APPENDIX A

Appendix A

Whittier Narrows Splash Pad – Soils Report
Dated 04/1/22

GEOTECHNICAL INVESTIGATION
PROPOSED SPLASH PAD AREA D
AND SITE IMPROVEMENTS
750 S. SANTA ANITA AVENUE
SOUTH EL MONTE, CALIFORNIA

Prepared for:
County of Los Angeles Department of Public Works
Project Management Division
900 S. Fremont Avenue, 5th Floor
Alhambra, California 91803-1331

Prepared by:
Geotechnical Professionals Inc.
5736 Corporate Avenue
Cypress, California 90630
(714) 220-2211

Project No. 2780.47I

April 1, 2022

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County of Los Angeles Department of Public Works
Project Management Division
900 S. Fremont Avenue, 5th Floor
Alhambra, California 91803-1331

Attention: Courtney Tossounian, Architect
Project Manager

Subject: Geotechnical Investigation
Proposed Splash Pad and Site Improvements
Area D - Whittier Narrows Regional Recreational Area
750 S. Santa Anita Avenue, South El Monte, California
Contract PW-15015
GPI Project No. 2780.47I

Dear Courtney:

Transmitted herewith is our report of geotechnical investigation for the subject project. The report presents the results of our evaluation of the subsurface conditions at the site and recommendations for design and construction.

We are providing this report in an electronic format. Further wet-signed originals of the report can be provided to the County upon request.

We appreciate the opportunity of offering our services on this project and look forward to seeing the project through its successful completion. Feel free to call us if you have questions regarding our report or need further assistance.

Very truly yours,
Geotechnical Professionals Inc.

Paul R. Schade, G.E. 2371
Principal

5736 Corporate Avenue • Cypress, CA 90630 • (714) 220-2211 , FAX (714) 220-2122

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1.0 INTRODUCTION

1.1 GENERAL

This report presents the results of a geotechnical investigation performed by Geotechnical Professionals Inc. (GPI) for a proposed splash pad and site improvements in El Monte, California. The general site location is shown on the Site Location Map, Figure 1.

1.2 PROJECT DESCRIPTION

A new splash pad is proposed in Area D of the Whittier Narrows Regional Recreational Area. Details of the project are limited but based on the information provided (Project Concept Report, June 2021), we understand that the project will include the proposed splash pad, storm water basin, and improvements to the adjacent parking lot, walks, and restroom/equipment storage building. The splash pad will include concrete hardscape, perimeter seating walls, screen walls, shade sails, a mechanical pad, and a pump vault. Finished grades are expected to be within a few feet of the existing grades. The project will also include a storm water infiltration system. Details for the structural loads have not been provided. The Site Plan, Figure 2, shows the configuration of these improvements.

Our recommendations are based upon the above structural and grading information. We should be notified if the actual loads and/or grades change during the project design to either confirm or modify our recommendations. When the project structural and grading plans become available, we should be provided with copies for review and comment.

1.3 PURPOSE OF INVESTIGATION

The primary purpose of this investigation and report is to provide an evaluation of the existing geotechnical and seismic conditions at the site, as they relate to the design and construction of the proposed improvements. More specifically, this investigation was aimed at providing geotechnical recommendations for earthwork and design of foundations and pavements.

2.0 SCOPE OF WORK

Our scope of work included a review of existing data, field investigation, field infiltration testing, laboratory testing, geotechnical analyses, and preparation of this report.

Our field exploration consisted of four exploratory borings drilled to depths between 10 and 60 feet below grade. Descriptions of the field procedures and logs of the explorations are presented in Appendix A.

Laboratory soil tests were performed on selected representative samples as an aid in soil classification and to evaluate the engineering properties of the soils. The geotechnical laboratory testing program included determinations of moisture content and dry density, fines content, shear strength, corrosivity, maximum density and optimum moisture content, expansion index, and Atterberg limits. Corrosivity testing was performed by HDR under subcontract to GPI. Laboratory testing procedures and results are summarized in Appendix B.

Engineering evaluations were performed to provide earthwork criteria, foundation design parameters, and assessments of seismic hazards. The results of our evaluations are presented in the remainder of the report.

Our investigation was prepared in general accordance with the County of Los Angeles, Department of Public Works Manual (County, 2013).

3.0 SITE CONDITIONS

3.1 SURFACE CONDITIONS

The existing site is currently occupied by lawn adjacent to restroom number 12. The project area is approximately 10,000 square feet. The site location is bound to the north by the parking lot that is adjacent to the 60 freeway, to the west by a lake, to the south by Whittier Narrows Park Trail, to the east by Santa Anita Avenue.

3.2 SUBSURFACE SOIL CONDITIONS

Our field investigation disclosed a subsurface profile consisting of fill soils overlying natural soils. Detailed descriptions of the conditions encountered are shown on the Logs of Borings in Appendix A.

We encountered fill soils consisting of silty sands and sandy silts extending to depths of approximately 4 feet below the existing ground surface. The fill soils are generally medium dense or very stiff, with variable moisture contents ranging from slightly moist to wet. Documentation is not available regarding the placement and compaction of the fill soils.

The underlying natural materials consist predominately of medium dense silty sands and stiff to very stiff silts and clays to a depth of approximately 10 feet. The moisture content of the upper natural soils is highly variable, ranging from slightly moist to wet. At depths of 10 to 15 feet, the natural soils consist of wet, very stiff silts. The natural soils in the upper 10 feet exhibit low to moderate strength and moderate compressibility characteristics. The upper soils have a very low expansion potential.

3.3 GROUNDWATER AND CAVING

Groundwater was encountered in our boring at a depth of 43 feet below existing grade. The historical high groundwater has been determined to be approximately 0 to 5 feet deep in the vicinity of the site by the State of California (CDMG, 1998).

Caving was not observed in our relatively small diameter, hollow stem auger borings. However, caving may be anticipated in the wet sands and silty sands in the upper 10 feet of the existing soil profile.

4.0 CONCLUSIONS AND RECOMMENDATIONS

4.1 OVERVIEW

Based on the results of our investigation, it is our opinion that from a geotechnical engineering viewpoint it is feasible to develop the site as proposed. Furthermore, in accordance with the County of Los Angeles Statement 111, it is our opinion that the project will be safe for its intended use against hazard from landslide, settlement, or slippage and the project will not have adverse effect on the stability of the site or adjoining properties.

The proposed restroom/equipment storage building can be supported on shallow footing and other improvements can be supported on shallow footings or drilled piles following the mitigation of the geotechnical constraints discussed below. The most significant geotechnical issues that will affect the design and construction of the proposed improvements are as follows:

- Undocumented fills were reported to depths of up to 4 feet below existing grade at the site. The fill soils are not considered to be suitable for direct support of foundations or floor slabs without remedial earthwork. For the proposed improvements, we recommend removal and recompaction of the fill and a portion of the upper low-density natural soils to provide uniform support for the planned foundations and floor slab.

- Proposed shade structures may be supported on drilled pile foundations extending into the undisturbed natural soils or shallow footings supported on properly compacted fill. Remedial grading is not required for drilled pile foundations.

- Current moisture contents of the upper soils vary widely from under to well over optimum moisture content. Earthwork will likely require drying/mixing of the on-site soils prior to placing as properly compacted fill and localized deeper removals or subgrade stabilization should be anticipated.

- The site is located within a zone identified as having a potential for liquefaction by the State. Soil liquefaction is likely to occur at this site during a design earthquake event. The estimated magnitude of induced existing ground surface settlement is on the order of 1¼ inch with differential settlement across 40 feet on the order of ½ inch.

- The upper on-site sandy soils are considered to be susceptible to caving in open cuts and excavations. Care should be taken to maintain support of the soils and structures left in-place adjacent to planned excavations.

Our recommendations related to the geotechnical aspects of the development of the site are presented in the subsequent sections of this report.

4.2 SEISMIC DESIGN

4.2.1 General

The site is located in a seismically active area typical of Southern California and is likely to be subjected to strong ground shaking due to earthquakes on nearby faults.

We assume the seismic design of the proposed development will be in accordance with the 2019 California Building Code (CBC) criteria. For the 2019 CBC, a Site Class D (stiff soil) may be used. Using the Site Class, which is dependent on geotechnical issues, and the appropriate SEAOC/OSHPD website (OSHPD), the corresponding seismic design parameters from the CBC are as follows:

<u>2019 CBC:</u>

$S_S = 1.894g$	$S_{MS} = F_a * S_S = 1.894g$	$S_{DS} = 2/3 * S_{MS} = 1.263g$
$S_1 = 0.678g$	$S_{M1} = F_V * S_1 = 1.153g$	$S_{D1} = 2/3 * S_{M1} = 0.768g$

The seismic design parameters above should be confirmed by the Project Structural Engineer.

