Geotechnical Evaluation City Heights Pool 4380 Landis Street San Diego, California

Platt/Whitelaw Architects, Inc. 4034 30th Street | San Diego, California 92104

February 12, 2019 | Project No. 108716001



Geotechnical | Environmental | Construction Inspection & Testing | Forensic Engineering & Expert Witness Geophysics | Engineering Geology | Laboratory Testing | Industrial Hygiene | Occupational Safety | Air Quality | GIS





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1 INTRODUCTION

In accordance with your request, we have performed a geotechnical evaluation of the City Heights Recreation Center pool located at 4380 Landis Street in San Diego, California (Figure 1). Due to plumbing failures and cracks in the pool deck, we understand that the pool will be repaired and portions may be replaced. The purpose of our study was to evaluate the subsurface conditions below and adjacent to the concrete pool and deck as part of a scoping study. This report presents the results of our field explorations and laboratory testing as well as our conclusions regarding the geotechnical conditions at the pool and our recommendations to address those conditions.

2 SCOPE OF SERVICES

Our scope of services for this evaluation included the following:

- Reviewing readily available published and in-house geotechnical literature including previous geotechnical reports for the pool (Geocon, 2018a and 2018b), topographic maps, geologic and geologic hazard maps, fault maps, and stereoscopic aerial photographs.
- Attendance at a kick-off meeting to discuss the scope of the project with the client and City
 of San Diego personnel.
- Performing a field reconnaissance to observe site conditions and to mark the locations of the exploratory borings.
- Notifying Underground Service Alert (USA) to clear excavation locations for the potential presence of underground utilities. In addition, a private utility locating company was used to clear the locations for the potential presence of underground utilities.
- Performing a subsurface exploration program consisting of the drilling, logging, and sampling
 of three exploratory borings using a limited access drill rig. Due to site access constraints, a
 crane was used to place the drill rig on the pool deck. Relatively undisturbed drive and bulk
 soil samples of the materials encountered were collected at selected intervals from the
 borings and transported to our in-house geotechnical laboratory for testing.
- Performing geotechnical laboratory testing of representative soil samples to evaluate soil characteristics and geotechnical design parameters.
- Compiling and performing an engineering analysis of the information obtained from our background review, subsurface exploration, and laboratory testing.
- Preparing this geotechnical report presenting our findings, conclusions, and geotechnical recommendations for repair/replacement of the pool.

3 SITE DESCRIPTION

The site is located within the City Heights Recreation Center located at 4380 Landis Street in San Diego, California (Figure 1). The center was constructed in the late 1990's as part of the City Heights Urban Village project. The subject pool is situated between the recreation center building to the south and a park area to the north (Figure 2). Topographically, the site is relatively level with a surface elevation of approximately 365 feet above mean sea level (MSL). While much of the site vicinity is also relatively level, the City Heights area exhibits a very gentle gradient down to the south.

The subject swimming pool is approximately 85 feet long by 75 feet wide with an entry pool on the southwest side. Depths within the pool range from approximately 3¹/₂ feet in the shallow (southern) portion of the pool to approximately 10 feet in the deep (northern) portion. A gutter system that collects and recirculates overflow is located along the perimeter of the pool. An approximately 3-inch wide strip drain is located around the pool, approximately 6 feet from the pool edge. Other improvements along the pool deck include a separate tot pool on the southeast side of the pool, a slide on the west side of the pool, light standards, and fencing. A restroom building that serves the park area is located northwest of the pool.

4 SITE BACKGROUND

The City Heights Recreation Center was constructed in the late 1990's as part of the City Heights Urban Village project. The subject pool was constructed in 1998 (MCM, 1999). In March 2018, plumbing pipes beneath the pool slab were broken and water was observed coming out of cracks in the pool decking. The pool was subsequently drained and a trench was saw-cut down the middle of the pool shell and removed. The shell saw-cut, as well as a saw-cut in the deck on the south side of the pool, were performed to facilitate plumbing inspection. We understand that testing of the pipes beneath the pool indicated extensive leakage.

A limited geotechnical investigation was performed by Geocon in 2018 that included subsurface exploration (Geocon, 2018a), which consisted of three small-diameter borings, two of which were drilled within the drained pool. The results of their evaluation indicated the presence of highly expansive clay underlying the pool decking and the shallow portion of the pool. Geocon concluded that significant hydrostatic pressure had built up within the pipe backfill and caused distress. The findings of the Geocon report are incorporated herein, where appropriate.

Since draining of the pool, bulging of the pool wall was reported on the north side (deep end of the pool). Geocon concludes that the bulging is associated with the change in loading conditions on the pool wall following the removal of the water (Geocon, 2018b). In addition, according to City of San Diego staff, water accumulates at the bottom of the pool and is pumped out each week. This condition also occurred after draining the nearby tot pool. Water has also recently been observed seeping from the pool lights.

5 SUBSURFACE EXPLORATION

Our subsurface exploration was conducted on January 15, 2019 and included the drilling, logging, and sampling of three small-diameter borings (B-1 through B-3). The purpose of the borings was to further evaluate subsurface conditions and to collect soil samples for laboratory testing. Prior to commencing the subsurface exploration, the locations were cleared of underground utilities of Underground Service Alert. In addition, a private utility locator was retained to locate existing utilities in the area of our exploratory borings.

Prior to drilling, the concrete deck at each boring location was cored. The borings were drilled to depths up to approximately 19 feet using a limited-access, track-mounted drill rig equipped with 6-inch diameter, continuous-flight, hollow-stem augers. Due to site access constraints, a crane was used to lower the drill rig into the pool area. Drilling refusal was encountered in our borings B-1 and B-2 at depths of 13 feet and 11.9 feet, respectively. Ninyo & Moore personnel logged the borings in general accordance with the Unified Soil Classification System (USCS) and ASTM International (ASTM) Test Method D 2488 by observing cuttings and drive samples. Representative bulk and in-place soil samples were collected at selected depths from within the exploratory borings and transported to our in-house geotechnical laboratory for analysis. The approximate locations of the borings are presented on Figure 2. The boring logs are presented in Appendix A.

6 LABORATORY TESTING

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

7 GEOLOGY AND SUBSURFACE CONDITIONS

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

7.1 Regional Geologic Setting

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

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

7.2 Site Geology

Geologic units encountered during our field reconnaissance and subsurface exploration included fill and very old paralic deposits. Generalized descriptions of the earth units encountered during our subsurface exploration are provided below. The geology of the site vicinity is shown on Figure 4. Additional descriptions are provided on the boring logs in Appendix A. Geologic cross sections are shown on Figures 5A and 5B. Selected site photographs are presented in Appendix C. The geologic conditions encountered during Geocon's previous subsurface exploration at the site (2018a) are incorporated herein, where appropriate. Copies of their reports are presented in Appendix D.

7.2.1 Concrete Pool Deck

Our borings were performed within the reinforced concrete pool deck which is underlain by fill materials and very old paralic deposits. Table 1 below summarizes the thickness of the concrete pool deck as encountered in our exploratory borings.

Table 1 – Encountered Concrete Sections			
Boring	Encountered Concrete Thickness (inches)		
B-1	4		
B-2	6		
B-3	7		

7.2.2 Fill

Fill materials were encountered underlying the concrete deck pavements in each of our borings. The depth of the fill materials varied from approximately 4 feet to 7½ feet. As encountered, the fill materials generally consisted of various shades of brown, moist to wet, loose to medium dense, sandy silt and clayey and silty sand. Gravel and cobbles were encountered in the fill materials. Plumbing trench backfill that extended to depths of approximately 3 feet was also encountered in two of the borings previously performed by Geocon (2018a). Documentation regarding placement of fill materials was not available for review.

7.2.3 Very Old Paralic Deposits

Materials of the middle to early Pleistocene-aged very old paralic deposits, previously known as the Lindavista Formation (Kennedy, 1975), were encountered in our exploratory borings underlying the fill and extending to the total depths explored. As encountered, these materials generally consisted of brown, reddish brown, and gray to brownish gray, moist to wet, weakly to moderately cemented, silty and clayey sandstone, and sandy gravel conglomerate, along with weakly to moderately indurated, sandy claystone. Cobbles were also encountered in the very old paralic deposits. Drilling refusal within the very old paralic deposits occurred in two of our borings (B-1 and B-2). Similar materials were also encountered in the borings previously performed by Geocon (2018a).

The claystone encountered within the very old paralic deposits in borings B-2 and B-3 are characteristic of the Normal Heights Mudstone, a sub-unit that caps the very old paralic deposits in the City Heights, University Heights, Normal Heights, and North Park neighborhoods of San Diego (Reed, 1991). Regional geologic maps (Kennedy and Tan, 2008) currently consider the Normal Heights Mudstone to be a member of the very old paralic deposits. Based on our experience in the area, the Normal Heights Mudstone is

relatively impermeable and often possesses a high expansion index. The clay and sandy clay described by Geocon in their limited geotechnical investigation report (2018a) are also generally consistent with the Normal Heights Mudstone.

