Geotechnical Evaluation
Tijuana River Valley Regional Park Campgrounds
San Diego, California

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1 INTRODUCTION

In accordance with your request and authorization, we are providing this geotechnical evaluation report for the proposed Tijuana River Valley Regional Park Campgrounds project located north of Monument Road and west of Saturn Boulevard in San Diego, California (Figure 1). Presented in this report are the results of our background review, field exploration, geotechnical laboratory testing, our conclusions regarding the geotechnical conditions at the site, and our recommendations for the design and earthwork construction aspects of this project.

2 SCOPE OF SERVICES

Ninyo & Moore’s scope of services for this project included review of pertinent background data, performance of a geologic reconnaissance, subsurface exploration, and engineering analysis with regard to the proposed construction. These services generally follow the scope outlined in our proposal dated November 20, 2017. Specifically, we performed the following tasks:

- Reviewing readily available pertinent geotechnical reports, published in-house geotechnical literature, topographic, geologic, and fault maps, historic stereoscopic aerial photographs, and conceptual site plans (MW Peltz & Associates, 2018) of the proposed project.

- Attending a kick-off meeting to discuss the project scope.

- Performing a field reconnaissance with County of San Diego staff to observe site conditions and to mark the locations of our exploratory borings. The locations of the borings were approved in the field by the County of San Diego project manager.

- Coordinating with Underground Service Alert (USA) for utility clearance of our boring locations.

- Obtaining a boring permit from the County of San Diego Department of Environmental Health (DEH) prior to performing our subsurface exploration.

- Performing a subsurface exploration consisting of the drilling, logging, and sampling of eight small diameter exploratory borings. The soil borings were drilled to depths of up to approximately 61½ feet using a truck-mounted drill rig and manual techniques. Bulk and relatively undisturbed samples of the materials encountered were collected at selected interval from the borings and transported to our in-house geotechnical laboratory for testing.

- Installing a temporary piezometer in one of the borings.

- Performing infiltration testing within four of the borings.

- Performing geotechnical laboratory testing on representative samples to evaluate soil parameters for design and classification purposes.
• Performing engineering analyses of the site geotechnical conditions based on data obtained from our background review, field exploration, and laboratory testing.

• Preparing this geotechnical evaluation report describing the findings and conclusions of our study and providing recommendations for design and construction of the proposed improvements.

3 SITE AND PROJECT DESCRIPTION

The site is located north of Monument Road and west of Saturn Boulevard within the Tijuana River Valley in the southern part of San Diego, California (Figure 1). The project site consists of two adjacent parcels of land that are owned and operated by the County of San Diego which total approximately 80 acres and are currently undeveloped. The site is bounded by Monument Road to the south, Saturn Boulevard to the east, and undeveloped open space to the west and north. The project site is relatively level with a gentle gradient down to the west. Onsite elevations range from approximately 15 feet above mean sea level (MSL) in the western portion of the site to approximately 20 feet above MSL in the eastern portion of the site. The project site is generally undeveloped and sparsely vegetated with grasses and shrubs. Several dirt roads and trails transect portions of the site. Berms comprised of soil and concrete debris are present along the northern and southern perimeters of the site. Portions of the site have been used for agricultural purposes in the past (Ninyo & Moore, 2018a).

Based on our review of conceptual plans (MW Peltz & Associates, 2018), we understand that the project will consist of the construction of a campground with new restroom and shower buildings, an office building, an entry booth, camping areas for tents and trailers, parking areas, and an equestrian center. Additional improvements are anticipated to consist of circulation roads and trails with aggregate base or decomposed granite (DG) surfaces, underground utilities, concrete American with Disabilities Act (ADA) walkways and ramps, and landscaping.

It is our understanding that due to the site being within a flood zone, the proposed structural buildings will be elevated above the ground surface. Specifically, our conversations with the design team indicate that the restroom/shower buildings and the office building will be constructed 6 feet above grade to account for the anticipated flood height. While the project is located within the City of San Diego, we understand that the site is owned and operated by the County of San Diego and that general development of the project will primarily be reviewed and overseen by the County of San Diego.
4 SUBSURFACE EXPLORATION

Our subsurface exploration was conducted on May 22 and 23, 2018, and included the drilling, logging, and sampling of eight small-diameter borings (Borings B-1 through B-8). Prior to commencing the subsurface exploration, USA was notified for marking of the existing site utilities. The purpose of the borings was to evaluate subsurface conditions and to collect soil samples for laboratory testing.

The borings were drilled to depths ranging from approximately 3 feet to 61½ feet using manual equipment and a truck-mounted drill rig equipped with 8-inch diameter, continuous-flight, hollow-stem augers. During the drilling operations, the borings were logged and sampled by personnel from Ninyo & Moore. Representative bulk and in-place soil samples were obtained from the borings. The samples were then transported to our in-house geotechnical laboratory for testing. The approximate locations of the exploratory borings are shown on Figure 2. Logs of the borings are included in Appendix A.

5 LABORATORY TESTING

Geotechnical laboratory testing was performed on representative soil samples collected during our subsurface exploration. Testing included an evaluation of in-situ dry density and moisture content, gradation (sieve) analysis, Atterberg limits, shear strength, expansion index, soil corrosivity, and R-value. The results of the in-situ dry density and moisture content tests are presented 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 May 22 and March 23, 2018 in general accordance with the County of San Diego BMP Design Manual (2016). The infiltration test holes (B-2, B-5, B-7, and B-8) were excavated with manual equipment to depths of approximately 3 to 5 feet at the locations shown on Figure 2. The infiltration test at B-2 was performed prior to additional drilling. The infiltration tests were performed in general accordance with the County of San Diego BMP Design Manual (2016). 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 May 22, 2018 to represent adverse conditions for infiltration. The presoak consisted of maintaining approximately 1 foot of water in each boring for approximately
4 hours. Infiltration testing was then performed in the presoaked test borings. Due to the high infiltration rate during the presoak (12 inches of water infiltrated in less than 30 minutes), the test was modified to measure and record the water depth every 5 or 10 minutes. As necessary, the borings were refilled to maintain the water level until the infiltration rate stabilized.

### 6.1 Infiltration Test Results

Infiltration rates were calculated using the Porchet method. Infiltration test results and calculations are included in Appendix C and summarized in Table 1. Appendix C of this report also includes a copy of the Categorization of Infiltration Feasibility worksheet (Worksheet C.4-1).

<table>
<thead>
<tr>
<th>Infiltration Test</th>
<th>Approximate Test Depth (feet)</th>
<th>Description</th>
<th>Adjusted Infiltration Rate (in/hr)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-2</td>
<td>5</td>
<td>Silty Sand (Alluvium)</td>
<td>3.43</td>
</tr>
<tr>
<td>B-5</td>
<td>3</td>
<td>Silty Sand (Alluvium)</td>
<td>10.42</td>
</tr>
<tr>
<td>B-7</td>
<td>5</td>
<td>Silty Sand (Alluvium)</td>
<td>8.17</td>
</tr>
<tr>
<td>B-8</td>
<td>3</td>
<td>Silty Sand (Alluvium)</td>
<td>2.61</td>
</tr>
</tbody>
</table>

Note: in/hr = inches per hour

We note that the in-situ infiltration rates presented in Table 1 represent the infiltration rates at the specific locations and depths indicated in the table. Variation in the infiltration rates can be expected at different depths and/or locations from those shown in the table.

Based on the County of San Diego BMP Design Manual (2016), infiltration rates of less than 0.5 inches per hour may be suitable for partial infiltration and infiltration rates of 0.5 inches per hour or greater per hour may be considered suitable for full infiltration design. The infiltration rates presented above are based on in-situ testing (i.e., factor of safety of 1.0). The design engineer should evaluate and apply an appropriate factor of safety when designing the improvements. The County of San Diego BMP Design Manual (2016) provides additional discussion and considerations for applying an infiltration factor of safety.

The above results are considered preliminary and are for informational purposes. Final design of infiltration should be based on infiltration test results at the planned locations of infiltration basins. General recommendations for the placement, design, and construction of storm water BMPs are in the recommendations section of this report.
Other areas of the site not specifically tested may or may not accommodate full infiltration of storm water. Additional infiltration testing would be needed in these other areas to evaluate whether infiltration in these areas/depths are feasible. Our services did not include an evaluation of specific design of infiltration devices at the site or their potential impact on improvements at the site. Should specific devices be proposed, we can evaluate those upon request.

7 GEOLOGY AND SUBSURFACE CONDITIONS

Our findings regarding regional and site geology and groundwater conditions are provided in the following sections.

7.1 Regional Geologic Setting

The project 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 portion of the province in San Diego County that includes the project area consists generally of uplifted and dissected Quaternary and Tertiary age sedimentary rock.

The Peninsular Ranges Province is traversed by a group of sub-parallel faults and fault zones trending approximately northwest. The Elsinore, San Jacinto, and San Andreas 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 fault located west of the project area. The Rose Canyon fault zone is the nearest active fault system and has been mapped approximately 3 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

The geology of the site vicinity is shown on Figure 3. Geologic units encountered during our site reconnaissance and subsurface exploration included fill and alluvium. Generalized descriptions of the earth units encountered during our field reconnaissance and subsurface exploration and mapped in the vicinity of the project site are provided in the subsequent sections. Additional descriptions of the subsurface units are provided on the boring logs in Appendix A. Due to the relatively level ground surface and generally consistent nature of underlying geologic materials, geologic cross sections were not prepared at the proposed structural building locations.

7.2.1 Fill

Fill materials were encountered in boring B-7 at the ground surface and extending down to a depth of approximately 1½ feet. As encountered, the fill generally consists of brown, moist, medium dense, silty sand with gravel. Based on our site observations, fill within the onsite berms consists of similar material. Additionally, debris and boulder-sized pieces of concrete were observed at the onsite berms.

