June 1, 2023 Project No. 21291-01

Ms. Lovisa Kjerrgren
SWA Group
570 Glenneyre Street
Laguna Beach, CA 92651

Subject: ***Addendum to Updated Geotechnical Evaluation for Proposed Parnell Park Improvements, 15390 Lambert Road, Whittier, California***

Introduction

In accordance with your request, LGC Geotechnical, Inc. has prepared this letter as an addendum to the Updated Geotechnical Evaluation for Proposed Parnell Park Improvements, 15390 Lambert Road, Whittier California. A concrete balance tank is proposed for the Parnell Park Splash Pad. It is our understanding that the proposed retaining structure will have an approximate depth of 11 feet.

This addendum should be considered as part of the project design documents in conjunction with our previous geotechnical report (LGC Geotechnical, 2023). In the case of conflict, the recommendations contained herein should supersede those provided in our previous report. The remaining recommendations provided in our previous geotechnical report (LGC Geotechnical, 2023) remain valid and applicable.

Lateral Earth Pressures and Retaining Wall Design Considerations

Lateral earth pressures are provided as equivalent fluid unit weights, in pound per square foot (psf) per foot of depth or pcf. The values for the active or at-rest condition do not contain an appreciable factor of safety, so the retaining wall designer should apply the applicable factors of safety and/or load factors during design. A soil unit weight of 120 pcf may be assumed for calculating the actual weight of soil over the wall footing.

Lateral earth pressures are presented in Tables 1 and 2 on the following page. Tables 1 and 2 contain values for approved select sandy backfill and native backfill for both drained and undrained conditions. Typically, retaining walls are designed for a drained condition with select sandy soils used as backfill. Should retaining walls have constraints limiting the ability to replace the retained soil with select sandy soils, or have an undrained condition, design parameters in Table 2 should be used. Approved select sandy soils should have a sand equivalent of 30 or greater as determined by ASTM D2419. Please note, in order to utilize the lateral earth pressures of select sandy soils, the width of the sand zone should be a minimum of half the retained height.

The retaining wall designer should clearly indicate on the retaining wall plans if select sandy soil backfill is required. Soil meeting these criteria may be present at the site. Possible sources should be sampled and tested for compliance.

If the wall can yield enough to mobilize the full shear strength of the soil, it can be designed for "active" pressure. If the wall cannot yield under the applied load, the earth pressure will be higher. This would include 90-degree corners of retaining walls. Such walls should be designed for "at-rest." The equivalent fluid pressure values assume free-draining conditions and a drainage system will be installed and maintained to prevent the build-up of hydrostatic pressures. If conditions other than those assumed above are anticipated, the equivalent fluid pressure values should be provided on an individual-case basis by the geotechnical engineer.

TABLE 1

Lateral Earth Pressures – Select Sand Backfill

Condition	**Equivalent Fluid Weight (pcf)**	
	Level Backfill	
	Approved Sandy Backfill Material - Drained	**Approved Sandy Backfill Material - Undrained**
Active	35	80
At-Rest	55	90

TABLE 2

Lateral Earth Pressures – Native Backfill

Condition	**Equivalent Fluid Weight (pcf)**	
	Level Backfill	
	Native Backfill Material - Drained	**Native Backfill Material - Undrained**
Active	45	85
At-Rest	65	95

The drained equivalent fluid pressure values assume free-draining conditions and a drainage system will be installed and maintained to prevent the build-up of hydrostatic pressures. A relatively sandy backfill along with a subdrain pipe wrapped in drainage aggregate and filter fabric (e.g., "burrito" subdrain) properly outletted to a suitable discharge point is typically used for conventional retaining walls. Retaining wall structures should be provided with appropriate drainage and appropriately waterproofed. If conditions other than those assumed above are anticipated, the equivalent fluid pressure values should be provided on an individual-case basis by the geotechnical consultant.

Surcharge loading effects from any adjacent structures should be evaluated by the retaining wall designer. In general, structural loads within a 1:1 (horizontal to vertical) upward projection from the bottom of the proposed retaining wall footing will surcharge the proposed retaining structure. Uniform surcharges may be estimated using the applicable coefficient of lateral earth pressure using a rectangular distribution. A factor of 0.5 and 0.33 may be used for at-rest and active conditions, respectively. The retaining wall designer should contact the geotechnical engineer for any required geotechnical input in estimating any applicable surcharge loads.

If required, the retaining wall designer may use a seismic lateral earth pressure increment of 25 pcf for retaining walls up to a maximum of 11 feet in height with level backfill. This increment should be applied in addition to the applicable static lateral earth pressure using a triangular distribution with the resultant acting at H/3 in relation to the base of the retaining structure (where H is the retained height). For the restrained, at-rest condition, the seismic increment may be added to the applicable active lateral earth pressure (in lieu of the at-rest lateral earth pressure) when analyzing short duration seismic loading. Per Section 1803.5.12 of the 2019 CBC, the seismic lateral earth pressure is applicable to structures assigned to Seismic Design Category D through F for retaining wall structures supporting more than 6 feet of backfill height. This seismic lateral earth pressure is estimated using the procedure outlined by the Structural Engineers Association of California (Lew, et al, 2010). The provided seismic lateral earth pressure is for a level backfill condition and a maximum of 11 feet in height; a sloping backfill condition and greater retaining wall heights are not anticipated. However, if a sloping backfill condition or retaining walls greater than 11 feet in height are proposed, the retaining wall designer should contact the geotechnical engineer for specific seismic lateral earth pressure increments based on the configuration of the planned retaining wall structures.

Resistance to lateral loads can be provided by friction acting at the base of foundations and by passive earth pressure. For concrete/soil frictional resistance, an allowable coefficient of friction of 0.30 may be assumed with dead-load forces. An allowable passive lateral earth pressure of 250 psf per foot of depth (or pcf) to a maximum of 2,500 psf may be used for the sides of footings poured against properly compacted fill. This passive pressure is applicable for level (ground slope equal to or flatter than 5H:1V) conditions only. The passive pressure may be increased by one-third due to wind or seismic forces. We recommend that the upper foot of passive resistance be neglected if finished grade will not be covered with concrete or asphalt. Frictional resistance and passive pressure may be used in combination without reduction. These lateral and frictional resistance values represent ultimate values, so appropriate safety and/or load factors should be applied by the structural engineer. The provided allowable passive pressures are based on a factor of safety of 1.5 and 1.1 for static and seismic loading conditions, respectively.

An allowable soil bearing pressure of 1,500 pounds per square foot (psf) may be used for the design of footings having a minimum width of 12 inches and minimum embedment of 12 inches below lowest adjacent ground surface. This value may be increased by 300 psf for each additional foot of embedment or 100 psf for each additional foot of foundation width to a maximum value of 3,000 psf. These allowable bearing pressures are applicable for level (ground slope equal to or flatter than 5H:1V) conditions only. Bearing values indicated are for total dead loads and frequently applied live loads and may be increased by ⅓ for short duration loading (i.e., wind or seismic loads).

In utilizing the above-mentioned allowable bearing capacity, and provided our earthwork recommendations are implemented, foundation settlement due to structural loads is anticipated to be 1-inch. Differential settlement may be taken as half of the total settlement (i.e., ½-inch over a horizontal span of 40 feet).

Closure

Our services were performed using the degree of care and skill ordinarily exercised, under similar circumstances, by reputable soils engineers and geologists practicing in this or similar localities. No other warranty, expressed or implied, is made as to the conclusions and professional advice included in this report.

Should you have any questions regarding this letter, please do not hesitate to contact our office. We appreciate this opportunity to be of service.

Sincerely,

LGC Geotechnical, Inc.

Blake J. Elliott

REGISTERED PROFESSIONAL ENGINEER
BLAKE J. ELLIOTT
No. 70705
CIVIL
STATE OF CALIFORNIA

Blake J. Elliott, RCE 70705
Project Engineer

BJE/CMP/amm

Attachment: References

Distribution: (1) Addressee (electronic copy)

<u>References</u>

California Building Standards Commission, 2022 California Building Code, California Code of Regulations Title 24, Volumes 1 and 2, dated July 2019.

LGC Geotechnical, 2023, Updated Geotechnical Evaluation for Proposed Parnell Park Improvements, 15390 Lambert Road, Whittier, California, Project No. 21291-01, dated May 5, 2023

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________, 2014, Unified Hazard Tool, Dynamic: Conterminous U.S. 2014 (update) (v4.2.0), Retrieved July 26, 2022, from: https://earthquake.usgs.gov/hazards/interactive/

SWA, 2022a, Parnell Park Concept Plan, dated April 18, 2022

________, 2022b, Preliminary Grading Model, undated

LGC
Geotechnical, Inc.

May 5, 2023 Project No. 21291-01

Ms. Lovisa Kjerrgren
SWA Group
570 Glenneyre Street
Laguna Beach, CA 92651

Subject: ***Updated Geotechnical Evaluation for Proposed Parnell Park Improvements, 15390 Lambert Road, Whittier, California***

In accordance with your request, LGC Geotechnical, Inc. is providing a geotechnical evaluation for the proposed Parnell Park improvements, located at 15390 Lambert Road, in the City of Whittier, California. This report presents the results of our limited subsurface explorations and geotechnical analysis and provides a summary of our conclusions and recommendations relative to the proposed improvements.

Should you have any questions regarding this report, please do not hesitate to contact our office. We appreciate this opportunity to be of service.

Sincerely,

LGC Geotechnical, Inc.

REGISTERED PROFESSIONAL ENGINEER
BLAKE J. ELLIOTT
No. 70705
CIVIL
STATE OF CALIFORNIA

Blake J. Elliott, RCE 70705
Project Engineer

CERTIFIED ENGINEERING GEOLOGIST
BARRY GRAHAM
No. 2749
STATE OF CALIFORNIA

Barry Graham, CEG
Project Geologist

Clarissa Pappo, EIT
Staff Engineer

BJE/BPG/CMP/amm

Distribution: (1) Addressee (electronic copy)

131 Calle Iglesia, Suite 200, San Clemente, CA 92672 (949) 369-6141 www.lgcgeotechnical.com

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1.0 INTRODUCTION

1.1 Purpose and Scope of Services

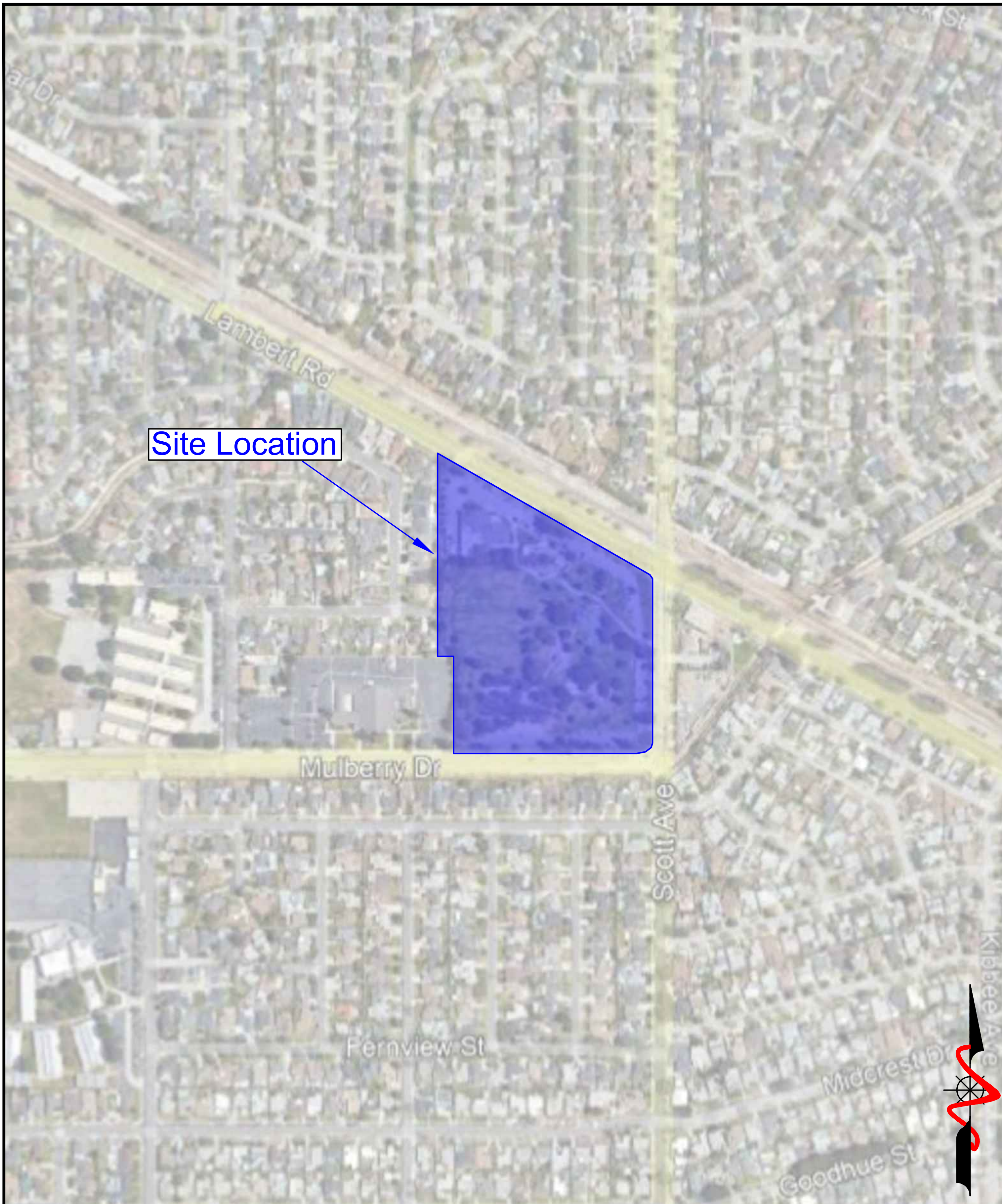
The purpose of our work was to evaluate site geotechnical conditions and to provide geotechnical recommendations with respect to the proposed improvements.