7.3 Groundwater

Groundwater was not encountered at the time of our subsurface exploration. However, seepage was encountered within the fill in borings B-2 and B-3. Seepage in boring B-2 was generally heavy and standing water was observed in the cored concrete pool deck prior to drilling of the boring. Reported extensive leaks in piping at the site likely contributed to the observed seepage. According to our review of readily available data from the Geotracker (2018) website, groundwater is anticipated at depths greater than 100 feet. Existing utility trench lines may act as conduits for perched water conditions and seepage should be anticipated. Fluctuations in the groundwater level and perched conditions may occur due to variations in ground surface topography, subsurface geologic conditions and structure, rainfall, irrigation, and other factors.

8 FAULTING AND SEISMICITY

Based on our review of the referenced geologic maps and stereoscopic aerial photographs, as well as on our geologic field mapping, the subject site is not underlain by known active or potentially active faults (i.e., faults that exhibit evidence of ground displacement in the last 11,000 years and 2,000,000 years, respectively). However, like the majority of southern California, the site is located in a seismically active area and the potential for strong ground motion is considered significant during the design life of the proposed structures. The nearest known active fault is the Rose Canyon fault, located approximately 4½ miles west of the site. Table 2 lists selected principal known active faults that may affect the subject site, including the approximate fault-to-site distances, and the maximum moment magnitudes (Mmax) as published by the USGS (2019).

Table 2 – Principal Active Faults			
Fault	Approximate Fault-to-Site Distance miles (kilometers)	Maximum Moment Magnitude (Mmax)	
Rose Canyon	4½ (7)	6.9	
Coronado Bank	16 (26)	7.4	
Newport-Inglewood (Offshore)	34 (55)	7.0	
Elsinore (Julian Segment)	38 (61)	7.4	
Elsinore (Temecula Segment)	41 (66)	7.1	
Earthquake Valley	42 (68)	6.8	
Elsinore (Coyote Mountain)	47 (78)	6.9	
San Jacinto (Coyote Creek)	59 (94)	7.0	

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

8.1 Surface Ground Rupture

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

8.2 Strong Ground Motion

The 2016 California Building Code (CBC) specifies that the Risk-Targeted, Maximum Considered Earthquake (MCER) ground motion response accelerations be used to evaluate seismic loads for design of buildings and other structures. The MCER 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 MCER for the segments was calculated as 0.440g using the Structural Engineers Association of California and California Office of Statewide Health Planning and Development (SEAOC/OSHPD, 2019) seismic design tool (web-based).

The 2016 CBC specifies that the potential for liquefaction and soil strength loss be evaluated, where applicable, for the Maximum Considered Earthquake Geometric Mean (MCE_G) peak ground acceleration with adjustment for site class effects in accordance with the American Society of Civil Engineers (ASCE) 7-10 Standard. The MCE_G peak ground acceleration is based on the geometric mean peak ground acceleration with adjustment for site class effects (PGA_M) was calculated as 0.451g using the USGS (SEAOC/OSHPD, 2019) seismic design tool that yielded a mapped MCE_G peak ground acceleration of 0.417g for the site and a site coefficient (F_{PGA}) of 1.083 for Site Class D.

8.3 Liquefaction

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

8.4 Geologic Hazard Map

Per the City of San Diego's Seismic Safety Study (2008), the site is located within an area designated as Category 52, which is described as "other level areas, gently sloping to steep terrain with favorable geologic structure, low risk." A portion of the Seismic Safety Study map that includes the site and vicinity is presented in Figure 6.

9 CONCLUSIONS

Based on our review of the referenced background data, the subsurface exploration, and geotechnical laboratory testing, it is our opinion the reconstruction of the pool shell is considered feasible from a geotechnical standpoint provided the recommendations presented in this report are incorporated into its design and construction. In general, the following conclusions were made:

- The project site is generally underlain by fill and very old paralic deposits. An expansive
 claystone layer within the very old paralic deposits is present beneath the pool decking and
 the shallow end of the pool. The sandstone and conglomerate members of the very old paralic
 deposits encountered are considered suitable for structural support. The existing fill is
 anticipated to be removed to address the underlying expansive materials. Recommendations
 for the remedial grading of the upper soils are presented in the following sections.
- Groundwater was not encountered during our subsurface exploration. However, seepage
 was encountered in the fill materials in borings B-2 and B-3. As noted in Section 4, City of
 San Diego staff notes that water regularly accumulates at the bottom of the drained pool and
 seepage has been observed in the pool lights. The seepage condition is the result of
 perched subsurface water migration along the top of the relatively impermeable claystone.
 Once the subsurface water reaches the soils' interface with pool shell, it can flow down and
 around the pool shell, resulting in seepage at the lights and at the pool bottom. We note that
 the potential source of the water could include the following:
 - Existing water pipes and drain lines associated with the pool that are leaking and introducing water into the subsurface materials.
 - Existing water, sewer, and/or irrigation lines associated with the nearby restroom building and park that are leaking and introducing water into the subsurface materials.
 - Accumulated precipitation, irrigation, and surface runoff in the vicinity of the site that has infiltrated into the subsurface materials.

- Gravel and cobbles were encountered in the fill and very old paralic deposits and drilling refusal within the very old paralic deposits occurred in two of our borings (B-1 and B-2).
- The on-site fill materials and very old paralic deposits are considered to be generally excavatable with heavy-duty earth moving equipment in good working condition. Gravel, cobbles, and strongly cemented soils may be encountered and additional efforts including heavy ripping within these materials should be anticipated.
- Soils derived from on-site excavations are anticipated to generate gravel, cobbles, and oversize pieces of cemented sandstone. On-site soils may be suitable for reuse as engineered fill, provided they are processed in accordance with the following recommendations. Additional processing and handling of materials including screening and crushing should be anticipated.
- The closest known active fault, the Rose Canyon fault, has been mapped approximately 4½ miles west of the site. No active faults are reported underlying the subject site. Therefore, potential for ground rupture due to faulting at the site is considered low.
- Results of our laboratory testing indicate that the upper soils at the site possess a very low to medium expansion potential. Expansion index testing of on-site soils previously performed by Geocon (2018a, Appendix D) indicated a medium to very high expansion index.
- Based on the results of our limited geotechnical laboratory testing presented in Appendix B, as compared to the Caltrans (2018) corrosion guidelines, and ACI 318, the on-site soils would be classified as corrosive.

10 RECOMMENDATIONS

The following recommendations are provided for the design and construction of the proposed pool reconstruction and repairs to its associated improvements. These recommendations are based on our evaluation of the site geotechnical conditions and our assumptions regarding the planned development. Ninyo & Moore should be contacted for questions regarding the recommendations or guidelines presented herein.

10.1 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.

10.1.1 Site Preparation

Site preparation should begin with the removal of existing improvements, vegetation, utility lines, asphalt, concrete, and other deleterious debris from the pool areas to be replaced and/or repaired. Clearing and grubbing should extend to the outside of the proposed excavation and fill areas. The debris and unsuitable material generated during clearing and

grubbing should be removed from areas to be graded and disposed of at a legal dumpsite away from the project area, unless noted otherwise in the following sections.

10.1.2 Excavation Characteristics

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

10.1.3 Remedial Grading

Prior to replacement of the pool shell, we recommend that the existing fill soils and the upper portions of the very old paralic deposits be removed down to the competent sandstone and conglomerate materials of the very old paralic deposits. Based on our geotechnical evaluation, these excavations should extend to a depth of 3 feet below the bottom of the new pool shell. In areas to receive new decking and/or general sidewalks or retaining walls that are not tied into or that do not support the swimming pool, the existing upper soils should be overexcavated a depth of 1 foot below the subgrade elevation, or 1 foot below the bottom of the retaining wall footing. These removals should extend a lateral distance of 3 feet or more beyond the pool shell or decking/flatwork. The extent and depths of removals and overexcavations should be evaluated by Ninyo & Moore's representative in the field to observe that the upper soils have been sufficiently removed. Based on our observations, deeper removals may be recommended.

The resulting surface should be scarified to a depth of approximately 8 inches, moisture conditioned, and recompacted to a relative compaction of 90 percent as evaluated by the ASTM D 1557 prior to placing new fill. Once the resulting removal surface has been recompacted, the overexcavation should be backfilled with generally granular soils that possess a very low to low expansion potential (i.e., an expansion index [EI] less than 50). These materials can consist of the soils derived from on-site excavations that have been adequately processed to meet the soils characteristics recommended in the "Materials for Fill" section of this report.

10.1.4 Temporary Excavations

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

Fill	Туре С
Very Old Paralic Deposits	Туре В

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

10.1.5 Materials For Fill

Soils derived from on-site excavations are anticipated to generate gravel, cobbles, and oversize pieces of cemented sandstone. On-site soils may be suitable for reuse as engineered fill, provided they are processed in accordance with the following recommendations. Additional processing and handling of materials including screening and crushing should be anticipated. Engineered fill soils should be comprised of granular soils with a very low to low expansion potential (i.e., an expansion index [EI] of 50 or less), and possess an organic content of less than approximately 3 percent by volume (or 1 percent by weight). In general, engineered fill material should not contain rocks or lumps over approximately 3 inches in diameter, and not more than approximately 30 percent larger than ³/₄ inch. Oversize materials should be separated from material to be used for fill and removed from the site. Materials comprising the Normal Heights Mudstone should not be utilized as engineered fill.