7.2.2 Alluvium

Quaternary-age alluvium is mapped as underlying the project site and was encountered within our borings from the ground surface or underlying the fill and extending to the total depths explored of 61½ feet. As encountered, the alluvium generally consisted of various shades of gray and brown, moist to wet, stiff to hard, sandy clay and very loose to very dense, sandy silt, silty sand, poorly graded sand with silt, and poorly graded sand. Scattered gravel and cobbles were encountered in the alluvium.

7.2.3 Landslide Deposits

While not observed in our borings, landslide deposits are mapped adjacent to the southern portion of the project site. We anticipate that these deposits are derived from materials of the San Diego Formation and consist of brown silty and clayey sand with gravel and cobbles.

7.2.4 San Diego Formation

The early Pleistocene and late Pliocene–age San Diego Formation is mapped on the hillside south of the project site. While not encountered in our borings, we observed during our site reconnaissance that the San Diego Formation south of the project area is predominantly composed of gray and brown, weakly to strongly cemented, silty sandstone.
7.3 Groundwater
During our field evaluation, groundwater was encountered at depths ranging from approximately 5.5 to 9 feet. A temporary piezometer was installed in boring B-1. Measurements of groundwater depth within the temporary piezometer at the time of installation and approximately 23 hours later were 5.6 feet and 5.7 feet, respectively. Based on our review of groundwater monitoring well data in the site vicinity using the California Department of Water Resources (DWR) Water Data Library (California DWR, 2018), groundwater is anticipated in the project area at elevations of approximately 6 to 10 feet above mean sea level. Fluctuations in groundwater typically occur due to variations in precipitation, ground surface topography, subsurface stratification, irrigation, groundwater pumping, flooding, and other factors.

7.4 Faulting and Seismicity
The numerous faults in southern California include active, potentially active, and inactive faults. As defined by the California Geological Survey, active faults are faults that have ruptured within Holocene time, or within approximately the last 11,000 years. Potentially active faults are those that show evidence of movement during Quaternary time (approximately the last 1.6 million years), but for which evidence of Holocene movement has not been established. Inactive faults have not ruptured in the last approximately 1.6 million years. The approximate locations of major active and potentially active faults in the vicinity of the site and their geographic relationship to the site are shown on Figure 4.

The site is located in a seismically active area, as is the majority of southern California, and the potential for strong ground motion is considered significant during the design life of the proposed structure. Based on our review of the referenced geologic maps, as well as on our site reconnaissance, several faults are mapped as underlying the project site. These faults are not considered active and are not part of a State of California Earthquake Fault Zone (EFZ) (formerly known as an Alquist-Priolo Special Studies Zone) (Hart and Bryant, 1997). The nearest known active fault is the Rose Canyon fault, located approximately 3 miles west of the site.

Table 2 lists selected principal known active faults that may affect the site and the maximum moment magnitude $M_{max}$ calculated from the USGS National Seismic Hazard Maps - Fault Parameters website (USGS, 2008).
### Table 2 – Principal Active Faults

<table>
<thead>
<tr>
<th>Fault</th>
<th>Approximate Fault-to-Site Distance (miles (kilometers))</th>
<th>Maximum Moment Magnitude (Mmax)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rose Canyon</td>
<td>2.8 (3.5)</td>
<td>6.9</td>
</tr>
<tr>
<td>Coronado Bank</td>
<td>10 (16)</td>
<td>7.4</td>
</tr>
<tr>
<td>Newport-Inglewood (Offshore)</td>
<td>46 (74)</td>
<td>7.0</td>
</tr>
<tr>
<td>Elsinore (Julian Segment)</td>
<td>48 (78)</td>
<td>7.4</td>
</tr>
</tbody>
</table>

As noted previously and shown on Figure 4, several faults are mapped at the project site trending in a general northerly direction. These faults are mapped as being concealed by the alluvium and are not considered active. According to the City of San Diego Seismic Safety Study (2008), two faults are mapped at the site that are potentially active, inactive, presumed inactive, or activity unknown (Figure 5). These faults are mapped as trending north across the central portion of the site and along the eastern perimeter. The proposed structural buildings are located over 500 feet from these faults.

Based on this information, we consider the seismic parameters associated with the closest known active fault, the Rose Canyon fault, more appropriate for design purposes. In general, hazards associated with seismic activity include strong ground motion, ground rupture, liquefaction, seismically induced settlement, and tsunamis. These hazards, along with landsliding, are discussed in the following sections.

#### 7.4.1 Ground Motion

The 2016 California Building Code (CBC) specifies that the Risk-Targeted, Maximum Considered Earthquake (MCE\textsubscript{R}) ground motion response accelerations be used to evaluate seismic loads for design of buildings and other structures. The MCE\textsubscript{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\textsubscript{R} for the site was calculated as 0.46g using the United States Geological Survey (USGS, 2018) 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\textsubscript{G}) peak ground acceleration with adjustment for site class effects in accordance with the American Society of Civil Engineers (ASCE) 7-10 Standard. The MCEG peak ground acceleration is based on the geometric mean peak ground acceleration with a 2 percent probability of exceedance in 50 years. The MCEG peak ground acceleration with
adjustment for site class effects (PGA_m) was calculated as 0.48g using the USGS (USGS, 2018) seismic design tool that yielded a mapped MCE_G peak ground acceleration of 0.46g for the site and a site coefficient (F_PGA) of 1.04 for Site Class D. Site Class D is based on an evaluation of the representative blow counts from a combination of the borings performed at the site.

7.4.2 Ground Rupture
Based on our review of the referenced literature and our site reconnaissance, no active faults are known to cross the project vicinity. While several north-trending faults are mapped at the project site (Figures 3 and 5), they are not considered active by the State of California or the City of San Diego Seismic Safety Study (2008). Therefore, the potential for ground rupture due to faulting at the site is considered low. However, lurching or cracking of the ground surface as a result of nearby seismic events is possible.

7.4.3 Liquefaction and Seismically Induced Settlement
Liquefaction is the phenomenon in which loosely deposited granular soils with silt and clay contents of less than approximately 35 percent and non-plastic silts located below the water table undergo rapid loss of shear strength when subjected to strong earthquake-induced ground shaking. Ground shaking of sufficient duration results in the loss of grain-to-grain contact due to a rapid rise in pore water pressure, and causes the soil to behave as a fluid for a short period of time. Liquefaction is known generally to occur in saturated or near-saturated cohesionless soils at depths shallower than 60 feet below the ground surface. Factors known to influence liquefaction potential include composition and thickness of soil layers, grain size, relative density, groundwater level, degree of saturation, and both intensity and duration of ground shaking.

According to the City of San Diego Seismic Safety Study (2008), the project site is located in geologic hazard category 31, which has a high potential for liquefaction. Accordingly, the liquefaction potential of the subsurface soils was evaluated using subsurface data from borings B-1 and B-3. The liquefaction analysis was based on the National Center for Earthquake Engineering Research (NCEER) procedure (Youd, et al., 2001) developed from the methods originally recommended by Seed and Idriss (1982) and Robertson and Wride (1997) using the computer program LiquefyPro (CivilTech Software, 2007). The historic high groundwater table was considered to be at the ground surface, such as during a flooding event.
As noted in Section 1803A.5.12 of the 2016 CBC, liquefaction analyses are to be based on a site-specific study or in accordance with Section 11.8.3 of ASCE 7. Section 11.8.3 of ASCE 7-10 states that the potential for liquefaction may be evaluated using the peak ground acceleration adjusted for Site Class effects (PGA\textsubscript{m}). Accordingly, our liquefaction analysis been performed using a magnitude of 6.7 and a peak ground acceleration of 0.48g. Our liquefaction analysis indicates that the relatively loose, granular soil layers up to a depth of approximately 60 feet are susceptible to liquefaction during the design seismic event (Appendix D).

7.4.4 Dynamic Settlement of Saturated Soils

As a result of liquefaction, the proposed improvements may be subject to several hazards, including liquefaction-induced settlement. In order to estimate the amount of post-earthquake settlement, the method proposed by Tokimatsu and Seed (1987) was used in which the seismically induced cyclic stress ratios and corrected N-values are related to the volumetric strain of the soil. The amount of soil settlement during a strong seismic event depends on the thickness of the liquefiable layers and the density and/or consistency of the soils. Based on our knowledge and experience with the site soils, the occasional clay layers that were encountered are not sensitive enough to undergo cyclic strain softening.

We evaluated dynamic settlement at the site using the LiquefyPro software (CivilTech Software, 2007). Based on our evaluation, which assumes a peak ground acceleration of 0.48g and a modal magnitude of 6.7, a post-earthquake total settlement of up to approximately 2 to 6½ inches is estimated for the site (Appendix D). Based on the guidelines presented in CGS Special Publication 117A (2008) and assuming relatively uniform subsurface stratigraphy across the site, we estimate differential settlement on the order of 4½ inches over a horizontal distance of approximately 40 feet. Some of the dynamic settlement can be reduced by remedial grading as described in the Recommendations section of this report.

7.4.5 Lateral Spread

Lateral spreading of ground surface during an earthquake usually takes place along weak shear zones that have formed within a liquefiable soil layer. Lateral spread has generally been observed to take place in the direction of a free-face (i.e., retaining wall, slope, channel) but has also been observed to a lesser extent on ground surfaces with very gentle slopes. An empirical model developed by Bartlett and Youd (1995, revised 1999) is typically used to predict the amount of horizontal ground displacement within a site. For a site
located in proximity to a free-face, the amount of lateral ground displacement is strongly correlated with the distance of the site from the free-face. Other factors such as earthquake magnitude, distance from the earthquake epicenter, thickness of the liquefiable layers, and the fines content and particle sizes of the liquefiable layers also affect the amount of lateral ground displacement.

The project site is relatively flat and the Tijuana River channel is located approximately 1,800 feet north of the site. Accordingly, lateral displacement is not a design consideration. As discussed above, our liquefaction analysis (Appendix D) indicates that the site soils from the ground surface to depths of approximately 60 feet are liquefiable.