As part of this report, we have: 1) reviewed available geotechnical reports and in-house geologic maps pertinent to the site; 2) performed a subsurface geotechnical evaluation of the site consisting of the excavation of 4 small-diameter borings ranging from approximately 7 to 51.5 feet below existing ground surface; 3) performed infiltration testing on one of the small diameter borings; 4) excavated two hand auger borings 5) performed laboratory testing of soil samples obtained during our subsurface evaluation; and 6) prepared this summary report presenting our findings, conclusions, and geotechnical recommendations with respect to the proposed site improvements.

1.2 Existing Site Conditions and Proposed Development

The subject site is bound to the north by Lambert Road, to the south by Mulberry Drive, to the west by Scott Avenue and to the east by a residential and commercial area. The site is currently a park with a Civic Center building and petting zoo in the northern portion and various structures exist throughout the park including an office building, concrete paths to the playground and basketball court, and a water feature. Parking areas are currently bound the northern, eastern, and southern perimeter of the site. The site is relatively flat-lying with grades dropping approximately 10 feet to the west across the site.

Based upon the Parnell Park Concept Plan (SWA, 2022a) and preliminary grading model (SWA, 2022b), we understand that the proposed improvements will consist of minor grading, expansion of the petting zoo, construction of various hardscape improvements and event areas with synthetic turf and overhead lighting, and additional recreational spaces will be developed.



LGC Geotechnical, Inc.

FIGURE 1
Site Location Map

PROJECT NAME	SWA-Parnell Park
PROJECT NO.	21291-01
ENG. / GEOL.	BJE / BPG
SCALE	Not To Scale
DATE	May 2023

1.3 Subsurface Exploration

A geotechnical evaluation of the site was performed by LGC Geotechnical on July 21 and 22, 2022. The exploration program consisted of drilling and sampling four small-diameter exploratory borings and two hand-auger borings to evaluate onsite geotechnical conditions. The subsurface exploration was performed to evaluate the general engineering characteristics of the onsite materials.

Borings HS-1, HS-2, HS-3, and I-1 were drilled by MR Drilling under subcontract to LGC Geotechnical. The depth of borings HS-1 and HS-2 were approximately 50 feet below existing grade. Boring HS-3 was drilled at approximately 10 feet below existing grade. Boring I-1 was also drilled and used for shallow boring percolation testing, with a depth of approximately 7 feet below existing grade. An LGC Geotechnical representative observed the drilling operations, logged the borings, and collected soil samples for laboratory testing. The borings were performed using a truck mounted hollow stem rig equipped with 8-inch diameter hollow-stem augers. Driven soil samples were collected by means of the Modified California Drive (MCD) sampler generally obtained at 2.5 and 5-foot vertical increments. The MCD is a split-barrel sampler with a tapered cutting tip and lined with a series of 1-inch-tall brass rings. The MCD sampler (2.4-inch ID, 3.0-inch OD) was driven using a 140-pound automatic hammer falling 30 inches to advance the sampler a total depth of 18 inches. The raw blow counts for each 6-inch increment of penetration were recorded on the boring logs. Bulk samples of the near-surface soils were also collected and logged at select borings for laboratory testing. At the completion of drilling and/or infiltration testing, the borings were backfilled with the native soil cuttings, and tamped. Some settlement of the backfill soils may occur over time.

Two hand auger borings were excavated to depths ranging from 1 to 3 feet below ground surface to gather additional subsurface information.

Boring Logs are presented in Appendix B and their approximate locations are depicted on Figure 2 – Boring Location Map.

1.4 Field Infiltration Testing

One field infiltration test was performed in boring I-1. A 3-inch-diameter perforated PVC pipe was placed in the borehole, and the annulus was backfilled with gravel including placement of approximately 2 inches of gravel at the bottom of the borehole. The infiltration well was pre-soaked prior to testing. Estimation of the infiltration rate was accomplished in general accordance with the guidelines set forth by the County of Los Angeles (2021). At completion of infiltration testing, the pipe was abandoned and backfilled with cuttings and tamped. Some settlement of the backfill should be expected.

In general, three-dimensional flow out of the test well (percolation), as observed in the field, is mathematically corrected to one-dimensional flow out of the bottom of the test well (infiltration). Infiltration testing was performed using relatively clean water, free of particulates, silt, etc. The results are presented in Appendix D.

The tested infiltration rates provided in this report are considered a general representation of the infiltration rates at the location of the proposed infiltration boring. Please note, the testing of infiltration rates is highly dependent upon the materials encountered at the point of testing (i.e., location of depth of testing). Varying subsurface conditions may exist outside of the test location which could alter the calculated infiltration rate. Please refer to section 4.2.

1.5 Laboratory Testing

Representative driven and bulk samples were retained for laboratory testing during our field evaluation. Laboratory testing included in-situ density and moisture content, fines content, Atterberg limits, direct shear, expansion index, R-value, and corrosion sulfate. A summary of the laboratory test results is presented in Appendix C.

- Dry density of the samples collected ranged from approximately 102 pounds per cubic foot (pcf) to 118 pcf, with an average of 109 pcf. Field moisture contents ranged from approximately 8 to 30 percent, with an average of 17 percent.
- Two fines content tests were performed and indicated a fines content (passing No. 200 sieve) ranging from approximately 61 to 82 percent. Based on the Unified Soils Classification System (USCS), the tested samples would be classified as “fine-grained.”
- Four Atterberg Limit (liquid and plastic limit) tests were performed. Results indicated Plasticity Index (PI) values ranging from 12 to 28.
- One Direct shear test was performed on a driven ring sample. The Shear vs. Normal Stress plot is provided in Appendix C.
- Two expansion potential tests were performed and indicated expansion index values of 55 and 94 corresponding to a “Medium” and “High” expansion potentials, respectively.
- One R-value test was performed on a bulk sample collected and resulted in an R-Value of 16.
- Corrosion testing indicated a soluble sulfate content of less than 0.02 percent, a chloride content of approximately 144 parts per million (ppm), pH of 7.62, and a minimum resistivity of 498 ohm-centimeters.

A summary of the laboratory test results is presented in Appendix C. The moisture and dry unit weight results are presented on the boring logs in Appendix B.

2.0 GEOTECHNICAL CONDITIONS

2.1 Regional Geology

The subject site is located within the Peninsular Ranges Geomorphic Province of California, more specifically within the Downey Plains region, south of the Puente Hills. The site is located on laterally extensive alluvial fan deposits generated from the nearby canyons in the Puente Hills to the north and from the Rio Hondo and San Gabriel River drainages that run south from the San Gabriel Valley through an area called the Whittier Narrows. Regional topography is mostly flat lying to the south of the site, with hills to the north of the site defined by the steeper and overturned stratigraphy of the Whittier fault zone. The region has a complex geologic history influenced by periods of uplift, folding, faulting, and alluvial deposition; however, no faults are known to transect the site.

2.2 Generalized Subsurface Conditions

The subsurface evaluation indicates that site soils underlying the site generally consist of medium dense to very dense silty to clayey sands with varying amounts of stiff to hard silts and sandy clays to the maximum explored depth of approximately 51.5 feet below existing grade.

It should be noted that borings are only representative of the location and time where/when they are performed, and varying subsurface conditions may exist outside of the performed location. In addition, subsurface conditions can change over time. The soil descriptions provided above should not be construed to mean that the subsurface profile is uniform, and that soil is homogeneous within the project area. For details on the stratigraphy at the exploration locations, refer to Appendix B.

2.3 Groundwater

During our subsurface evaluation, groundwater was encountered at approximate depths of 18 and 21 feet below the ground surface. Historic high groundwater is estimated to be at approximately 30 feet below existing ground surface (CDMG, 1998).

Seasonal fluctuations of groundwater elevations should be expected over time. In general, groundwater levels fluctuate with the seasons and local zones of perched groundwater may be present due to local seepage caused by irrigation and/or recent precipitation. Local perched groundwater conditions or surface seepage may develop once site development is completed.

2.4 Faulting

Prompted by damaging earthquakes in California, State legislation and policies concerning the classification and land-use criteria associated with faults have been developed. Their purpose was to prevent the construction of urban developments across the trace of active faults, resulting in the Alquist-Priolo Earthquake Fault Zoning Act. Earthquake Fault Zones have been delineated along the traces of active faults within California. Where developments for human occupation are proposed within these zones, the State requires detailed fault evaluations be performed so that engineering geologists can mitigate the hazards associated with active faulting by identifying the location of active faults and allowing for a setback from zones of previous ground rupture.

The subject site is <u>not</u> located within an Alquist-Priolo Earthquake Fault Zone and no faults were identified on the site during our site evaluation (CGS, 2018). The possibility of damage due to ground rupture is considered low since no active faults are known to cross the site.

There is a potential for significant ground shaking across the entire site during a strong seismic event. ***New improvements will need to be designed for seismic forces in accordance with current building codes and regulations.***

Secondary effects of seismic shaking resulting from large earthquakes on the major faults in the Southern California region, which may affect the site, include ground lurching, shallow ground rupture, soil liquefaction and dynamic settlement. These secondary effects of seismic shaking are a possibility throughout the Southern California region and are dependent on the distance between the site and causative fault and the onsite geology. Some of the major active nearby faults that could produce these secondary effects include the Whittier Fault Zone, San Andreas Fault Zone, Compton Fault and Elysian Park Fault, among others. A discussion of these secondary effects is provided in the following sections.

2.4.1 <u>Liquefaction and Dynamic Settlement</u>

Liquefaction is a seismic phenomenon in which loose, saturated, granular soils behave similarly to a fluid when subject to high-intensity ground shaking. Liquefaction occurs when three general conditions coexist: 1) shallow groundwater; 2) low density non-cohesive (granular) soils; and 3) high-intensity ground motion. Studies indicate that saturated, loose to medium dense, near surface cohesionless soils exhibit the highest liquefaction potential, while dry, dense, cohesionless soils and cohesive soils exhibit low to negligible liquefaction potential. In general, cohesive soils are not considered susceptible to liquefaction. Effects of liquefaction on level ground include settlement, sand boils, and bearing capacity failures below structures.

Based on our review of the State of California Seismic Hazard Zone for liquefaction potential (CDMG, 1998), a portion of the subject site is located within a liquefaction hazard zone. Liquefaction potential was evaluated using the procedures outlined by Special Publication 117A (SCEC, 1999 & CGS, 2008) and the applicable seismic criteria (e.g., 2022 CBC).

Historic high groundwater is mapped at a depth of approximately 30 feet below existing grade (CDMG, 2001). Due to shallower groundwater conditions relative to the historic high groundwater, a 15-foot depth to groundwater was used in our liquefaction analysis. The alluvial soils encountered below a depth of approximately 15 feet were generally found to be very dense and generally not susceptible to liquefaction. The soils tested are cohesive and not considered to be susceptible to liquefaction based on their Plasticity Index and saturated moisture content compared to their Liquid Limit (Bray & Sancio, 2006). The potential for liquefaction and liquefaction-induced settlement is considered low. Based on the data obtained from our field evaluation, liquefaction settlement analysis is provided in Appendix F.

2.4.2 Lateral Spreading

Lateral spreading is a type of liquefaction-induced ground failure associated with the lateral displacement of surficial blocks of sediment resulting from liquefaction in a subsurface layer. Once liquefaction transforms the subsurface layer into a fluid mass, gravity plus the earthquake inertial forces may cause the mass to move downslope towards a free face (such as a river channel or an embankment). Lateral spreading may cause large horizontal displacements and such movement typically damages pipelines, utilities, bridges, and structures.

Due to the lack of liquefiable materials, the potential for lateral spreading is considered very low.