Imported fill material, if needed, should generally be granular soils with a very low to low expansion potential (i.e., an expansion index [EI] of 50 or less). Import material should also be non-corrosive in accordance with the Caltrans (2018) corrosion guidelines and American Concrete Institute (ACI) 318. Materials for use as fill should be evaluated by Ninyo & Moore's representative prior to filling or importing. To reduce the potential of importing contaminated materials to the site, prior to delivery, soil materials obtained from off-site

sources should be sampled and tested in accordance with standard practice (Department of Toxic Substances Control [DTSC], 2001). Soils that exhibit a known risk to human health, the environment, or both, should not be imported to the site.

10.1.6 Compacted Fill

Prior to placement of compacted fill, the contractor should request an evaluation of the exposed ground surface by Ninyo & Moore. Unless otherwise recommended, the exposed ground surface should then be scarified to a depth of approximately 8 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.

10.1.7 Pipe Bedding and Modulus of Soil Reaction (E')

We recommend 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 and pipe zone backfill should have a Sand Equivalent (SE) of 30 or greater, and be placed around the sides and top of the pipe. In addition, the pipe zone backfill should extend 1 foot or more above the top of the pipe.

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 excavation depths up to 5 feet and 1,800 psi may be used for excavation depths more than 5 feet, backfilled with granular soil and compacted to 90 percent of the modified Proctor density (based on ASTM D 1557).

10.1.8 Drainage

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

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

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

10.2 Seismic Design Parameters

Design of the proposed improvements should be performed in accordance with the requirements of governing jurisdictions and applicable building codes. Table 3 presents the seismic design parameters for the site in accordance with the CBC (2016) guidelines and adjusted MCE spectral response acceleration parameters (SEAOC/OSHPD, 2019).

Table 3 – 2016 California Building Code Seismic Design Criteria			
Seismic Design Factors	Values		
Site Class	D		
Site Coefficient, Fa	1.101		
Site Coefficient, Fv 1.640			
Mapped Spectral Acceleration at 0.2-second Period, Ss 0.998g			
Mapped Spectral Acceleration at 1.0-second Period, S10.380g			
Spectral Acceleration at 0.2-second Period Adjusted for Site Class, SMS 1.099g			
Spectral Acceleration at 1.0-second Period Adjusted for Site Class, SM1 0.623g			
Design Spectral Response Acceleration at 0.2-second Period, SDS 0.732g			
Design Spectral Response Acceleration at 1.0-second Period, SD1 0.416g			

10.3 Swimming Pool Recommendations

Detailed design plans for the existing pool were not available for our review. Based on our geotechnical evaluation, we recommend that the pool be reconstructed with a new shell. As such, we are providing the following recommendations.

10.3.1 Swimming Pool Bottom

To reduce the potential for differential settlement of the pool, Section 10.1.3 of this report recommends that the existing upper be removed and replaced with 3 feet or more fill of compacted fill. In addition, a 3 foot thickness of fill should be placed adjacent to the sides of the pool shell to provide consist geotechnical conditions around and below the shell. Sections 10.1.5 and 10.6.1 of this report provide recommendations for the types of materials to be utilized as fill and their placement/compaction in these areas.

10.3.2 Bearing Capacity

Structures bearing on compacted fill materials may be designed using an allowable bearing capacity of 3,000 pounds per square foot (psf). The allowable bearing capacity may be increased by one-third when considering loads of short duration such as wind or seismic

forces. The pool wall and floor should be reinforced in accordance with the recommendations of the project structural engineer.

10.3.3 Temporary Access Ramps

Backfill materials placed within temporary access ramps extending into the pool excavations should be properly compacted and tested. This will mitigate excessive settlement of the backfill and subsequent damage to pool decking or other structures placed on the backfill.

10.3.4 Pool Decking

To reduce the potential for differential movement between the edge of the pool and the adjacent pool decking, we recommend that the pool decking within 10 of the edge of the pool, be doweled into the sidewalls of the pool. From a geotechnical standpoint, we recommend that the pool decking be 5 inches or more in thickness thick. The dowel sizing and spacing should be evaluated by the project structural engineer.

For pool decking and general site sidewalks, to reduce the potential for shrinkage cracking, the pool decking should be 5 inches thick and should be reinforced with No. 3 reinforcing bars placed at 18 inches on-center (both ways). We recommend that crack control joints be provided at an interval of every 6 feet or less to help reduce the potential for distress due to soil movement and concrete shrinkage. As a further measure to reduce cracking of pool decking, the subgrade soils to a depth of approximately 12 inches below the pool decking and general sidewalks should be compacted to a relative compaction of 90 percent or more in accordance with the latest edition of ASTM D 1557 at moisture contents generally above the laboratory optimum. The subgrade soils should be shaped to provide a minimum gradient of one percent away from the pool shell and towards a subsurface drainage system.

10.3.5 Plumbing Fixtures

Leakage from the swimming pool or the appurtenant plumbing fixtures could create adverse saturated conditions of the surrounding subgrade soils. Areas of saturation can lead to differential settlement of the subgrade soils and subsequent shifting of pool decking. Therefore, it is recommended that the plumbing at the site (including that for the restroom building) and pool fixtures be monitored, tested, and maintained during the design life of the project. For similar reasons, drainage from the pool deck areas should be directed to area drains and/or swales designed to carry runoff water to suitable discharge locations.

10.3.6 Swimming Pool Retaining Walls

Retaining walls that are tied into or that support the swimming may be supported on a continuous footing bearing on compacted fill materials. Allowable bearing capacities of 3,500 psf may be used for the design of retaining wall foundations. The allowable bearing capacity may be increased by one-third when considering loads of short duration, such as wind or seismic forces.

For the design of a yielding retaining wall that is not restrained against movement by rigid corners or structural connections, lateral pressures are presented on Figure 7. Restrained walls (non-yielding) may be designed for lateral pressures presented on Figure 8. These pressures assume low-expansive backfill and free draining conditions. Measures should be taken to reduce the potential for build-up of moisture behind the retaining walls. A drain should be provided behind the retaining wall as shown on Figure 9. The drain should be connected to an appropriate outlet.

10.3.7 Hydrostatic Relief Valves

It has been our experience that accumulation of subsurface water adjacent to a pool shell can result in the build-up of unbalanced hydrostatic forces when then pool is empty. Geocon (2018a; 2018b) previously noted that hydrostatic pressures have caused noticeable distress at the pool, including the observed bulging of the pool wall in the deep end of the pool. To mitigate the potential for hydrostatic uplift pressures on the pool shell on those occasions when the pool is empty, we recommend the pool be equipped with an underdrain system and a hydrostatic relief valve.

10.3.8 Site Drainage

Surface drainage should be provided to direct water away from the pool and off concrete decking surfaces. Surface water should not be permitted to pond or drain toward the pool. Positive drainage is defined as a slope of 1 percent or more over a distance of 5 feet or greater away from the pool.

10.4 Soil Corrosivity

Laboratory testing was performed on representative samples of near-surface soil to evaluate soil pH, electrical resistivity, water-soluble chloride contents, and water-soluble sulfate contents. The soil pH and electrical resistivity tests were performed in general accordance with California

Test Method (CT) 643. The chloride content tests were performed in general accordance with CT 422. Sulfate testing was performed in general accordance with CT 417.

The results of the corrosivity testing indicated electrical resistivity of 500 and 1,000 ohm-centimeters (ohm-cm), soil pH of 8.8, chloride contents of 50 and 335 parts per million (ppm), and sulfate contents of 0.001 and 0.018 percent (i.e., 10 and 180 ppm). A comparison with the Caltrans corrosion (2018) criteria and ACI 318 indicates that the on-site soils would be classified as corrosive. Based on the Caltrans (2018) criteria, a project site is classified as corrosive if one or more of the following conditions exist for the representative soil samples retrieved from the site: chloride concentration of 500 ppm or greater, soluble sulfate concentration of 1,500 ppm or greater, an electrical resistivity of 1,100 ohm-centimeters or less, and a pH 5.5 or less.

10.5 Concrete

Concrete in contact with soil or water that contains high concentrations of water-soluble sulfates can be subject to premature chemical and/or physical deterioration. Soil samples tested during this evaluation indicated water-soluble sulfate contents of 0.001 and 0.018 percent (i.e., 10 and 180 ppm). Based on the ACI 318 criteria, the potential for sulfate attack is considered negligible for water-soluble sulfate contents in soil ranging from 0 to 0.10 percent by weight (0 to 1,000 ppm), indicating that soils underlying the site may be considered to have a negligible potential for sulfate attack. However, due to the potential for variability of on-site soils, we recommend that Type II/V or V cement be used for concrete in contact with soil.

