

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.

DATE DRILLED 8/16/18 BORING NO. IT-1

GROUND ELEVATION 420' ± (MSL) SHEET 1 OF 1

METHOD OF DRILLING 8" Diameter Hollow Stem Auger (CME-95) (Baja)

DRIVE WEIGHT 140 lbs. (Auto-Trip) DROP 30"

SAMPLED BY GSW LOGGED BY GSW REVIEWED BY NMM

DESCRIPTION/INTERPRETATION

Depth (feet)	Classification U.S.C.S.	Description/Interpretation
0	SM	TOPSOIL: Brown, dry to moist, medium dense, silty SAND; scattered roots.
		VERY OLD PARALIC DEPOSITS: Reddish brown, dry to moist, moderately cemented, silty fine- to medium-grained SANDSTONE; few gravel and cobbles.
5		Total Depth = 5 feet. Groundwater not encountered. Backfilled shortly after testing on 8/17/18.
10		
15		
20		

Note: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report.

The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED 8/16/18 BORING NO. IT-2 GROUND ELEVATION 420' ± (MSL) SHEET 1 OF 1 METHOD OF DRILLING 8" Diameter Hollow Stem Auger (CME-95) (Baja) DRIVE WEIGHT 140 lbs. (Auto-Trip) DROP 30" SAMPLED BY GSW LOGGED BY GSW REVIEWED BY NMM **DESCRIPTION/INTERPRETATION**
0							SM	TOPSOIL: Brown, dry to moist, medium dense, silty SAND; scattered roots.
	X							VERY OLD PARALIC DEPOSISTS: Reddish brown, dry to moist, moderately cemented, silty fine- to medium-grained SANDSTONE.
5								Total Depth = 5 feet. Groundwater not encountered. Backfilled shortly after testing on 8/17/18. Note: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.
10								
15								
20								

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0								ASPHALT CONCRETE: Approximately 2-1/2 inches thick.
							GC	AGGREGATE BASE: Brown, moist, medium dense, clayey GRAVEL; approximately 3-1/2 inches thick.
							SC	FILL: Brown, moist, loose to medium dense, clayey SAND; few cobbles.
								VERY OLD PARALIC DEPOSITS: Reddish brown, moist, moderately cemented, silty fine- to medium-grained SANDSTONE; trace gravel and cobbles.
5								Total Depth = 5 feet. Groundwater not encountered. Backfilled and patched shortly after testing on 8/17/18. Note: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.
10								
15								
20								

DATE DRILLED 8/16/18 BORING NO. IT-3

GROUND ELEVATION 420' ± (MSL) SHEET 1 OF 1

METHOD OF DRILLING 8" Diameter Hollow Stem Auger (CME-95) (Baja)

DRIVE WEIGHT 140 lbs. (Auto-Trip) DROP 30"

SAMPLED BY GSW LOGGED BY GSW REVIEWED BY NMM

DATE DRILLED 8/16/18	BORING NO. IT-4	
GROUND ELEVATION 420' ± (MSL)	SHEET 1 OF 1	
METHOD OF DRILLING 8" Diameter Hollow Stem Auger (CME-95) (Baja)		
DRIVE WEIGHT 140 lbs. (Auto-Trip)	DROP 30"	
SAMPLED BY GSW	LOGGED BY GSW	REVIEWED BY NMM

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0								ASPHALT CONCRETE: Approximately 2-1/2 inches thick.
							GC	AGGREGATE BASE: Brown, moist, medium dense, clayey GRAVEL; approximately 9-1/2 inches thick.
	■							VERY OLD PARALIC DEPOSITS: Reddish brown, moist, moderately cemented, silty fine- to medium-grained SANDSTONE; few gravel and cobbles.
5								Total Depth = 5 feet. Groundwater not encountered. Backfilled and patched shortly after testing on 8/17/18. Note: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.
10								
15								
20								

APPENDIX B

Laboratory Testing

APPENDIX B

LABORATORY TESTING

Classification
Soils were visually and texturally classified in accordance with the Unified Soil Classification System (USCS) in general accordance with ASTM D 2488. Soil classifications are indicated on the logs of the exploratory borings in Appendix A.

In-Place Moisture Tests
The moisture contents of relatively undisturbed samples obtained from the exploratory borings were evaluated in general accordance with ASTM D 2937. The test results are presented on the logs of the exploratory borings in Appendix A.