7.4.6 Surface Manifestation of Liquefaction
Based on the design curves developed by Ishihara (1985) the potential for surface manifestation of liquefaction (i.e., ground subsidence, sand boils, and/or seismically induced bearing failure) is a design consideration at this site.

7.4.7 Tsunamis
Tsunamis are long wavelength seismic sea waves (long compared to the ocean depth) generated by sudden movements of the ocean bottom during submarine earthquakes, landslides, or volcanic activity. Seiches are similar oscillating waves on inland or enclosed bodies of water. Based on our review of the tsunami inundation map prepared by the California Geological Survey (2009), a tsunami inundation area is present within the Tijuana River Valley west of the project site. However, as mapped, the project site is not subject to inundation by tsunamis (Figure 6).

7.4.8 Landsliding
Based on our review of referenced geologic maps, literature and topographic maps, and subsurface exploration, landslides or indications of deep-seated landsliding were not noted underlying the project site. A mapped landslide is present south of the central portion of the site (Figures 3 and 5). The mapped landslide lies at a higher elevation that the project site and the toe of the landslide extends along the central southern perimeter of the site (approximately 1,000 to 2,600 feet west of Saturn Boulevard), where buildings are not planned. In our opinion, the potential for significant large-scale slope instability at the site is not a design consideration.
7.5  Flood Hazards

Based on review of Federal Emergency Management Agency (FEMA) Flood Insurance Rate Maps (FIRM), the site is located within the limits of the mapped 100-year floodplain. Mapped base flood elevations at the site range from approximately 16 feet above MSL in the western portion of the site to 24 feet above MSL in the eastern portion, as shown in Figure 7.

7.6  Geologic Hazards

Based on a review of the City of San Diego Geologic Hazards map (City of San Diego, 2008), the site is located in Hazard Zone 31 (Figure 5). Hazard Zone 31 designates areas with shallow groundwater, within major drainages, and/or underlain by hydraulic fill where the potential for liquefaction is high. In addition, Hazard Zone 12 is mapped in the central portion of the site and the eastern perimeter. Hazard Zone 12 designated areas underlain by faults that are potentially active, inactive, presumed inactive, or activity unknown. We note that the proposed structural buildings are located over approximately 500 feet from Hazard Zone 12.

8  CONCLUSIONS

Based on our review of the referenced background data, subsurface exploration, and geotechnical laboratory testing, it is our opinion that construction of the proposed improvements is feasible from a geotechnical standpoint. In general, the following conclusions were made:

- Based on the results of our field and laboratory evaluations, the subsurface soils at the project site consist of fill and alluvial deposits.

- Fill and alluvial materials are loose and considered unsuitable for structural support in their present condition. Accordingly, recommendations are presented herein for remedial grading of these materials in preparation for new construction.

- Based on our subsurface exploration, excavation of the subsurface materials should be feasible with heavy-duty excavation equipment in good working condition. However, the contractor should anticipate caving and/or sloughing conditions due to the very loose and granular nature of the on-site materials and the presence of shallow groundwater.

- Groundwater was encountered during our evaluation at depths ranging from approximately 5½ feet to 9 feet below the ground surface. Therefore, groundwater is a significant design and construction consideration. Planned excavations should anticipate construction dewatering.

- The project site is located within a mapped 100-year flood zone.

- Faults are mapped at the site. However, active faults have not been mapped on or adjacent to the site and evidence of active faulting was not observed during our site visits. The closest known active fault, the Rose Canyon fault, has been mapped approximately 3 miles west of the site. Accordingly, the potential for relatively strong seismic accelerations will need to be considered in the design of the proposed improvements.
The project site is located in City of San Diego Seismic Safety Study geologic hazard category 31, which has a high potential for liquefaction. Our liquefaction analysis indicates that the relatively loose, granular soil layers occurring below the historic high groundwater level are susceptible to liquefaction during the design seismic event that may result in approximately 2 to 6½ inches of seismically-induced settlement.

Although mapped in the vicinity of the site, based on our field evaluation and a review of referenced maps, landslide deposits do not underlie the proposed improvements.

On-site soils, derived from the earthwork operations are generally considered suitable for reuse as compacted fill and trench backfill. However, due to the presence of gravel and cobble, processing of the on-site soils (including screening) should be anticipated.

Based on the results of our laboratory testing, surface soils possess a low potential for expansion.

Based on the results of our soil corrosivity tests presented in Appendix B of this report as compared to the Caltrans corrosion guidelines (2018), the site would be classified as corrosive.

9  RECOMMENDATIONS

The following recommendations are provided for the design and construction of the proposed project. These recommendations are based on our evaluation of the site geotechnical conditions and our assumptions regarding the planned development. The proposed site improvements should be constructed in accordance with the requirements of the applicable governing agencies.

9.1  Earthwork

In general, earthwork should be performed in accordance with the recommendations presented in this report. The geotechnical consultant should be contacted for questions regarding the recommendations or guidelines presented herein.

9.1.1  Pre-Construction Conference

We recommend that a pre-construction meeting be held prior to commencement of grading. The owner or his representative, the Project Inspector, the agency representatives, the architect, the civil engineer, Ninyo & Moore, and the contractor should attend to discuss the plans, the project, and the proposed construction schedule.
9.1.2 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.

9.1.3 Excavation Characteristics
The result of our field exploration program indicates that the project site is underlain by fill and alluvial soils. Excavation of the subsurface materials should be feasible with heavy-duty excavation equipment in good working condition. However, due to the presence of cohesionless sands and shallow groundwater, the following ground conditions are anticipated and the contractor should be prepared to address them:

- Caving soils;
- Flowing sands;
- Sloughing of excavations;
- Unstable excavation bottoms;
- Pumping subgrade conditions.

Recommendations for dewatering and mitigation of unstable excavation bottoms are presented in the following sections.

Additionally, due to the presence of large concrete debris in the noted earthen berms at the site, excavations into the fill materials of these berms may encounter difficulty with performing excavations and will generate oversize materials.

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

\[
\text{Fill and Alluvium} \quad \text{Type C}
\]

Upon making the excavations, the soil classifications and excavation performance should be evaluated in the field in accordance with the OSHA regulations. Temporary excavations should be constructed in accordance with OSHA recommendations. For trench or other excavations, OSHA requirements regarding personnel safety should be met using appropriate shoring (including trench boxes) or by laying back the slopes to a slope ratio no


steeper than 1½:1 (horizontal to vertical) in fill and alluvium. Temporary excavations that encounter seepage may require shoring or may be 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.

9.1.5 Mitigation of Unstable Bottoms
During the performance of the earthwork operations, loose/soft and wet, yielding subgrade materials are anticipated to be encountered. For areas to receive structural buildings, the area should be prepared in accordance with the following sections. For other site improvements or to aid in the placement of compacted fills, yielding subgrade conditions may be mitigated using a combination of aggregate base materials, geogrid, and/or geosynthetic fabric. However, these recommendations should be reviewed and adjusted by the project geotechnical consultant based on conditions exposed in the field on a case-by-case basis.

9.1.6 Remedial Grading – Structural Buildings
Due to the presence of soils susceptible to liquefaction during a seismic event and the potential for surface manifestation of liquefaction, we recommend that remedial grading be performed within structural building areas. For the purpose of this report, structural building areas are defined as the areas underlying the buildings and extending a horizontal distance of 5 feet beyond the footprints of the structures. To help mitigate the effects of liquefaction and lateral displacement of the soils underlying the structural buildings, our remedial earthwork includes the placement of a reinforced fill mat beneath the structural buildings. The excavation bottoms may encounter wet, loose material which may exhibit pumping under heavy equipment loads. To address the noted conditions, we recommend that the remedial grading include the following:

- Removal of the existing site soils within the building pad as described above, including fill and alluvium, to a depth of 2 feet below the bottom of the foundation system for structural buildings.

- Subsequent to the removal, a low ground pressure bulldozer or excavator should be used to fill in ruts and dress the surface at the removal bottom elevation.

- A layer of geosynthetic woven fabric (Mirafi HP 570 or an equivalent) should then be rolled out over the dressed excavation bottom. The geosynthetic woven fabric material should overlap approximately 2 feet. Heavy equipment traffic, including trucks, should not be allowed on the geosynthetic woven fabric.
• An initial 12 inches of aggregate base materials should be pushed out over the geosynthetic woven fabric with low-pressure construction equipment. Compaction effort should then be made to this aggregate base layer using smaller, low-pressure construction equipment. Again, heavy equipment traffic, including trucks, should not be allowed on the initial lift of aggregate.

• Then a layer of Triaxial geogrid TX-7 (or equivalent) should be placed over the initial layer of aggregate base materials and the geogrid should overlap approximately 2 feet. And heavy equipment traffic, including trucks, should not be allowed on the geogrid.

• After placement of the geogrid, another 12 inches of aggregate base materials should be pushed out over the geosynthetic woven fabric with low-pressure construction equipment. Compaction effort should then be made to this aggregate base layer using smaller, low-pressure construction equipment. This layer of aggregate base materials should be compacted to a relative compaction of 90 percent as evaluated by ASTM International (ASTM) D 1557.

Subsequent to the remedial grading efforts, the structural building foundations may be constructed on the aggregate base materials.

9.1.7 Construction Dewatering
Due to the presence of shallow groundwater, we anticipate that dewatering may be required during excavations. A specialty contractor should be utilized in design and construction of the dewatering system. Discharge of water from excavations may involve securing a National Pollution Discharge Elimination System (NPDES) permit. Compliance with the permit requirements and the current guidelines of the Regional Water Quality Control Board may involve testing and treatment of water prior to discharge.