In accordance with the 2019 CBC, site-specific response spectra are required for structures located in a Site Class D (with S_1 greater than or equal to 0.2) unless, per the exceptions detailed in Section 11.4 8 of ASCE 7-16 (ASCE, 2017), the structure is designed using seismic response coefficient (Cs) determined by either:

- Equation 12.8-2 for values of $T \leq 1.5\ T_{S,}$
- 1.5 times the value computed by Equation 12.8-3 for values of $T_L \geq T > 1.5\ T_S$, or
- 1.5 times the value computed by Equation 12.8-4 for values of $T > T_L$.

If this exception is not taken and the structure will still be designed in accordance with the 2019 CBC, GPI should be notified that site-specific response spectra is requested.

The actual method of seismic design should be determined by the Project Structural Engineer.

4.2.2 Strong Ground Motion Potential

Based on published information (USGS, 2008), the most significant fault in the proximity of the site is the East Montebello Fault, which is located about 2 miles from the subject site.

During the life of the project, the site will likely be subject to strong ground motions due to earthquakes on nearby faults. Based on the SEAOC/OSHPD website (OSHPD), we computed that the site could be subject to a peak ground acceleration (PGA_m) of 0.84g for a magnitude 6.87 earthquake. This acceleration has been computed using the mapped Maximum Considered Geometric Mean peak ground acceleration from ASCE 7-16 (ASCE, 2017) and a site coefficient (F_{PGA}) based on Site Class. The structural design of the facility will need to incorporate measures to mitigate the effects of strong ground motion.

4.2.3 Potential for Ground Rupture

There are no known active faults crossing or projecting through the site. The site is not located in an Alquist-Priolo Earthquake Fault Zone. Therefore, ground rupture due to faulting is considered unlikely at this site.

4.2.4 Liquefaction and Seismic Ground Settlement

Soil liquefaction is a phenomenon in which saturated cohesionless soils undergo a temporary loss of strength during severe ground shaking and acquire a degree of mobility sufficient to permit ground deformation. In extreme cases, the soil particles can become suspended in groundwater, resulting in the soil deposit becoming mobile and fluid-like. Liquefaction is generally considered to occur primarily in loose to medium dense deposits of saturated soils. Thus, three conditions are required for liquefaction to occur: (1) a cohesionless soil of loose to medium density; (2) a saturated condition; and (3) rapid large strain, cyclic loading, normally provided by earthquake motions.

The site is located within an area mapped as having a potential for soil liquefaction (CGS, 1999). Inclusion of a site on the maps does not mean that a hazard actually exists at the site. It simply means that the characteristics of the site require investigation of the hazard.

The California Building Code (CBC), ASCE 7-16 (ASCE, 2017), and Special Publication 117A (CGS, 2008) requires that the ground motion used for liquefaction evaluation be based on the Peak Ground Acceleration (PGA_M) adjusted for site class effects. Accordingly, we used a ground acceleration of 0.84g for a magnitude 6.8 earthquake for our analyses and SPT blow counts obtained from boring B-1. The potential for liquefaction was evaluated using the methods presented by the 2008 Idriss and Boulanger methodology (Idris and Boulanger, 2008) and modifications provided in Special Publication 117A.

To evaluate the potential for liquefaction at the site, we considered recent and historic groundwater levels. Groundwater was encountered at depths of 43 feet in our recent borings. Historical high ground water depth is 0-5 feet. Therefore, we used a groundwater depth of 5 feet below existing grade, which corresponds to the historical high level as determined by the State (CDMG, 1998) for our evaluation.

Based on our evaluations and the anticipated remedial earthwork, the soils within about 8 to 15 feet exhibit a potential for liquefaction if the groundwater rises to historic levels. Should liquefaction of these upper layers occur, the estimated magnitude of induced settlement would be approximately 1¼ inch or less. Differential settlement across 40 feet could be on the order of ½-inch or less.

4.3 EARTHWORK

The earthwork anticipated at the project site will consist of demolition of existing improvements, clearing, excavation of undocumented fills and disturbed soils, subgrade preparation, and placement and compaction of fill.

4.3.1 Clearing and Grubbing

Prior to grading, performing excavations or constructing the proposed improvements, the areas to be developed should be stripped of vegetation and cleared of existing structures, debris, and pavements. Buried obstructions, such as footings, abandoned utilities, and tree roots should be removed from areas to be developed. Deleterious material generated during the clearing operation, including organic topsoil, should be removed from the site. If approved by the owner and regulatory agency, inert demolition debris, such as concrete and asphalt, may be crushed for reuse in engineered fills and in a stabilization layer in accordance with the criteria presented in the "Materials for Fill" section of this report.

Although not encountered in our explorations, if cesspools or septic systems are encountered during grading they should be removed in their entirety. The resulting excavation should be backfilled as recommended in the “Subgrade Preparation” and “Placement and Compaction of Fills” sections of this report. As an alternative, cesspools can be backfilled with lean sand-cement slurry. At the conclusion of the clearing operations, a representative of GPI should observe and accept the site prior to further grading.

4.3.2 Excavations

Excavations at this site will include removals of the undocumented fills, removals of disturbed soils and a portion of natural soils, foundation excavations, and trenching for new utility lines.

Prior to placing fills or construction of the structures, the existing undocumented fill, portions of the upper compressible soils, and soils disturbed during demolition should be removed and replaced as properly compacted fill. For planning purposes, we anticipate the following removal depths:

- For the single-story restroom/equipment storage building and retaining walls taller than 5 feet, removals should extend at least 4 feet below existing grades or 2 feet below the base of foundations and slabs, whichever is deeper;

- For minor structures (e.g. monument signs, site walls, perimeter seating, mechanical pads, and the pedestrian splash pad footprint, removals should extend 3 feet below grade or 2 feet below the base of foundations and slabs, whichever is deeper;

- For new pedestrian hardscape outside of the splash pad footprint, vehicular pavements, and ADA ramps, removals for should extend at least 1-foot below the existing or proposed subgrade, whichever is deeper.

The actual depths of removals should be determined in the field during grading by a representative of GPI. The soils exposed at the base of the overexcavation should be processed in place as described in the “Subgrade Preparation” section of this report.

The Project Surveyor should accurately stake the corners of the areas to be overexcavated in the field. Where space is available, the base of the excavations should extend laterally at least 5 feet beyond the building lines or edge of foundations, or a minimum distance equal to the depth of overexcavation/compaction below finish grade (i.e., a 1:1 projection below the top outside edge of footings), whichever is greater. Building lines include the footprint of the building and other foundation supported improvements, such as canopies and attached site walls.

Where not removed by the aforementioned excavations, existing utility trench backfill should be removed and replaced as properly compacted fill within the building pads. This is especially important for deeper fills associated with existing sewers and storm drains. We recommend that known utilities be shown on the grading plan. Wet utilities left in-place outside building areas should be capped to reduce the potential for water to infiltrate into the building pad.

The upper soils at the site are expected to have a moderate caving potential when exposed in open cuts. Temporary construction excavations may be made vertically into the undisturbed natural soils without shoring to a depth of 4 feet below adjacent grade. For cuts up to 10 feet deep, the slopes should be properly shored or sloped back to at least 1:1 or flatter. The allowable slope inclinations are measured from the toe to the top of the cut. The exposed slope face should be kept moist (but not saturated) during construction to reduce local sloughing. Surcharge loads should not be permitted within a horizontal distance equal to the height of cut from the top of the excavation or 5 feet from the top of the slopes, whichever is greater, unless the cut is properly shored. Excavations that extend below an imaginary plane inclined at 45 degrees below the edge of adjacent existing site facilities should be properly shored to maintain support of adjacent elements. Excavations and shoring systems should meet the minimum requirements given in the State of California Occupational Safety and Health Standards.

Deeper removals along property lines or adjacent to existing improvements will require shoring or slot cuts. Removals that will undermine existing adjacent pavements or hardscape may utilize “ABC” slot cuts to depths not greater than 8 feet. Unsurcharged slot cuts up to 8 feet in height should not be wider than 6 feet and should be backfilled to finished grade prior to excavation of the adjacent four slots (two on each side of the excavated slot). We should be provided details of the building and excavation to provide final recommendations before making the excavations. A test slot should be performed prior to production slots to confirm the stability of the planned cuts.

4.3.3 Subgrade Preparation

Prior to placing fills, the subgrade soils at the bottom of overexcavations should be scarified to a depth of 8 inches, moisture-conditioned, and compacted to at least 90 percent of the maximum dry density determined in accordance with ASTM D1557. Where the exposed soils are well over the optimum moisture content, subgrade processing (scarification) and be omitted.

The soils encountered within the upper 10 feet below existing grades have moisture contents varying from slightly moist to wet. The wet soils appear to be limited to thin layers of fine-grained soils (silts and clays). Therefore, localized areas of the subgrade soils to be exposed at the base of excavations are anticipated to exhibit well over-optimum moisture conditions. Subjecting these materials to heavy rubber-tired equipment will induce pumping/rutting, likely requiring deeper removals or subgrade stabilization. For planning purposes, stabilization can be performed by placing a geogrid such as Tensar BX1100 or equivalent and 12 inches of crushed miscellaneous base (CMB) placed in one lift and compacted to at least 90 percent.