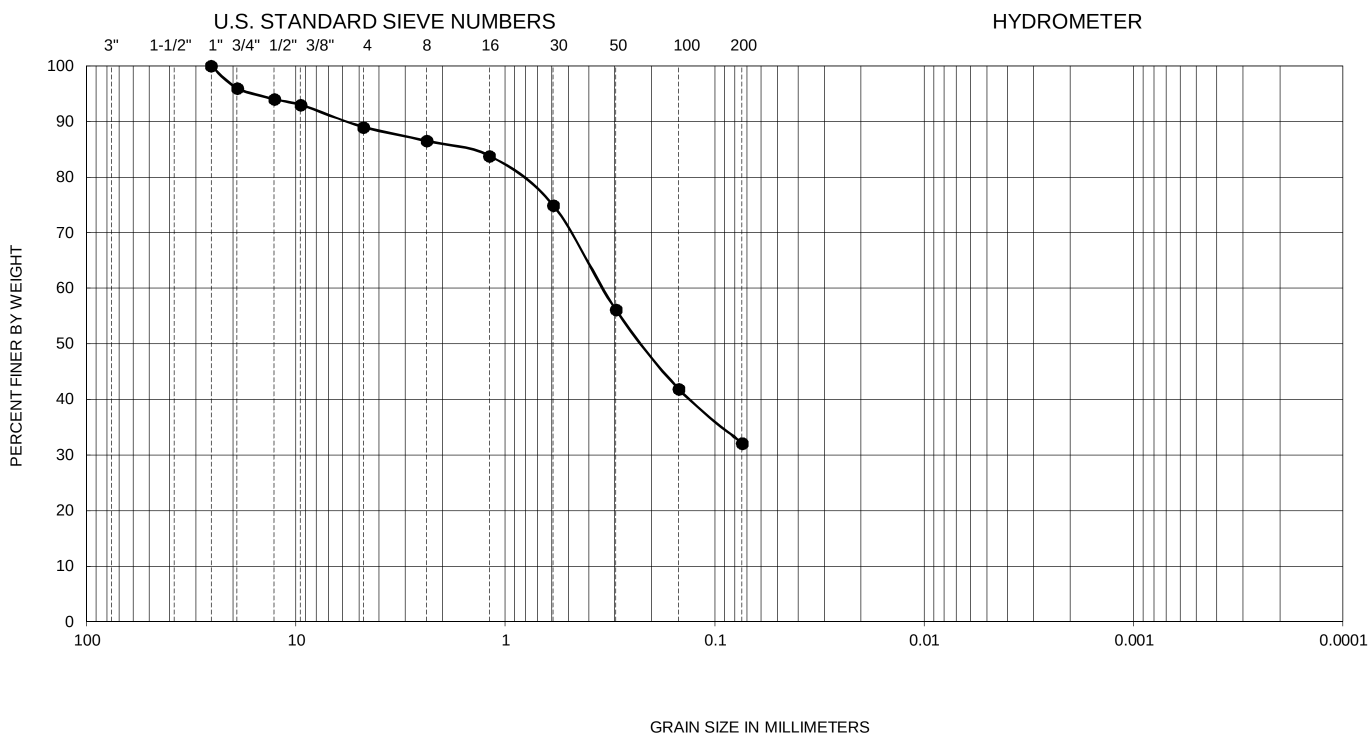
Gradation Analysis
Gradation analysis tests were performed on selected representative soil samples in general accordance with ASTM D 422. The grain size distribution curves are shown on Figures B-1 through B-3. These test results were utilized in evaluating the soil classifications in accordance with the USCS.

Expansion Index Tests
The expansion indices of selected materials were evaluated in general accordance with ASTM D 4829. The specimens were molded under a specified compactive energy at approximately 50 percent saturation. The prepared 1-inch thick by 4-inch diameter specimens were loaded with a surcharge of 144 pounds per square foot and were inundated with tap water. Readings of volumetric swell were made for a period of 24 hours. The results of the tests are presented on Figure B-4.

Soil Corrosivity Tests
Soil pH and electrical resistivity tests were performed on a representative sample in general accordance with CT 643. The sulfate and chloride contents of the selected sample were evaluated in general accordance with CT 417 and 422, respectively. The results of these tests are presented on Figure B-5.

R-Value
The resistance value (R-value) for site soils was evaluated in general accordance with CT 301. Samples were prepared and evaluated for exudation pressure and expansion pressure. The equilibrium R-value is reported as the lesser or more conservative of the two calculated results. The test results are presented in Figure B-6.

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY



Symbol	Hole No.	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (percent)	USCS
●	B-3	1.0-5.0	--	--	--	--	--	--	--	--	32	SC