9.1.8 Materials for Fill
Materials for fill may be processed from on-site excavations, or may consist of import materials. On-site soils with an organic content of less than approximately 3 percent by volume (or 1 percent by weight) are suitable for reuse as general fill material. Fill soils should be free of trash, debris, roots, vegetation, organics, or other deleterious materials. Due to the shallow groundwater moisture conditioning of on-site materials, including drying and/or aerating, should be anticipated. Fill and utility trench backfill materials should not contain rocks or lumps over 3 inches, and not more than 30 percent larger than ¾ inch. Larger chunks, if generated during excavation, may be broken into acceptably sized pieces or disposed of offsite.
Imported fill material, if needed, should meet the criteria described above and be granular soil possessing a low or very low expansion potential (i.e., an EI of 50 or less as evaluated by ASTM D 4829). Imported materials should also be non-corrosive in accordance with the Caltrans (2018) corrosion guidelines. Materials for use as fill should be evaluated by the geotechnical consultant's representative prior to filling or importing.

9.1.9 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.
9.1.10 Pipe Bedding and Modulus of Soil Reaction ($E'$)

It is our recommendation that new pipelines (pipes), where constructed in open excavations, be supported on 6 or more inches of granular bedding material. Granular pipe bedding should be provided to distribute vertical loads around the pipe. Bedding material and compaction requirements should be in accordance with this report. Pipe bedding typically consists of graded aggregate with a coefficient of uniformity of three or greater.

Pipe bedding and pipe zone backfill should have a Sand Equivalent of 30 or more, and be placed around the sides and the crown of the pipe. In addition, the pipe zone backfill should extend 1 foot or more above the crown of the pipe. Special care should be taken not to allow voids beneath and around the pipe. Compaction of the pipe zone backfill should proceed up both sides of the pipe.

It has been our experience that the voids within a crushed rock material are sufficiently large to allow fines to migrate into the voids, thereby creating the potential for sinkholes and depressions to develop at the ground surface. If open-graded gravel is utilized as pipe zone backfill, this material should be separated from the adjacent trench sidewalls and overlying trench backfill with a geosynthetic filter fabric.

The modulus of soil reaction ($E'$) is used to characterize the stiffness of soil backfill placed at the sides of buried flexible pipes for the purpose of evaluating deflection caused by the weight of the backfill over the pipe (Hartley and Duncan, 1987). A soil reaction modulus of 1,200 pounds per square inch (psi) may be used for an excavation depth of up to approximately 5 feet when backfilled with granular soil compacted to a relative compaction of 90 percent as evaluated by the ASTM D 1557. A soil reaction modulus of 1,500 psi may be used for trenches deeper than 5 feet.

9.1.11 Utility Trench Zone Backfill

Utility trench zone backfill should be free of organic material, clay lumps, debris, and meet the following recommendations. Trench backfill should not contain rocks or lumps over approximately 3 inches in diameter and not more than approximately 30 percent larger than ⅜ inch. Backfill materials should be moisture-conditioned to generally at or slightly above the laboratory optimum. Trench backfill should be compacted to a relative compaction of 90 percent as evaluated by ASTM D 1557 except for the upper 12 inches of the backfill beneath pavement areas which should be compacted to a relative compaction of 95 percent as evaluated by ASTM D 1557. Wet soils should be allowed to dry to moisture contents near the optimum prior to their placement as backfill. Lift thickness for backfill will
depend on the type of compaction equipment utilized, but fill should generally be placed in lifts not exceeding 8 inches in loose thickness. Special care should be exercised to avoid damaging the pipe during compaction of the backfill.

9.1.12  Thrust Blocks
Thrust restraint for buried pipelines may be achieved by transferring the thrust force to the soil outside the pipe through a thrust block. Thrust blocks may be designed using the magnitude and distribution of passive lateral earth pressures presented on Figure 8. Thrust blocks should be backfilled following the recommendations presented in this report.

9.1.13  Drainage
Roof, pad, and slope drainage should be directed 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 final 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 final 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.

9.2  Seismic Design Considerations
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 MCER spectral response acceleration parameters (USGS, 2018).
Table 3 – 2016 California Building Code Seismic Design Criteria

<table>
<thead>
<tr>
<th>Seismic Design Factors</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Site Class</td>
<td>D</td>
</tr>
<tr>
<td>Site Coefficient, Fa</td>
<td>1.075</td>
</tr>
<tr>
<td>Site Coefficient, Fv</td>
<td>1.602</td>
</tr>
<tr>
<td>Mapped Spectral Acceleration at 0.2-second Period, Ss</td>
<td>1.063 g</td>
</tr>
<tr>
<td>Mapped Spectral Acceleration at 1.0-second Period, S1</td>
<td>0.399 g</td>
</tr>
<tr>
<td>Spectral Acceleration at 0.2-second Period Adjusted for Site Class, SMS</td>
<td>1.143 g</td>
</tr>
<tr>
<td>Spectral Acceleration at 1.0-second Period Adjusted for Site Class, SM1</td>
<td>0.639 g</td>
</tr>
<tr>
<td>Design Spectral Response Acceleration at 0.2-second Period, SDS</td>
<td>0.762 g</td>
</tr>
<tr>
<td>Design Spectral Response Acceleration at 1.0-second Period, SD1</td>
<td>0.426 g</td>
</tr>
</tbody>
</table>

9.3 Foundations – Structural Buildings

As described earlier, the site is underlain by alluvial soils that are susceptible to liquefaction during a seismic event. Our analyses indicate that the site may undergo approximately 2 to 6½ inches of seismically-induced settlement during the design seismic event. Per the California Geological Survey Special Publication 117A (2008), large scale displacements include seismically-induced settlements of 4 inches or more. Additionally, for larger scale displacement CGS 117A suggests that ground improvement methods be implemented to mitigate the effects of liquefaction. However, due to the nature of the project, we understand that proposed campground improvements may be categorized to be similar to ancillary structures. Accordingly, the intent of the recommendations below are to mitigate the effects of liquefaction such that the structural buildings are not susceptible to collapse, however they may be susceptible to damage during the design earthquake and would potentially be in need of repair.

Accordingly, we are providing recommendations to support the structural buildings on mat foundations underlain by reinforced fill soils. Design of foundations should also be designed in accordance with structural considerations. In addition, requirements of the governing jurisdictions, practices of the Structural Engineers Association of California, and applicable building codes should be considered in the design of structures. In the event the structural are categorized such that they are to withstand the liquefaction effects during the design seismic event, additional recommendation can be provided for the use of deep foundation systems or the implementation of ground improvement techniques.
9.3.1 Mat Foundations

As noted above, mat foundations are anticipated to be used for the support of structural buildings. To provide consistent bearing conditions for the mat foundations, we recommend that they bear on the reinforced fill mat as described in Section 9.1.6 of this report and that no utilities, piping, or duct banks be constructed within 18 inches of the zone of influence of the bottom of each mat foundation. The zone of influence is defined by a 1:1 (horizontal to vertical) downward projection that extends outward from the bottom outside edge of the mat.

For the design of mats bearing on the reinforced fill mat during above groundwater conditions, a net allowable bearing pressure of 2,500 pounds per square foot (psf) may be used. For design of the mats during submerged water conditions, such as flooding, a net allowable bearing pressure of 1,500 psf may be used. These allowable bearing capacities may be increased by one-third when considering loads of a short duration such as wind or seismic forces. We recommend that mats be designed constructed with an embedment of 18 inches or more. Thickness and reinforcement of the mat foundation should be in accordance with the recommendations of the project structural engineer.

Mat foundations typically experience some deflection due to loads placed on the mat and the reaction of the soils directly underlying the mat. A design modulus of subgrade reaction (K) of 200 pounds per cubic inch (pci) should be used for the reinforced fill mat above groundwater and 90 pci for submerged conditions when evaluating such deflections. This value is based on a unit square foot area and should be adjusted for large mats. Adjusted values of the modulus of subgrade reaction, Kv, can be obtained from the following equation for mats of various widths:

\[ K_v = K [(B + 1)/2B]^2 \text{ (pci)} \]

B in the above equation represents the width (i.e., the lesser dimension of the width and length) of the mat in feet.

For frictional resistance to lateral loads on mat, we recommend a coefficient of friction of 0.35 at the concrete-soil interface. For a mat with an embedment depth shallower than 18 inches, passive earth pressure should be ignored while evaluating lateral resistance; only frictional resistance should be considered. For mats with embedment depths greater than 18 inches, passive earth pressure may be combined with frictional resistance to evaluate the total lateral resistance. The passive resistance values may be increased by one-third when considering loads of short duration such as wind or seismic forces.
9.3.2 Lateral Earth Pressures

Lateral bearing pressures (i.e., passive resistance) equal to an equivalent fluid weight of 350 pounds per cubic foot (pcf) may be used provided the footings are placed neat against compacted fill for above groundwater conditions and a value of 150 pcf may be used for submerged conditions. Footings may also be designed using a coefficient of friction between soil and concrete of 0.35. The allowable lateral resistance can be taken as the sum of the frictional resistance and passive resistance. The passive resistance may be increased by one-third when considering loads of short duration such as wind or seismic forces.

9.3.3 Static Settlement

We estimate that the proposed structures, designed and constructed as recommended herein, will undergo total static settlement of less than approximately 1 inch and differential static settlement of approximately 1/2–inch over a horizontal distance of 40 feet. Note, this does not include the seismically-induced settlements resulting from liquefaction during the design seismic event.

9.4 Preliminary Gravel Road Design

As part of the new construction, we anticipate that new gravel vehicular roads and parking areas will be constructed. Our laboratory testing of a near surface soil sample at the project site indicated an R-value of 57. In accordance with Caltrans Highway Design Manual (2017), our preliminary gravel road has utilized a design R-Value of 50. This R-value, along with assumed design Traffic Indices (TI) of 4.5, 5, 6, and 7 has been the basis of our preliminary road design. These assumed TIs should be evaluated by the Civil Engineer based on anticipated traffic loading at the site. Actual gravel road recommendations should be based on R-value tests performed on bulk samples of the soils that are exposed at the finished subgrade elevations across the site at the completion of the grading operations. The preliminary recommended flexible pavement sections are presented in Table 4.