2.5 Seismic Design Criteria

The site seismic characteristics were evaluated per the guidelines set forth in Chapter 16, Section 1613 of the 2022 California Building Code (CBC) and applicable portions of ASCE 7-16 which has been adopted by the CBC. Please note that the following seismic parameters are only applicable for code-based acceleration response spectra and are not applicable for where site-specific ground motion procedures are required by ASCE 7-16. Representative site coordinates of latitude 33.9376 degrees north and -118.0030 degrees west were utilized in our analyses. The maximum considered earthquake (MCE) spectral response accelerations (SMS and SM1) and adjusted design spectral response acceleration parameters (SDS and SD1) for Site Class D are provided in Table 1 on the following page. Since site soils are Site Class D, additional adjustments are required to code acceleration response spectrums as outlined below and provided in ASCE 7-16. The structural designer should contact the geotechnical consultant if structural conditions (e.g., number of stories, seismically isolated structures, etc.) require site-specific ground motions.

A deaggregation of the PGA based on a 2,475-year average return period (MCE) indicates that an earthquake magnitude of 6.8 at approximately 9.84 km from the site would contribute the most to this ground motion. A deaggregation of the PGA based on a 475-year average return period (Design Earthquake) indicates that an earthquake magnitude of 6.69 at a distance of approximately 15.00 km from the site would contribute the most to this ground motion (USGS, 2014).

Section 1803.5.12 of the 2022 CBC (per Section 11.8.3 of ASCE 7) states that the maximum considered earthquake geometric mean (MCE_G) Peak Ground Acceleration (PGA) should be used for liquefaction potential. The PGA_M for the site is equal to 0.835g (SEAOC, 2022).

TABLE 1

Seismic Design Parameters

Selected Parameters from 2022 CBC, Section 1613 - Earthquake Loads	Seismic Design Values	Notes/Exceptions
Distance to applicable faults classifies the site as a "Near-Fault" site.		Section 11.4.1 of ASCE 7
Site Class	D*	Chapter 20 of ASCE 7
Ss (Risk-Targeted Spectral Acceleration for Short Periods)	1.750g	From SEAOC, 2022
S_1 (Risk-Targeted Spectral Accelerations for 1-Second Periods)	0.623g	From SEAOC, 2022
F_a (per Table 1613.2.3(1))	1.000	For Simplified Design Procedure of Section 12.14 of ASCE 7, F_a shall be taken as 1.4 (Section 12.14.8.1)
F_v (per Table 1613.2.3(2))	1.700	Value is only applicable per requirements/exceptions per Section 11.4.8 of ASCE 7
S_{MS} for Site Class D [Note: $S_{MS} = F_aS_S$]	1.750g	-
S_{M1} for Site Class D [Note: $S_{M1} = F_vS_1$]	1.059g	Value is only applicable per requirements/exceptions per Section 11.4.8 of ASCE 7
S_{DS} for Site Class D [Note: $S_{DS} = (^2/_3)S_{MS}$]	1.167g	-
S_{D1} for Site Class D [Note: $S_{D1} = (^2/_3)S_{M1}$]	0.706g	Value is only applicable per requirements/exceptions per Section 11.4.8 of ASCE 7
C_{RS} (Mapped Risk Coefficient at 0.2 sec)	0.901	ASCE 7 Chapter 22
C_{R1} (Mapped Risk Coefficient at 1 sec)	0.901	ASCE 7 Chapter 22
*Since site soils are Site Class D and S_1 is greater than or equal to 0.2, the seismic response coefficient Cs is determined by Eq. 12.8-2 for values of $T \leq 1.5T_s$ and taken equal to 1.5 times the value calculated in accordance with either Eq. 12.8-3 for $T_L \geq T > T_s$, or Eq. 12.8-4 for $T > T_L$. Refer to ASCE 7-16.		

2.6 Expansion Potential

Based on the results of our laboratory testing, site soils are anticipated to have a "Medium" to "High" expansion potentials.

3.0 CONCLUSIONS

Based on the results of our geotechnical evaluation, it is our opinion that the proposed improvements are feasible from a geotechnical standpoint, provided the following conclusions and recommendations are implemented.

The following is a summary of the primary geotechnical factors that may affect future improvements at the site:

- Based on our subsurface exploration and regional geologic mapping, the site is underlain by Quaternary older alluvium deposits.
- Groundwater was encountered during our subsurface evaluation at depths of 18 and 2 feet below the ground surface. Historic high groundwater is estimated to be approximately 30 feet below current grade (CDMG, 1998).
- The subject site is not located within an Alquist-Priolo Earthquake Fault Zone and no faults were identified on the site during our site evaluation. The possibility of damage due to ground rupture is considered low since no active faults are known to cross the site.
- The main seismic hazard that may affect the site is ground shaking from one of the active regional faults. The subject site will likely experience strong seismic ground shaking during its design life.
- The subject site is located in a seismic hazard zone for liquefaction (CDMG, 1998). However, the potential for liquefaction is considered low.
- Based on field observations and findings, site soils have “Medium” to “High” expansion potentials.
- From a geotechnical perspective, the existing onsite soils are suitable material for use as fill, provided they are relatively free from rocks (larger than 8 inches in maximum dimension), construction debris, and significant organic material.

4.0 PRELIMINARY RECOMMENDATIONS

The following recommendations are to be considered preliminary and should be confirmed upon completion of grading and earthwork operations. In addition, they should be considered minimal from a geotechnical viewpoint, as there may be more restrictive requirements from the architect, structural engineer, building codes, governing agencies, or the owner.

It should be noted that the following geotechnical recommendations are intended to provide sufficient information to develop the site in general accordance with the 2022 CBC requirements. With regard to the potential occurrence of potentially catastrophic geotechnical hazards such as fault rupture, earthquake-induced landslides, liquefaction, etc. the following geotechnical recommendations should provide adequate protection for the proposed development to the extent required to reduce seismic risk to an "acceptable level." The "acceptable level" of risk is defined by the California Code of Regulations as "that level that provides reasonable protection of the public safety, though it does not necessarily ensure continued structural integrity and functionality of the project" [Section 3721(a)]. Therefore, repair and remedial work of the proposed improvements may be required after a significant seismic event. With regards to the potential for less significant geologic hazards to the proposed development, the recommendations contained herein are intended as a reasonable protection against the potential damaging effects of geotechnical phenomena such as expansive soils, fill settlement, groundwater seepage, etc. It should be understood, however, that although our recommendations are intended to maintain the structural integrity of the proposed development and structures given the site geotechnical conditions, they cannot preclude the potential for some cosmetic distress or nuisance issues to develop as a result of the site geotechnical conditions.

The geotechnical recommendations contained herein must be confirmed to be suitable or modified based on the actual as-graded conditions.

4.1 Site Earthwork

We anticipate that earthwork at the site will consist of the required earthwork removals, precise grading, and construction of the proposed new improvements.

We recommend that earthwork onsite be performed in accordance with the following recommendations, the 2022 CBC and the General Earthwork and Grading Specifications for Rough Grading included in Appendix E. In case of conflict, the following recommendations shall supersede those included in Appendix E. The following recommendations should be considered preliminary and may be revised within the future grading plan review report or based on the actual conditions encountered during site grading.

4.1.1 Site Preparation

Prior to grading of areas to receive structural fill or engineered improvements, the areas should be cleared of existing concrete, surface obstructions, and demolition debris. Vegetation and debris should be removed and properly disposed of off-site. Holes resulting from the removal of buried obstructions, which extend below proposed finish grades, should be replaced with suitable compacted fill material.

4.1.2 Removal Depths and Subgrade Preparation

In general, removal bottom areas, and any areas to receive compacted fill and the subgrade below the synthetic turf should be cross-ripped to a minimum depth of 12 inches, brought to a near-optimum moisture condition, and re-compacted per project recommendations. Removal bottoms and areas to receive fill should be observed and accepted by the geotechnical consultant prior to subsequent fill placement.

Local conditions may be encountered during excavation that could require additional over-excavation beyond the above-noted minimum in order to obtain an acceptable subgrade including localized areas of undocumented fill. The actual depths and lateral extents of grading will be determined by the geotechnical consultant, based on subsurface conditions encountered during grading. Removal bottoms and areas to receive fill should be observed and accepted by the geotechnical consultant prior to subsequent fill placement.

4.1.3 Temporary Excavations

Temporary excavations may be necessary during the construction of the infiltration system. Temporary excavations should be performed in accordance with project plans, specifications, and all Occupational Safety and Health Administration (OSHA) requirements. Excavations should be laid back or shored in accordance with OSHA requirements before personnel or equipment are allowed to enter.

Based on our field evaluation, we anticipate OSHA Type "C" soils. Soil conditions should be regularly evaluated during construction to verify conditions are as anticipated. The contractor shall be responsible for providing the "competent person," required by OSHA standards, to evaluate soil conditions. Sandy soils are present and should be considered susceptible to caving. Close coordination with the geotechnical consultant should be maintained to facilitate construction while providing safe excavations. Excavation safety is the sole responsibility of the contractor.

Vehicular traffic, stockpiles, and equipment storage should be set back from the perimeter of excavations a distance equivalent to a 1:1 projection from the bottom of the excavation. Once an excavation has been initiated, it should be backfilled as soon as practical. Prolonged exposure of temporary excavations may result in some localized instability. Excavations should be planned so that they are not initiated without sufficient time to shore/fill them prior to weekends, holidays, or forecasted rain.

It should be noted that any excavation that extends below a 1:1 (horizontal to vertical) projection of an existing foundation will remove existing support of the structure foundation. If requested, temporary shoring parameters will be provided.

4.1.4 Material for Fill

From a geotechnical perspective, the onsite soils are generally considered suitable for use as general compacted fill, provided they are screened of organic materials, construction debris and oversized material (8 inches in greatest dimension).

From a geotechnical viewpoint, any required import soils for general fill (i.e., non-retaining wall backfill) should consist of clean, granular soils of "Medium" expansion potential (expansion index between 51 and 90 based on ASTM D 4829), and free of organic materials, construction debris and any material greater than 3 inches in maximum dimension. Source samples should be provided to the geotechnical consultant for laboratory testing a minimum of four working days prior to any planned importation.

Although not shown on the preliminary site plan, if retaining walls are later proposed, any required retaining wall backfill should consist of sandy soils with a sand equivalent equal to or greater than 30 (ASTM D2419). Soils should also be screened of organic materials, construction debris, and any material greater than 3 inches in maximum dimension. The site contains soils are not suitable for retaining wall backfill due to their expansion potential.

Aggregate base (crushed aggregate base or crushed miscellaneous base) should conform to the requirements of Section 200-2 of the Standard Specifications for Public Works Construction ("Greenbook") for untreated base materials (except processed miscellaneous base) or Caltrans Class 2 aggregate base.

4.1.5 Placement and Compaction of Fills

Material to be placed as fill should be brought to near-optimum moisture content (generally within optimum and 2 percent above optimum moisture content) and recompacted to at least 90 percent relative compaction (per ASTM D1557). Moisture conditioning of site soils will be required in order to achieve adequate compaction. The optimum lift thickness to produce a uniformly compacted fill will depend on the type and size of compaction equipment used. In general, when using larger equipment, fill should be placed in uniform lifts not exceeding 8 inches in compacted thickness. For smaller equipment, lifts should not exceed 6 inches in thickness. Each lift should be thoroughly compacted and accepted prior to subsequent lifts. Generally, placement and compaction of fill should be performed in accordance with local grading ordinances and with observation and testing performed by the geotechnical consultant. Oversized material as previously defined should be removed from site fills.

During backfill of excavations, the fill should be properly benched into firm and competent soils of temporary backcut slopes as it is placed in lifts.

Aggregate base material should be compacted to at least 95 percent relative compaction at or slightly above optimum moisture content per ASTM D1557. Subgrade below aggregate base should be compacted to at least 90 percent relative compaction per ASTM D1557 at or slightly above optimum moisture content.

4.1.6 Trench Backfill and Compaction

The onsite soils may generally be suitable as trench backfill, provided the soils are screened of rocks and other material greater than 6 inches in diameter and organic matter. If trenches are shallow or the use of conventional equipment may result in damage to the utilities, sand having a sand equivalent (SE) of 30 or greater (per California Test Method [CTM] 217) may be used to bed and shade the pipes. Sand backfill within the pipe bedding zone may be densified by jetting or flooding and then tamping to ensure adequate compaction. Subsequent trench backfill should be compacted in uniform thin lifts by mechanical means to at least 90 percent relative compaction (per ASTM D1557).

4.2 Subsurface Water Infiltration

Recent regulatory changes have occurred that mandate that storm water be infiltrated below grade into subsurface soils rather than be collected in a conventional storm drain system. Typically, a combination of methods may be implemented to reduce surface water runoff and increase infiltration including permeable pavements/pavers for roadways and walkways, directing surface water runoff to grass-lined swales, retention areas, drywells, etc.