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422

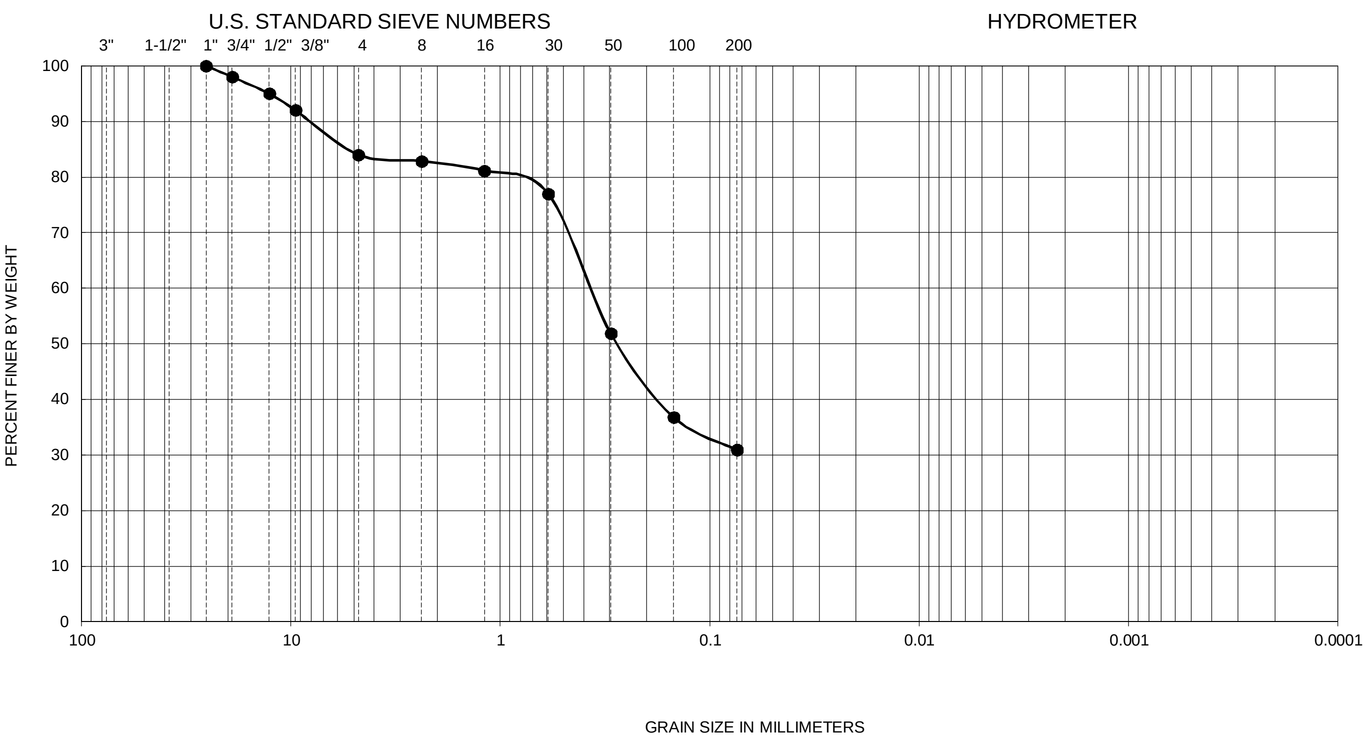
FIGURE B-1

GRADATION TEST RESULTS

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

SAN DIEGO FIRE-RESCUE AIR OPERATIONS HANGARS
MONTGOMERY-GIBBS EXECUTIVE AIRPORT, SAN DIEGO, CALIFORNIA

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY



Symbol	Hole No.	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (percent)	USCS
●	IT-2	1.0-5.0	--	--	--	--	--	--	--	--	31	SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422

FIGURE B-2

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

GRADATION TEST RESULTS

SAN DIEGO FIRE-RESCUE AIR OPERATIONS HANGARS
MONTGOMERY-GIBBS EXECUTIVE AIRPORT, SAN DIEGO, CALIFORNIA

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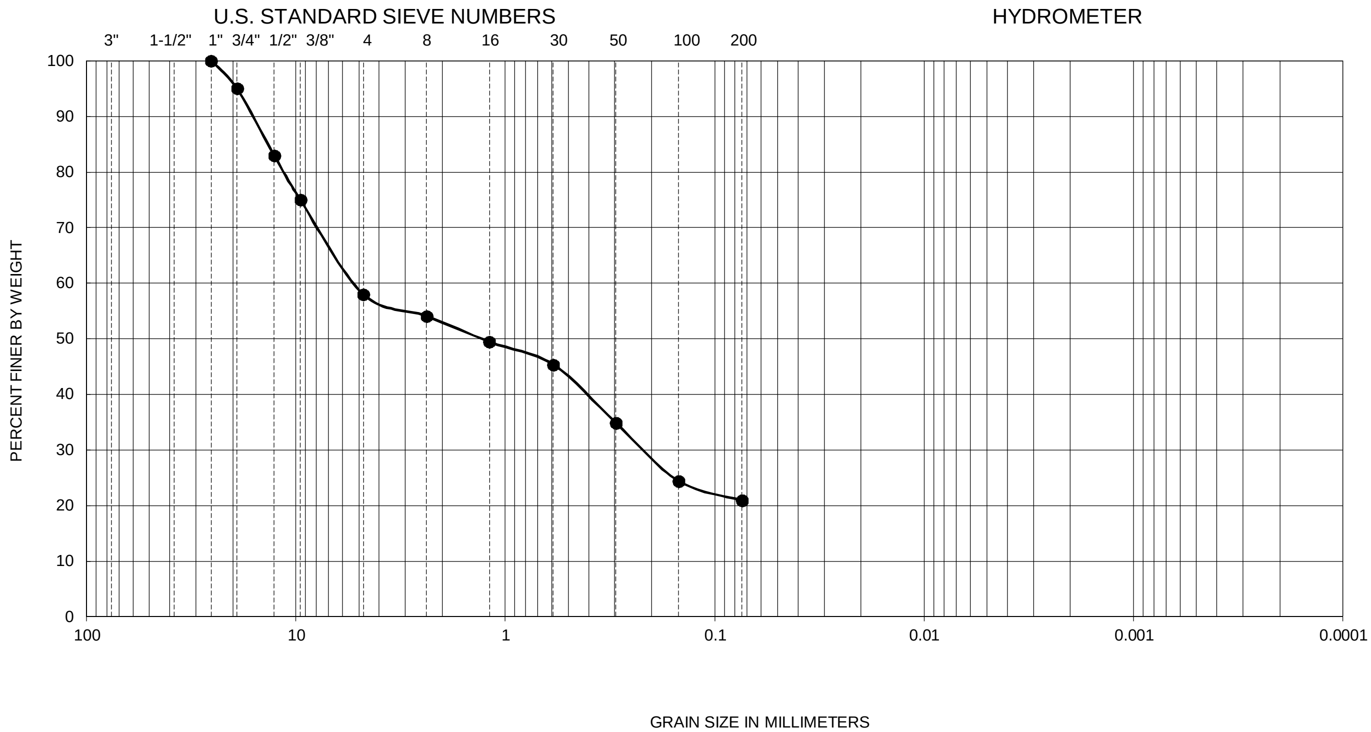
GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY



Symbol	Hole No.	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (percent)	USCS
●	IT-4	1.0-5.0	--	--	--	--	--	--	--	--	21	SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422

FIGURE B-3

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

GRADATION TEST RESULTS

SAN DIEGO FIRE-RESCUE AIR OPERATIONS HANGARS
MONTGOMERY-GIBBS EXECUTIVE AIRPORT, SAN DIEGO, CALIFORNIA

SAMPLE LOCATION	SAMPLE DEPTH (ft)	INITIAL MOISTURE (percent)	COMPACTED DRY DENSITY (pcf)	FINAL MOISTURE (percent)	VOLUMETRIC SWELL (in)	EXPANSION INDEX	POTENTIAL EXPANSION
B-1	0.5-2.5	7.6	120.2	14.3	0.002	2	Very Low
B-3	1.0-5.0	9.5	111.8	17.2	0.014	14	Very Low
B-4	0.0-4.0	10.5	106.6	17.9	0.009	9	Very Low

PERFORMED IN GENERAL ACCORDANCE WITH ☐ UBC STANDARD 18-2 ☑ ASTM D 4829

FIGURE B-4

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

EXPANSION INDEX TEST RESULTS

SAN DIEGO FIRE-RESCUE AIR OPERATIONS HANGARS
MONTGOMERY-GIBBS EXECUTIVE AIRPORT, SAN DIEGO, CALIFORNIA

SAMPLE LOCATION	SAMPLE DEPTH (ft)	pH [1]	RESISTIVITY [1] (ohm-cm)	SULFATE CONTENT [2]		CHLORIDE CONTENT [3] (ppm)
				(ppm)	(%)	
B-4	0.0-4.0	8.6	880	110	0.011	400

[1] PERFORMED IN GENERAL ACCORDANCE WITH CALIFORNIA TEST METHOD 643
[2] PERFORMED IN GENERAL ACCORDANCE WITH CALIFORNIA TEST METHOD 417
[3] PERFORMED IN GENERAL ACCORDANCE WITH CALIFORNIA TEST METHOD 422

FIGURE B-5

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

CORROSIVITY TEST RESULTS

SAN DIEGO FIRE-RESCUE AIR OPERATIONS HANGARS
MONTGOMERY-GIBBS EXECUTIVE AIRPORT, SAN DIEGO, CALIFORNIA

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SAMPLE LOCATION	SAMPLE DEPTH (ft)	SOIL TYPE	R-VALUE
B-1	0.5-2.5	Sandy CLAY	13

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2844/CT 301

FIGURE B-6

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

R-VALUE TEST RESULTS

SAN DIEGO FIRE-RESCUE AIR OPERATIONS HANGARS
MONTGOMERY-GIBBS EXECUTIVE AIRPORT, SAN DIEGO, CALIFORNIA

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APPENDIX C

Infiltration Test Data

Test Date:	8/17/2018	Infiltration Test No.:	IT-1
Test Hole Diameter, D (inches):	8.0	Excavation Depth (feet):	5.0
Test performed and recorded by:	GSW	Pipe Length (feet):	5.0

t_1	d_1 (feet)	t_2	d_2 (feet)	Δt (min)	ΔH (feet)	Percolation Rate (min/in)	H_{avg} (feet)	Infiltration Rate (in/hr)
7:00	2.90	7:25	2.90	25	0.00	---	2.10	<0.01
7:25	2.90	7:50	2.90	25	0.00	---	2.10	<0.01
7:50	2.90	8:20	2.90	30	0.00	---	2.10	<0.01
8:20	2.90	8:50	2.90	30	0.00	---	2.10	<0.01
8:50	2.90	9:20	2.91	30	0.01	250	2.10	0.02
9:20	2.90	9:50	2.90	30	0.00	---	2.10	<0.01
9:50	2.90	10:20	2.90	30	0.00	---	2.10	<0.01
10:20	2.90	10:50	2.90	30	0.00	---	2.10	<0.01
10:50	2.90	11:20	2.90	30	0.00	---	2.10	<0.01
11:20	2.90	11:50	2.90	30	0.00	---	2.10	<0.01
11:50	2.90	12:20	2.90	30	0.00	---	2.10	<0.01
12:20	2.90	12:50	2.90	30	0.00	---	2.10	<0.01