<table>
<thead>
<tr>
<th>Traffic Index (Pavement Usage)</th>
<th>Design R-Value</th>
<th>Aggregate Base Thickness (inches)</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.5 (Parking Stalls)</td>
<td>50</td>
<td>9</td>
</tr>
<tr>
<td>5 (Drive Aisles)</td>
<td>50</td>
<td>10</td>
</tr>
<tr>
<td>6 (Heavy Traffic Areas)</td>
<td>50</td>
<td>12</td>
</tr>
<tr>
<td>7 (Fire Access)</td>
<td>50</td>
<td>14</td>
</tr>
</tbody>
</table>
As indicated, these values assume TIs of 7.0 or less for site roads. If traffic loads are different from those assumed, the pavement design should be re-evaluated. 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. Gravel access roads will require periodic maintenance.

### 9.5 Rigid Concrete Pavements

We understand that rigid concrete pavements may be used for the American with Disabilities Act (ADA) parking stalls and their associated walkways. We recommend that these ADA parking stalls and walkways be 6 inches in thickness and should be reinforced with No. 3 reinforcing bars placed at 18 inches on-center both ways. To reduce the potential manifestation of distress to rigid concrete pavements due to movement of the underlying soil, we recommend that such flatwork be installed with crack-control joints at appropriate spacing as designed by the structural engineer. Before placement of concrete, the subgrade soils should be scarified to a depth of 6 inches, moisture conditioned to generally above the laboratory optimum moisture content, and compacted to a relative compaction of 95 percent as evaluated by ASTM D 1557. Positive drainage should be established and maintained adjacent to flatwork.

We also suggest that consideration be given to using Portland cement concrete pavements in areas where dumpsters will be stored and where refuse and delivery trucks will stop and load. Experience indicates that refuse and other heavy truck traffic can significantly shorten the useful life of asphalt concrete or gravel road sections. We recommend that in these areas, a pavement section consisting of a 7-inch thickness of Portland cement concrete underlain by 4 inches of compacted aggregate base be placed. We recommend that the Portland cement concrete have a 600 pounds per square inch (psi) flexural strength and that it be reinforced with No. 4 bars that are placed 18 inches on center (both ways). The rigid pavement and aggregate base should be placed on compacted subgrade that is at a relative compaction of 95 percent as evaluated by ASTM D 1557.

### 9.6 Corrosion

Laboratory testing was performed on a select representative sample of the on-site earth materials to evaluate pH and electrical resistivity, as well as chloride and sulfate contents. The pH and electrical resistivity tests were performed in accordance with California Test (CT) 643 and the sulfate and chloride content tests were performed in accordance with CT 417 and CT 422, respectively. These laboratory test results are presented in Appendix B.
The results of the corrosivity testing indicated electrical resistivity of 250 ohm-cm, soil pH of 8.8, chloride content of 1,200 parts per million (ppm), and sulfate content of 0.093 percent (i.e., 930 ppm). Based on a comparison with the Caltrans corrosion criteria (2018), the on-site soils would be classified as corrosive. Corrosive soils are defined as soil with electrical resistivity less than 1,100 ohm-cm, a chloride content more than 500 ppm, more than 0.15 percent sulfates (1,500 ppm), and/or a pH less than 5.5.

9.7 Concrete
Concrete in contact with soil or water that contains high concentrations of water-soluble sulfates that can be subject to premature chemical and/or physical deterioration. As noted, the soil sample tested in this evaluation indicated a water-soluble sulfate content of 0.093 percent by weight (i.e., about 930 ppm). Based on the ACI 318 criteria, the potential for sulfate attack is negligible for water-soluble sulfate contents in soils ranging from about 0.00 to 0.10 percent by weight. Therefore, the site soils may be considered to have a negligible potential for sulfate attack. However, due to the potential variability of site soils, consideration should be given to using Type II/V cement for normal weight concrete in contact with soil.

9.8 Storm Water BMPs
We understand that the project will include permanent storm water BMPs. As described earlier, results of in-situ testing of the underlying materials indicate infiltration rates greater than 0.5 inches per hour. As previously noted, the groundwater is present at the site at depths ranging from approximately 5½ feet to 9 feet from the ground surface. Accordingly, the bottoms of the proposed infiltration BMPs are anticipated to be less than 10 feet from groundwater. However, based on our review of the California State Water Resources Control Board Basin Plan for San Diego, beneficial uses do not apply to the portion of the groundwater basin underlying the project site (i.e., west of Hollister Street). Based on our review of Appendix C of the County of San Diego BMP Design Manual (2016), infiltration BMPs may be designed and constructed at the site provided that the groundwater quality is maintained. The appropriate governing agencies should be consulted and coordinated with during design of storm water BMPs.

As presented earlier, site infiltration testing indicated infiltration rates of 2.62 to 10.42 in/hr, based on a factor of safety of 1.0. The design should include an appropriate factor of safety in accordance with the County of San Diego BMP Design Manual (2016). Due to the variability and unknown conditions of the existing fill materials, infiltration BMPs should be laterally set back approximately 20 feet from proposed structures. Consideration should be given to including appropriate overflow controls.
10 PLAN REVIEW AND CONSTRUCTION OBSERVATION

The conclusions and recommendations presented in this report are based on analysis of observed conditions in widely spaced exploratory borings. If conditions are found to vary from those described in this report, Ninyo & Moore should be notified, and additional recommendations will be provided upon request. Ninyo & Moore should review the final project drawings and specifications prior to the commencement of construction. Ninyo & Moore should perform the needed observation and testing services during construction operations.

The recommendations provided in this report are based on the assumption that Ninyo & Moore will provide geotechnical observation and testing services during construction. In the event that it is decided not to utilize the services of Ninyo & Moore during construction, we request that the selected consultant provide the client and Ninyo & Moore with a letter indicating that they fully understand Ninyo & Moore’s recommendations, and that they are in full agreement with the design parameters and recommendations contained in this report. Construction of proposed improvements should be performed by qualified subcontractors utilizing appropriate techniques and construction materials.

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
NOTE: DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE.


FIGURE 3

GEOLOGY

TIJUANA RIVER VALLEY REGIONAL PARK CAMPGROUNDS
SAN DIEGO, CALIFORNIA

108572001 | 6/18
NOTE: DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE.
FIGURE 7
FLOOD HAZARDS
TIJUANA RIVER VALLEY REGIONAL PARK CAMPGROUNDS
SAN DIEGO, CALIFORNIA
108572001 | 6/18


LEGEND
SPECIAL FLOOD HAZARD AREAS
REGULATORY FLOODWAY
1% ANNUAL CHANCE FLOOD HAZARD
OTHER AREAS OF FLOOD HAZARD
0.2% ANNUAL CHANCE FLOOD HAZARD

NOTE: DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE.
NOTES:

1. GROUNDWATER BELOW BLOCK
   \[ P_p = 175 (D^2 - d^2) \text{ lb/ft} \]

2. GROUNDWATER ABOVE BLOCK
   \[ P_p = 1.5 \left( D - d \right) \left[ 124.8h + 58 \left( D + d \right) \right] \text{ lb/ft} \]

3. ASSUMES BACKFILL IS GRANULAR MATERIAL

4. ASSUMES THRUST BLOCK IS ADJACENT TO COMPETENT MATERIAL

5. D, d AND h ARE IN FEET

6. GROUNDWATER TABLE
NOTES:

1. APPARENT LATERAL EARTH PRESSURES, $P_{o1}$ AND $P_{o2}$
   
   $P_{o1} = 38 \ h_1 \text{ psf}$
   $P_{o2} = 38 \ h_1 + 18 \ h_2 \text{ psf}$

2. WATER PRESSURE, $P_w$
   $P_w = 62.4 \ h_2 \text{ psf}$

3. DYNAMIC LATERAL EARTH PRESSURE, $P_E$, IS BASED
   ON A PEAK GROUND ACCELERATION, OF 0.48 g
   $P_E = 19 \ H \text{ psf}$

4. $P_E$ IS CALCULATED IN ACCORDANCE WITH THE
   RECOMMENDATIONS OF MONONOBE AND MATSUO
   (1929), AND ATIK AND SITAR (2010).

5. UPLIFT PRESSURE, $P_u$
   $P_u = 62.4 \ h_2 \text{ psf}$

6. SURCHARGE PRESSURES CAUSED BY VEHICLES
   OR NEARBY STRUCTURES ARE NOT INCLUDED

7. H, $h_1$, AND $h_2$ ARE IN FEET

8. ▽ GROUNDWATER TABLE
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 Modified Split-Barrel Drive Sampler. The sampler, with an external diameter of 3.0 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 140-pound 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.
Bulk sample.

Modified split-barrel drive sampler.

2-inch inner diameter split-barrel drive sampler.

No recovery with modified split-barrel drive sampler, or 2-inch inner diameter split-barrel drive sampler.

Sample retained by others.

Standard Penetration Test (SPT).

No recovery with a SPT.

Shelby tube sample. Distance pushed in inches/length of sample recovered in inches.

No recovery with Shelby tube sampler.

Continuous Push Sample.

Seepage.
Groundwater encountered during drilling.
Groundwater measured after drilling.

MAJOR MATERIAL TYPE (SOIL):
Solid line denotes unit change.

Dashed line denotes material change.