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**

- American Concrete Institute (ACI), 2014, ACI 318 Building Code Requirements for Structural Concrete and Commentary.
- American Society of Civil Engineers (ASCE), 2010, Minimum Design Loads for Buildings and Other Structures, ASCE 7-10.
- Atik, Linda L., and Sitar, N., 2010, Seismic Earth Pressures on Cantilever Retaining Structures, ASCE Journal of Geotechnical and Geoenvironmental Engineering, Vol. 136, No. 10, October 1.
- BNi Building News, 2018, "Greenbook," Standard Specifications for Public Works Construction: BNI Publications.
- California Building Standards Commission (CBSC), 2016, California Building Code (CBC), Title 24, Part 2, Volumes 1 and 2.
- California Department of Transportation (Caltrans), 2018, Corrosion Guidelines (Version 3.0), Division of Engineering and Testing Services, Corrosion Technology Branch: dated March.
- California Geological Survey, 2008a, Guidelines for Evaluating and Mitigating Seismic Hazards in California, CGS Special Publication 117A.
- California Geological Survey, 2008b (revised), Earthquake Shaking Potential for California: Map Sheet 48.
- City of San Diego, 1952, Topographic Survey, Sheet 210-1737, Scale 1:2,400.
- City of San Diego, 1976, Orthotopographic Survey, Sheet 210-1737, Scale 1:2,400.
- City of San Diego, 2008, Seismic Safety Study, Grid 22, Scale 1:9,600.
- Department of Toxic Substances Control, 2001, Information Advisory Clean Import Fill Material, <u>http://www.dtsc.ca.gov/Schools/index.cfm</u>: dated October.
- Geocon, 2018a, Limited Geotechnical Investigation, City Heights Pool, Contract No. 156367, Task No. 15GG20, San Diego, California: dated July 11.
- Geocon, 2018b, Miscellaneous Geotechnical Consultation, City Heights Pool, Contract No. 156367, Task No. 15GG20, San Diego, California: dated August 7.

GeoTracker, 2018, <u>http://geotracker.waterboards.ca.gov/;</u> accessed in December.

Google Inc., 2019, https://www.google.com/earth/; accessed in January.

Harden, D.R., 2004, California Geology, Second Edition: Prentice Hall, Inc.

- Hart, E.W., and Bryant, W.A., 1997, Fault-Rupture Hazard Zones in California, Alquist-Priolo Earthquake Fault Zoning Act with Index to Earthquake Fault Zone Maps: California Geological Survey, Special Publication 42, with Supplements 1 and 2 added in 1999.
- Hartley, J.D., and Duncan, J.M., 1987, E' and Its Variation with Depth: American Society of Civil Engineers (ASCE), Journal of Transportation Engineering, Vol. 113, No. 5: dated September.
- Kennedy, M.P., 1975, Geology of the San Diego Metropolitan Area, California: California Geological Survey, Bulletin 200, Scale 1:24,000.

- Kennedy, M.P., and Tan, S.S., 2008, Geologic Map of the San Diego 30' x 60' Quadrangle, California: California Geological Survey, Regional Geologic Map No. 3, Scale 1:100,000.
- Martinez Cutri and McArdle (MCM) Architects, 1999, City Heights Urban Village, San Diego, California, As-Built Plan Set: dated July 20.
- Mononobe, N. and Matsuo, H., 1929, On the Determination of Earth Pressure during Earthquakes, Proceedings of the World Engineering Conference, No. 9.

Ninyo & Moore, in-house proprietary data.

Ninyo & Moore, 2018, Proposal for Geotechnical Evaluation Services, City Heights Pool, 4380 Landis Street, San Diego, California: Proposal No. 02-01476: dated September 26.

Norris, R.M. and Webb, R.W., 1990, Geology of California, Second Edition: John Wiley & Sons, Inc.

- Reed, L.D., 1991, A New Upper Pleistocene Marine Sedimentary Unit, San Diego, California, in Geotechnical Engineering Case Histories in San Diego County, San Diego Association of Geologists, p. 1-27.
- SEAOC/OSHPD, 2019, Seismic Design Maps, <u>http://seismicmaps.org</u>: accessed in February.
- United States Geological Survey, 2018, National City Quadrangle 7.5 minute, San Diego County, California.
- United States Geological Survey (USGS), 2019, 2008 National Seismic Hazard Maps Fault Parameters website, <u>https://earthquake.usgs.gov/cfusion/hazfaults_2008_search/query_main.cfm</u>.

FIGURES

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CITY HEIGHTS POOL 4380 LANDIS STREET, SAN DIEGO, CALIFORNIA





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GEOLOGIC CROSS SECTION A-A'

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NOTES:

- 1. ASSUMES NO HYDROSTATIC PRESSURE BUILD-UP BEHIND THE RETAINING WALL
- 2. STRUCTURAL, GRANULAR BACKFILL MATERIALS AS SPECIFIED IN SECTION 10.1.5 SHOULD BE USED FOR RETAINING WALL BACKFILL
- 3. DRAINS AS RECOMMENDED IN THE RETAINING WALL DRAINAGE DETAIL SHOULD BE INSTALLED BEHIND THE RETAINING WALL
- 4. DYNAMIC LATERAL EARTH PRESSURE IS BASED ON A PEAK GROUND ACCELERATION OF 0.45g
- 5. P_E IS CALCULATED IN ACCORDANCE WITH THE RECOMMENDATIONS OF MONONOBE AND MATSUO (1929), AND ATIK AND SITAR (2010).
- 6. SURCHARGE PRESSURES CAUSED BY VEHICLES OR NEARBY STRUCTURES ARE NOT INCLUDED
- 7. H AND D ARE IN FEET

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8. SETBACK SHOULD BE IN ACCORDANCE WITH FIGURE 1808.7.1 OF THE IBC (2015) RECOMMENDED GEOTECHNICAL DESIGN PARAMETERS

Lateral Earth Pressure	Equivalent Fluid Pressure (Ib/ft ² /ft) ⁽¹⁾	
P.	Level Backfill with Granular Soils ⁽²⁾	2H:1V Sloping Backfill with Granular Soils ⁽²⁾
• 4	35 H	55 H
P _E	18 H	
P.	Level Ground	2H:1V Descending Ground
• •	350 D	150 D

NOT TO SCALE

FIGURE 7

LATERAL EARTH PRESSURES FOR YIELDING RETAINING WALLS

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- 1. ASSUMES NO HYDROSTATIC PRESSURE BUILD-U BEHIND THE RETAINING WALL
- 2. STRUCTURE BACKFILL MATERIALS AS SPECIFIED IN SECTION 10.1.5 SHOULD BE USED FOR RETAINING WALL BACKFILL
- 3. DRAINS AS RECOMMENDED IN THE RETAINING WALL DRAINAGE DETAIL SHOULD BE INSTALLED BEHIND THE RETAINING WALL
- 4. DYNAMIC LATERAL EARTH PRESSURE IS BASED ON A PEAK GROUND ACCELERATION OF 0.45g
- 5. P_E IS CALCULATED IN ACCORDANCE WITH THE RECOMMENDATIONS OF MONONOBE AND MATSUO (1929), AND ATIK AND SITAR (2010).
- 6. SURCHARGE PRESSURES CAUSED BY VEHICLES OR NEARBY STRUCTURES ARE NOT INCLUDED
- 7. H AND D ARE IN FEET

RECOMMENDED GEOTECHNICAL DESIGN PARAMETERS

Lateral Earth Pressure	Equivalent Fluid Pressure (lb/ft ² /ft) ⁽¹⁾	
Po	Level Backfill with Granular Soils ⁽²⁾	2H:1V Sloping Backfill with Granular Soils ⁽²⁾
.0	55 H	79 H
P _E	18 H	
P _n	Level Ground	2H:1V Descending Ground
- P	350 D	150 D

NOT TO SCALE

FIGURE 8

LATERAL EARTH PRESSURES FOR RESTRAINED RETAINING WALLS

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APPENDIX A

Boring Logs

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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 1^{*}/₆ inches. The sampler was driven into the ground with a 140-pound hammer free-falling from a height of 30 inches in general accordance with ASTM D 1586. The blow counts were recorded for every 6 inches of penetration; the blow counts reported on the logs are those for the last 12 inches of penetration. Soil samples were observed and removed from the sampler, bagged, sealed and transported to the laboratory for testing.