Test Date:	8/17/2018	Infiltration Test No.:	IT-2
Test Hole Diameter, D (inches):	8.0	Excavation Depth (feet):	5.0
Test performed and recorded by:	GSW	Pipe Length (feet):	5.0

t_1	d_1 (feet)	t_2	d_2 (feet)	Δt (min)	ΔH (feet)	Percolation Rate (min/in)	H_{avg} (feet)	Infiltration Rate (in/hr)
7:01	2.50	7:26	2.50	25	0.00	---	2.50	<0.01
7:26	2.50	7:51	2.50	25	0.00	---	2.50	<0.01
7:51	2.50	8:21	2.50	30	0.00	---	2.50	<0.01
8:21	2.50	8:51	2.50	30	0.00	---	2.50	<0.01
8:51	2.50	9:21	2.50	30	0.00	---	2.50	<0.01
9:21	2.50	9:51	2.51	30	0.01	250	2.50	0.02
9:51	2.50	10:21	2.50	30	0.00	---	2.50	<0.01
10:21	2.50	10:51	2.50	30	0.00	---	2.50	<0.01
10:51	2.50	11:21	2.50	30	0.00	---	2.50	<0.01
11:21	2.50	11:51	2.50	30	0.00	---	2.50	<0.01
11:51	2.50	12:21	2.50	30	0.00	---	2.50	<0.01
12:21	2.50	12:51	2.50	30	0.00	---	2.50	<0.01

Notes:

t_1 = initial time when filling or refilling is completed

d_1 = initial depth to water in hole at t_1

t_2 = final time when incremental water level reading is taken

d_2 = final depth to water in hole at t_2

Δt = change in time between initial and final water level readings

ΔH = change in depth to water or change in height of water column (i.e., $d_2 - d_1$)

H_0 = Initial height of water column

in/hr = inches per hour

<u>Percolation Rate to Infiltration Rate Conversion</u> [1]

$$I_t = \frac{\Delta H \times 60 \times r}{\Delta t\left(r + 2H_{avg}\right)}$$

I_t = tested infiltration rate, inches/hour

ΔH = change in head over the time interval, inches

Δt = time interval, minutes

r = effective radius of test hole

H_{avg} = average head over the time interval, inches

[1] Based on the "Porchet Method" as presented in:
Riverside County Flood Control, 2011, Design Handbook for Low Impact Development Best Management Practices: dated September.

Test Date:	8/17/2018						Infiltration Test No.:	IT-3
Test Hole Diameter, D (inches):			8.0				Excavation Depth (feet):	5.0
Test performed and recorded by:			GSW				Pipe Length (feet):	5.0

t_1	d_1 (feet)	t_2	d_2 (feet)	Δt (min)	ΔH (feet)	Percolation Rate (min/in)	H_{avg} (feet)	Infiltration Rate (in/hr)
7:04	2.85	7:29	2.85	25	0.00	---	2.15	<0.01
7:29	2.85	7:54	2.85	25	0.00	---	2.15	<0.01
7:54	2.85	8:24	2.85	30	0.00	---	2.15	<0.01
8:24	2.85	8:54	2.85	30	0.00	---	2.15	<0.01
8:54	2.85	9:24	2.85	30	0.00	---	2.15	<0.01
9:24	2.85	9:54	2.86	30	0.01	250	2.15	0.02
9:54	2.85	10:24	2.85	30	0.00	---	2.15	<0.01
10:24	2.85	10:54	2.85	30	0.00	---	2.15	<0.01
10:54	2.85	11:24	2.85	30	0.00	---	2.15	<0.01
11:24	2.85	11:54	2.85	30	0.00	---	2.15	<0.01
11:54	2.85	12:24	2.85	30	0.00	---	2.15	<0.01
12:24	2.85	12:54	2.85	30	0.00	---	2.15	<0.01

Test Date:	8/17/2018						Infiltration Test No.:	IT-4
Test Hole Diameter, D (inches):			8.0				Excavation Depth (feet):	5.0
Test performed and recorded by:			GSW				Pipe Length (feet):	5.0

t_1	d_1 (feet)	t_2	d_2 (feet)	Δt (min)	ΔH (feet)	Percolation Rate (min/in)	H_{avg} (feet)	Infiltration Rate (in/hr)
7:05	2.20	7:30	2.20	25	0.00	---	2.80	<0.01
7:30	2.20	7:55	2.20	25	0.00	---	2.80	<0.01
7:55	2.20	8:25	2.20	30	0.00	---	2.80	<0.01
8:25	2.20	8:55	2.20	30	0.00	---	2.80	<0.01
8:55	2.20	9:25	2.20	30	0.00	---	2.80	<0.01
9:25	2.20	9:55	2.20	30	0.00	---	2.80	<0.01
9:55	2.20	10:25	2.20	30	0.00	---	2.80	<0.01
10:25	2.20	10:55	2.20	30	0.00	---	2.80	<0.01
10:55	2.20	11:25	2.21	30	0.01	250	2.80	0.01
11:25	2.20	11:55	2.20	30	0.00	---	2.80	<0.01
11:55	2.20	12:25	2.20	30	0.00	---	2.80	<0.01
12:25	2.20	12:55	2.20	30	0.00	---	2.80	<0.01

Notes:

t_1 = initial time when filling or refilling is completed

d_1 = initial depth to water in hole at t_1

t_2 = final time when incremental water level reading is taken

d_2 = final depth to water in hole at t_2

Δt = change in time between initial and final water level readings

ΔH = change in depth to water or change in height of water column (i.e., $d_2 - d_1$)

H_0 = Initial height of water column

in/hr = inches per hour

<u>Percolation Rate to Infiltration Rate Conversion</u> [1]

$$I_t = \frac{\Delta H \times 60 \times r}{\Delta t\left(r + 2H_{avg}\right)}$$

I_t = tested infiltration rate, inches/hour

ΔH = change in head over the time interval, inches

Δt = time interval, minutes

r = effective radius of test hole

H_{avg} = average head over the time interval, inches

[1] Based on the "Porchet Method" as presented in:
Riverside County Flood Control, 2011, Design Handbook for Low Impact Development Best Management Practices: dated September.