4.3.4 Material for Fill

In general, the on-site soils can be used in general compacted fill except as discussed below. Moisture conditioning will likely be required prior to placing some of the in-place soils as compacted fill. While not anticipated at the site, clayey soils should not be used as retaining wall backfill, or within 12-inches below exterior hardscape and flatwork. Retaining wall backfill should consist of on-site or imported granular soils (containing no more than 40 percent fines – portion passing No. 200 sieve) and be relatively non-expansive (E.I. of 20 or less). These granular soils should be readily available in the near-surface soils at the site.

Imported fill material should be predominately granular (contain no more than 40 percent fines - portion passing No. 200 sieve) and non-expansive (E.I. less than 20). GPI should be provided with a sample (at least 50 pounds) and notified of the location of soils proposed for import at least 72 hours prior to importing. Each proposed import source should be sampled, tested and accepted for use prior to delivery of the soils to the site. Soils imported prior to acceptance by GPI may be rejected if not suitable.

Both imported and existing on-site soils to be used as fill should be free of debris and any pieces larger than 6 inches in greatest dimension. Fill materials placed within the depths of planned foundations should not contain particles larger than 3 inches in greatest dimension.

If open-graded gravel (such as pea gravel or ¾-inch crushed rock) is placed as backfill for the project, it should be separated from the on-site soils with a suitable filter fabric (Mirafi 140 N or equivalent) to mitigate the potential for migration of soil into the voids of the gravel. Open-graded gravel should be placed in lifts of 2 feet or less and densified with vibratory equipment prior to placing subsequent lifts.

On-site inert demolition debris, such as concrete and asphalt, may be reused in the compacted fills provided approval is obtained from the reviewing regulatory agency and the owner. The material should be crushed to the consistency of aggregate base and blended with the on-site or imported soils.

4.3.5 Placement and Compaction of Fills

Fill soils should be placed in horizontal lifts, moisture-conditioned, and mechanically compacted to densities equal to at least 90 percent of the maximum dry density, determined in accordance with ASTM D1557. The optimum lift thickness will depend on the compaction equipment used and can best be determined in the field. The following uncompacted lift thickness can be used as preliminary guidelines.

Plate compactors	4-6 inches
Small vibratory or static rollers (5-ton±), or track equipment	6-8 inches
Heavy loaders or heavy vibratory rollers	8-12 inches

The maximum lift thickness should not be greater than 8 inches and each lift should be thoroughly compacted and accepted prior to placement of subsequent lifts.

Fills consisting of on-site or imported granular soils should be placed at a moisture content of 0 to 2 percent over the optimum moisture content in order to achieve the required compaction and reduce the potential for future swelling. Fills consisting of silts and clays should be placed at a moisture content of 1 to 3 percent over the optimum moisture content.

Once moisture conditioned and properly compacted, the exposed soils should not be allowed to dry out prior to covering. If exposed soils are allowed to dry out, additional processing and moisture conditioning will be required. A representative of GPI should confirm the moisture content of the subgrade soils immediately prior to placement of concrete or additional fill.

During backfill of excavations, the fill should be properly benched into the construction slopes as it is placed in lifts.

4.3.6 Shrinkage and Subsidence

Shrinkage is the loss of soil volume caused by compaction of fills to a higher density than before grading. Subsidence is the settlement of in-place subgrade soils caused by loads generated by large earthmoving equipment. For earthwork volume estimating purposes, an average shrinkage value of 20 to 25 percent. Subsidence is expected to be less than 0.1 foot. These values are estimates only and exclude losses due to removal of vegetation or debris. Actual shrinkage and subsidence will depend on the types of earthmoving equipment used and should be determined during grading.

4.3.7 Trench/Wall Backfill

Utility trench backfill should be mechanically compacted in lifts. Retaining wall backfill should consist of on-site or imported granular soils. Lift thickness should not exceed those values given in the "Compacted Fill" section of this report. Jetting or flooding of backfill materials should not be permitted. A representative of GPI should observe and test trench and wall backfills as they are placed.

In backfill areas where mechanical compaction of soil backfill is impractical due to space constraints, sand-cement slurry may be substituted for compacted backfill. The slurry should contain at least one sack of cement per cubic yard and have a maximum slump of 5 inches.

4.3.8 Observation and Testing

A representative of GPI should observe excavations, subgrade preparation, and fill placement activities. Sufficient in-place field density tests should be performed during fill placement and in-place compaction to evaluate the overall compaction of the soils. Soils that do not meet minimum compaction requirements should be reworked and tested prior to placement of additional fill.

4.4 FOUNDATIONS

4.4.1 Foundation Type

The proposed restroom/equipment storage building may be supported on conventional isolated and/or continuous shallow footings. Other improvements related to the splash pad can be supported on shallow footings or drilled piles.

The subsurface soils under shallow footings for the building and minor structures should be prepared in accordance with the recommendations given in this report. Shallow footings should be supported on properly compacted fill. Footing bottoms should be moistened immediately prior to placement of concrete.

4.4.2 Conventional Shallow Footings

Allowable Bearing Pressure

Based on the shear strength and elastic settlement characteristics of the properly compacted fill and natural soils, a static allowable net bearing pressure of up to 2,500 pounds per square foot (psf) may be used for both continuous footings and isolated column footings.

The bearing pressures provided below are for dead-load-plus-live-load, and may be increased one-third for short-term, transient, wind and seismic loading. The actual bearing pressure used may be less than the value presented above and can be based on economics and structural loads to determine the minimum width for footings as discussed below. The maximum edge pressures induced by eccentric loading or overturning moments should not be allowed to exceed these recommended values.

The following minimum footing widths and embedments are recommended for the corresponding allowable bearing pressure for the building.

STATIC BEARING PRESSURE (psf)	MINIMUM FOOTING WIDTH (inches)	MINIMUM FOOTNG* EMBEDMENT (inches)
2,500	24	24
2,000	18	18
1,500	15	15

* Refers to minimum depth below lowest adjacent grade at the time of foundation construction.

A minimum footing width of 15 inches should be used even if the actual bearing pressure is less than 1,500 psf.

Estimated Settlements

We calculated settlements for the restroom/equipment storage building based on the assumed wall loads. Based on our analyses, the total static settlement under a maximum wall load of 1 to 2 kips per lineal foot is expected to be on the order of ¾-inch or less. Differential settlement along a 40-foot span of wall footing is anticipated to be on the order of ½-inch or less.

Total settlements of ½-inch or less are anticipated for any minor at-grade structures supported on properly compacted fill derived from on-site soils. Differential settlement between similarly loaded adjacent footings or along a 40-foot span is anticipated to be on the order of ¼-inch or less.

The above settlements should be included with the anticipated seismic settlement caused by seismic ground subsidence or liquefaction presented previously when evaluating the total and differential settlement of the building or other lightly loaded minor structures.

The above estimates are based on the assumption that the recommended earthwork will be performed and that the footings will be sized in accordance with our recommendations. If the structural loads change as the design of the project progresses, we should be notified in order to confirm the estimated settlement values provided above.

Lateral Load Resistance

Soil resistance to lateral loads will be provided by a combination of frictional resistance between the bottom of footings and underlying soils and by passive soil pressures acting against the embedded sides of the footings. For frictional resistance, a coefficient of friction of 0.3 may be used for design. In addition, an allowable lateral bearing pressure equal to an equivalent fluid weight of 200 pounds per cubic foot may be used, provided the footings are poured tight against competent natural soils or compacted fill. These values may be used in combination without reduction.

4.4.3 Drilled Pile Foundation Design Parameters

Axial Capacity

Drilled pile foundations will develop their frictional capacity with relatively small displacement (about ¼-inch). The allowable capacities will depend on the drilled pile diameter and the total depth of the drilled pile embedment below the finished grade. The values presented are for static loads and can be increased by one-third for short-term wind and seismic loads. The allowable uplift capacity for a given drilled pile may be taken as one-half of the compressive capacity calculated using the drilled pile parameters provided below.

Based upon the results of our subsurface explorations and laboratory testing, the drilled pile design parameters shown in the following tables may be used for axial pile design:

Recommended Design Parameters - Axial

Depth Below Existing Grade (feet)	Allowable Pile Skin Friction (psf)
0-4	0
4-8	200
8-12	300
12-16	350
>16	500

We utilized a factor of safety of 3 to determine the allowable skin friction values. Due to the difficulties associated with cleaning the bottom of the drilled excavations, we recommend that end bearing should not be used in the design of the drilled piles. The settlement of the drilled piles designed using the above allowable skin friction values only is anticipated to be on the order of ½-inch.

The uplift capacity of the piles may be taken as one-half of the downward capacities derived from skin friction only.