Field Procedure for the Collection of Relatively Undisturbed Samples

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

The Modified Split-Barrel Drive Sampler

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

	Soil Clas	sification C	hart	Per AST	M D 2488	Grain Size					
	lana		Seco	ndary Divisions		Dosc	rintion	Sieve	Grain Sizo	Approximate	
F	rimary Divis	sions	Gro	up Symbol	Group Name		Desci	Πριιοπ	Size	Grain Size	Size
		CLEAN GRAVEL		GW	well-graded GRAVEL		Bou	Iders	> 12"	> 12"	Larger than
		less than 5% fine		GP	poorly graded GRAVEL						
	GRAVEL			GW-GM	well-graded GRAVEL with silt		Cot	bles	3 - 12"	3 - 12"	Fist-sized to basketball-sized
	more than	GRAVEL with DUAL		GP-GM	poorly graded GRAVEL with silt						
	coarse	CLASSIFICATIONS 5% to 12% fine		GW-GC	well-graded GRAVEL with clay			Coarse	3/4 - 3"	3/4 - 3"	Thumb-sized to fist-size
	retained on			GP-GC	poorly graded GRAVEL with		Gravel				Pop sized to
COARSE- GRAINED SOILS more than 50% retained on No. 200 sieve	NO. 4 SIEVE	GRAVEL with		GM	silty GRAVEL			Fine	#4 - 3/4"	0.19 - 0.75"	thumb-sized
		FINES more than	12	GC	clayey GRAVEL			Caaraa	#10 #1	0.070 0.10"	Rock-salt-sized to
		12% fine		GC-GM	silty, clayey GRAVEL			Coarse	#10 - #4	0.079 - 0.19	pea-sized
		CLEAN SAND		SW	well-graded SAND		Sand	Medium	#40 - #10	0.017 - 0.079"	Sugar-sized to
		less than 5% fine		SP	poorly graded SAND						rock-salt-sized
		SAND with DUAL CLASSIFICATIONS 5% to 12% fine		SW-SM	well-graded SAND with silt			Fine	#200 - #40	0.0029 -	Flour-sized to
	SAND 50% or more			SP-SM	poorly graded SAND with silt					0.017	Sugai-Sizeu
	of coarse fraction passes No. 4 sieve		[]]]	SW-SC	well-graded SAND with clay		Fii	nes	Passing #200	< 0.0029"	Flour-sized and smaller
				SP-SC	poorly graded SAND with clay						
		SAND with FINES		SM	silty SAND				Plastic	ity Chart	
				SC	clayey SAND						
		12 % 11110		SC-SM	silty, clayey SAND		70				
				CL	lean CLAY		% 60				
	SILT and	INORGANIC		ML	SILT		(Id) 50				
	CLAY liquid limit			CL-ML	silty CLAY		H 40			CH or C	рн
FINE-	less than 50%	OPCANIC		OL (PI > 4)	organic CLAY		∠ 30				
SOILS		OROANIC		OL (PI < 4)	organic SILT		101 20		CL o	r OL	MH or OH
50% or more passes				СН	fat CLAY		.SP 10				
No. 200 sieve	SILT and CLAY			MH	elastic SILT		۹ م ۹	CL-I	ML ML o	r OL	
	liquid limit 50% or more	ORGANIC		OH (plots on or above "A"-line)	organic CLAY		0	0 10	20 30 40	0 50 60 7	70 80 90 100
		UNGANIC		OH (plots below "A"-line)	organic SILT				LIQUI	D LIMIT (LL),	%
	Highly	Organic Soils		PT	Peat						

Apparent Density - Coarse-Grained Soil

	parone bo											
	Spooling C	able or Cathead	Automatic	Trip Hammer		Spooling Ca	ble or Cathead	Automatic Trip Hammer				
Density	SPT (blows/foot)	Modified Split Barrel (blows/foot)	SPT (blows/foot) Modified (blows/foot) (blows/foot)		Consis- tency	SPT (blows/foot)	Modified Split Barrel (blows/foot)	SPT (blows/foot)	Modified Split Barrel (blows/foot)			
Very Loose	≤ 4	≤ 8	≤3	≤ 5	Very Soft	< 2	< 3	< 1	< 2			
Loose	5 - 10	9 - 21	4 - 7 6 - 14		Soft	2 - 4	3 - 5	1 - 3	2 - 3			
Medium	11 - 30	22 - 63	8 - 20	15 - 42	Firm	5 - 8	6 - 10	4 - 5	4 - 6			
Dense		00			Stiff	9 - 15	11 - 20	6 - 10	7 - 13			
Dense	31 - 50	64 - 105	21 - 33	43 - 70	Very Stiff	16 - 30	21 - 39	11 - 20	14 - 26			
Very Dense	> 50 > 105 > 33 :		> 70	Hard	> 30	> 39	> 20	> 26				



USCS METHOD OF SOIL CLASSIFICATION

Consistency - Fine-Grained Soil

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



BORING LOG

DEPTH (feet)	Bulk SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED 1/15/19 BORING NO. B-1 GROUND ELEVATION 365' ± (MSL) SHEET 1 OF 1 METHOD OF DRILLING 6" Diameter Hollow Stem Auger (Pacific Drilling - Fraste) DRIVE WEIGHT 140 lbs. (Auto-Trip) DROP 30" SAMPLED BY NMM LOGGED BY NMM REVIEWED BY CAT
0						SC	CONCRETE: Approximately 4 inches thick.
			 17.8			SM ML SC	FILL: Grayish brown, moist, loose to medium dense, clayey SAND; few cobbles up to 5-1/2 inches in diameter; scattered zones of clay. Grayish brown, moist, loose to medium dense, silty fine to medium SAND; trace gravel; trace shell pieces. Yellowish brown, moist, loose to medium dense, fine sandy SILT. Yellowish brown and brown, moist, medium dense, clayey fine to medium SAND; trace gravel.
	-		22.4	93.0			VERY OLD PARALIC DEPOSITS:
10 -		- <u>1</u> 2					Brown, moist, weakly cemented, silty fine-grained SANDSTONE. Reddish brown to brown, moist, weakly indurated, sandy CLAYSTONE; trace gravel; scattered lenses of silty sandstone.
		_ <u>///11"</u> _ 78					Reddish brown, moist, weakly to moderately cemented, sandy gravel CONGLOMERATE; few cobbles.
					<u>2219</u>		Total Depth = 13 feet. (Refusal)
							Backfilled and capped with concrete shortly after drilling on 1/15/19.
							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.
20 -							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.
30 -							
40 -				l			BORING LOG FIGURE A- 1
٨	lin	40 & /	Noo	re			CITY HEIGHTS POOL 4380 LANDIS STREFT, SAN DIEGO, CALIFORNIA
Geot	echnical 8	Environmenta	I Sciences Co	onsultants			108716001 2/19

F

et)	SAMPLES	OT	(%)	(PCF)		NOI	DATE DRILLED 1/15/19 BORING NO. B-2 GROUND ELEVATION 365' ± (MSL) SHEET 1 OF 1
DEPTH (fee	Bulk Driven	BLOWS/FO	MOISTURE	DRY DENSITY	SYMBOL	CLASSIFICAT U.S.C.S.	METHOD OF DRILLING 6" Diameter Hollow Stem Auger (Pacific Drilling - Fraste) DRIVE WEIGHT 140 lbs. (Auto-Trip) DROP 30" SAMPLED BY NMM LOGGED BY NMM REVIEWED BY CAT
0							DESCRIPTION/INTERPRETATION
	-		26.4			SM	Approximately 6 inches thick. <u>FILL:</u> Grayish brown, wet, loose, silty SAND; few clay; few gravel. @ 1': Heavy seepage; perched water after core removal. Some gravel/cobble
		6					<u>VERY OLD PARALIC DEPOSITS:</u> Brownish gray, wet, weakly indurated, sandy CLAYSTONE; trace gravel.
		13	17.8	110.			Moderately indurated.
10 -	-7	<u>56</u>	17.7	110.8			Reddish brown, moist, weakly to moderately cemented, silty fine- to medium-grained
20 -		<u>36</u>	13.4				Reddish brown, moist, weakly cemented, sandy gravel CONGLOMERATE; few cobbles. @ 11.5': Difficult drilling. Total Depth = 11.9 feet. (Refusal) Seepage encountered at approximately 1 to 4 feet. Groundwater not encountered during drilling. Backfilled and capped with concrete shortly after drilling on 1/15/19. 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.
30 -							
	lin						BORING LOG FIGURE A- 2 CITY HEIGHTS POOL
Geot	technical 8	Environmenta	I Sciences Co	onsultants			4380 LANDIS STREET, SAN DIEGO, CALIFORNIA 108716001 2/19

DEPTH (feet)	Bulk SAMPLES	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED 1/15/19 BORING NO. B-3 GROUND ELEVATION 365' ± (MSL) SHEET 1 OF 1 METHOD OF DRILLING 6" Diameter Hollow Stem Auger (Pacific Drilling - Fraste) DRIVE WEIGHT 140 lbs. (Auto-Trip) DROP 30" SAMPLED BY NMM LOGGED BY NMM REVIEWED BY CAT
0						SM	CONCRETE: Approximately 7 inches thick
			Ş			OW	FILL: Grayish brown and yellowish brown, moist, loose to medium dense, silty fine to medium SAND; scattered zones of clay; few gravel. @ 3.0': Seepage encountered.
		15	12.7	119.1			VERY OLD PARALIC DEPOSITS: Gray to brownish gray, moist to wet, weakly to moderately indurated, silty CLAYSTONE; few to some gravel. Brown moist to wet, weakly cemented, clavey fine- to medium-grained SANDSTONE; few
		37	17.1	110.9			gravel. Beddish brown, moist, weakly to moderately camented, silty fine, to medium grained
·		28					SANDSTONE.
10 -		50 50/6"					Gravel/cobbles.
		E0/2"					Slow drilling.
20 -		<u> </u>					 Total Depth = 18.8 feet. Seepage encountered at approximately 3 feet. Groundwater not encountered during drilling. Backfilled and capped with concrete shortly after drilling on 1/15/19. <u>Note:</u> 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.
40 -							BORING LOG FIGURE A- 3
Λ	lin	40 &	Noc	ne			CITY HEIGHTS POOL 4380 LANDIS STREET, SAN DIEGO, CALIFORNIA
Geot	echnical &	Environmental	Sciences Co	onsultants			108716001 2/19

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 log of the exploratory boring 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 testing was performed on a selected representative soil sample in general accordance with ASTM D 422. The grain size distribution curve is shown on Figure B-1. The test results were utilized in evaluating the soil classifications in accordance with the USCS.