Worksheet C.4-1: Categorization of Infiltration Feasibility Condition Based on Geotechnical Conditions[9]

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[10]
Part 1 - Full Infiltration Feasibility Screening Criteria		
DMA(s) Being Analyzed:		Project Phase:
San Diego Fire-Rescue Air Operations Hangars		Design
Criteria 1: Infiltration Rate Screening		
1A	Is the mapped hydrologic soil group according to the NRCS Web Soil Survey or UC Davis Soil Web Mapper Type A or B and corroborated by available site soil data[11]? ☐ Yes; the DMA may feasibly support full infiltration. Answer "Yes" to Criteria 1 Result or continue to Step 1B if the applicant elects to perform infiltration testing. ☐ No; the mapped soil types are A or B but is not corroborated by available site soil data (continue to Step 1B). ☑ No; the mapped soil types are C, D, or "urban/unclassified" and is corroborated by available site soil data. Answer "No" to Criteria 1 Result. ☐ No; the mapped soil types are C, D, or "urban/unclassified" but is not corroborated by available site soil data (continue to Step 1B).	
1B	Is the reliable infiltration rate calculated using planning phase methods from Table D.3-1? ☐ Yes; Continue to Step 1C. ☐ No; Skip to Step 1D.	
1C	Is the reliable infiltration rate calculated using planning phase methods from Table D.3-1 greater than 0.5 inches per hour? ☐ Yes; the DMA may feasibly support full infiltration. Answer "Yes" to Criteria 1 Result. ☐ No; full infiltration is not required. Answer "No" to Criteria 1 Result.	
1D	**Infiltration Testing Method.** Is the selected infiltration testing method suitable during the design phase (see Appendix D.3)? Note: Alternative testing standards may be allowed with appropriate rationales and documentation. ☐ Yes; continue to Step 1E. ☐ No; select an appropriate infiltration testing method.	

[9] Note that it is not required to investigate each and every criterion in the worksheet, a single "no" answer in Part 1, Part 2, Part 3, or Part 4 determines a full, partial, or no infiltration condition.

[10] This form must be completed each time there is a change to the site layout that would affect the infiltration feasibility condition. Previously completed forms shall be retained to document the evolution of the site storm water design.

[11] Available data includes site-specific sampling or observation of soil types or texture classes, such as obtained from borings or test pits necessary to support other design elements.

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[10]
1E	**Number of Percolation/Infiltration Tests.** Does the infiltration testing method performed satisfy the minimum number of tests specified in Table D.3-2? ☐ Yes; continue to Step 1F. ☐ No; conduct appropriate number of tests.	
IF	**Factor of Safety.** Is the suitable Factor of Safety selected for full infiltration design? See guidance in D.5; Tables D.5-1 and D.5-2; and Worksheet D.5-1 (Form I-9). ☐ Yes; continue to Step 1G. ☐ No; select appropriate factor of safety.	
1G	**Full Infiltration Feasibility.** Is the average measured infiltration rate divided by the Factor of Safety greater than 0.5 inches per hour? ☐ Yes; answer "Yes" to Criteria 1 Result. ☐ No; answer "No" to Criteria 1 Result.	
Criteria 1 Result	Is the estimated reliable infiltration rate greater than 0.5 inches per hour within the DMA where runoff can reasonably be routed to a BMP? ☐ Yes; the DMA may feasibly support full infiltration. Continue to Criteria 2. ☑ No; full infiltration is not required. Skip to Part 1 Result.	
Summarize infiltration testing methods, testing locations, replicates, and results and summarize estimates of reliable infiltration rates according to procedures outlined in D.5. Documentation should be included in project geotechnical report. In-situ infiltration testing of site soils indicated that the water level at all four test locations generally remained constant over the 30 minute testing intervals and did not infiltrate. For infiltration test method, locations, and results, refer to the project preliminary geotechnical evaluation report (2018) prepared by Ninyo & Moore.		

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[10]	
Criteria 2: Geologic/Geotechnical Screening			
2A	If all questions in Step 2A are answered "Yes," continue to Step 2B. For any "No" answer in Step 2A answer "No" to Criteria 2, and submit an "Infiltration Feasibility Condition Letter" that meets the requirements in Appendix C.1.1. The geologic/geotechnical analyses listed in Appendix C.2.1 do not apply to the DMA because one of the following setbacks cannot be avoided and therefore result in the DMA being in a no infiltration condition. The setbacks must be the closest horizontal radial distance from the surface edge (at the overflow elevation) of the BMP.		
2A-1	Can the proposed full infiltration BMP(s) avoid areas with existing fill materials greater than 5 feet thick below the infiltrating surface?	☐ Yes	☐ No
2A-2	Can the proposed full infiltration BMP(s) avoid placement within 10 feet of existing underground utilities, structures, or retaining walls?	☐ Yes	☐ No
2A-3	Can the proposed full infiltration BMP(s) avoid placement within 50 feet of a natural slope (>25%) or within a distance of 1.5H from fill slopes where H is the height of the fill slope?	☐ Yes	☐ No
2B	When full infiltration is determined to be feasible, a geotechnical investigation report must be prepared that considers the relevant factors identified in Appendix C.2.1. If all questions in Step 2B are answered "Yes," then answer "Yes" to Criteria 2 Result. If there are "No" answers continue to Step 2C.		
2B-1	**Hydroconsolidation.** Analyze hydroconsolidation potential per approved ASTM standard due to a proposed full infiltration BMP. Can full infiltration BMPs be proposed within the DMA without increasing hydroconsolidation risks?	☐ Yes	☐ No
2B-2	**Expansive Soils.** Identify expansive soils (soils with an expansion index greater than 20) and the extent of such soils due to proposed full infiltration BMPs. Can full infiltration BMPs be proposed within the DMA without increasing expansive soil risks?	☐ Yes	☐ No