It should be noted that collecting and concentrating surface water for the purpose of intentionally infiltrating it below grade, conflicts with the geotechnical engineering objective of directing surface water away from slopes, structures, and other improvements. The geotechnical stability and integrity of a site is reliant upon appropriately handling surface water. In general, we do not recommend that surface water be intentionally infiltrated into subsurface soils. Due to the shallow groundwater at this site, if there is an option to not infiltrate storm water, we would recommend this.

If it is determined that water must be infiltrated due to regulatory requirements, we recommend the absolute minimum amount of water be infiltrated and that the infiltration areas not be located near slopes or near settlement sensitive existing/proposed improvements. We recommend the design of any infiltration system include at least one redundancy or overflow system. It may be prudent to provide an overflow system connected directly to a storm drain system in order to prevent failure of the infiltration system, either as a result of lower than anticipated infiltration with time and/or very high flow volumes.

As with all systems that are designed to concentrate surface flow and direct the water into the subsurface soils, some minor settlement, nuisance type localized saturation and/or other water related issues should be expected. Due to variability in geologic and hydraulic conductivity characteristics, these effects may be experienced at the onsite location and/or potentially at other locations beyond the physical limits of the subject site. Infiltrated water may enter underground utility pipe zones or flow along heterogeneous soil layers or geologic structure and migrate laterally impacting other improvements which may be located far away or at an elevation much lower than the infiltration source.

Adequate distances should be maintained between infiltration locations and structures. The invert of any storm water infiltration system should be set back a minimum of 15 feet from building structures and outside a 1:1(horizontal to vertical) plane drawn up from the bottom of adjacent foundations.

The observed infiltration rate (no factor of safety) of 0.0 inches per hour was obtained from field infiltration testing. The design infiltration rate is unchanged due to an observed rate of 0.0 inches per hour, however other site suitability and design considerations including factors of safety can be provided upon request.

4.3 Synthetic Turf

Per reviewed plans, the proposed sports field will be made of synthetic turf. We defer to manufacturer specifications for turf underlayment. Base material and subgrade soils placed under the turf should be compacted with heavy earthmoving equipment and placed under observation by a representative of LGC Geotechnical. Subgrade and aggregate base should be compacted to at least 90 percent relative compaction per ASTM D1557 at or slightly above optimum moisture content. The subgrade/perimeter drainage trench separation barrier should be modified for specific site conditions (see Synthetic Turf Base Courses 2.1.B.1.c for plastic and moisture sensitive soils).

All other specifications shown on the plan set should be followed during construction of the synthetic turf fields.

4.4 Preliminary Foundation Design Parameters

Given that the expansion index exceeds 20, the foundation systems shall be designed for effects of expansive soil. Preliminary conventional and post-tensioned foundation recommendations are provided in the following sections. Recommended soil bearing and estimated static settlement are provided in Section 4.5. Please note that the following foundation recommendations are preliminary and must be confirmed by LGC Geotechnical at the completion of project plans (i.e., foundation, grading and site layout plans) as well as completion of earthwork.

4.4.1 Provisional Conventional Foundation Design Parameters

Conventional foundations may be designed in accordance with Wire Reinforcement Institute (WRI) procedure for slab-on-ground foundations per Section 1808 of the 2022 CBC to resist expansive soils. The following preliminary soil parameters may be used:

- Effective Plasticity Index: 30
- Climatic Rating: Cw = 15
- Reinforcement: Per structural designer.
- Moisture condition subgrade soils to 100 % of optimum moisture content to a depth of 24 inches prior to trenching for footings.

4.4.2 Preliminary Post-Tensioned Foundation Design Recommendations

Given that the expansion index exceeds 20, the foundation system shall be designed for effects of expansive soil. It is our understanding that a post-tensioned foundation is preferred over a conventionally reinforced foundation. The geotechnical parameters provided herein may be used for post-tensioned slab foundations with a deepened perimeter footing or a post-tensioned mat slab. These parameters have been determined in general accordance with the Post-Tensioning Institute (PTI) Standard Requirements for Design of Shallow Post-Tensioned Concrete Foundations on Expansive Soils, referenced in Chapter 18 of the 2022 CBC. In utilizing these parameters, the foundation engineer should design the foundation system in accordance with the allowable deflection criteria of applicable codes and the requirements of the structural designer/architect. Other types of stiff slabs may be used in place of the CBC post-tensioned slab design provided that, in the opinion of the foundation structural designer, the alternative type of slab is at least as stiff and strong as that designed by the CBC/PTI method.

Our design parameters are based on our experience with similar projects, test results, and the anticipated nature of the soil (with respect to expansion potential). Please note that implementation of our recommendations will not eliminate foundation movement (and related distress) should the moisture content of the subgrade soils fluctuate. It is the intent of these recommendations to help maintain the integrity of the proposed structures and reduce (not eliminate) movement, based upon the anticipated site soil conditions. Should future owners and/or property maintenance personnel not properly maintain the areas surrounding the foundation, for example by overwatering, then we anticipate for highly expansive soils the maximum differential movement of the perimeter of the foundation to the center of the foundation to be on the order of a couple of inches. Soils of lower expansion potential are anticipated to show less movement.

TABLE 2

Preliminary Geotechnical Foundation Design Parameters

Parameter	PT Slab with Perimeter Footing	PT Mat with Thickened Edge
Expansion Index	High[1]	High[1]
Thornthwaite Moisture Index	-20	-20
Constant Soil Suction	PF 3.9	PF 3.9
Center Lift Edge moisture variation distance, e_m Center lift, y_m	 7.7 feet 0.75 inch	 7.7 feet 0.90 inch
Edge Lift Edge moisture variation distance, e_m Edge lift, y_m	 4.0 feet 1.65 inches	 4.0 feet 2.0 inch
Modulus of Subgrade Reaction, k (assuming presoaking as indicated below)	100 pci	100 pci
Minimum perimeter footing/thickened edge embedment below finish grade	24 inches	6 inches

1. Assumed for preliminary design purposes. Further evaluation is needed at the completion of grading.
2. Recommendations for foundation reinforcement and slab thickness are ultimately the purview of the foundation engineer/structural engineer based upon geotechnical criteria and structural engineering considerations.
3. Recommendations for sand below slabs have traditionally been included with geotechnical foundation recommendations, although they are not the purview of the geotechnical consultant. The sand layer requirements are the purview of the foundation engineer/structural engineer, and should be provided in accordance with ACI Publication 302 "Guide for Concrete Floor and Slab Construction".
4. Recommendations for vapor retarders below slabs are also the purview of the foundation engineer/structural engineer and should be provided in accordance with applicable code requirements.
5. Moisture condition to 140 % of optimum moisture content to a depth of 24 inches prior to trenching.

4.4.3 Post-Tensioned Foundation Subgrade Preparation and Maintenance

Pre-soaking of the subgrade for building slabs will be required due to site expansive soils. The duration of this process varies greatly based on the chosen method and is also dependent on factors such as soil type and weather conditions. Time duration for presoaking from completion of rough grading to trenching of foundations should be accounted for in the construction schedule (typically 1 to 3 weeks). The recommendations, specific to anticipated site soil conditions, are presented in Table 2. The subgrade moisture condition of the building pad soils should be maintained at the recommended moisture content up to the time of concrete placement. This moisture content should be maintained around the immediate perimeter of the slab during construction and up to occupancy of the building structures.

The geotechnical parameters provided in Table 2 assume that if the areas adjacent to the foundation are planted and irrigated, these areas will be designed with proper drainage and adequately maintained so that ponding, which causes significant moisture changes below the foundation, does not occur. Our recommendations do not account for excessive irrigation and/or incorrect landscape design. Plants should only be provided with sufficient irrigation for life and not overwatered to saturate subgrade soils. Sunken planters placed adjacent to the foundation should either be designed with an efficient drainage system or liners to prevent moisture infiltration below the foundation. Some lifting of the perimeter foundation beam should be expected even with properly constructed planters.

In addition to the factors mentioned above, future owners/property management personnel should be made aware of the potential negative influences of trees and/or other large vegetation. Roots that extend near the vicinity of foundations can cause distress to foundations. Future owners (and the owner's landscape architect) should not plant trees/large shrubs closer to the foundations than a distance equal to half the mature height of the tree or 20 feet, whichever is more conservative, unless specifically provided with root barriers to prevent root growth below the building foundation.

It is the homeowner's responsibility to perform periodic maintenance during hot and dry periods to ensure that adequate watering has been provided to keep soil from separating or pulling back from the foundation. Future owners and property management personnel should be informed and educated regarding the importance of maintaining a constant level of soil-moisture. The owners should be made aware of the potential negative consequences of both excessive watering, as well as allowing potentially expansive soils to become too dry. Expansive soils can undergo shrinkage during drying, and swelling during the rainy winter season, or when irrigation is resumed. This can result in distress to building structures and hardscape improvements. The builder should provide these recommendations to future homeowners and property management personnel.

4.5 Soil Bearing and Lateral Resistance

Provided our earthwork recommendations are implemented, an allowable soil bearing pressure of 1,500 pounds per square foot (psf) may be used for the design of footings having a minimum width of 12 inches and minimum embedment of 12 inches below lowest adjacent ground surface. This value may be increased by 300 psf for each additional foot of embedment or 100 psf for each additional foot of foundation width to a maximum value of 3,000 psf. An allowable soil bearing pressure of 1,200 psf may be used for a mat post-tensioned slab a minimum of 6 inches below lowest adjacent grade. These allowable bearing pressures are applicable for level (ground slope equal to or flatter than 5H:1V) conditions only. Bearing values indicated above are for total dead loads and live loads. The above vertical bearing may be increased by one-third for short durations of loading which will include the effect of wind or seismic forces.

In utilizing the above-mentioned allowable bearing capacity and provided our earthwork recommendations are implemented, foundation settlement due to structural loads is anticipated to be 1-inch or less. Differential settlement may be taken as half of the total settlement (i.e., ½-inch over a horizontal span of 40 feet).

Resistance to lateral loads can be provided by friction acting at the base of foundations and by passive earth pressure. For concrete/soil frictional resistance, an allowable coefficient of friction of 0.35 may be assumed with dead-load forces. For slabs constructed over a moisture barrier, a friction coefficient of 0.3 may be used. An allowable passive lateral earth pressure of 270 psf per foot of depth (or pcf) to a maximum of 2,700 psf may be used for lateral resistance. Allowable passive pressure may be increased to 360 pcf to a maximum of 3,600 psf for short duration seismic or wind loading. These passive pressures are applicable for level (ground slope equal to or flatter than 5H:1V) conditions only.

Frictional resistance and passive pressure may be used in combination without reduction. The provided allowable passive pressures are based on a factor of safety of 1.5 and 1.1 for static and seismic loading conditions, respectively. The structural designer should incorporate appropriate factors of safety and/or load factors in their design.

4.6 __Pier Footing Design__

Foundations for bleachers, scoreboards, light poles, backstops, shade structures etc. may consist of either Cast-In-Drilled Hole (CIDH) piers or spread footings. If drilled piers are selected, it would be prudent to terminate the pile tips at a depth of about 15 feet below existing grade due to site groundwater. This may require larger diameter piers. These footings should be designed in accordance with Section 1803 of the 2022 CBC. utilizing the following parameters.

- To resist axial dead and live loads, an allowable skin friction of 400 pounds per square foot (psf) may be used for the design of Cast-In-Drilled Hole (CIDH) piers. The upper 1 foot of skin friction should be neglected. Piers should generally be spaced at a minimum on-center spacing of three times the pier diameter. Passive resistance is provided in Section 4.5.

- For allowable axial capacity see *Figure 3: Allowable Axial Compressive Capacity for 30" Diameter CIDH Pier*. Please note that a figure for an alternate pier diameter can be provided upon request.

- Based on our laboratory test results of representative site soil samples, onsite soils have a designated sulfate exposure class of "S0" per ACI 318-19, Table 19.3.1.1. As a result, per Table 19.3.2.1 the minimum compressive strength of structural concrete shall be 2,500 psi.

- Passive pressure for pier footings is only applicable for the upper 15 feet (above groundwater). For isolated pier footings generally spaced a minimum of 3 pile diameters on-center above groundwater, an allowable passive pressure of 500 pcf may be used for passive resistance. The provided passive pressure is based on an arching factor of 2 (e.g., 250 pcf x 2) and should be limited to a maximum of 10 times the value provided above (e.g., 500 pcf to a maximum of 5,000 psf). Passive pressure should be reduced for any piers extending below estimated design groundwater at 15 feet below grade. Below groundwater, an allowable passive pressure of 240 pcf (e.g., 120 pcf x 2) to a maximum of 10 times (e.g., 240 pcf to a maximum of 2,400 psf) may be used for passive resistance. These passive pressure values are applicable for level (ground slope equal to or flatter than 5 horizontal feet to 1-foot vertical) conditions only.