Atterberg Limits

Testing was 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-2.

Consolidation Tests

Consolidation testing was performed on a selected relatively undisturbed soil sample in general accordance with ASTM D 2435. The sample was inundated during testing to represent adverse field conditions. The percent of consolidation for each load cycle was recorded as a ratio of the amount of vertical compression to the original height of the sample. The results of the testing are summarized on Figure B-3.

Direct Shear Tests

Direct shear tests were performed on relatively undisturbed and remolded samples in general accordance with ASTM D 3080 to evaluate the shear strength characteristics of selected materials. The samples were inundated during shearing to represent adverse field conditions. The results are shown on Figures B-4 through B-6.

Expansion Index Tests

The expansion indices of selected material were evaluated in general accordance with ASTM D 4829. Specimens were molded under a specified compactive energy at approximately 50 percent saturation. The prepared 1-inch thick by 4-inch diameter specimens were loaded with a surcharge of 144 pounds per square foot and were inundated with tap water. Readings of volumetric swell were made for a period of 24 hours. The results of the tests are presented on Figure B-7.

Proctor Density Tests

The maximum dry density and optimum moisture content of selected representative soil samples were evaluated using the Modified Proctor method in general accordance with ASTM D 1557. The results of these tests are summarized on Figure B-6.

Soil Corrosivity Tests

Soil pH and electrical resistivity tests were performed on a representative sample in general accordance with CT 643. The sulfate and chloride contents of the selected sample were evaluated in general accordance with CT 417 and 422, respectively. The results of these tests are presented on Figure B-9.

GRAVEL SAND FINES Fine Fine Medium SILT CLAY Coarse Coarse U.S. STANDARD SIEVE NUMBERS HYDROMETER 3" 2' 11/2" 1" 3/4" 1/2" 3/8" 50 4 16 30 100 200 100.0 90.0 80.0 70.0 PERCENT FINER BY WEIGHT 60.0 50.0 40.0 30.0 20.0 10.0 0.0 100 10 1 0.1 0.01 0.001 0.0001 **GRAIN SIZE IN MILLIMETERS** Passing Sample Depth Liquid Plastic Plasticity Equivalent C_{u} C_{c} D₁₀ D₃₀ D₆₀ Symbol No. 200 uscs Location (ft) Limit Limit Index (percent) . B-3 5.0-6.5 30 17 13 ---------___ ---48 SC PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422

FIGURE B-1

GRADATION TEST RESULTS



SYMBOL	LOCATION	DEPTH (ft)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	USCS CLASSIFICATION (Fraction Finer Than No. 40 Sieve)	USCS
•	B-3	5.0-6.5	30	17	13	CL	SC

NP - INDICATES NON-PLASTIC



PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4318



FIGURE B-2

ATTERBERG LIMITS TEST RESULTS



Geotechnical & Environmental Sciences Consultants

4380 LANDIS STREET, SAN DIEGO, CALIFORNIA 108716001 | 2/19



PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 3080

FIGURE B-4

DIRECT SHEAR TEST RESULTS

CITY HEIGHTS POOL 4380 LANDIS STREET, SAN DIEGO, CALIFORNIA 108716001 | 2/19

Ninyo & Moore

Geotechnical & Environmental Sciences Consultants



PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 3080

FIGURE B-5

DIRECT SHEAR TEST RESULTS





PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 3080

FIGURE B-6

DIRECT SHEAR TEST RESULTS



SAMPLE LOCATION	SAMPLE DEPTH (ft)	INITIAL MOISTURE (percent)	COMPACTED DRY DENSITY (pcf)	FINAL MOISTURE (percent)	VOLUMETRIC SWELL (in)	EXPANSION INDEX	POTENTIAL EXPANSION
B-1	0.4-2.5	8.0	118.4	13.7	0.003	3	Very Low
B-2	6.0-8.0	10.5	108.5	22.3	0.064	64	Medium
			ue	SC STANDARD 18-	2 🛛 ASTM D	4829	

PERFORMED IN GENERAL ACCORDANCE WITH

FIGURE B-7



EXPANSION INDEX TEST RESULTS



SAMPLE	SAMPLE		RESISTIVITY ¹	SULFATE (CONTENT ²	CHLORIDE CONTENT ³ (ppm) 50 335
LOCATION	DEPTH (ft)	рп	(ohm-cm)	(ppm)	(%)	(ppm)
B-1	0.4-2.5	8.8	1,000	10	0.001	50
B-3	4.0-7.0	8.8	500	180	0.018	335

¹ PERFORMED IN ACCORDANCE WITH CALIFORNIA TEST METHOD 643

- ² PERFORMED IN ACCORDANCE WITH CALIFORNIA TEST METHOD 417
- ³ PERFORMED IN ACCORDANCE WITH CALIFORNIA TEST METHOD 422

FIGURE B-9

Ninyo & Moore Geotechnical & Environmental Sciences Consultants

CORROSIVITY TEST RESULTS CITY HEIGHTS POOL 4380 LANDIS STREET, SAN DIEGO, CALIFORNIA

APPENDIX C

Selected Site Photographs

Ninyo & Moore | 4380 Landis Street, San Diego, California | 108716001 | February 12, 2019



Photograph 1: Drained pool with sawcut area, view to northwest. 12/19/18



Photograph 2:

North side of pool, view to west. Pool wall exhibits bowing. 12/19/18

FIGURE C-1

PHOTOGRAPHS CITY HEIGHTS POOL 4380 LANDIS STREET, SAN DIEGO, CALIFORNIA

Geotechnical & Environmental Sciences Consultants



Photograph 3: Concrete deck on north side of pool. Pool deck is cracked. Concrete cores in view are by others. 12/19/18



Photograph 4:

Water in deep end of the pool (north side). A pump was being used to remove the water. 01/15/19

FIGURE C-2

PHOTOGRAPHS

CITY HEIGHTS POOL 4380 LANDIS STREET, SAN DIEGO, CALIFORNIA





Photograph 5: Water within pool light with seepage at corner. 01/15/19



Photograph 6:

Standing water after pavement core at B-2. 01/15/19

FIGURE C-3

PHOTOGRAPHS

CITY HEIGHTS POOL 4380 LANDIS STREET, SAN DIEGO, CALIFORNIA





Photograph 7: Drilling of B-2 on north side of pool, view to east. 01/15/19



Photograph 8:

FIGURE C-4

PHOTOGRAPHS

CITY HEIGHTS POOL 4380 LANDIS STREET, SAN DIEGO, CALIFORNIA



APPENDIX D

Geocon 2018a and Geocon 2018b

Ninyo & Moore | 4380 Landis Street, San Diego, California | 108716001 | February 12, 2019

LIMITED GEOTECHNICAL INVESTIGATION

CITY HEIGHTS POOL CONTRACT NO. H156367 TASK NO. 15GG20 SAN DIEGO, CALIFORNIA

PREPARED FOR

CITY OF SAN DIEGO PARKS AND RECREATION DEPARTMENT SAN DIEGO, CALIFORNIA

> JULY 11, 2018 PROJECT NO. G1849-52-15



GEOTECHNICAL ENVIRONMENTAL MATERIALS GEOCON

GEOTECHNICAL . ENVIRONMENTAL MATERIALS



Project No. G1849-52-15 July 11, 2018

City of San Diego Parks and Recreation Department 1250 Sixth Avenue, Fourth Floor MS 804A San Diego, California 92101

Attention: Mr. Julian Spinoza

Subject: LIMITED GEOTECHNICAL INVESTIGATION CITY HEIGHTS POOL CONTRACT NO. 156367 TASK NO. 15GG20 SAN DIEGO, CALIFORNIA

Dear Mr. Spinoza:

In accordance with your request and authorization, we have performed a limited geotechnical investigation for the subject pool located at 4380 Landis Street in the City Heights area of San Diego, California (Figure 1, Vicinity Map). We understand that in March of 2018, the plumbing pipes beneath the pool slab were broken and water started coming out of the localized cracks within the pool decking. The pool was then emptied and a section of the concrete slab in the middle portion of the pool, including localized cracking was saw cut and removed to facilitate plumbing inspection. A section of the pool decking at the south side the pool was also saw cut and removed. A review of the as-built plans for the pool indicates that the pool was constructed in 1998.

SITE VISIT

We initially visited the site on June 7, 2018, accompanied by Parks and Recreation Department representatives. The purpose of the site visit was to observe site conditions at the pool and evaluate the cause of the distress.