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[10]	
2B-3	**Liquefaction**. If applicable, identify mapped liquefaction areas. Evaluate liquefaction hazards in accordance with Section 6.4.2 of the City of San Diego's Guidelines for Geotechnical Reports (2011 or most recent edition). Liquefaction hazard assessment shall take into account any increase in groundwater elevation or groundwater mounding that could occur as a result of proposed infiltration or percolation facilities. Can full infiltration BMPs be proposed within the DMA without increasing liquefaction risks?	☐ Yes	☐ No
2B-4	**Slope Stability**. If applicable, perform a slope stability analysis in accordance with the ASCE and Southern California Earthquake Center (2002) Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Landslide Hazards in California to determine minimum slope setbacks for full infiltration BMPs. See the City of San Diego's Guidelines for Geotechnical Reports (2011) to determine which type of slope stability analysis is required. Can full infiltration BMPs be proposed within the DMA without increasing slope stability risks?	☐ Yes	☐ No
2B-5	**Other Geotechnical Hazards.** Identify site-specific geotechnical hazards not already mentioned (refer to Appendix C.2.1). Can full infiltration BMPs be proposed within the DMA without increasing risk of geologic or geotechnical hazards not already mentioned?	☐ Yes	☐ No
2B-6	**Setbacks.** Establish setbacks from underground utilities, structures, and/or retaining walls. Reference applicable ASTM or other recognized standard in the geotechnical report. Can full infiltration BMPs be proposed within the DMA using established setbacks from underground utilities, structures, and/or retaining walls?	☐ Yes	☐ No

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[10]	
2C	**Mitigation Measures.** Propose mitigation measures for each geologic/geotechnical hazard identified in Step 2B. Provide a discussion of geologic/geotechnical hazards that would prevent full infiltration BMPs that cannot be reasonably mitigated in the geotechnical report. See Appendix C.2.1.8 for a list of typically reasonable and typically unreasonable mitigation measures. Can mitigation measures be proposed to allow for full infiltration BMPs? If the question in Step 2 is answered "Yes," then answer "Yes" to Criteria 2 Result. If the question in Step 2C is answered "No," then answer "No" to Criteria 2 Result.	☐ Yes	☐ No
Criteria 2 Result	Can infiltration greater than 0.5 inches per hour be allowed without increasing risk of geologic or geotechnical hazards that cannot be reasonably mitigated to an acceptable level?	☐ Yes	☐ No
Summarize findings and basis; provide references to related reports or exhibits.			
Part 1 Result – Full Infiltration Geotechnical Screening [12]		Result	
If answers to both Criteria 1 and Criteria 2 are "Yes", a full infiltration design is potentially feasible based on Geotechnical conditions only. If either answer to Criteria 1 or Criteria 2 is "No", a full infiltration design is not required.		☐ Full infiltration Condition ☑ Complete Part 2	

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

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[10]
Part 2 – Partial vs. No Infiltration Feasibility Screening Criteria		
DMA(s) Being Analyzed:		Project Phase:
San Diego Fire-Rescue Air Operations Hangars		Design
Criteria 3 : Infiltration Rate Screening		
3A	**NRCS Type C, D, or "urban/unclassified":** Is the mapped hydrologic soil group according to the NRCS Web Soil Survey or UC Davis Soil Web Mapper is Type C, D, or "urban/unclassified" and corroborated by available site soil data? ☐ Yes; the site is mapped as C soils and a reliable infiltration rate of 0.15 in/hr. is used to size partial infiltration BMPS. Answer "Yes" to Criteria 3 Result. ☐ Yes; the site is mapped as D soils or "urban/unclassified" and a reliable infiltration rate of 0.05 in/hr. is used to size partial infiltration BMPS. Answer "Yes" to Criteria 3 Result. ☑ No; infiltration testing is conducted (refer to Table D.3-1), continue to Step 3B.	
3B	**Infiltration Testing Result:** Is the reliable infiltration rate (i.e. average measured infiltration rate/2) greater than 0.05 in/hr. and less than or equal to 0.5 in/hr? ☐ Yes; the site may support partial infiltration. Answer "Yes" to Criteria 3 Result. ☑ No; the reliable infiltration rate (i.e. average measured rate/2) is less than 0.05 in/hr., partial infiltration is not required. Answer "No" to Criteria 3 Result.	
Criteria 3 Result	Is the estimated reliable infiltration rate (i.e., average measured infiltration rate/2) greater than or equal to 0.05 inches/hour and less than or equal to 0.5 inches/hour at any location within each DMA where runoff can reasonably be routed to a BMP? ☐ Yes; Continue to Criteria 4. ☑ No: Skip to Part 2 Result.	
Summarize infiltration testing and/or mapping results (i.e. soil maps and series description used for infiltration rate). A total of four infiltration tests were conducted at the site. Each test was performed at a depth of approximately 5 feet in very old paralic deposits consisting of silty sandstone. In-situ infiltration rates were measured as follows: IT-1: did not infiltrate IT-2: did not infiltrate IT-3: did not infiltrate IT-4: did not infiltrate		