4.7 *Pier Footing Construction*

Pier borings should be plumb and free of loose or softened material. Extreme care in drilling, placement of the precast, pre-stressed concrete pole sections and the pouring of concrete will be essential to avoid excessive disturbance of pier boring walls. The concrete poles should be installed, and the concrete pumped immediately after drilling is completed. Concrete mix design should include provisions to minimize shrinkage which can reduce frictional resistance of the pile shaft. Concrete placement by pumping or tremie tube to the bottom of the excavations is recommended. No pier boring should be left open overnight. Pier borings should not be drilled immediately adjacent to another pier until the concrete in the other pier has attained its initial set. Contractor should know that caving soils is possible. A representative from LGC Geotechnical should be onsite full-time during the drilling of piers to verify the assumptions made during the design stages.

If caving occurs during pier construction, a temporary casing may be required. Vibratory hammers and oversized predrill are not allowed for casing installation. The temporary casing should be pulled as the concrete is being poured while always maintaining at least a 5-foot head of concrete inside the casing.

4.8 *Control of Surface Water and Drainage Control*

Positive drainage of surface water away from structures is very important. Water should not be allowed to pond adjacent to buildings or to flow freely down a graded slope. Per section 1804.3 of the 2022 CBC, positive drainage may be accomplished by providing drainage away from buildings at a gradient of at least 5 percent for earthen surfaces for a distance of at least 10 feet away from the face of wall. If a distance of 10 feet cannot be achieved, an alternative of a gradient of at least 5 percent to an area drain or swale having a gradient of 2 percent is acceptable. Where necessary, drainage paths may be shortened by use of area drains and collector pipes. Eave gutters are recommended and should reduce water infiltration into the subgrade soils if the downspouts are properly connected to appropriate outlets.

Planters with open bottoms adjacent to buildings should be avoided. Planters should not be designed adjacent to buildings unless provisions for drainage, such as catch basins, liners, and/or area drains, are made. Overwatering must be avoided.

4.9 Soil Corrosivity

Although not corrosion engineers (LGC Geotechnical is not a corrosion consultant), several governing agencies in Southern California require the geotechnical consultant to determine the corrosion potential of soils to buried concrete and metal facilities. We therefore present the results of our testing with regard to corrosion for the use of the client and other consultants, as they determine necessary.

Based on laboratory sulfate test results, the near surface soils are designated to a class "S0" per ACI 318, Table 19.3.1.1 with respect to sulfates. Concrete in direct contact with the onsite soils can be designed according to ACI 318, Table 19.3.2.1 using the "S0" sulfate classification.

Additional laboratory testing may need to be performed at the completion of grading by the project corrosion engineer to further evaluate the as-graded soil corrosivity characteristics. Accordingly, revision of the corrosion potential may be needed, should future test results differ substantially from the conditions reported herein. The client and/or other members of the development team should consider this during the design and planning phase of the project and formulate an appropriate course of action.

4.10 Preliminary Portland Cement Concrete Pavement

The provided preliminary Portland Cement concrete section is based on the guidelines of the American Concrete Institute (ACI 330R-08). For the final design section, we recommend a traffic study be performed as LGC Geotechnical does not perform traffic engineering. A traffic study should include the design vehicle (number of axles and load per axle) and estimated number of daily repetitions/trips. Based on an assumed Traffic Category C with an assumed Average Daily Truck Traffic (ADTT) of 100, we recommend a preliminary section of a minimum of 6.5 inches of concrete over 4 inches of compacted aggregate base over compacted subgrade. In areas such as the food truck staging where a concrete section will see daily vehicular traffic but less than ADTT of 100, we recommend 6 inches in thickness with 2 inches of base underneath.

The concrete should have a minimum compressive strength of 4,000 psi and a minimum flexural strength of 550 psi at the time the pavement is subjected to traffic. Steel reinforcement is not required (ACI, 2013). This pavement section assumes that edge restraints like a curb and gutter will be provided. To reduce the potential (but not eliminate) for cracking, paving should provide control joints at regular intervals not exceeding 10 feet in each direction. Decreasing the spacing of these joints will further reduce, but not eliminate the potential for unsightly cracking. Preliminary pavement section is based on a 20-year design. Truck loading is defined one 16-kip axle and two 32-kip tandem axles. LGC Geotechnical does not perform traffic engineering and determination of traffic loading is not the purview of the geotechnical consultant.

The thicknesses shown are for minimum thicknesses. Increasing the thickness of any or all of the above layers will reduce the likelihood of the pavement experiencing distress during its service life. The above recommendations are based on the assumption that proper maintenance and irrigation of the areas adjacent to the roadway will occur through the design life of the pavement.

Failure to maintain a proper maintenance and/or irrigation program may jeopardize the integrity of the pavement.

Earthwork recommendations regarding aggregate base and subgrade are provided in the previous section "Site Earthwork" and the related sub-sections of this report.

4.11 Nonstructural Concrete Flatwork

Nonstructural concrete (such as flatwork, sidewalks etc.) has a potential for cracking due to changes in soil volume related to soil-moisture fluctuations. To reduce the potential for excessive cracking and lifting, concrete should be designed in accordance with the minimum guidelines outlined in Table 3 below. These guidelines will reduce the potential for irregular cracking and promote cracking along construction joints but will not eliminate all cracking or lifting. Thickening the concrete and/or adding additional reinforcement will further reduce cosmetic distress. Please note that these are preliminary recommendation that will need to be confirmed and/or modified based on as-graded conditions at the completion of grading.

TABLE 3

Nonstructural Concrete Flatwork for High Expansion Potential

	Perimeter Sidewalks	Private Drives	Interior Walkways and Plazas	City Sidewalk Curb and Gutters
Minimum Thickness (in.)	4 (nominal)	5 (full)	5 (full)	City/Agency Standard
Presaturation	Presoak to 12 inches	Presoak to 12 inches	Presoak to 12 inches	City/Agency Standard
Reinforcement	—	No. 3 at 24 inches on centers	No. 3 at 24 inches on centers	City/Agency Standard
Thickened Edge (in.)	—	8 x 8	8 x 8	City/Agency Standard
Crack Control Joints	Saw cut or deep open tool joint to a minimum of 1/3 the concrete thickness	Saw cut or deep open tool joint to a minimum of 1/3 the concrete thickness	Saw cut or deep open tool joint to a minimum of 1/3 the concrete thickness	City/Agency Standard
Maximum Joint Spacing	5 feet	10 feet or quarter cut whichever is closer	6 feet	City/Agency Standard
Aggregate Base Thickness (in.)	—	2	2	City/Agency Standard

4.12 Preliminary Asphalt Pavement Sections

For the purpose of these preliminary recommendations, we have selected a preliminary design R-value of 16 and calculated pavement sections for Traffic Index (TI) of 5.0 or less, 5.5, and 6.0. The California Department of Transportation Highway Design Manual (Caltrans, 2017) allows for a maximum R-Value of 50 to be used in pavement design. These recommendations must be confirmed with R-Value testing of representative near-surface soils at the completion of grading and after underground utilities have been installed and backfilled. Final street sections should be confirmed by the project civil engineer based upon the final design Traffic Index. Determination of the TI is not the purview of the geotechnical consultant. If requested, LGC Geotechnical will provide sections for alternate TI values.

TABLE 4

Preliminary Pavement Sections

Assumed Traffic Index	5.0 or less	5.5	6.0
R -Value Subgrade	16	16	16
AC Thickness	4.0 inches	4.0 inches	4.0 inches
Base Thickness	5.5 inches	8.0 inches	9.0 inches

The thicknesses shown are for minimum thicknesses. Increasing the thickness of any or all of the above layers will reduce the likelihood of the pavement experiencing distress during its service life. The above recommendations are based on the assumption that proper maintenance and irrigation of the areas adjacent to the roadway will occur through the design life of the pavement. Failure to maintain a proper maintenance and/or irrigation program may jeopardize the integrity of the pavement.

Earthwork recommendations regarding aggregate base and subgrade are provided in Section 4.1 “Site Earthwork” and the related sub-sections of this report.

4.13 Vehicular Pavers

An assumed TI of 5.0 has been used for the proposed paver areas depicted on the Parnell Park Concept Plan (SWA, 2022a). R-Value testing will be performed on finished grade soils at the completion of grading, however, for these preliminary geotechnical recommendations an R-Value of 16 has been assumed.

Concrete pavers should be a minimum of 3 and 1/8 inches (80 mm) thick, rated for vehicular traffic and placed in a herringbone pattern. Manufacturer’s specific recommendations regarding the pavers (required bedding and jointing sand, etc.) should be implemented during construction. It should be noted that pavers are typically underlain by 1 to 1½ inches of bedding sand followed by compacted aggregate base. Concrete pavers may be underlain with a minimum of 10.0 inches of compacted aggregate base over compacted subgrade soils. As an alternative, the 10-inch layer of compacted crushed base may be substituted with 6 inches of structural concrete over 4 inches of compacted crushed base. If concrete is utilized, it should have a minimum 28-day strength requirement of 2,500 psi and be reinforced minimally with No. 3 rebars at 24 inches on-center. We also recommend weep holes be constructed within the concrete to reduce the potential for ponding of water above the concrete. We recommend a minimum 2-inch diameter drain hole be placed at the lowest elevations in the concrete slab and be backfilled with pea gravel.

Concrete bands around the perimeter of the pavers are recommended. The concrete bands should be at least 6 inches thick, with two No. 4 rebars placed longitudinally at approximately mid-height. The concrete should be underlain by a minimum of 4 inches of crushed base material. This base thickness can be increased to match the same subgrade elevation as the pavers if it is easier from a construction logistics standpoint.

The thicknesses above are minimum thicknesses. Increasing the thickness of any or all of the above layers will reduce the likelihood of the pavement experiencing distress during its service life. The above recommendations are based on the assumption that proper maintenance and irrigation of the areas adjacent to the roadway will occur throughout the design life of the pavement. Failure to maintain a proper maintenance and/or irrigation program may jeopardize the integrity of the pavement and pavers.

Aggregate base should meet the requirements of the latest edition of the *Standard Specifications for Public Works Construction* ("Greenbook") or the specifications for Caltrans Class 2 aggregate base. Aggregate base should be compacted to a minimum of 95 percent relative compaction over subgrade compacted to a minimum of 90 percent relative compaction per ASTM Test Method D1557.

4.14 Preliminary Pedestrian Pavers Section

Concrete pavers for pedestrian traffic should be a minimum of 2 and 3/8 inches (60 mm) thick and placed in a herringbone pattern. Manufacturer's specific recommendations regarding the pavers (required bedding and jointing sand, etc.) should be implemented during construction. It should be noted that pavers are typically underlain by 1 to 1½ inches of bedding sand followed by compacted aggregate base. Concrete pavers may be underlain with a minimum of 4 inches of compacted aggregate base over compacted subgrade soils. Pavers subjected to maintenance vehicular loading should be underlain with a minimum of 6 inches of base.

Concrete bands around the perimeter of the pavers are recommended. The concrete bands should be at least 6 inches thick, with two No. 4 rebars placed longitudinally at approximately mid-height. The concrete should be underlain by a minimum of 4 inches of crushed base material.

The thicknesses above are minimum thicknesses. Increasing the thickness of any or all of the above layers will reduce the likelihood of the pavement experiencing distress during its service life. The above recommendations are based on the assumption that proper maintenance and irrigation of the areas adjacent to the roadway will occur throughout the design life of the pavement. Failure to maintain a proper maintenance and/or irrigation program may jeopardize the integrity of the pavement and pavers.

Aggregate base should meet the requirements of the latest edition of the *Standard Specifications for Public Works Construction* ("Greenbook") or the specifications for Caltrans Class 2 aggregate base. Aggregate base should be compacted to a minimum of 95 percent relative compaction over subgrade compacted to a minimum of 90 percent relative compaction per ASTM Test Method D1557.

4.15 <u>Decomposed Granite (DG) Paths</u>

From a geotechnical perspective, we recommend that the designed pedestrian trail consist of 4 inches of DG compacted to a minimum relative compaction of 90 percent overlaying compacted subgrade. Subgrade soils should also be compacted to a minimum of 90 percent relative compaction.

From a geotechnical perspective, we recommend that the designed maintenance trails that will be subject to light load traffic (light pickups, quads etc.) incorporate the following: the top surface consist of 4 inches of DG compacted to 90 percent relative compaction overlaying 12 inches of CMB compacted to a minimum of 95 percent relative compaction; the subgrade soils should be compacted to a minimum of 90 percent relative compaction.

We understand that stabilization of the DG Paths will be completed with the addition of a binder. The addition of binder will not impact our recommendations regarding placement, thickness, or compaction of the material.