Lateral Capacity

Soil resistance to lateral loads can be provided by the piles. The design of the piles will likely be governed by lateral force considerations. For design by the simplified pole formula presented in Section 1807A.3.2.1 of the 2019 California Building Code, a unit passive resistance of 200 pounds per square may be used for the piles with level ground in lieu of the presumptive lateral bearing values presented in Table 1806A.2. As stated in the code, a passive resistance of 400 pounds per square foot per foot (to a maximum of 4,000 pounds per square foot) may be used for isolated piles as determined by the Project Structural Engineer. This value incorporates the allowable increase stated in the Section 1806A.3.4 of the code for single poles that can tolerate a ½-inch of deflection under short-term loads.

The above piles design parameters for shallow depths assume that the ground surface surrounding the piles will be paved with asphalt concrete. If the surface surrounding the pile is not paved, we recommend the upper 1 foot of soil be ignored in developing axial capacity and lateral resistance. An increased depth should be used where the piles will extend through a planter.

Installation

Since the drilled piles will be designed to derive resistance from soil friction only, rigorous cleaning of the loose materials from the bottom of the excavation prior to placement of steel and concrete is not considered essential. However, effort should be made to clean the bottom with the drill rig-mounted equipment.

Pile excavations should be filled with concrete on the same day that they are drilled. The concrete should be placed with special equipment so that it is not allowed to fall freely more than 5 feet or strike the walls of the excavations. Drilling should not be performed within 5 feet of recently excavated or recently poured piles until the concrete has been allowed to set for at least 6 hours. The piles should be poured in a manner that will not result in concrete flowing into adjacent drilled pile excavations and prevent segregation of aggregate. Drilled pile construction should be performed in accordance with the latest edition of ACI 336.1, “Standard Specifications for the Construction of Drilled Piles.”

Pile excavations should be observed by a representative of GPI to confirm and document the depth, diameter, and embedment in suitable materials.

4.4.4 Foundation Concrete

Laboratory testing by HDR indicates that the near surface soils exhibit a soluble sulfate content of 93 mg/kg (0.004 percent by weight). For the 2019 CBC, foundation concrete should conform to the requirements outlined in ACI 318, Section 4.3, for negligible levels of soluble sulfate exposure from the on-site soils, (Category S0). Chloride levels in the on-site soils were found to be low (85 ppm). For concrete exposed to soil moisture, such as footings and floor slabs, we recommend a chloride Category C1.

4.4.5 Foundation Excavation Observation

Prior to placement of concrete and steel, a representative of GPI should be observe and approve foundation excavations including pile excavations. Shallow footing excavations should be moistened immediately prior to concrete placement.

4.5 RETAINING WALLS

Based on information available to us at the time this report was prepared, retaining walls are not anticipated to be taller than 5 feet. The following recommendations are provided for walls less than 5 feet in height. We recommend that walls be properly drained and backfilled with sandy soils (less than 40 percent passing the No. 200 sieve).

Active earth pressures can be used for designing walls that can yield at least ½-inch laterally per 10 feet of wall height under the imposed loads. For level, drained backfill, derived from non-expansive granular soils (EI < 20), a lateral pressure of an equivalent fluid weighing of 38 pounds per cubic foot may be used. At-rest pressures should be used for restrained walls that remain rigid enough to be essentially non-yielding. For select, non-expansive, level, drained backfill, a lateral pressure of an equivalent fluid weighing 55 pounds per cubic foot can be used. If the wall backfill is not drained, the combined earth and water pressures will be much higher.

Walls subject to surcharge loads should be designed for an additional uniform lateral pressure equal to one-third and one-half the anticipated surcharge pressure for unrestrained and restrained walls, respectively.

The recommended pressures assume that the supported earth will be fully drained, preventing the build-up of hydro-static pressures. For traditional backfilled retaining walls, a drain consisting of perforated pipe and gravel, wrapped in a suitable filter fabric should be used. As a minimum, one cubic foot of rock should be used for each lineal foot of drain. The fabric (non-woven filter fabric, Mirafi 140N or equivalent) should be lapped at the top.

The Structural Engineer should specify the use of select, granular wall backfill on the plans. Wall footings should be designed as discussed in the "Foundations" section.

4.6 CORROSIVITY

Resistivity testing of a representative sample of the on-site soils indicates that they are corrosive to buried metals. We do not practice corrosion protection engineering. If buried metallic structures or pipe are planned, a Corrosion Engineer such as HDR should be consulted.

4.7 SURFACE DRAINAGE

Positive surface gradients should be provided adjacent to structures so as to direct surface water run-off and roof drainage away from foundations and slabs toward suitable discharge facilities. Long-term ponding of surface water should not be allowed on pavements or adjacent to buildings.

4.8 EXTERIOR CONCRETE FLATWORK

Exterior concrete or masonry pedestrian flatwork and concrete pavers should be supported on non-expansive compacted fill. The use of clayey soils, if encountered at the site, is not recommended within one foot of the flatwork subgrade. Prior to placement of concrete, the subgrade should be prepared as recommended in “Subgrade Preparation” section.

We recommend the Project Civil Engineer design the concrete hardscape and sidewalks, including determination of thickness and reinforcing. For exterior flatwork, hardscape, and sidewalks, we recommend a minimum slab thickness of 4 inches with minimum slab reinforcement of No. 3 rebar placed at 16 inches on-center, in both directions. Control joints to direct shrinkage cracking in exterior slabs and sidewalks should be provided at maximum spacing of 8 and 6 feet on center in two directions, respectively. These recommendations should be considered as minimums based on the geotechnical site conditions, and the Project

Civil Engineer should confirm if more stringent recommendations are needed for other purposes.

The concrete hardscape, stairs, flatwork, sidewalks, and pavers should be underlain by at least 4 inches of crushed aggregate base (CAB). Base course materials should be compacted to at least 95 percent of the maximum dry density (ASTM D1557). Aggregate base should conform to the requirements of Section 200-2 of the Standard Specifications for Public Works Construction (Green Book) for crushed aggregate base (CAB) or crushed miscellaneous base (CMB) materials. We recommend that concrete pavers have a minimum thickness of 60 mm (2.375 inches) for pedestrian applications in accordance with Interlocking Concrete Pavement Institute (ICPI), Tech Spec 10 (ICPI, 1999). We recommend that the pavers be placed on 25 mm (1 inch) thick of bedding sand in accordance.

The bedding sand for concrete pavers should be in accordance with the gradation provided in ASTM C-33. Mason sand should not be used as bedding because it can be slow draining and unstable. Bedding sand or mason sand may be used in the joints between pavers. Mason sand should be in accordance with the gradation provided in ASTM C-144. Installation of the pavers should be in accordance with the guidance in ICPI, 1999. We do not recommend a maintenance program with high pressure washing of the pavers be implemented unless a stabilized joint sand is used for the pavers.

4.9 PAVED AREAS

Preliminary vehicular pavement design has been based on an assumed R-value of 25. The California Division of Highways Design Method was used for design of the recommended preliminary pavement sections. The following pavement sections are recommended for planning purposes only.

PAVEMENT AREA	TRAFFIC INDEX	SECTION THICKNESS (inches)	
		ASPHALT/PORTLAND CONCRETE	AGGREGATE BASE COURSE
Asphalt Concrete	4.0	3.0	4.0
	5.0	3.0	7.0
	6.0	3.0	10.0
Portland Cement Concrete	4.0	6.0	4.0
	5.0	6.5	4.0
	6.0	7.0	4.0

The pavement subgrade underlying the aggregate base should be properly prepared and compacted in accordance with the recommendations outlined under "Subgrade Preparation”.

The concrete used for paving should have a modulus of rupture of at least 570 psi (equivalent to an approximate compressive strength of 3,500 psi) at the time the pavement is subjected to truck traffic.

The pavement base course should be compacted to at least 95 percent of maximum dry density (ASTM D-1557). Aggregate base should conform to the requirements of Section 26 of the California Department of Transportation Standard Specifications for Class II aggregate base (three-quarter inch maximum) or Section 200-2 of the Standard Specifications for Public Works Construction (Green Book) for untreated base materials (except processed miscellaneous base).

The above recommendations are based on the assumption that the base course and compacted subgrade will be properly drained. The design of paved areas should incorporate measures to prevent moisture build-up within the base course, which can otherwise lead to premature pavement failure. For example, curbing adjacent to landscaped areas should be deep enough to act as a barrier to infiltration of irrigation water into the adjacent base course.

4.10 INFILTRATION TESTING

To evaluate the infiltration characteristics for the proposed infiltration system, we performed two field infiltration tests (P-1 and P-2) located in the center of the splash pad. We performed the field infiltration test between approximate depths of 9 and 12 feet below the existing grades.