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[10]	
Criteria 4: Geologic/Geotechnical Screening			
4A	If all questions in Step 4A are answered "Yes," continue to Step 2B. For any "No" answer in Step 4A answer "No" to Criteria 4 Result, and submit an "Infiltration Feasibility Condition Letter" that meets the requirements in Appendix C.1.1. The geologic/geotechnical analyses listed in Appendix C.2.1 do not apply to the DMA because one of the following setbacks cannot be avoided and therefore result in the DMA being in a no infiltration condition. The setbacks must be the closest horizontal radial distance from the surface edge (at the overflow elevation) of the BMP.		
4A-1	Can the proposed partial infiltration BMP(s) avoid areas with existing fill materials greater than 5 feet thick?	☐ Yes	☐ No
4A-2	Can the proposed partial infiltration BMP(s) avoid placement within 10 feet of existing underground utilities, structures, or retaining walls?	☐ Yes	☐ No
4A-3	Can the proposed partial infiltration BMP(s) avoid placement within 50 feet of a natural slope (>25%) or within a distance of 1.5H from fill slopes where H is the height of the fill slope?	☐ Yes	☐ No
4B	When full infiltration is determined to be feasible, a geotechnical investigation report must be prepared that considers the relevant factors identified in Appendix C.2.1 If all questions in Step 4B are answered "Yes," then answer "Yes" to Criteria 4 Result. If there are any "No" answers continue to Step 4C.		
4B-1	**Hydroconsolidation.** Analyze hydroconsolidation potential per approved ASTM standard due to a proposed full infiltration BMP. Can partial infiltration BMPs be proposed within the DMA without increasing hydroconsolidation risks?	☐ Yes	☐ No
4B-2	**Expansive Soils.** Identify expansive soils (soils with an expansion index greater than 20) and the extent of such soils due to proposed full infiltration BMPs. Can partial infiltration BMPs be proposed within the DMA without increasing expansive soil risks?	☐ Yes	☐ No

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[10]	
4B-3	**Liquefaction**. If applicable, identify mapped liquefaction areas. Evaluate liquefaction hazards in accordance with Section 6.4.2 of the City of San Diego's Guidelines for Geotechnical Reports (2011). Liquefaction hazard assessment shall take into account any increase in groundwater elevation or groundwater mounding that could occur as a result of proposed infiltration or percolation facilities. Can partial infiltration BMPs be proposed within the DMA without increasing liquefaction risks?	☐ Yes	☐ No
4B-4	**Slope Stability**. If applicable, perform a slope stability analysis in accordance with the ASCE and Southern California Earthquake Center (2002) Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Landslide Hazards in California to determine minimum slope setbacks for full infiltration BMPs. See the City of San Diego's Guidelines for Geotechnical Reports (2011) to determine which type of slope stability analysis is required. Can partial infiltration BMPs be proposed within the DMA without increasing slope stability risks?	☐ Yes	☐ No
4B-5	**Other Geotechnical Hazards.** Identify site-specific geotechnical hazards not already mentioned (refer to Appendix C.2.1). Can partial infiltration BMPs be proposed within the DMA without increasing risk of geologic or geotechnical hazards not already mentioned?	☐ Yes	☐ No
4B-6	**Setbacks.** Establish setbacks from underground utilities, structures, and/or retaining walls. Reference applicable ASTM or other recognized standard in the geotechnical report. Can partial infiltration BMPs be proposed within the DMA using recommended setbacks from underground utilities, structures, and/or retaining walls?	☐ Yes	☐ No
4C	**Mitigation Measures.** Propose mitigation measures for each geologic/geotechnical hazard identified in Step 4B. Provide a discussion on geologic/geotechnical hazards that would prevent partial infiltration BMPs that cannot be reasonably mitigated in the geotechnical report. See Appendix C.2.1.8 for a list of typically reasonable and typically unreasonable mitigation measures. Can mitigation measures be proposed to allow for partial infiltration BMPs? If the question in Step 4C is answered "Yes," then answer "Yes" to Criteria 4 Result. If the question in Step 4C is answered "No," then answer "No" to Criteria 4 Result.	☐ Yes	☐ No

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[10]	
Criteria 4 Result	Can infiltration of greater than or equal to 0.05 inches/hour and less than or equal to 0.5 inches/hour be allowed without increasing the risk of geologic or geotechnical hazards that cannot be reasonably mitigated to an acceptable level?	☐ Yes	☐ No
Summarize findings and basis; provide references to related reports or exhibits.			

Part 2 – Partial Infiltration Geotechnical Screening Result[13]	Result
If answers to both Criteria 3 and Criteria 4 are "Yes", a partial infiltration design is potentially feasible based on geotechnical conditions only. If answers to either Criteria 3 or Criteria 4 is "No", then infiltration of any volume is considered to be infeasible within the site.	☐ Partial Infiltration Condition ☑ No Infiltration Condition

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

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