4.16 <u>Playground Design Recommendations</u>

Given that the expansion index exceeds 20, the proposed playground should be designed for the effects of expansive soils. Playgrounds have the potential for cracking due to changes in soil volume related to soil-moisture fluctuations. These guidelines will reduce the potential for irregular cracking but will not eliminate all cracking or lifting.

To reduce the potential for excessive cracking and lifting, the proposed playground should be designed in accordance with a minimum thickness gunite concrete sub slab of 4 inches beneath the rubberized play surfaces. The concrete slab should have a minimum welded wire mesh reinforcement of 4-inch grid pattern with W4 wire size (4x4-W4xW4).

Concrete can be placed upon aggregate base and/or subgrade for all flat areas or playground mounds that is compacted to at least 90 percent relative compaction per ASTM D1557 at or slightly above optimum moisture content. Moisture conditioning of the subgrade soils is recommended prior to construction of the playground. The subgrade moisture condition of the playground soils should be maintained at the recommended moisture content up to the time of concrete placement. This moisture content should be maintained around the immediate perimeter of the playground during construction.

The following is for informational purposes only since slab underlayment (e.g., moisture retarder, sand or gravel layers for concrete curing and/or capillary break) is unrelated to the geotechnical performance of the foundation and thereby not the purview of the geotechnical consultant. Post-construction moisture migration should be expected below the playground slab. The playground slab designer should determine whether the use of a capillary break (sand or gravel layer), in conjunction with the vapor retarder, is necessary or required by code. Sand layer thickness and location (above and/or below vapor retarder) should also be determined by the foundation engineer/architect.

4.17 ***Structures without Foundations***

We recommend that structures constructed on the proposed ground surface, which do not have a subsurface foundation, be placed on compacted base. The compacted base will help reduce the potential for settlement if the subsurface becomes saturated. Aggregate base should be a minimum of 6 inches in thickness and compacted to 95% relative compaction as compared to ASTM 1557.

4.18 ***Grading and Foundation Plan Review***

When available, foundation and any updated plans should be reviewed by LGC Geotechnical in order to verify our geotechnical recommendations are implemented. Updated recommendations and/or additional field work may be necessary.

4.19 ***Geotechnical Observation and Testing During Construction***

The recommendations provided in this report are based on limited subsurface observations and geotechnical analysis. The interpolated subsurface conditions should be checked in the field during construction by a representative of LGC Geotechnical. Geotechnical observation and testing are required per Section 1705 of the 2022 California Building Code (CBC).

Geotechnical observation and/or testing should be performed by LGC Geotechnical at the following stages:

- During grading (removal bottoms, fill placement, etc.);
- During utility trench and retaining wall backfill and compaction;
- During excavation for wall foundations;
- After presoaking concrete-flatwork subgrades, and prior to placement of aggregate base or concrete;
- Placement of aggregate base and asphalt;
- During pier foundation excavation and prior to placing reinforcement and/or concrete; and
- When any unusual soil conditions are encountered during any construction operation subsequent to issuance of this report.

5.0 LIMITATIONS

Our services were performed using the degree of care and skill ordinarily exercised, under similar circumstances, by reputable soils engineers and geologists practicing in this or similar localities. No other warranty, expressed or implied, is made as to the conclusions and professional advice included in this report.

This report is based on data obtained from limited observations of the site, which have been extrapolated to characterize the site. While the scope of services performed is considered suitable to adequately characterize the site geotechnical conditions relative to the proposed development, no practical evaluation can completely eliminate uncertainty regarding the anticipated geotechnical conditions in connection with a subject site. Variations may exist and conditions not observed or described in this report may be encountered during grading and construction.

This report is issued with the understanding that it is the responsibility of the owner, or of his/her representative, to ensure that the information and recommendations contained herein are brought to the attention of the other consultants (at a minimum the civil engineer, structural engineer, landscape architect) and incorporated into their plans. The contractor should properly implement the recommendations during construction and notify the owner if they consider any of the recommendations presented herein to be unsafe, or unsuitable.

The findings of this report are valid as of the present date. However, changes in the conditions of a site can and do occur with the passage of time, whether they be due to natural processes or the works of man on this or adjacent properties. The findings, conclusions, and recommendations presented in this report can be relied upon only if LGC Geotechnical has the opportunity to observe the subsurface conditions during grading and construction of the project, in order to confirm that our preliminary findings are representative for the site. This report is intended exclusively for use by the client, any use of or reliance on this report by a third party shall be at such party's sole risk.

In addition, changes in applicable or appropriate standards may occur, whether they result from legislation or the broadening of knowledge. Accordingly, the findings of this report may be invalidated wholly or partially by changes outside our control. Therefore, this report is subject to review and modification.

LEGEND

HS-3 T.D. = 50' — Approximate Location of Hollow Stem Auger Boring by LGC Geotechnical, With Total Depth in Feet

I-1 T.D. = 7' — Approximate Location of Hollow Stem Auger Infiltration Boring by LGC Geotechnical, With Total Depth in Feet

HA-2 T.D. = 3' — Approximate Location of Hand Auger Boring by LGC Geotechnical, With Total Depth in Feet

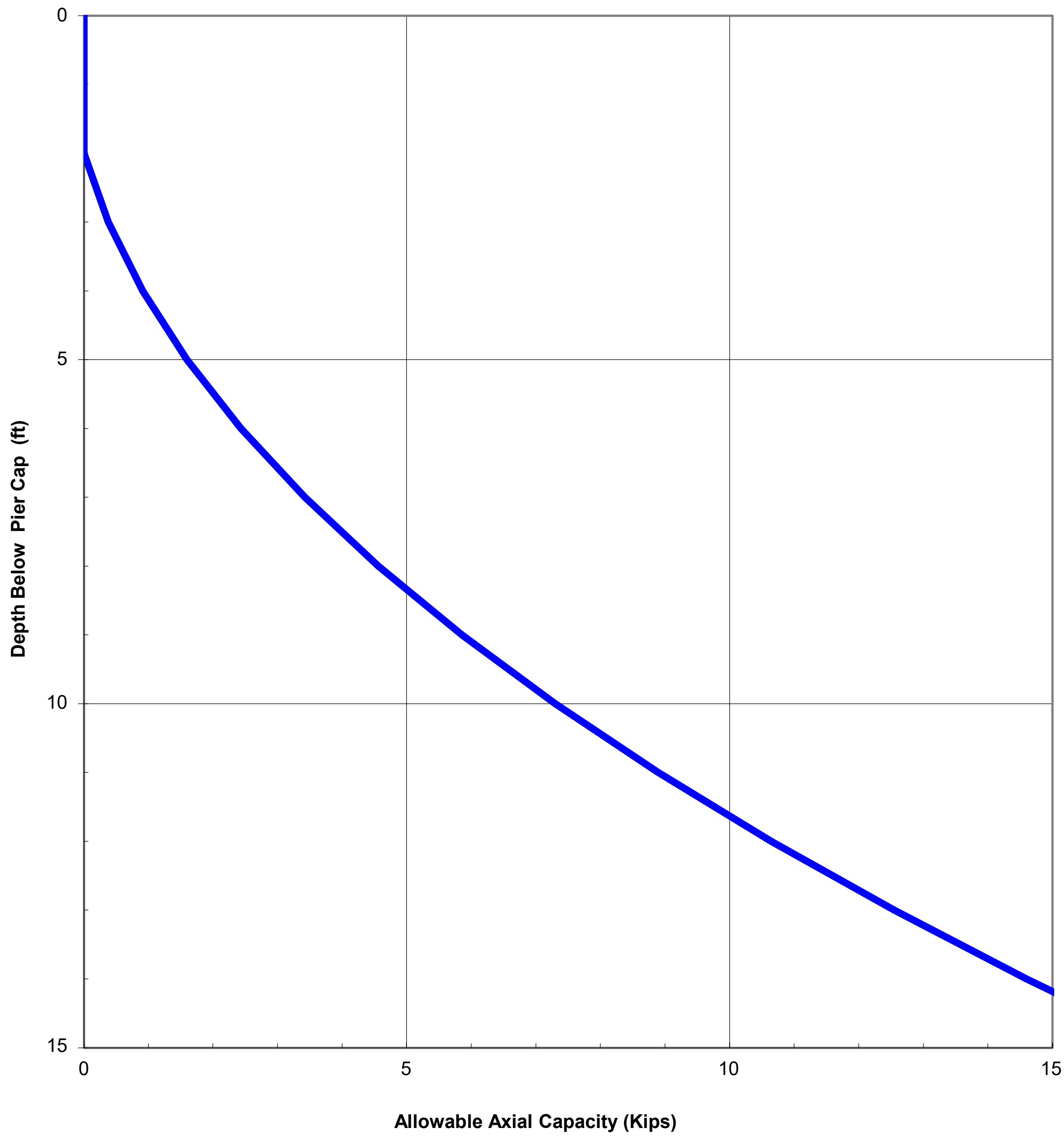
Approximate Limits of This Report

LGC Geotechnical, Inc.

LGC Geotechnical, Inc.
131 Calle Iglesia, Ste. 200
San Clemente, CA 92672
TEL (949) 369-6141 FAX (949) 369-6142

FIGURE 2
Boring Location Map

PROJECT NAME	SWA-Parnell Park
PROJECT NO.	21291-01
ENG. / GEOL.	BJE / BPG
SCALE	1" = 80'
DATE	May 2023



FS = 2

Refer to text discussion

Figure 3

LGC Geotechnical, Inc.

Allowable Axial Compressive Capacity for 30" Diameter CIDH Pier

Project Number:	21291-01
Date:	May-23

SWA - Parnell Park

Appendix A
References

APPENDIX A

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Appendix B
Boring Logs

Geotechnical Boring Log Borehole HS-1

Date: 7/21/2022	**Drilling Company:** MR
Project Name: SWA- Parnell Park	**Type of Rig:** Truck-Mounted
Project Number: 21291-01	**Drop:** 30" **Hole Diameter:** 8"
Elevation of Top of Hole: ~211' MSL	**Drive Weight:** 140 pounds
Hole Location: See Boring Location Map	Page 1 of 2

Logged By JJV
Sampled By JJV
Checked By BJE

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	0							@ 0'- parking lot, asphalt concrete to 3" over 5" of aggregate base	AL EI
			R-1	4 9 16	106.0	16.5	CL	@ 2.5'- Silty CLAY with Sand: dark brown, moist, very stiff	
95	5	B-1	R-2	5 6 15	103.8	15.7	SC	@ 5'- Clayey SAND: light brown, moist, medium dense	
			R-3	5 17 24	114.4	14.2		@ 7.5'- Clayey SAND: reddish brown, moist, dense	
90	10		R-4	7 9 18	115.1	13.9	SM	@ 10'- Silty SAND: reddish brown, very moist, medium dense	
85	15		SPT-1	5 10 20		17.6		@ 15'- Silty SAND: reddish brown, wet, dense	
80	20		R-5	9 15 26	105.2	22.3	CL-ML	@ 20'- Silty CLAY with Sand: light brown, very moist, hard	
75	25		SPT-2	4 5 9		29.5		@ 25'- Silty CLAY: reddish brown, wet, very stiff	
	30								

Geotechnical, Inc.

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 THE ACTUAL CONDITIONS ENCOUNTERED. THE DESCRIPTIONS PROVIDED ARE QUALITATIVE FIELD DESCRIPTIONS AND ARE NOT BASED ON QUANTITATIVE ENGINEERING ANALYSIS.

SAMPLE TYPES:
- B BULK SAMPLE
- R RING SAMPLE (CA Modified Sampler)
- G GRAB SAMPLE
- SPT STANDARD PENETRATION TEST SAMPLE
- GROUNDWATER TABLE

TEST TYPES:
- DS DIRECT SHEAR
- MD MAXIMUM DENSITY
- SA SIEVE ANALYSIS
- S&H SIEVE AND HYDROMETER
- EI EXPANSION INDEX
- CN CONSOLIDATION
- CR CORROSION
- AL ATTERBERG LIMITS
- CO COLLAPSE/SWELL
- RV R-VALUE
- -#200 % PASSING # 200 SIEVE

Last Edited: 7/26/2022

Geotechnical Boring Log Borehole HS-1

Date: 7/21/2022	**Drilling Company:** MR
Project Name: SWA- Parnell Park	**Type of Rig:** Truck-Mounted
Project Number: 21291-01	**Drop:** 30" **Hole Diameter:** 8"
Elevation of Top of Hole: ~211' MSL	**Drive Weight:** 140 pounds
Hole Location: See Boring Location Map	

Logged By JJV
Sampled By JJV
Checked By BJE

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	30		R-6	4 9 14	104.6	21.6	SC	@ 30'- Clayey SAND: light brown, very moist, medium dense	
70	35		SPT-3	4 4 12		22.8	CL-ML	@ 35'- Silty CLAY: light brown, very moist, very stiff, sticky	
65	40		R-7	6 10 15	105.4	21.3	CL	@ 40'- Sandy CLAY: light reddish brown, very moist, very stiff	
60	45		SPT-4	4 7 11		20.4		@ 45'- Sandy CLAY: reddish brown, very moist, very stiff, very sticky	
55	50		R-8	4 4 7	107.2	20.1		@ 50'- Sandy CLAY: reddish brown, very moist, stiff	
50	55							Total Depth = 51.5' Groundwater Encountered at approximately 18' Backfilled with Cuttings and Capped with AC to 3 inches on 7/21/2022	
	60								

LGC Geotechnical, Inc.