The test wells were installed in boreholes drilled using truck-mounted, 8-inch diameter hollow-stem auger drill equipment. The well consisted of 2-inch diameter PVC casing installed in the 8-inch diameter borehole and the annular space was backfilled with well sand to maintain an open hole. Prior to testing, the well was presoaked in general accordance with the requirements of the County. Evaluation of the infiltration rate began during the initial soak.

The test wells were filled with at least 2-feet of water at the initiation of each test. The tests were repeated at least 8 times and until a stabilized rate of water level drop was determined. A stabilized rate is defined as three or more consecutive readings in which the highest and lowest readings are within 10 percent of each other.

Once the stabilized field flowrate (volume per unit time) or drop was determined, we calculated the raw percolation rate by dividing the volume of water discharged by the wetted surface area of the test section in a given amount of time. The wetted area is composed of the sidewalls and bottom of the boring. The average of the stabilized raw percolation rate over the last three consecutive readings is the *measured percolation rate* at the test location and is typically expressed in units of inches per hour.

The measured percolation rates are provided below:

TEST WELL	APPROXIMATE DEPTH OF TEST WELL (feet)	CORRECTED INFILTRATION RATE (in./hr.)
P-1	12	2.8
P-2	9	5.1

The above measured percolation rate must be reduced to determine the design infiltration rate for the proposed infiltration BMP. Reduction factors (RF) account for the test method, site variability, and long-term siltation of the proposed system that should be determined during the design phase of the project. Typical values for these reduction factors are provided in the following table:

DESCRIPTION	REDUCTION FACTORS (RF)*
Boring Percolation	$RF_t = 2$
Site variability and thoroughness of subsurface investigation	RF_v = 1 to 3
Long-term siltation and maintenance	RF_s = 1 to 3

* Range of values provided in "Guidelines for Geotechnical Investigation and Reporting Low Impact Development Stormwater Infiltration", GS200.2, dated June 30, 2017

The reduction factor for test method, RF_t, should equal 2 because the boring percolation test method was used. Based on the number of borings performed and the uniformity of the subsurface conditions, it is our opinion that a reduction factor for site variability, RF_v, of 2 may be used. The reduction factor for long-term siltation and maintenance, RF_s, should be selected by the system designer.

Well No. and Approximate Depth	**P-1 (9.0 ft)**	**P-2 (12 ft)**
Measured Percolation Rate (in/hr)	2.8	5.1
Test Method: $\mathbf{RF_t}$	2	2
Site Variability and Thoroughness of Subsurface Investigation: $\mathbf{RF_v}$	2	2
Long-term Siltation and Maintenance: $\mathbf{RF_s}$	1 – 3	1 – 3
Design Infiltration Rate Range (inches/hour)	0.56-0.40	1.02-0.73

The infiltration rates at the locations and depths tested, indicate that stormwater infiltration at the site is feasible. The results of the infiltration tests are presented in Table 1, Borehole Infiltration Test Results.

4.11 GEOTECHNICAL OBSERVATION AND TESTING

We recommend that a representative of GPI observe earthwork during construction to confirm that the recommendations provided in our report are applicable during construction. The earthwork activities include grading, compaction of fills, subgrade preparation, and foundation excavations. If conditions are different than expected, we should be afforded the opportunity to provide an alternate recommendation based on the actual conditions encountered.

5.0 LIMITATIONS

This report, exploration logs, and other materials resulting from my efforts were prepared exclusively for use by the County of Los Angeles and their consultants in designing the proposed development. The report is not intended to be suitable for reuse on extensions or modifications of the project or for use on project other than the currently proposed development as it may not contain sufficient or appropriate information for such uses.

Soil deposits may vary in type, strength, and many other important properties between points of exploration due to non-uniformity of the geologic formations or to man-made cut and fill operations. While we cannot evaluate the consistency of the properties of materials in areas not explored, the conclusions drawn in this report are based on the assumption that the data obtained in the field and laboratory are reasonably representative of field conditions and are conducive to interpolation and extrapolation.

Furthermore, our recommendations were developed with the assumption that a proper level of field observation and construction review will be provided during grading, excavation, and foundation construction. If field conditions during construction appear to be different than is indicated in this report, we should be notified immediately so that we may assess the impact of such conditions on our recommendations. If construction phase services are performed by others, the client and new geotechnical firm must accept full responsibility for geotechnical aspects of the project, including this report.

Our investigation and evaluations were performed using generally accepted engineering approaches and principles available at this time and the degree of care and skill ordinarily exercised under similar circumstances by reputable geotechnical engineers practicing in this area. No other representation, either expressed or implied, is included or intended in our report.

Respectfully submitted,
Geotechnical Professionals Inc.

Jose Arellano
Staff Engineer

Paul R. Schade, G.E. 2371
Principal

REGISTERED PROFESSIONAL ENGINEER
PAUL R. SCHADE
No. 2371
GEOTECHNICAL
STATE OF CALIFORNIA

REFERENCES

American Society of Civil Engineers (2017), “Minimum Design Loads for Buildings and Other Structures,” ASCE/SEI 7-16.

California Department of Conservation, Division of Mines and Geology (1998), “Seismic Hazard Zone Report for the El Monte 7.5-Minute Quadrangle, Los Angeles County, California, Seismic Hazard Zone Report 024”.

California Geologic Survey, 1999, Earthquake Zones of Required Investigation, El Monte Quadrangle, released 1999

County of Los Angeles, Department of Public Works (2013), “Manual for Preparation of Geotechnical Reports”, dated July 1, 2013.

California Office of Statewide Health Planning and Development (OSHPD), Seismic Design Maps Website, https://seismicmaps.org/

United States Geologic Survey, 2008 National Seismic Hazard Maps – Source Parameters, http://earthquake.usgs.gov/cfusion/hazfaults_search

California Geological Survey, 2008, “Guidelines for Evaluating and Mitigating Seismic Hazards in California,” Special Publication 117A

Idriss, I.M. and Boulanger, R.W., 2008, “Soil Liquefaction during Earthquakes”, Earthquake Engineering Research Institute.

Interlocking Concrete Pavement Institute (ICPI), “Application Guide for Interlocking Concrete Pavements, Tech Spec 10, 1999.

http://www.historicaerials.com, Aerial Photography from the Past and Present”, National Environmental Title Research, LLC.

BASE PLAN REPRODUCED FROM CALTOPO © 2021

GEOTECHNICAL PROFESSIONALS, INC.

WHITTIER NARROWS SPLASH PAD

GPI PROJECT NO.: 2780.47I

SCALE: 1" = 2000'

SITE LOCATION MAP

FIGURE 1

N

0 50 100 FEET

EXPLANATION

B-5 APPROXIMATE LOCATION AND NUMBER OF EXPLORATORY BORING

P-2 APPROXIMATE LOCATION AND NUMBER OF INFILTRATION WELL

BASE PLAN REPRODUCED FROM SITE TEST FIT DIAGRAM
PROVIDED LAC PARKS AND RECREATION.: DATED FEBRUARY 2022

GPI GEOTECHNICAL PROFESSIONALS, INC.

WHITTIER NARROWS SPLASH PAD

GPI PROJECT NO.: 2780.47I | SCALE: 1" = 50'

SITE PLAN

FIGURE 2

APPENDIX A

APPENDIX A

EXPLORATORY BORINGS

The subsurface conditions at the site were investigated by drilling and sampling four exploratory borings. These borings were advanced to depths of 10 to 60 feet below the existing ground surface. The locations of the explorations are shown on the Exploration Location Plan, Figure 2.

Relatively undisturbed samples were obtained using a brass-ring lined sampler (ASTM D 3550). The brass-rings have an inside diameter of 2.42 inches. The ring samples were driven into the soil by a 140-pound hammer dropping 30 inches. The number of blows needed to drive the sampler into the soil was recorded as the penetration resistance.

At selected locations, disturbed samples were obtained using a split-spoon sampler by means of the Standard Penetration Test (SPT, ASTM D 6066). The spoon sampler was driven into the soil by a 140-pound hammer dropping 30 inches, employing the “free-fall” hammer described above. After an initial seating drive of 6 inches, the number of blows needed to drive the sampler into the soil a depth of 12 inches was recorded as the penetration resistance. These values are the raw uncorrected blow counts.

The field explorations for the investigation were performed under the continuous technical supervision of GPI's representative, who visually inspected the site, maintained detailed logs of the borings, classified the soils encountered, and obtained relatively undisturbed samples for examination and laboratory testing. The soils encountered in the borings were classified in the field and through further examination in the laboratory in accordance with the Unified Soils Classification System. Detailed logs of the borings are presented in Figures A-1 through A-5 in this appendix.

The boring and sampling locations were laid out in the field by measuring from existing site features. Ground surface elevations at the exploration locations were estimated from the Google Earth and should be considered very approximate.

MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	*DESCRIPTION OF SUBSURFACE MATERIALS* This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	ELEVATION (FEET)
			B	0	Fill: **SANDY SILT (ML)** grey brown, moist, trace clay	
21.9	77	31	D		@ 2 feet, wet, very stiff	210
8.7	92	34	D	5	Natural: **SILTY SAND (SM)** grey brown, moist, medium dense	
3.5	96	37	D		**SAND (SP)** brown, slightly moist, medium dense, trace gravel	205
41.3	79	15	D	10	**SILT (ML)** grey, wet, stiff, with sand	
35.1		6	S		@ 11 feet, firm	200
44.0		16	S			
4.5		38	S	15	**SAND (SP)** light brown, slightly moist, dense	
3.8		41	S		@ 17 feet, dry	195
1.6		63	S	20	@ 19 feet,very dense, trace gravel	
						190
2.1		51	S	25		
						185
3.7		72	S	30		
						180
38.9		29	S	35	**SANDY SILT (ML)** brown, wet, very stiff	
						175
12.7		50/6"	S		**SILTY SAND (SM)** brown, very moist, very dense	

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED:
2-25-22

EQUIPMENT USED:
8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft):
43

GPI

PROJECT NO.: 2780.47I

WHITTIER NARROWS SPLASH PAD

LOG OF BORING NO. B-1

FIGURE A-1

MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
				40	**SILTY SAND (SM)** brown, wet, very dense	170
6.8		50/6"	S	45	**SAND WITH SILT (SP-SM)** brown, moist, very dense, with gravel	165
7.8		92/11"	S	50		160
				55		155
20.2		50/5"	S		@ 58.5 feet, wet	
				60	Total Depth 60 feet	

This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED: 2-25-22

EQUIPMENT USED: 8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft): 43

GPI

PROJECT NO.: 2780.47I

WHITTIER NARROWS SPLASH PAD

LOG OF BORING NO. B-1

FIGURE A-1

MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
				0	Fill: **SILTY SAND (SM)** brown, slightly moist, trace clay	210
11.9			B		@ 2 feet, moist	
8.0	94	51	D		Natural: **SAND (SP)** brown, moist, dense	
				5	**CLAYEY SILT (ML)** dark brown, slightly moist, hard, with sand	205
5.5	102	39	D		**SAND (SP)** light brown, slightly moist, medium dense	
4.5	84	31	D	10		
					Total Depth 10 feet	

DESCRIPTION OF SUBSURFACE MATERIALS: This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED: 2-25-22

EQUIPMENT USED: 8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft): Not Encountered

GPI

PROJECT NO.: 2780.47I

WHITTIER NARROWS SPLASH PAD

LOG OF BORING NO. B-2

FIGURE A-2

	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	*DESCRIPTION OF SUBSURFACE MATERIALS* This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	ELEVATION (FEET)
					0	Fill: **SILTY SAND (SM)** brown, slightly moist, trace clay	
	4.9			B			210
	4.6	95	46	D	5	Natural: **SAND (SP)** brown, slightly moist, dense	
	34.8	81	22	D		**SILTY CLAY (CL)** brown, wet, stiff to very stiff, trace sand	205
	3.8	94	53	D	10	**SAND (SP)** light brown, dry, dense	
						Total Depth 10 feet	

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED:
2-25-22

EQUIPMENT USED:
8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft):
Not Encountered

GPI

PROJECT NO.: 2780.47I

WHITTIER NARROWS SPLASH PAD

LOG OF BORING NO. B-3

FIGURE A-3

	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	ELEVATION (FEET)
					0	Fill: **SILTY SAND (SM)** brown, moist, trace clay	210
	30.0			B		@ 2 feet, wet	
	12.0	77	35	D	5	Natural: **SILTY SAND (SM)** brown, moist, medium dense	
	38.8	59	14	D		**SILT (ML)** brown, wet, stiff, trace clay, with roots	205
	13.9	80	18	D	10	**SILTY SAND (SM)** brown, very moist, medium dense	
						Total Depth 10 feet	

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED:
2-25-22

EQUIPMENT USED:
8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft):
Not Encountered

GPI

PROJECT NO.: 2780.47I

WHITTIER NARROWS SPLASH PAD

LOG OF BORING NO. B-4

FIGURE A-4

MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	*DESCRIPTION OF SUBSURFACE MATERIALS* This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	ELEVATION (FEET)
				0	Fill: **SILTY SAND (SM)** brown, slightly moist, trace clay	
						210
25.2			B		@ 2 feet, wet	
10.3	80	17	D		Natural: **SILTY SAND (SM)** brown, moist, medium dense	
				5		
16.0	84	18	D		**SANDY SILT (ML)** dark grey, moist, stiff	
					SILTY SAND (SM) brown, very moist, medium dense	205
16.0	84	25	D		@ 9 feet, very stiff	
				10	Total Depth 10 feet	

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED:
2-25-22

EQUIPMENT USED:
8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft):
Not Encountered

GPI

PROJECT NO.: 2780.47I

WHITTIER NARROWS SPLASH PAD

LOG OF BORING NO. B-5

FIGURE A-5

APPENDIX B

APPENDIX B

LABORATORY TESTS

INTRODUCTION

Representative undisturbed soil samples, and bulk samples were carefully packaged in the field and sealed to prevent moisture loss. The samples were then transported to our Cypress office for examination and testing assignments. Laboratory tests were performed on selected representative samples as an aid in classifying the soils and to evaluate the physical properties of the soils affecting foundation design and construction procedures. Detailed descriptions of the laboratory tests are presented below under the appropriate test headings. Test results are presented in the figures that follow.

MOISTURE CONTENT AND DRY DENSITY

Moisture content and dry density were determined from a number of the ring samples from the boring. The samples were first trimmed to obtain volume and wet weight and then were dried in accordance with ASTM D 2216. After drying, the weight of each sample was measured, and moisture content and dry density were calculated. Moisture content and dry density values are presented on the boring log in Appendix A.

PERCENT PASSING NO. 200 SIEVE

A number of soil samples were dried, weighed, soaked in water until individual soil particles were separated, and then washed on the No. 200 sieve. That portion of the material retained on the No. 200 sieve was oven-dried and weighed to determine the percentage of the material passing the No. 200 sieve. A summary of the percentages passing the No. 200 sieve is presented below.

BORING NO.	DEPTH (ft)	SOIL DESCRIPTION	PERCENT PASSING No. 200 SIEVE
B-1	0-5	Sandy Silt (ML)	62
B-1	6	Sand (SP)	2
B-1	13	Silt (ML)	85
B-1	24	Sand (SP)	5

ATTERBERG LIMITS

The liquid and plastic limits were determined for three select cohesive soil samples in accordance with ASTM D4318. The results of the Atterberg Limits tests are presented in Figure B-1.

EXPANSION INDEX TEST

An expansion index was performed on a bulk sample. The test was performed in accordance with ASTM D4829 to assess the expansion potential of the on-site fill soils. The results of the test are summarized below.

BORING NO.	DEPTH (ft)	SOIL DESCRIPTION	EXPANSION INDEX, EI	EXPANSION POTENTIAL
B-1	0-5	Sandy Silt (ML)	5	Very Low

DIRECT SHEAR

Two direct shear tests were performed on undisturbed and remolded samples in accordance with ASTM D 3080. The bulk sample was remolded to approximately 90 percent of maximum density (ASTM D1557). The samples were placed in the shear machine, and pre-selected normal loads were applied. The samples were inundated, allowed to consolidate, and then were sheared to failure at a strain rate of 0.001 to 0.002 inches per minute. The tests were repeated on additional test specimens under increased normal loads. Shear stress and sample deformation were monitored throughout the test. The results of the direct shear tests are presented in Figures B-2 to B-3.