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

The total depth line is a solid line that is drawn at the bottom of the boring.
SOIL CLASSIFICATION CHART PER ASTM D 2488

<table>
<thead>
<tr>
<th>PRIMARY DIVISIONS</th>
<th>SECONDARY DIVISIONS</th>
<th>GROUP SYMBOL</th>
<th>GROUP NAME</th>
</tr>
</thead>
<tbody>
<tr>
<td>GRAVEL more than 50% of coarse fraction retained on No. 4 sieve</td>
<td>CLEAN GRAVEL less than 5% fines</td>
<td>GW</td>
<td>well-graded GRAVEL</td>
</tr>
<tr>
<td></td>
<td>GRAVEL with DUAL CLASSIFICATIONS 5% to 12% fines</td>
<td>GW-GM</td>
<td>well-graded GRAVEL with silt</td>
</tr>
<tr>
<td></td>
<td>GRAVEL with FINES more than 12% fines</td>
<td>GM</td>
<td>silty GRAVEL</td>
</tr>
<tr>
<td></td>
<td>CLEAN SAND less than 5% fines</td>
<td>SW</td>
<td>well-graded SAND</td>
</tr>
<tr>
<td></td>
<td>SAND with DUAL CLASSIFICATIONS 5% to 12% fines</td>
<td>SW-SM</td>
<td>well-graded SAND with silt</td>
</tr>
<tr>
<td></td>
<td>SAND with FINES more than 12% fines</td>
<td>SM</td>
<td>silty SAND</td>
</tr>
<tr>
<td></td>
<td>Silt and CLAY liquid limit less than 50%</td>
<td>CL</td>
<td>lean CLAY</td>
</tr>
<tr>
<td></td>
<td>ORGANIC</td>
<td>ML</td>
<td>SILT</td>
</tr>
<tr>
<td></td>
<td>ORGANIC</td>
<td>CL-ML</td>
<td>silty CLAY</td>
</tr>
<tr>
<td></td>
<td>ORGANIC</td>
<td>OL (PI &gt; 4)</td>
<td>organic CLAY</td>
</tr>
<tr>
<td></td>
<td>ORGANIC</td>
<td>OL (PI &lt; 4)</td>
<td>organic SILT</td>
</tr>
<tr>
<td></td>
<td>Highly Organic Soils</td>
<td>OH (plots on or above &quot;A&quot;-line)</td>
<td>organic CLAY</td>
</tr>
<tr>
<td></td>
<td>ORGANIC</td>
<td>OH (plots below &quot;A&quot;-line)</td>
<td>organic SILT</td>
</tr>
<tr>
<td></td>
<td>INORGANIC</td>
<td>CH</td>
<td>fat CLAY</td>
</tr>
<tr>
<td></td>
<td>INORGANIC</td>
<td>MH</td>
<td>elastic SILT</td>
</tr>
<tr>
<td></td>
<td>FINE-GRAINED SOILS 50% or more passes No. 200 sieve</td>
<td>Fines</td>
<td>Passing #200</td>
</tr>
</tbody>
</table>

GRAIN SIZE

<table>
<thead>
<tr>
<th>DESCRIPTION</th>
<th>SIEVE SIZE</th>
<th>GRAIN SIZE</th>
<th>APPROXIMATE SIZE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Boulders</td>
<td>&gt; 12&quot;</td>
<td>&gt; 12&quot;</td>
<td>Larger than basketball-sized</td>
</tr>
<tr>
<td>Cobble</td>
<td>3 - 12&quot;</td>
<td>3 - 12&quot;</td>
<td>Fist-sized to basketball-sized</td>
</tr>
<tr>
<td>Gravel</td>
<td>Coarse: 3/4 - 3&quot;</td>
<td>3/4 - 3&quot;</td>
<td>Thumb-sized to fist-sized</td>
</tr>
<tr>
<td></td>
<td>Fine: #4 - 3/4&quot;</td>
<td>0.19 - 0.75&quot;</td>
<td>Pea-sized to thumb-sized</td>
</tr>
<tr>
<td>Sand</td>
<td>Medium: #40 - #10</td>
<td>0.017 - 0.079&quot;</td>
<td>Rock-salt-sized to pea-sized</td>
</tr>
<tr>
<td></td>
<td>Fine: #200 - #40</td>
<td>0.0029 - 0.017&quot;</td>
<td>Flour-sized to rock-salt-sized</td>
</tr>
<tr>
<td></td>
<td>Fines: Passing #200</td>
<td>&lt; 0.0029&quot;</td>
<td>Flour-sized and smaller</td>
</tr>
</tbody>
</table>

CONSISTENCY - FINE-GRAINED SOIL

<table>
<thead>
<tr>
<th>APPARENT DENSITY - COARSE-GRAINED SOIL</th>
<th>SPOOLING CABLE OR CATHEAD</th>
<th>AUTOMATIC TRIP HAMMER</th>
</tr>
</thead>
<tbody>
<tr>
<td>APPARENT DENSITY</td>
<td>SPT (blows/foot)</td>
<td>MODIFIED SPLIT BARREL (blows/foot)</td>
</tr>
<tr>
<td>Very Loose</td>
<td>≤ 4</td>
<td>≤ 8</td>
</tr>
<tr>
<td>Loose</td>
<td>5 - 10</td>
<td>9 - 21</td>
</tr>
<tr>
<td>Medium Dense</td>
<td>11 - 30</td>
<td>22 - 63</td>
</tr>
<tr>
<td>Dense</td>
<td>31 - 50</td>
<td>64 - 105</td>
</tr>
<tr>
<td>Very Dense</td>
<td>&gt; 50</td>
<td>&gt; 105</td>
</tr>
</tbody>
</table>

CONSISTENCY - FINE-GRAINED SOIL

<table>
<thead>
<tr>
<th>CONSISTENCY</th>
<th>SPOONING CABLE OR CATHEAD</th>
<th>AUTOMATIC TRIP HAMMER</th>
</tr>
</thead>
<tbody>
<tr>
<td>CONSISTENCY</td>
<td>SPT (blows/foot)</td>
<td>MODIFIED SPLIT BARREL (blows/foot)</td>
</tr>
<tr>
<td>Very Soft</td>
<td>&lt; 2</td>
<td>&lt; 3</td>
</tr>
<tr>
<td>Soft</td>
<td>2 - 4</td>
<td>3 - 5</td>
</tr>
<tr>
<td>Firm</td>
<td>5 - 8</td>
<td>6 - 10</td>
</tr>
<tr>
<td>Stiff</td>
<td>9 - 15</td>
<td>11 - 20</td>
</tr>
<tr>
<td>Very Stiff</td>
<td>16 - 30</td>
<td>21 - 39</td>
</tr>
<tr>
<td>Hard</td>
<td>&gt; 30</td>
<td>&gt; 39</td>
</tr>
</tbody>
</table>

USCS METHOD OF SOIL CLASSIFICATION

Explanation of USCS Method of Soil Classification

- Project No.
- Date
- Figure
**SM**

ALLUVIUM: Brown, moist, loose, silty SAND.

Grayish brown; wet; scattered layers of poorly graded SAND; scattered gravel.

**ML**

Gray, wet, medium dense, sandy SILT.

**CL**

Gray, wet, stiff, silty CLAY; scattered sand.
ALLUVIUM: (Continued)

Gray, wet, stiff, sandy CLAY.
Light brown, wet, dense, silty SAND; micaceous.

SP-SM

Gray, wet, medium dense, poorly graded SAND with silt; scattered gravel up to 1 inch; scattered clay; micaceous.

Dense.

CL

Gray, wet, very stiff, sandy CLAY; organic odor.
Total Depth = 61.5 feet.
Temporary piezometer installed with 30 feet of PVC screen.
Groundwater encountered at approximately 5.6 feet from ground surface during drilling on 5/22/18 at 3:00 pm.
Groundwater encountered at approximately 5.7 feet from ground surface on 5/23/18 at 10:35 am.
Backfilled with approximately 20 cubic feet of bentonite grout on 5/23/18.

Note: Groundwater may rise to a level higher than that measured in borehole 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.
ALLUVIUM:
Brown, moist, medium dense, clayey SAND; micaceous.

Loose.

Brown, wet, medium dense, poorly graded SAND with clay; micaceous.

Grayish brown, wet, hard, sandy CLAY; scattered gravel.

Grayish brown, wet, medium dense, clayey SAND.

Total Depth = 21.5 feet.
Groundwater encountered at approximately 8 feet from ground surface during drilling. Backfilled with approximately 6 cubic feet of bentonite grout shortly after drilling on 5/23/18.

Note: Groundwater may rise to a level higher than that measured in borehole 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.
ALLUVIUM:

- Light brown, moist, loose, poorly graded SAND with silt.
- Brown, moist, loose, silty SAND; scattered clay layers; micaceous.
- Wet; medium dense.
- Brown, wet, dense, poorly graded SAND with silt.
- Very dense.
- Gray, wet, dense, silty SAND; micaceous.
ALLUVIUM: (Continued)
Gray, wet, dense, silty SAND; micaceous.

Very dense; scattered gravel up to 1/4 inch.

Gray, wet, very dense; poorly graded SAND with silt; scattered gravel.

Gray, wet, very dense, poorly graded SAND; scattered gravel.

Gray, wet, dense, poorly graded SAND with silt; scattered gravel; trace cobbles.

Total Depth = 61.5 feet.
Groundwater encountered at approximately 9 feet from ground surface during drilling. Backfilled with approximately 20 cubic feet of bentonite grout shortly after drilling on 5/22/18.

Note: Groundwater may rise to a level higher than that measured in borehole 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.
<table>
<thead>
<tr>
<th>DEPTH (feet)</th>
<th>SAMPLES</th>
<th>BLOWS/FOOT</th>
<th>MOISTURE (%)</th>
<th>DRY DENSITY (PCF)</th>
<th>SYMBOL</th>
<th>CLASSIFICATION</th>
<th>U.S.C.S.</th>
<th>DESCRIPTION/INTERPRETATION</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>SM</td>
<td></td>
<td>23.2</td>
<td>99.7</td>
<td>ALLUVIUM: Brown, moist, loose, silty SAND.</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>6</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Wet; micaceous.</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>SP</td>
<td></td>
<td>19</td>
<td></td>
<td>Grayish brown, wet, medium dense, poorly graded SAND; trace silt.</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>11</td>
<td>SC</td>
<td></td>
<td></td>
<td></td>
<td>Grayish brown, wet, medium dense, clayey SAND.</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>20</td>
<td>SM</td>
<td></td>
<td>22</td>
<td></td>
<td>Grayish brown, wet, dense, silty SAND.</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Total Depth = 21.5 feet.
Groundwater encountered at approximately 5.5 feet from ground surface during drilling. Backfilled with approximately 6 cubic feet of bentonite grout shortly after drilling on 5/23/18.