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- B BULK SAMPLE
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- GROUNDWATER TABLE

TEST TYPES:
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- MD MAXIMUM DENSITY
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- EI EXPANSION INDEX
- CN CONSOLIDATION
- CR CORROSION
- AL ATTERBERG LIMITS
- CO COLLAPSE/SWELL
- RV R-VALUE
- -#200 % PASSING # 200 SIEVE

Geotechnical Boring Log Borehole HS-2

Date: 7/21/2022	**Drilling Company:** MR
Project Name: SWA- Parnell Park	**Type of Rig:** Truck-Mounted
Project Number: 21291-01	**Drop:** 30" **Hole Diameter:** 8"
Elevation of Top of Hole: ~211' MSL	**Drive Weight:** 140 pounds
Hole Location: See Boring Location Map	Page 1 of 2

Logged By JJV
Sampled By JJV
Checked By BJE

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	0	B-1						@ 0'- grass/turf	AL EI
			R-1	8 24 38	109.3	12.0	SM	@ 2.5'- Silty SAND: light brown, moist, dense, contains calcium carbonate	
95	5		R-2	14 50/6"	111.2	8.9		@ 5'- Silty SAND: light brown, moist, very dense	
			R-3	14 42 50/5"	109.1	15.8	ML	@ 7.5'- Sandy SILT: light reddish brown, very moist, hard	
90	10		R-4	4 16 34	110.6	17.0		@ 10'- Sandy SILT: light reddish brown, very moist, hard	
85	15		SPT-1	7 10 16		16.5		@ 15'- Sandy SILT: light brown, very moist, hard	
80	20		R-5	10 13 37	113.7	17.8	CL	@ 20'- Sandy CLAY: light brown, very moist, hard	-#200 AL
75	25		SPT-2	5 9 20		20.1		@ 25'- CLAY: reddish brown, very moist, hard	
	30								

LGC Geotechnical, Inc.

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SAMPLE TYPES:
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- GROUNDWATER TABLE

TEST TYPES:
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- MD MAXIMUM DENSITY
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- S&H SIEVE AND HYDROMETER
- EI EXPANSION INDEX
- CN CONSOLIDATION
- CR CORROSION
- AL ATTERBERG LIMITS
- CO COLLAPSE/SWELL
- RV R-VALUE
- -#200 % PASSING # 200 SIEVE

Last Edited: 7/26/2022

Geotechnical Boring Log Borehole HS-2

Date: 7/21/2022	**Drilling Company:** MR
Project Name: SWA- Parnell Park	**Type of Rig:** Truck-Mounted
Project Number: 21291-01	**Drop:** 30" **Hole Diameter:** 8"
Elevation of Top of Hole: ~211' MSL	**Drive Weight:** 140 pounds
Hole Location: See Boring Location Map	

Logged By JJV
Sampled By JJV
Checked By BJE

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	30		R-6	6 15 18	106.4	21.0	CL	@ 30'- Sandy CLAY: brown, very moist, very stiff	-#200 AL
70	35		SPT-3	2 11 18		18.5		@ 35'- Sandy CLAY: light reddish brown, very moist, hard, sticky	
65	40		R-7	4 10 22	114.9	17.7		@ 40'- Sandy CLAY: light brown, very moist, very stiff	
60	45		SPT-4	5 10 13		21.6	CL-ML	@ 45'- Silty CLAY: brown, very moist, very stiff, sticky	
55	50		R-8	5 12 26	104.7	24.6	CL	@ 50'- CLAY: reddish brown, very moist, very stiff, sticky, disturbed	
								Total Depth = 51.5' Groundwater Encountered at approximately 21' Backfilled with Cuttings on 7/21/2022	
50	55								
	60								

LGC Geotechnical, Inc.

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 THE ACTUAL CONDITIONS ENCOUNTERED. THE DESCRIPTIONS PROVIDED ARE QUALITATIVE FIELD DESCRIPTIONS AND ARE NOT BASED ON QUANTITATIVE ENGINEERING ANALYSIS.

SAMPLE TYPES:
- B BULK SAMPLE
- R RING SAMPLE (CA Modified Sampler)
- G GRAB SAMPLE
- SPT STANDARD PENETRATION TEST SAMPLE
- GROUNDWATER TABLE

TEST TYPES:
- DS DIRECT SHEAR
- MD MAXIMUM DENSITY
- SA SIEVE ANALYSIS
- S&H SIEVE AND HYDROMETER
- EI EXPANSION INDEX
- CN CONSOLIDATION
- CR CORROSION
- AL ATTERBERG LIMITS
- CO COLLAPSE/SWELL
- RV R-VALUE
- -#200 % PASSING # 200 SIEVE

Geotechnical Boring Log Borehole HS-3

Date: 7/21/2022	**Drilling Company:** MR
Project Name: SWA- Parnell Park	**Type of Rig:** Truck-Mounted
Project Number: 21291-01	**Drop:** 30" **Hole Diameter:** 8"
Elevation of Top of Hole: ~207' MSL	**Drive Weight:** 140 pounds
Hole Location: See Boring Location Map	Page 1 of 1

Logged By JJV
Sampled By JJV
Checked By BJE

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	0							@ 0'- grass/turf	
95	5	B-1	R-1	4 17 23	113.0	14.6	ML	@ 5'- Sandy SILT: pale brown, very moist, hard, disturbed	
			R-2	4 16 24	118.4	11.9	SM	@ 7.5'- Silty SAND: light brown, moist, dense	
90	10		R-3	3 12 22	109.3	14.8	ML	@ 10'- Sandy SILT: light brown, very moist, very stiff	
								Total Depth = 11.5' Groundwater Not Encountered Backfilled with Cuttings on 7/21/2022	
85	15								
80	20								
75	25								
	30								

LGC Geotechnical, Inc.

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 THE ACTUAL CONDITIONS ENCOUNTERED. THE DESCRIPTIONS PROVIDED ARE QUALITATIVE FIELD DESCRIPTIONS AND ARE NOT BASED ON QUANTITATIVE ENGINEERING ANALYSIS.

SAMPLE TYPES:
B BULK SAMPLE
R RING SAMPLE (CA Modified Sampler)
G GRAB SAMPLE
SPT STANDARD PENETRATION TEST SAMPLE
GROUNDWATER TABLE

TEST TYPES:
DS DIRECT SHEAR
MD MAXIMUM DENSITY
SA SIEVE ANALYSIS
S&H SIEVE AND HYDROMETER
EI EXPANSION INDEX
CN CONSOLIDATION
CR CORROSION
AL ATTERBERG LIMITS
CO COLLAPSE/SWELL
RV R-VALUE
-#200 % PASSING # 200 SIEVE

Last Edited: 7/26/2022

Geotechnical Boring Log Borehole I-1

Date: 7/21/2022	**Drilling Company:** MR
Project Name: SWA- Parnell Park	**Type of Rig:** Truck-Mounted
Project Number: 21291-01	**Drop:** 30" **Hole Diameter:** 8"
Elevation of Top of Hole: ~210' MSL	**Drive Weight:** 140 pounds
Hole Location: See Boring Location Map	Page 1 of 1

Logged By JJV
Sampled By JJV
Checked By BJE

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	0							@ 0'- SAND with grains/fines	
			R-1	9 25 38	111.8	9.8	SM	@ 2.5'- Silty SAND: light brown, moist, dense	
95	5		R-2	4 22 50/4"	107.0	12.7	ML	@ 5'- Sandy SILT: light brown, moist, hard	
			R-3	17	102.2	13.3		@ 6.5' to 7'- Sandy SILT: light reddish brown, moist, stiff, disturbed	
90	10							Total Depth = 7.5' Groundwater Not Encountered Backfilled and Abandoned on 7/26/2022	
85	15								
80	20								
75	25								
	30								

Geotechnical, Inc.

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 THE ACTUAL CONDITIONS ENCOUNTERED. THE DESCRIPTIONS PROVIDED ARE QUALITATIVE FIELD DESCRIPTIONS AND ARE NOT BASED ON QUANTITATIVE ENGINEERING ANALYSIS.

SAMPLE TYPES:
B BULK SAMPLE
R RING SAMPLE (CA Modified Sampler)
G GRAB SAMPLE
SPT STANDARD PENETRATION TEST SAMPLE
GROUNDWATER TABLE

TEST TYPES:
DS DIRECT SHEAR
MD MAXIMUM DENSITY
SA SIEVE ANALYSIS
S&H SIEVE AND HYDROMETER
EI EXPANSION INDEX
CN CONSOLIDATION
CR CORROSION
AL ATTERBERG LIMITS
CO COLLAPSE/SWELL
RV R-VALUE
-#200 % PASSING # 200 SIEVE

Last Edited: 7/26/2022

Geotechnical Boring Log Borehole HA-1

Date: 7/22/2022	**Drilling Company:** LGC
Project Name: SWA- Parnell Park	**Type of Rig:** N/A
Project Number: 21291-01	**Drop:** N/A" **Hole Diameter:** 4"
Elevation of Top of Hole: ~214' MSL	**Drive Weight:** N/A pounds
Hole Location: See Boring Location Map	Page 1 of 1

Logged By JJV
Sampled By JJV
Checked By BJE

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	0							@ 0'- grass/turf	
								@ 7"- Top soil, roots	
							ML	@ 1.5'- SILT with Sand: brown, slightly moist, roots	
95	5								
								@ 3'- SILT with Sand: brown, slightly moist	
								Total Depth = 3' Groundwater Not Encountered Backfilled with Cuttings on 7/22/2022	
90	10								
85	15								
80	20								
75	25								
	30								

LGC Geotechnical, Inc.

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SAMPLE TYPES:

B	BULK SAMPLE
R	RING SAMPLE (CA Modified Sampler)
G	GRAB SAMPLE
SPT	STANDARD PENETRATION TEST SAMPLE
▽	GROUNDWATER TABLE

TEST TYPES:

DS	DIRECT SHEAR
MD	MAXIMUM DENSITY
SA	SIEVE ANALYSIS
S&H	SIEVE AND HYDROMETER
EI	EXPANSION INDEX
CN	CONSOLIDATION
CR	CORROSION
AL	ATTERBERG LIMITS
CO	COLLAPSE/SWELL
RV	R-VALUE
-#200	% PASSING # 200 SIEVE

Last Edited: 7/26/2022

Geotechnical Boring Log Borehole HA-2

Date: 7/22/2022	**Drilling Company:** LGC
Project Name: SWA- Parnell Park	**Type of Rig:** N/A
Project Number: 21291-01	**Drop:** N/A" **Hole Diameter:** 4"
Elevation of Top of Hole: ~211' MSL	**Drive Weight:** N/A pounds
Hole Location: See Boring Location Map	Page 1 of 1

Logged By JJV
Sampled By JJV
Checked By BJE

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	0							@ 0'- grass/turf	
							ML	@ 7"- SILT with Sand: brown, slightly moist	
								@ 1.5'- SILT with Sand: brown, slightly moist	
95	5								
								@ 3'- SILT with Sand: light brown, slightly moist	
								Total Depth = 3' Groundwater Not Encountered Backfilled with Cuttings on 7/22/2022	
90	10								
85	15								
80	20								
75	25								
	30								

LGC Geotechnical, Inc.

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- GROUNDWATER TABLE

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- CR CORROSION
- AL ATTERBERG LIMITS
- CO COLLAPSE/SWELL
- RV R-VALUE
- -#200 % PASSING # 200 SIEVE

Last Edited: 7/26/2022

Appendix C
Laboratory Testing Procedures and Test Results

APPENDIX C

Laboratory Testing Procedures and Test Results

The laboratory testing program was formulated towards providing data relating to the relevant engineering properties of the soils with respect to residential construction. Samples considered representative of site conditions were tested in general accordance with American Society for Testing and Materials (ASTM) procedure and/or California Test Methods (CTM), where applicable. The following summary is a brief outline of the test type and a table summarizing the test results.

Moisture and Density Determination Tests: Moisture content (ASTM D2216) and dry density determinations (ASTM D2937) were performed on relatively undisturbed samples obtained from the test borings and/or trenches. The results of these tests are presented in the boring logs. Where applicable, only moisture content was determined from undisturbed or disturbed samples.