During our site visit the following observations were made:

• The pool is approximately 102 feet long and 75 feet wide. The shallow section of the pool is approximately 4 feet deep. It transitions into the deepest section at approximately 10 feet in depth.

- A section in the bottom of the pool, approximately two feet wide and 60 feet long has been saw cut and removed. It reveals that the concrete slab is approximately 8 inches thick and incorporates reinforcing steel.
- Except for a few minor hairline cracks, we did not observe any sign of major cracking within the uncut pool slab, wall and decking.
- We were informed by the pool maintenance personnel that the pool decking was generally level except for the northeast corner that was approximately 1 to 2 inches higher.
- We were informed that the pipes beneath the pool have been pressure tested and the results indicated extensive leakage.

DOCUMENT REVIEW

- Geotechnical investigation and as-graded reports for the pool were not available for our use.
- A review of the Geologic Map of San Diego indicates that the area is underlain by the Very Old Paralic Deposits (formerly known as the Lindavista Formation). This material typically consists of interbedded orangish brown sandstone and conglomerate. A very expansive clay bed is associated with the upper portion of this formation in the Mid-City and the North Park area.
- We have also reviewed the Construction plans for the pool, deck and piping (As-Built, dated July 20, 1999). These plans do not reveal if the pool was excavated into the formational materials or the area was undercut and backfilled prior to construction of the pool.

SITE INVESTIGATION

Our subsurface investigation was conducted on June 14, 2018. The investigation consisted of drilling three small diameter borings inside and outside of the pool to assist in creating a profile of underlying soils. A limited access drill rig was lifted by a crane into the pool area. Boring B-1 was drilled in the shallow portion of the pool, and Boring B-2 was drilled in the deep portion of the pool. Boring B-3 was drilled on the south of the pool on the deck.

Boring B-1 was drilled in the area where the concrete slab was already removed. Formational materials consisting of dark grayish brown stiff saturated clay to an approximate depth of 4 feet were encountered over layers of dense to very dense conglomerate and sandstone to a depth of 21¹/₂ feet.

Boring B-2 was drilled through the cored concrete slab. The material encountered consisted of saturated trench backfill material associated with a drainage pipe. Formational sandstone and conglomerate were encountered below the trench backfill. Moisture content of the materials decreased with depth. The saturated clay layer was not present in this boring. The drilling was terminated at the depth of 18 feet due to practical refusal.

Boring B-3 was drilled on the pool deck within the area that the concrete slab had already been removed. We encountered shallow fill, perhaps associated with the plumbing trench backfill, overlying the undisturbed formational material. The formational material consisted of stiff, wet, dark gray sandy clay, to the depth of 8 ½ feet overlaying very dense cobble conglomerate. The boring was terminated at 18 feet due to practical refusal.

We logged the materials encountered in each boring and collected soil samples at various depths for laboratory testing. The boring logs are presented in Appendix A. The approximate locations of borings are shown on the Site Plan, Figure 2. A schematic geologic cross-section is presented on Figure 3.

We delivered the soil samples to our laboratory and tested selected soil samples for in-place density and moisture content, gradation, Plasticity Index (PI), Expansion Index (EI) and swell potential. The swell test was performed on relatively undisturbed samples of the clay to evaluate the degree of saturation and potential for additional heaving by increasing moisture content. The in-place density and moisture contents of the materials encountered are shown on the boring logs in Appendix A. The laboratory test results are presented in Appendix B.

FINDINGS

- The pool was excavated into the formational materials of the Very Old Paralic Deposits consisting of highly expansive clay overlying dense sandstone and conglomerate. The clay zone appears to be approximately 8 feet thick. See Figure 3, Geologic Cross-Section.
- Our field observation and laboratory test results indicate that the in- place moisture content of the underlying materials decreases with depth, indicating the source of high moisture content in underlying soils is not groundwater.
- The formational materials possess relatively low permeability, either due to clay content or cementation.
- The laboratory test results of the clay samples indicate that this material is highly expansive, Expansion Index (EI = 205). However, swell test on the relatively undisturbed samples indicated that the in-situ clay is nearly saturated and swelling due to additional exposure to water would be minimal.
- The clayey-silty sandstone/conglomerate encountered below the clay layer has a "medium" expansion potential.
- The plumbing pipes underneath the pool are likely embedded in sandy gravel fill material with high permeability.
- The concrete slab, inside the pool and at the pool decking, whether cored or saw cut was approximately 8 inches thick. The concrete slab is reinforced with rebars.
- The uncut concrete slab and walls of the pool are relatively free of significant cracking.

CONCLUSIONS

- The pool and surrounding areas are underlain by formational material of the Very Old Paralic Deposits with the highly expansive clay member at the surface to a depth of approximately 8 feet. Therefore, the decking and shallow portion of the pool are underlain by expansive, saturated clay and the deep end of the pool in underlain by dense sandstone and conglomerate.
- Although the clayey material is highly expansive when dried or disturbed, it not likely that has directly caused localized slab distress and water overflow.
- It appears that the cause of localized distress, water built up and overflow was associated with plumbing damage and extensive leakage, where significant water pressure was built up within the sandy gravel material surrounding the pipes beneath the slab.

RECOMMENDATIONS

- For repairing the damaged and removed concrete slabs, we recommend involvement of a structural engineer to design proper doweling and sealing procedures.
- The repairing and/or replacement of damaged pipes should be performed appropriately to facilitate long term maintenance and minimize disturbance of the underlying soils.

If you have any questions or we may be of further service, please contact the undersigned at your convenience.

Very truly yours,

GEOCON INCORPORATED

Yong Wang li Sadr SIONAL CEG 1778 GE 2775 GEO PROF AS:YW:dmc SADR No. 1778 CERTIFIED (3) Addressee GEOLOGIST FOFCAL



Plotted:07/11/2018 2:52PM | By: JONATHAN WILKINS | File Location:Y:\PROJECTS\G1849-52-15 City Hights Pool\DETAILS\G1849-52-15_Vicinity Map.dwg



Plotted:07/11/2018 2:56PM | By:JONATHAN WILKINS | File Location:Y:\PROJECTS\G1849-52-15 City Hights Pool\SHEETS\G1849-52-15 SitePlan.dwg



DISTANCE (FEET)

GEOLOGIC CROSS-SECTION A-A'

SCALE: 1" = 10' (Vert. = Horiz.)



CITY HEIGHTS POOL SAN DIEGO, CALIFORNIA



Plotted:07/11/2018 3:10PM | By: JONATHAN WILKINS | File Location:Y:\PROJECTS\G1849-52-15 City Hights Pool\SHEETS\G1849-52-15 Cross Section.dwg





APPENDIX A

FIELD INVESTIGATION

We performed our field investigation on June 14, 2018, which consisted of coring the concrete slab at 2 locations and drilling 3 small diameter borings to a maximum depth of 21 ½ feet below the concrete slab. The drill rig was a limited access drill rig that was lifted by a crane over the existing fence and was equipped with 6-inch diameter hollow stem auger. The locations of the exploratory borings are shown on the *Site Plan*, Figure 2. Boring logs are presented on Figures A-1 through A-3 following the text in this appendix. We located the borings in the field using a measuring tape and existing reference points (poles). Therefore, actual boring locations may deviate slightly.

We obtained samples during our boring excavations using a California split-spoon sampler or a Standard Penetration Test (SPT) sampler. Both samplers are composed of steel and are driven to obtain the soil samples. The California sampler has an inside diameter of 2.5 inches and an outside diameter of 2.875 inches. Up to 18 rings are placed inside the sampler that is 2.4 inches in diameter and 1 inch in height. The SPT sampler has an inside diameter of 1.5 inches and an outside diameter of 2 inches. Ring samples at appropriate intervals were retained in moisture-tight containers and transported to the laboratory for testing. We also retained bulk samples from the borings for laboratory testing. The type of sample is noted on the exploratory boring logs. The samplers were driven 12 and 18 inches using the California and SPT samplers, respectively, into the bottom of the excavations with the use of a Cathead hammer and the use of A rods. The sampler is connected to the A rods and driven into the bottom of the excavation using a 140-pound hammer with a 30-inch drop. Blow counts are recorded for every 6 inches the sampler is driven. The penetration resistances shown on the boring logs are shown in terms of blows per foot. The values indicated on the boring logs are the sum of the last 12 inches of the sampler if driven 18 inches. If the sampler was not driven for 18 inches, an approximate value is calculated in term of blows per foot or the final 6-inch interval is reported. These values are not to be taken as N-values, adjustments have not been applied.

We visually examined, classified, and logged the soil encountered in the borings in general accordance with American Society for Testing and Materials (ASTM) practice for Description and Identification of Soils (Visual-Manual Procedure D 2488). The logs depict the soil and geologic conditions observed and the depth where we obtained samples.