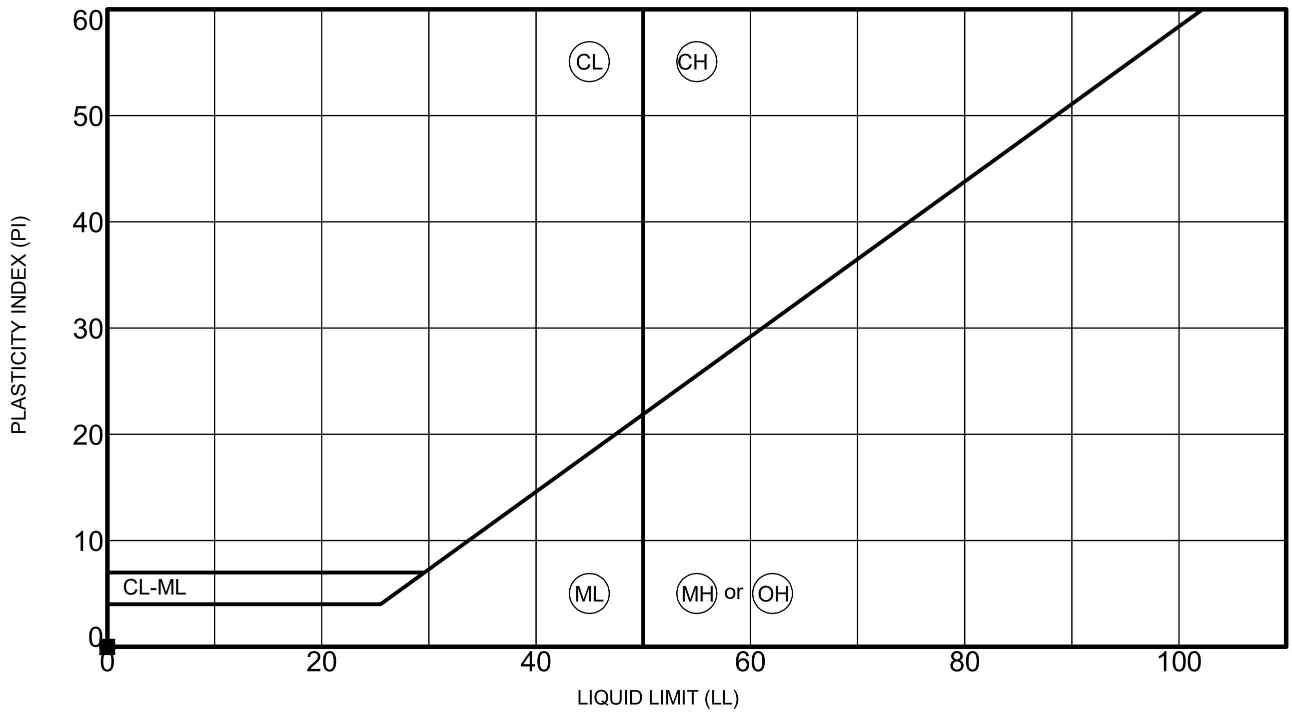
COMPACTION TEST

A maximum dry density/optimum moisture test was performed in accordance with ASTM D 1557 on a representative bulk sample of the on-site soils. The test results are as follows.

BORING NO.	DEPTH (ft)	SOIL DESCRIPTION	OPTIMUM MOISTURE (%)	MAXIMUM DRY DENSITY (pcf)
B-1	0-5	Sandy Silt (ML)	13.5	113

CORROSIVITY

Soil corrosivity testing was performed by HDR a soil sample provided by GPI. The test results are summarized in Table 1 of this Appendix.



	SAMPLE LOCATION		LL	PL	PI	Fines, %	Classification
●	B-1	11.0	NP	NP	NP		SILT (ML)
⧗	B-1	34.0	NP	NP	NP		SANDY SILT (ML)

PROJECT: WHITTIER NARROWS SPLASH PAD

PROJECT NO. 2780.47I

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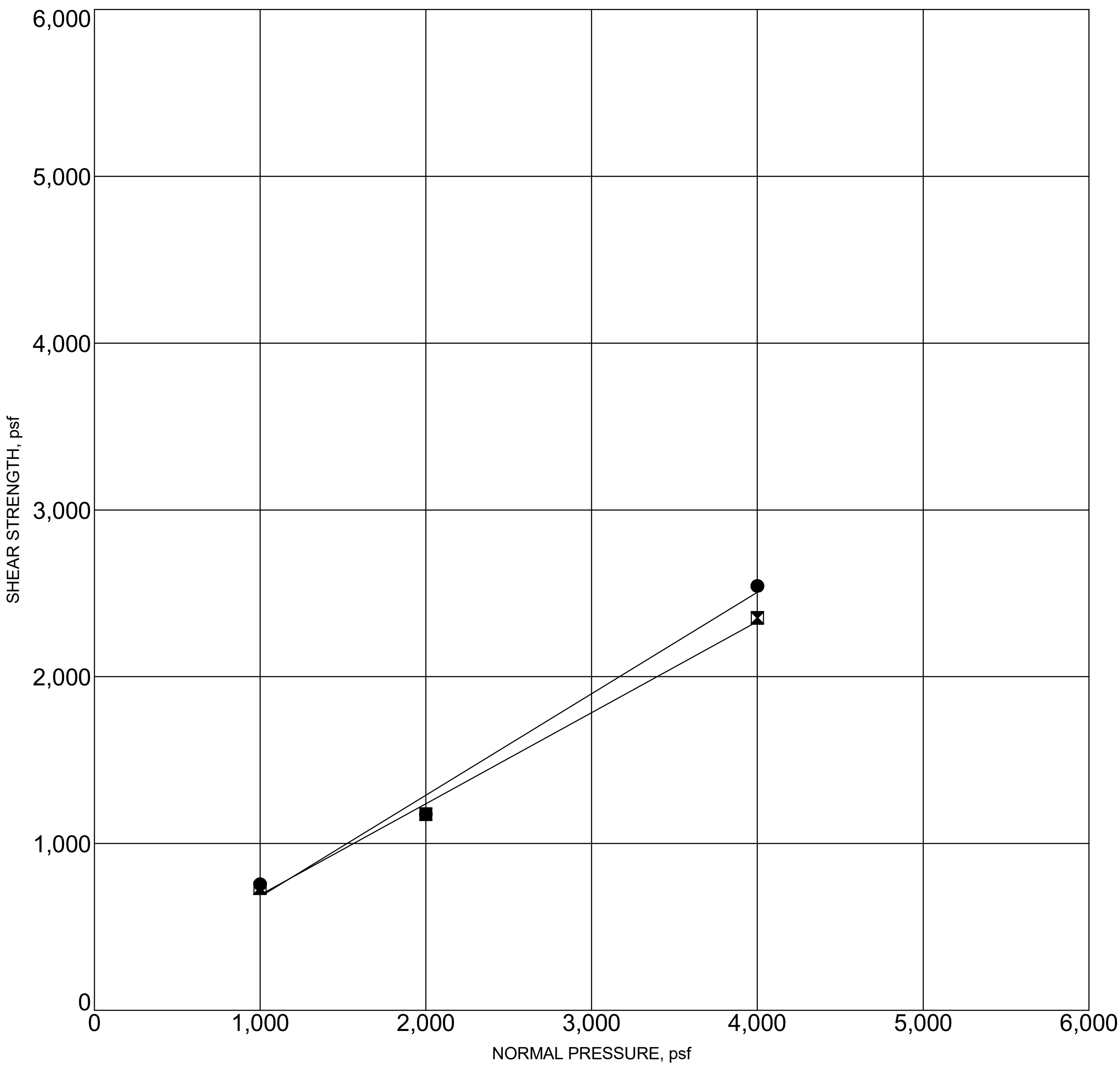
ATTERBERG LIMITS TEST RESULTS

FIGURE B-1



● **PEAK STRENGTH**
Friction Angle= 31 degrees
Cohesion= 72 psf

⧗ **ULTIMATE STRENGTH**
Friction Angle= 29 degrees
Cohesion= 144 psf

Note: Samples Remolded to 90% of maximum dry density.