Note: Groundwater may rise to a level higher than that measured in borehole 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.
SM  ALLUVIUM:
Brown, moist, medium dense, silty SAND; micaceous.

Total Depth = 3 feet.
Groundwater not encountered during drilling.
Backfilled after infiltration testing on 5/23/18.

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.
ALLUVIUM:
Brown, moist, medium dense, silty SAND; micaceous.

Total Depth = 9.2 feet.
Groundwater encountered at approximately 5.5 feet from ground surface during drilling.
Backfilled with approximately 3 cubic feet of bentonite grout shortly after drilling on 5/23/18.

Note: Groundwater may rise to a level higher than that measured in borehole 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.
FILL:
Brown, moist, medium dense, silty SAND; scattered gravel.

ALLUVIUM:
Brown, moist, medium dense, silty SAND; micaceous.

Total Depth = 5 feet.
Groundwater not encountered during drilling.
Backfilled shortly after infiltration testing on 5/23/18.

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 intrepretations 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.
SM ALLUVIUM:
Brown, moist, medium dense, silty SAND; micaceous.

Total Depth = 3 feet.
Groundwater not encountered during drilling.
Backfilled shortly after infiltration testing on 5/23/18.

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.
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 and Density Tests**
The moisture content and dry density 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-8. These test results were utilized in evaluating the soil classifications in accordance with the USCS.

**Atterberg Limits**
Tests were performed on a selected representative fine-grained soil sample to evaluate the liquid limit, plastic limit, and plasticity index in general accordance with ASTM D 4318. These test results were utilized in evaluating the soil classifications in accordance with the USCS. The test results and classifications are shown on Figure B-9.

**Direct Shear Test**
A direct shear test was performed on a relatively undisturbed sample in general accordance with ASTM D 3080 to evaluate the shear strength characteristics of the selected material. The sample was inundated during shearing to represent adverse field conditions. The results are shown on Figure B-10.

**Expansion Index Tests**
The expansion index of selected materials was evaluated in general accordance with ASTM D 4829. The specimen was molded under a specified compactive energy at approximately 50 percent saturation. The prepared 1-inch thick by 4-inch diameter specimen was 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 test are presented on Figure B-11.

**Soil Corrosivity Tests**
Soil pH and resistivity tests were performed on a representative sample in general accordance with CT 643. The soluble sulfate and chloride contents of the selected sample were evaluated in general accordance with CT 417 and CT 422, respectively. The test results are presented on Figure B-12.

**R-Value**
The resistance value, or R-value, for site soils was evaluated in general accordance with CT 301. A sample was 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 shown on Figure B-13.
PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422
<table>
<thead>
<tr>
<th>Symbol</th>
<th>Sample Location</th>
<th>Depth (ft)</th>
<th>Liquid Limit</th>
<th>Plastic Limit</th>
<th>D_{10}</th>
<th>D_{30}</th>
<th>D_{60}</th>
<th>C_{u}</th>
<th>C_{c}</th>
<th>Passing No. 200 (percent)</th>
<th>USCS</th>
</tr>
</thead>
<tbody>
<tr>
<td>●</td>
<td>B-1</td>
<td>25.0-26.5</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>34</td>
<td>SM</td>
</tr>
</tbody>
</table>

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422

FIGURE B-2

GRADATION TEST RESULTS
TIJUANA RIVER VALLEY REGIONAL PARK CAMPGROUNDS
SAN DIEGO, CALIFORNIA
108572001 | 6/18
PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422

FIGURE B-3

GRADATION TEST RESULTS
TIJUANA RIVER VALLEY REGIONAL PARK CAMPGROUNDS
SAN DIEGO, CALIFORNIA

108572001 | 6/18
FIGURE B-4

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422
Coarse       Fine       Coarse       Medium       Fine       SILT       CLAY
3"  2"  ¾"  ½"  ⅜"  4"  8"  30"  50"  100"  200"

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422

U.S. STANDARD SIEVE NUMBERS

HYDROMETER

GRAIN SIZE IN MILLIMETERS

PERCENT FINER BY WEIGHT

GRAVEL SAND FINES

PERCENT FINER BY WEIGHT

GRAIN SIZE IN MILLIMETERS

Symbol | Sample Location | Depth (ft) | Liquid Limit | Plastic Limit | Plasticity Index | D_{10} | D_{30} | D_{60} | C_{u} | C_{c} | Passing No. 200 (percent) | USCS
---|---|---|---|---|---|---|---|---|---|---|---|---|---|
● | B2 | 0.5-4.0 | -- | -- | -- | -- | -- | -- | -- | -- | 41 | SC

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422
Coarse | Fine | Coarse | Medium | Fine | SILT | CLAY
---|---|---|---|---|---|---
3" | 2" | ¾" | ½" | ⅜" | 4 | 8 | 30 | 50

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422

U.S. STANDARD SIEVE NUMBERS

HYDROMETER

GRAIN SIZE IN MILLIMETERS

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Sample Location</th>
<th>Depth (ft)</th>
<th>Liquid Limit</th>
<th>Plastic Limit</th>
<th>Plasticity Index</th>
<th>$D_{10}$</th>
<th>$D_{30}$</th>
<th>$D_{60}$</th>
<th>$C_u$</th>
<th>$C_c$</th>
<th>Passing No. 200 (percent)</th>
<th>USCS</th>
</tr>
</thead>
<tbody>
<tr>
<td>•</td>
<td>B-3</td>
<td>20.0-21.5</td>
<td>--</td>
<td>--</td>
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<td>0.13</td>
<td>0.35</td>
<td>0.62</td>
<td>4.9</td>
<td>1.6</td>
<td>7</td>
<td>SP-SM</td>
</tr>
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</table>

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422

FIGURE B-6

GRADATION TEST RESULTS

TIJUANA RIVER VALLEY REGIONAL PARK CAMPGROUNDS
SAN DIEGO, CALIFORNIA

108572001 | 6/18
FIGURE B-7

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422
**GRAVITY**

<table>
<thead>
<tr>
<th>GRAVEL</th>
<th>SAND</th>
<th>FINES</th>
</tr>
</thead>
<tbody>
<tr>
<td>Coarse</td>
<td>Fine</td>
<td>Coarse</td>
</tr>
</tbody>
</table>

**U.S. STANDARD SIEVE NUMBERS**

1. **GRAIN SIZE IN MILLIMETERS**

   | Symbol | Sample Location | Depth (ft) | Liquid Limit | Plastic Limit | Plasticity Index | $D_{10}$ | $D_{30}$ | $D_{60}$ | $C_{u}$ | $C_{c}$ | Passing No. 200 (percent) | USCS |
|--------|---------------|------------|-------------|---------------|----------------|-------------|-----------|-----------|-----------|----------|----------|---------------------------|------|
| ●      | B-7           | 3.0-5.0    | --          | --            | --             | --          | --        | --        | --        | --       | --       | 19                        | SM   |

**PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422**

**FIGURE B-8**

**GRADATION TEST RESULTS**

TIJUANA RIVER VALLEY REGIONAL PARK CAMPGROUNDS
SAN DIEGO, CALIFORNIA

108572001 | 6/18
NP - INDICATES NON-PLASTIC

<table>
<thead>
<tr>
<th>SYMBOL</th>
<th>LOCATION</th>
<th>DEPTH (ft)</th>
<th>LIQUID LIMIT</th>
<th>PLASTIC LIMIT</th>
<th>PLASTICITY INDEX</th>
<th>USCS CLASSIFICATION</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>B-2</td>
<td>5.0-6.5</td>
<td>31</td>
<td>23</td>
<td>8</td>
<td>CL</td>
</tr>
</tbody>
</table>

USCS (Fraction Finer Than No. 40 Sieve)  

<table>
<thead>
<tr>
<th>SYMBOL</th>
<th>LOCATION</th>
<th>DEPTH (ft)</th>
<th>LIQUID LIMIT</th>
<th>PLASTIC LIMIT</th>
<th>PLASTICITY INDEX</th>
<th>USCS</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>B-2</td>
<td>5.0-6.5</td>
<td>31</td>
<td>23</td>
<td>8</td>
<td>CL</td>
</tr>
</tbody>
</table>

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4318

FIGURE B-9

ATTERBERG LIMITS TEST RESULTS
TIJUANA RIVER VALLEY REGIONAL PARK CAMPGROUNDS  
SAN DIEGO, CALIFORNIA
PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 3080

SILTY SAND

<table>
<thead>
<tr>
<th>Description</th>
<th>Symbol</th>
<th>Sample Location</th>
<th>Depth (ft)</th>
<th>Shear Strength</th>
<th>Cohesion (psf)</th>
<th>Friction Angle (degrees)</th>
<th>Soil Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>Silty SAND</td>
<td></td>
<td>B-4</td>
<td>5.0-6.5</td>
<td>Peak</td>
<td>0</td>
<td>34</td>
<td>SM</td>
</tr>
<tr>
<td>Silty SAND</td>
<td></td>
<td>B-4</td>
<td>5.0-6.5</td>
<td>Ultimate</td>
<td>0</td>
<td>33</td>
<td>SM</td>
</tr>
</tbody>
</table>

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 3080

FIGURE B-10
DIRECT SHEAR TEST RESULTS
TIJUANA RIVER VALLEY REGIONAL PARK CAMPGROUNDS
SAN DIEGO, CALIFORNIA

108572001 | 6/18
<table>
<thead>
<tr>
<th>SAMPLE LOCATION</th>
<th>SAMPLE DEPTH (ft)</th>
<th>INITIAL MOISTURE (percent)</th>
<th>COMPACTED DRY DENSITY (pcf)</th>
<th>FINAL MOISTURE (percent)</th>
<th>VOLUMETRIC SWELL (in)</th>
<th>EXPANSION INDEX</th>
<th>POTENTIAL EXPANSION</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-2</td>
<td>0.0-5.0</td>
<td>11.0</td>
<td>105.8</td>
<td>18.6</td>
<td>0.039</td>
<td>39</td>
<td>Low</td>
</tr>
</tbody>
</table>