PROJECT NO. G1849-52-15

			_					
DEPTH IN	SAMPLE	огосу	DWATER	SOIL CLASS		'RATION STANCE VS/FT.)	DENSITY C.F.)	STURE ENT (%)
FEET	NO.	H H	NNO	(USCS)	ELEV. (MSL.) DATE COMPLETED 06-14-2018	ENET	RY D (Р.	
			GR		EQUIPMENT MOLE RIG BY: A. SADR	∃ R ⊃		0
					MATERIAL DESCRIPTION			
- 0 -		0.00.00			8-INCHES CONCRETE			
	<mark>B1-1</mark>			СН	VERY OLD PARALIC DEPOSITS (Qvop) Stiff, saturated, dark gravish brown, CLAY with little sand and gravel	-		
- 2 -	B1-2					- 9	92.9	28.3
						-		
- 4 -			<u> </u>	SC-GC	Dense wet dark reddish brown Clavey SAND with gravel			
L –		101		Se de				
- 6 -	B1-3	p/p			-No recovery due to gravel	- 28		
Ũ		0, 1						
		10/1				_		
- 8 -		9/1				-		
		6/				-		
- 10 -	B1-4			$-\frac{1}{SM}$	Very dense moist orangish brown Silty SANDSTONE: inter bedded with			119
					layers of gravelly conglomerate	_		
- 12 -						_		
- 14 -					-Cobbles	-		
	B1-5				-No recovery	50/3"		
- 16 -				GM-SM	Interbedded grayish brown, Silty, fine SANDSTONE some gravel			
					conglomerate	-		
- 18 -						_		
				- <u>-</u>	Dance maint light house fine to medium SAND, schericalers	┣		
- 20 -				5r	Dense, moist, light brown, line to medium SAND, conesionless			
20	B1-6					46		
					BORING TERMINATED AT 21.5 FEET			
					No groundwater encountered			
					Backfilled with bentonite grout			
								<u> </u>
	e A-1, f Borina	n R 1		2 ana 1	of 1		G184	9-52-15.GPJ
		90	, (
SAMF	PLE SYMB	OLS		SAMP	LING UNSUCCESSFUL I STANDARD PENETRATION TEST I DRIVE S. JRBED OR BAG SAMPLE I CHUNK SAMPLE I WATER '	AMPLE (UNDI: TABLE OR SE	STURBED) EPAGE	

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.
			-					
DEPTH	SAMPLE NO.	LITHOLOGY	ATER	5011	BORING B 2	TION NCE FT.)	SITY (IRE Г (%)
IN FEET			MDN	CLASS	ELEV. (MSL.) DATE COMPLETED 06-14-2018		P.C.F	DISTU
			GROL	(USCS)	EQUIPMENT MOLE RIG BY: A. SADR	RES (BL	DR)	CONC
			\vdash					
- 0 -					MATERIAL DESCRIPTION 9.INCHES CONCRETE			
L _								
- 2 -				GP	FILL Medium dense, saturated, dark gray, SAND and GRAVEL (plumbing backfill) -PVC Pine at 2 feet	-		
			•	SM	VERY OLD PARALIC DEPOSITS (Qvop)			
- 4 -	B2-1		• • •		Very dense, wet to moist, orangish brown, Silty, fine SANDSTONE	75	108.1	18.1
	1 [•					
- 6 -	B2-2		•			\vdash		
			•			-		
_ <u>o</u> _			°					
0	B2-3	$\begin{bmatrix} 0 \end{bmatrix}$		GM	Very dense, moist, dark reddish brown, GRAVEL CONGLOMERATE with	50/6"		
		$\int O $			-No recovery	-		
- 10 -	D2 4	P_{0}				- 25		
	D2-4	ρ						
	▎ ዞ							
- 12 -		b°			-Dense, wet to moist	-		
			1			-		
- 14 -			1					
				SP-SM	Becomes dense, moist, light brown, fine to medium SAND, little silt			
	B2-5					35		
- 16 -						\vdash		
						-		
- 18 -					-Cobbles refusal			
10					BORING TERMINATED AT 18 FEET No groundwater encountered Boring backfilled with bentonite grout			
Figure A-2, G1849-52-15.GPJ								
Log of Boring B 2, Page 1 of 1								
SAMPLE SYMPOLS								
SAMPLE SYMBOLS								

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.



			_						
DEPTH IN FEET	SAMPLE NO.	ITHOLOGY	UNDWATER	SOIL CLASS (USCS)	BORING B 3 ELEV. (MSL.) DATE COMPLETED 06-14-2018	NETRATION ESISTANCE LOWS/FT.)	YY DENSITY (P.C.F.)	AOISTURE DNTENT (%)	
			GRC		EQUIPMENT MOLE RIG BY: A. SADR	BEI (B	Ъ	≥0 0	
					MATERIAL DESCRIPTION				
- 0 -		0.00.00			8-INCHES CONCRETE TOP-6-INCHES CONCRETE				
				SM	FILL	-			
- 2 -	B3-1				(plumbing trench backfill)	- 15			
	<mark>B3-2</mark>			CL-SC	VERY OLD PARALIC DEPOSITS (Qvop) Stiff wet dark grou to bluich grou Sondy CLAV		91.8	31.4	
- 4 -	B3-3				Sun, we, dark gray to ordisin gray, Sandy CLAT	15			
	l l					-			
- 6 -	. 8	8//				-			
						-			
- 8 -									
	B3-4			SC-GC	Becomes firm, moist, brown and gray, Sandy CLAY	<u>57</u>			
		ρ_{0}			CONGLOMERATE with clayey sand matrix				
- 10 -		b_{0}				-			
		b_{0}				-			
- 12 -					-Very difficult drilling from 12 feet cobble in sample	-			
		b_{0}				-			
- 14 -						_			
10	B3-5	-No recovery		50/5"					
- 16 -	[\mathbf{b}_{0}							
					-No recovery	50/6"			
- 18 -					BORING TERMINATED AT 18 FEET				
					No groundwater encountered Boring terminated with bentonite grout				
					Boring terminated with bencome grout				
Figure A-3. G1849-52-15.GPJ									
Log of Boring B 3, Page 1 of 1									
CAMPLE SYMPOL C SAMPLING UNSUCCESSFUL STANDARD PENETRATION TEST DRIVE SAMPLE (UNDISTURBED)									
SAMPLE SYMBOLS									

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.



APPENDIX B

LABORATORY TESTING

We performed laboratory testing to evaluate the physical and mechanical properties of the soil encountered at the borings. We performed the laboratory tests in accordance with the current versions of the generally accepted *American Society for Testing Materials* (ASTM) procedures or other suggested procedures. We tested selected soil samples for in-situ dry density and moisture content, gradation, plasticity and expansion potential. The results of our laboratory tests are presented in Table B-I and on Figures B-1 through B-3. In addition, the in-situ dry density and moisture content are presented on the exploratory boring logs in Appendix A.

TABLE B-I SUMMARY OF LABORATORY EXPANSION INDEX TEST RESULTS ASTM D 4829

Sample	Moisture C	Content (%)	Drv Density	Expansion	Soil	2016 CBC Expansion Classification	
No.	Before Test	After Test	(pcf)	Index	Expansion Classification		
B1-1	11.2	28.6	105.5	205	Highly Expansive	Expansive	
B3-2	10.2	21.8	109.7	67	Medium	Expansive	





Figure B-2

GEOCON



Figure B-3

GEOCON

GEOTECHNICAL E ENVIRONMENTAL MATERIALS



Project No. G1849-52-15 August 7, 2018

ORPORATED

City of San Diego Public Works Department 525 B Street, Suite 750 San Diego, California 92101

Attention: Mr. Julian Spinoza

Subject: MISCELLANEOUS GEOTECHNICAL CONSULTATION CITY HEIGHTS POOL CONTRACT NO. 156367 TASK NO. 15GG20 SAN DIEGO, CALIFORNIA

Dear Mr. Spinoza:

Further to our recent communications regarding the subject project, please find our opinions below:

The wall of the pool is in a critical loading condition now. The plumbing failure resulted in new loads on the pool walls (erosion of material/free water), which was followed by quick draining of the pool that unloads one side of the pool wall. Following removal of pool water, soils outside the pool are drying slightly with the lack of new water through the deck and/or pool wall cracks. The bulge was originally observed to be rather minor. We assume the bulging has stabilized.

Based on the current information available to Geocon, it was our opinion that the cracks were associated with the plumbing failure and that any bulging was likely the result of the unloading of the inside of the pool by removal of pool water. Each change results in some new stresses and new stresses result in new movements (strain).

If the localized cracks and bulging were a result of soil expansion or settlement, we would expect to see similar distress throughout the pool. Since we see this distress in only localized areas, it is our opinion that the distress is more likely the result of plumbing failure.

With regard to collapse potential, it would be best to confer with the pool designers to confirm that they designed the wall for all loading conditions: (1) unsaturated soil with no water in pool during construction, (2) unsaturated soil with water in the pool upon first filling, (3) saturated soil with water in the pool as surface water slowly infiltrates into the soils, (4) saturated soil with no water in pool when water was removed to do repairs. If these scenarios were properly included in design and construction, there is no reason to be concerned about collapse.

Should you have any questions regarding this letter, or if we may be of further service, please contact the undersigned at your convenience.

Very truly yours,

GEOCON INCORPORATED

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(e-mail) Addressee



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