Sample Location		Classification	DD,pcf	MC,%
B-1	0-5	SANDY SILT (ML)	103	13.5

PROJECT: WHITTIER NARROWS SPLASH PAD PROJECT NO.2780.47I

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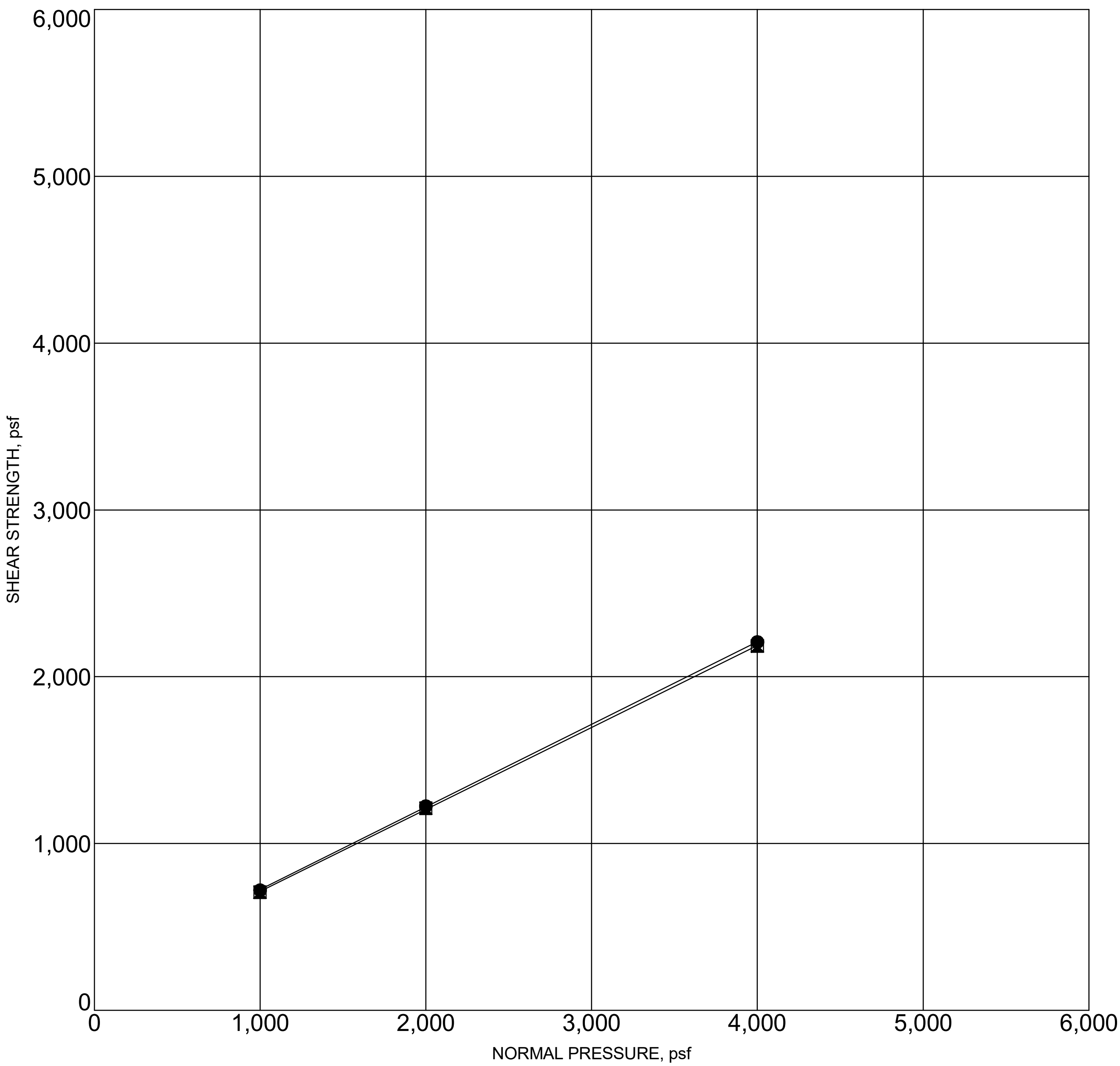
DIRECT SHEAR TEST RESULTS

FIGURE B-2



● **PEAK STRENGTH**
Friction Angle= 26 degrees
Cohesion= 228 psf

⧗ **ULTIMATE STRENGTH**
Friction Angle= 26 degrees
Cohesion= 222 psf

Sample Location		Classification	DD,pcf	MC,%
B-4	6.0	SILT (ML)	59	38.8

PROJECT: WHITTIER NARROWS SPLASH PAD PROJECT NO.2780.47I

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DIRECT SHEAR TEST RESULTS

FIGURE B-3

Appendix A

Whittier Narrows Splash Pad – Soils Report Letter
Dated 12/08/23

December 8, 2023

County of Los Angeles Department of Public Works
Project Management Division
900 S. Fremont Avenue, 5th Floor
Alhambra, California 91803-1331

Attention: Courtney Tossounian, Architect
Project Manager

Subject: Supplemental Geotechnical Recommendations – Below-Grade Tank
Proposed Splash Pad and Site Improvements
Area D - Whittier Narrows Regional Recreational Area
750 S. Santa Anita Avenue, South El Monte, California
Contract PW-15015
GPI Project No. 2780.47I

Dear Courtney:

As requested, this letter presents supplemental geotechnical recommendations for the subject project related to a below-grade balance tank. We performed our geotechnical investigation for the project and issued our findings and recommendations in a report dated April 1, 2022. Except where specifically appended or modified herein, the conclusions and recommendations presented in our prior report remain applicable and should be used for design and construction of the proposed development.

Based on updated information, a below-grade balance tank is planned for the project with the walls of the tank extending between 8 and 10 feet below the ground surface. Our prior report provided recommendations for retaining walls up to 5 feet in height, and this letter updates those recommendations for walls retaining up to 10 feet of soil. We recommend that walls be backfilled with sandy soils (less than 40 percent passing the No. 200 sieve). Sandy soils are anticipated to be readily available on-site.

Active earth pressures can be used for designing walls that can yield at least ½-inch laterally per 10 feet of wall height under the imposed loads. For level, drained backfill, derived from non-expansive granular soils (EI < 20), a lateral pressure of an equivalent fluid weighing of 38 pounds per cubic foot may be used. At-rest pressures should be used for restrained walls that remain rigid enough to be essentially non-yielding, which are more appropriate for tanks or vaults. For select, non-expansive, level, drained backfill, a lateral pressure of an equivalent fluid weighing 55 pounds per cubic foot can be used for the at-rest pressure. If the wall/tank backfill is not drained, the combined earth and water pressures will be higher, as discussed below.

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A seismic lateral pressure should be used for the design of retaining walls supporting 6 feet or more of backfill. We recommend a seismic lateral pressure of 25 pounds per cubic foot be added to the active earth pressure recommended above. If at-rest pressure is used to design the retaining walls/tank, the total lateral pressure used (at-rest plus seismic) is not required to exceed the total active plus seismic pressure (63 pounds per cubic foot).

Walls subject to surcharge loads should be designed for an additional uniform lateral pressure equal to one-third and one-half the anticipated surcharge pressure for unrestrained and restrained walls, respectively.

The recommended pressures above assume that the supported earth will be fully drained, preventing the build-up of hydro-static pressures. For traditional backfilled retaining walls, a drain consisting of perforated pipe and gravel, wrapped in a suitable filter fabric should be used. As a minimum, one cubic foot of rock should be used for each lineal foot of drain. The fabric (non-woven filter fabric, Mirafi 140N or equivalent) should be lapped at the top. If walls below grade are not drained, we recommend waterproofing and an additional lateral pressure of 47 pounds per cubic foot be added to the above lateral earth pressures.

The Structural Engineer should specify the use of select, granular wall backfill on the plans. Wall footings should be designed as discussed in the "Foundations" section.

Please contact us if you have questions regarding our letter or need further assistance.

Very truly yours,
Geotechnical Professionals Inc.

Paul R. Schade, G.E. 2371
Principal (pauls@gpi-ca.com)

REFERENCE

Geotechnical Professionals Inc. (2022), "Geotechnical Investigation, Proposed Splash Pad and Site Improvements, Area D – Whittier Narrows Regional Recreational Area, 750 S. Santa Anita Avenue, South El Monte, California," GPI Project No. 2780.47I, dated April 1, 2022

Appendix A

Whittier Narrows Splash Pad – Soils Report Letter
Dated 07/10/25

GEOTECHNICAL
PROFESSIONALS INC.

July 10, 2025

County of Los Angeles Department of Public Works
Project Management Division
900 S. Fremont Avenue, 5th Floor
Alhambra, California 91803-1331

Attention: Courtney Tossounian, Architect
Project Manager

Subject: Update to Geotechnical Investigation Report
Proposed Splash Pad and Site Improvements
Area D - Whittier Narrows Regional Recreational Area
750 S. Santa Anita Avenue, South El Monte, California
Contract PW-15015/GPI Project No. 2780.47I

Dear Courtney:

As requested, this letter presents an update to our geotechnical investigation report provided for the subject project, dated April 1, 2022. We understand that the project is currently in plan check and the County requires an update letter from us to confirm the suitability of our report to the currently planned project. You have provided a current set of the project plans for our review, dated March 14, 2025.

We reviewed the current project plans and confirmed that the currently proposed project is consistent with the project planned at the time of our initial investigation. We provided seismic design parameters for the 2019 CBC, but we understand that the current project plans reference the 2022 CBC/2023 LABC. Both versions of the code reference ASCE 7-16 for determination of the seismic design parameters, such that there is no change required in parameters. Given the above findings, it is our opinion that our April 1, 2022 report and supplemental letter dated December 8, 2023 continue to be suitable for the currently planned project.

We trust that this letter satisfies your current needs. Please contact us if you have questions regarding our letter or need further assistance.

Very truly yours,
Geotechnical Professionals Inc.

Jose Arellano

Jose Arellano
Staff Engineer

Paul R. Schade, G.E. 2371
Principal (pauls@gpi-ca.com)

REGISTERED PROFESSIONAL ENGINEER
PAUL R. SCHADE
No. 2371
GEOTECHNICAL
STATE OF CALIFORNIA

Attachment: References

5736 Corporate Avenue • Cypress, CA 90630 • (714) 220-2211 , FAX (714) 220-2122

REFERENCE

Geotechnical Professionals Inc. (2022), "Geotechnical Investigation, Proposed Splash Pad and Site Improvements, Area D – Whittier Narrows Regional Recreational Area, 750 S. Santa Anita Avenue, South El Monte, California," GPI Project No. 2780.47I, dated April 1, 2022

Geotechnical Professionals Inc. (2023), "Supplemental Geotechnical Recommendations – Below-Grade Tank, Proposed Splash Pad and Site Improvements, Area D – Whittier Narrows Regional Recreational Area, 750 S. Santa Anita Avenue, South El Monte, California," GPI Project No. 2780.47I, dated December 23, 2023