PERFORMED IN GENERAL ACCORDANCE WITH

- UBC STANDARD 18-2
- ASTM D 4829

FIGURE B-11

EXPANSION INDEX TEST RESULTS
TIJUANA RIVER VALLEY REGIONAL PARK CAMPGROUNDS
SAN DIEGO, CALIFORNIA

108572001 | 6/18
<table>
<thead>
<tr>
<th>SAMPLE LOCATION</th>
<th>SAMPLE DEPTH (ft)</th>
<th>pH (^1)</th>
<th>RESISTIVITY (^1) (ohm-cm)</th>
<th>SULFATE CONTENT (^2) (ppm)</th>
<th>SULFATE CONTENT (^2) (%)</th>
<th>CHLORIDE CONTENT (^3) (ppm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-2</td>
<td>0.0-5.0</td>
<td>8.8</td>
<td>250</td>
<td>930</td>
<td>0.093</td>
<td>1200</td>
</tr>
</tbody>
</table>

\(^1\) 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
<table>
<thead>
<tr>
<th>SAMPLE LOCATION</th>
<th>SAMPLE DEPTH (ft)</th>
<th>SOIL TYPE</th>
<th>R-VALUE</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-5</td>
<td>1.0-3.0</td>
<td>Silty SAND (SM)</td>
<td>57</td>
</tr>
</tbody>
</table>

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2844/CT 301
APPENDIX C
Infiltration Test Data
### Infiltration Test No.: B-2

<table>
<thead>
<tr>
<th>Test Date: 5/22/2018</th>
<th>Infiltration Test No.: B-2</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Test Hole Diameter, D (inches):</strong> 6.0</td>
<td><strong>Excavation Depth (feet):</strong> 5.0</td>
</tr>
<tr>
<td>Test performed and recorded by: BTM</td>
<td><strong>Pipe Length (feet):</strong> 5.0</td>
</tr>
<tr>
<td>$t_1$</td>
<td>$d_1$ (feet)</td>
</tr>
<tr>
<td>13:35</td>
<td>4.50</td>
</tr>
<tr>
<td>13:48</td>
<td>4.50</td>
</tr>
<tr>
<td>14:01</td>
<td>4.50</td>
</tr>
<tr>
<td>14:18</td>
<td>4.50</td>
</tr>
<tr>
<td>14:30</td>
<td>4.50</td>
</tr>
<tr>
<td>14:43</td>
<td>4.50</td>
</tr>
</tbody>
</table>

### Infiltration Test No.: B-5

<table>
<thead>
<tr>
<th>Test Date: 5/23/2018</th>
<th>Infiltration Test No.: B-5</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Test Hole Diameter, D (inches):</strong> 6.0</td>
<td><strong>Excavation Depth (feet):</strong> 3.0</td>
</tr>
<tr>
<td>Test performed and recorded by: BTM</td>
<td><strong>Pipe Length (feet):</strong> 3.0</td>
</tr>
<tr>
<td>$t_1$</td>
<td>$d_1$ (feet)</td>
</tr>
<tr>
<td>14:24</td>
<td>1.90</td>
</tr>
<tr>
<td>14:32</td>
<td>1.90</td>
</tr>
<tr>
<td>14:40</td>
<td>1.90</td>
</tr>
<tr>
<td>14:47</td>
<td>1.90</td>
</tr>
<tr>
<td>14:54</td>
<td>1.90</td>
</tr>
<tr>
<td>15:01</td>
<td>1.90</td>
</tr>
</tbody>
</table>

**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 $t_2$
- $\Delta t$: change in time between initial and final water level readings
- $\Delta H$: change in depth to water or change in height of water column (i.e., $d_2 - d_1$)
- $H_{avg}$: initial height of water column
- $\text{in/hr}$: inches per hour

**Percolation Rate to Infiltration Rate Conversion**

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

- $I_t$: tested infiltration rate, inches/hour
- $\Delta H$: change in head over the time interval, inches
- $\Delta t$: time interval, minutes
- $r$: effective radius of test hole
- $H_{avg}$: average head over the time interval, inches

Appendix C: Geotechnical and Groundwater Investigation Requirements

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

<table>
<thead>
<tr>
<th>Categorization of Infiltration Feasibility Condition</th>
<th>Worksheet C.4-1</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Part 1 - Full Infiltration Feasibility Screening Criteria</strong></td>
<td></td>
</tr>
<tr>
<td>Would infiltration of the full design volume be feasible from a physical perspective without any undesirable consequences that cannot be reasonably mitigated?</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Criteria</th>
<th>Screening Question</th>
<th>Yes</th>
<th>No</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Is the estimated reliable infiltration rate below proposed facility locations greater than 0.5 inches per hour? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2 and Appendix D.</td>
<td>✓</td>
<td></td>
</tr>
</tbody>
</table>

Provide basis:

As noted in Section 6 of the Geotechnical Evaluation report for the project (Ninyo & Moore, 2018), in-situ infiltration rates at the site were measured between approximately 2.6 and 10.4 inches per hour. Please note that any infiltration system utilizing these results should apply the appropriate factor of safety to determine applicable site infiltration rates prior to design. The design safety factor shall be determined by the design engineer.

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

| 2        | Can infiltration greater than 0.5 inches per hour be allowed without increasing risk of geotechnical hazards (slope stability, groundwater mounding, utilities, or other factors) that cannot be mitigated to an acceptable level? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2. | ✓   |    |

Provide basis:

As noted in Section 8 of the Geotechnical Evaluation report for the project (Ninyo & Moore, 2018), laboratory testing of the subsurface soils indicated presence of soils with a low expansion potential. Additionally, as noted in Section 7, the site is mapped within an area with a high potential for liquefaction. Provided that the recommendations (i.e. setback, etc) are incorporated into the design and construction of proposed storm water BMPs, it is anticipated that infiltration will not increase the risk of geotechnical hazards above the level already present at the site.

Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.
### Appendix C: Geotechnical and Groundwater Investigation Requirements

#### Worksheet C.4-1 Page 2 of 4

<table>
<thead>
<tr>
<th>Criteria</th>
<th>Screening Question</th>
<th>Yes</th>
<th>No</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>Can infiltration greater than 0.5 inches per hour be allowed without increasing risk of groundwater contamination (shallow water table, storm water pollutants or other factors) that cannot be mitigated to an acceptable level? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.</td>
<td>✓</td>
<td></td>
</tr>
</tbody>
</table>

Provide basis:

Shallow groundwater is present at the site. However, as noted in Section 9.9 of the Geotechnical Report, beneficial uses do not apply in the portion of the groundwater basin that underlies the project area. Therefore, infiltration may be allowed without increasing the risk of groundwater contamination.

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

| 4        | Can infiltration greater than 0.5 inches per hour be allowed without causing potential water balance issues such as change of seasonality of ephemeral streams or increased discharge of contaminated groundwater to surface waters? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3. | ✓   |    |

Provide basis:

Based on the location of the project site approximately 1,800 feet south of the Tijuana River channel, infiltration is not anticipated to cause water balance issues at the site.

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

**Part 1 Result***

If all answers to rows 1 - 4 are “Yes” a full infiltration design is potentially feasible. The feasibility screening category is **Full Infiltration**

If any answer from row 1-4 is “No”, infiltration may be possible to some extent but would not generally be feasible or desirable to achieve a “full infiltration” design. Proceed to Part 2

---

*To be completed using gathered site information and best professional judgment considering the definition of MEP in the MS4 Permit. Additional testing and/or studies may be required by [City Engineer] to substantiate findings.*
### Worksheet C.4-1 Page 3 of 4

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

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

<table>
<thead>
<tr>
<th>Criteria</th>
<th>Screening Question</th>
<th>Yes</th>
<th>No</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>Do soil and geologic conditions allow for infiltration in any appreciable rate or volume? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2 and Appendix D.</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Provide basis:

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

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

Provide basis:

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

### Worksheet C.4-1 Page 4 of 4

<table>
<thead>
<tr>
<th>Criteria</th>
<th>Screening Question</th>
<th>Yes</th>
<th>No</th>
</tr>
</thead>
<tbody>
<tr>
<td>7</td>
<td><strong>Can Infiltration in any appreciable quantity be allowed without posing significant risk for groundwater related concerns (shallow water table, storm water pollutants or other factors)?</strong> The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Provide basis:</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability and why it was not feasible to mitigate low infiltration rates.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>8</td>
<td><strong>Can infiltration be allowed without violating downstream water rights?</strong> The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Provide basis:</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability and why it was not feasible to mitigate low infiltration rates.</td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Part 2</strong></td>
<td><strong>Result</strong>&lt;sup&gt;*&lt;/sup&gt; If all answers from row 5-8 are yes then partial infiltration design is potentially feasible. The feasibility screening category is <strong>Partial Infiltration</strong>. If any answer from row 5-8 is no, then infiltration of any volume is considered to be <strong>infeasible</strong> within the drainage area. The feasibility screening category is <strong>No Infiltration</strong>.</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

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

Liquefaction Analysis
LIQUEFACTION ANALYSIS
Tijuana River Valley Regional Park Campground

Hole No.=B-1  Water Depth=0 ft  Surface Elev.=15

Magnitude=6.7
Acceleration=0.477g

Shaded Zone has Liquefaction Potential

S = 6.58 in.

CivilTech Corporation
108572001
Plate A-1
LIQUEFACTION ANALYSIS
Tijuana River Valley Regional Park Campground

Hole No.=B-3   Water Depth=0 ft   Surface Elev.=15
Magnitude=6.7
Acceleration=0.477g

Shear Stress Ratio

Shaded Zone has Liquefaction Potential

CRR   CSR   fs1

Settlement
S = 2.20 in.

Factor of Safety

fs1=1

Saturated
Unsaturated.