Grain Size Distribution: Representative samples were dried, weighed, and soaked in water until individual soil particles were separated (per ASTM D421) and then washed on a No. 200 sieve. The portion retained on the No. 200 sieve was dried and then sieved on a U.S. Standard brass sieve set in accordance with ASTM D422 (CTM 202). Where an appreciable amount of fines were encountered (greater than 20 percent passing the No. 200 sieve) a hydrometer analysis was done to determine the distribution of soil particles passing the No. 200 sieve.

Sample Location	**Description**	**% Passing # 200 Sieve**
HS-2, R-5 @ 20’	Light Brown Sandy Clay	82
HS-2, R-6 @ 30’	Brown Sandy Clay	61

Atterberg Limits: The liquid and plastic limits (“Atterberg Limits”) were determined in accordance with ASTM Test Method D4318 for engineering classification of fine-grained material and presented in the table below:

Sample Location	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	USCS Soil Classification
HS-1, B-1 @ 0-5’	37	18	19	CL
HS-2, B-1 @ 0-5’	47	19	28	CL
HS-2, R-5 @ 20’	33	15	18	CL
HS-2, R-6 @ 30’	30	18	12	CL

Direct Shear: A direct shear test was performed on a selected undisturbed ring sample, which was soaked for a minimum of 24 hours under a surcharge equal to the applied normal force during testing. After transfer of the sample to the shear box, and reloading the sample, pore pressures set up in the sample due to the transfer were allowed to dissipate for a period of approximately 1 hour prior to application of shearing force. The samples were tested under various normal loads, a motor-driven, strain-controlled, direct-shear testing apparatus at a strain rate of less than 0.05 inch per minute (for sandy soil). The results of this test are presented in the following table.

Sample Location	Description	Friction Angle Peak / At 0.3" Def.	Cohesion (psf) Peak / At 0.3" Def.
HS-3, R-3 @ 10'	Light Brown Sandy Silt	45.1° / 27.7°	120 / 120

Expansion Index: The expansion potential of selected samples was evaluated by the Expansion Index Test, Standard ASTM D4829. Specimens are molded under a set compactive effort to approximately the optimum moisture content and approximately 50 percent saturation or approximately 90 percent relative compaction. The prepared 1-inch-thick by 4-inch-diameter specimens are loaded to an equivalent 144 psf surcharge and are inundated with tap water until volumetric equilibrium is reached. The results of these tests are presented in the table below.

Sample Location	Expansion Index	Expansion Potential*
HS-1, B-1 @ 0-5 feet	55	Medium
HS-2, B-1 @ 0-5 feet	94	High

R-value Test: An R-value test was performed in general accordance with California Test Method 301. The plot is included in this Appendix C.

Sample Location	R-value
HS-1, B-1 @ 0-5 ft	16

Chloride Content: Chloride content was tested in accordance with Caltrans Test Method (CTM) 422. The results are presented below.

Sample Location	Chloride Content, ppm
HS-2, B-1 @ 0-5 feet	144

Soluble Sulfates: The soluble sulfate contents of selected samples were determined by standard geochemical methods (CTM 417). The soluble sulfate content is used to determine the appropriate cement type and maximum water-cement ratios. The test results are presented in the table below.

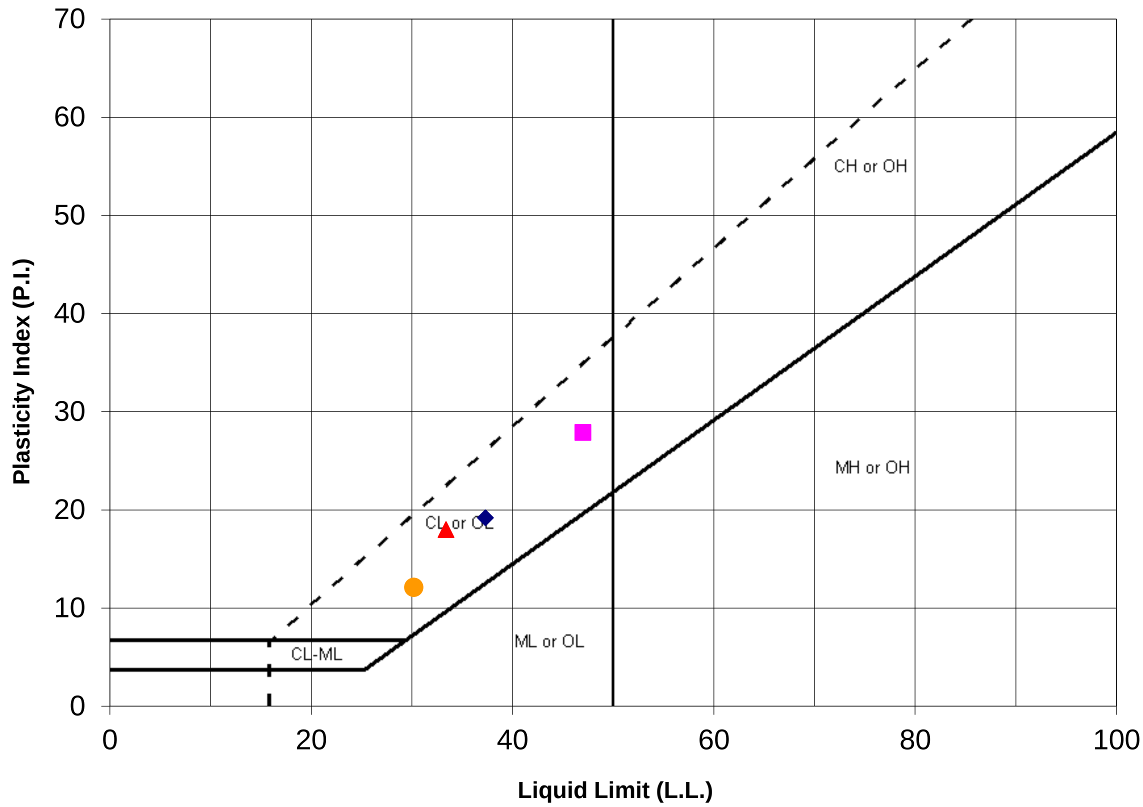
Sample Location	Sulfate Content (ppm)	Sulfate Exposure Class *
HS-2, B-1 @ 0-5 feet	157	S0

*Based on ACI 318R-14, Table 19.3.1.1

Minimum Resistivity and pH Tests: Minimum resistivity and pH tests were performed in general accordance with CTM 643 and standard geochemical methods. The results are presented in the table below.

Sample Location	pH	Minimum Resistivity (ohms-cm)
HS-2, B-1 @ 0-5 feet	7.62	498

PLASTICITY CHART - CLASSIFICATION OF FINE-GRAINED SOILS



Symbol	Location.:	Sample No.:	Depth (ft)	Passing No. 200 Sieve (%)	Liquid Limit (%) LL	Plastic Limit (%) PL	Plasticity Index (%) PI	USCS
◆	**HS-1**	**B-1**	**0-5'**	**-**	**37**	**18**	**19**	**CL**
■	**HS-2**	**B-1**	**0-5'**	**-**	**47**	**19**	**28**	**CL**
▲	**HS-2**	**R-5**	**20'**	**82**	**33**	**15**	**18**	**CL**
●	**HS-2**	**R-6**	**30'**	**61**	**30**	**18**	**12**	**CL**

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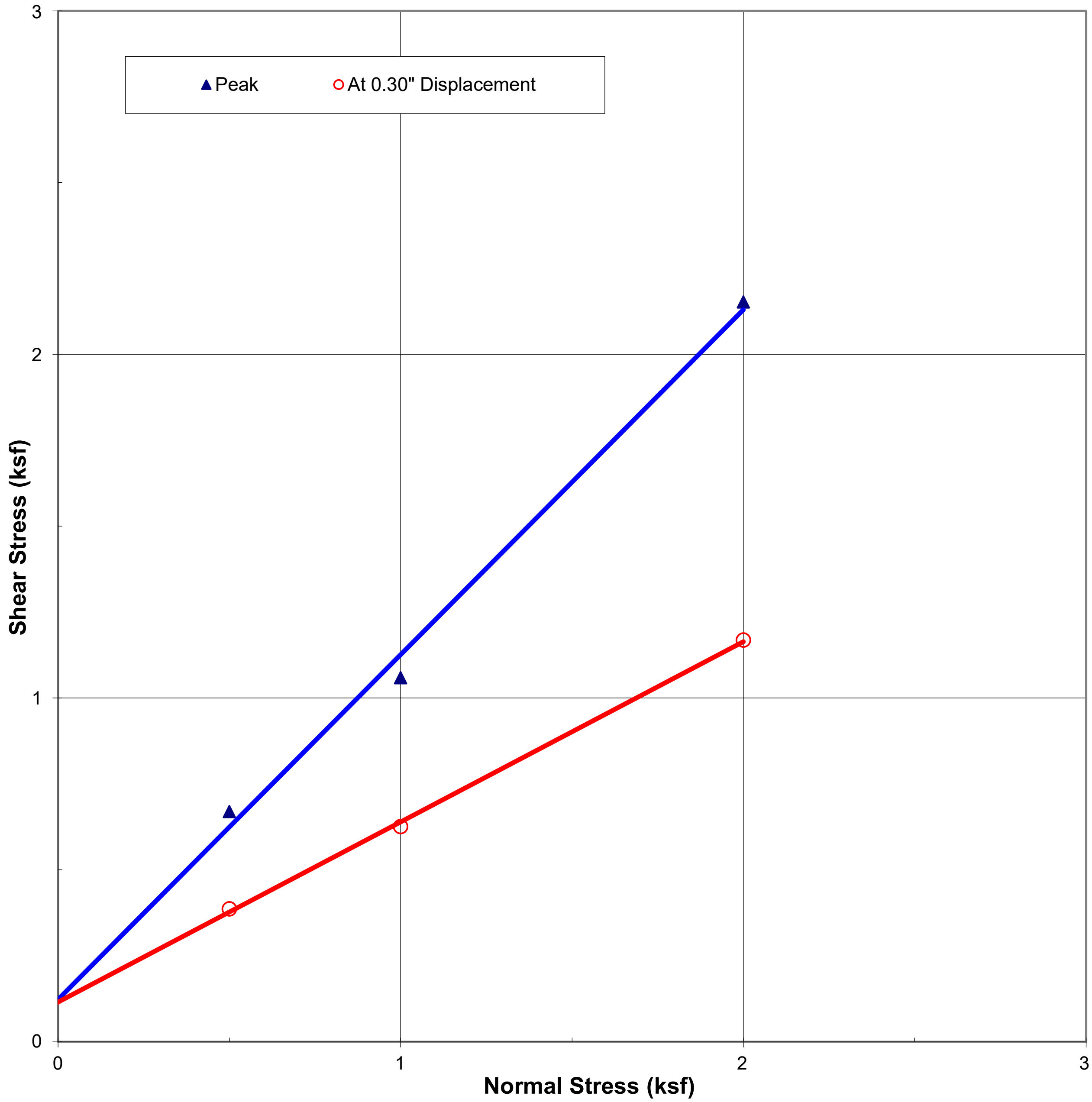
ATTERBERG LIMITS
(ASTM D 4318)

Project Number: 21291-01
Date: Aug-22

Parnell Park



Tested Samples:
HS-3 at 10 ft

	Peak:	At 0.30" Displacement:
	45.1 Degrees	27.7 Degrees
	0.12 ksf	0.12 ksf

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DIRECT SHEAR PLOT

Project Number: 21291-01
Date: Aug-22

Parnell Park

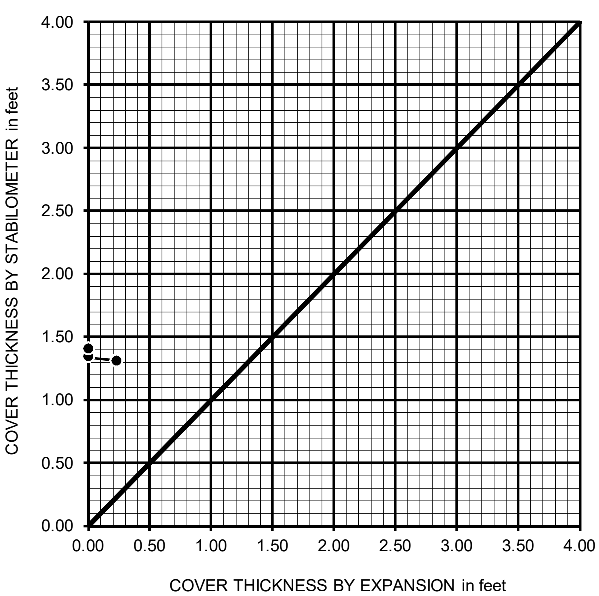
R-VALUE TEST RESULTS
DOT CA Test 301

PROJECT NAME:	SWA - Parnell Park	PROJECT NUMBER:	21291-01
BORING NUMBER:	HS-1	DEPTH (FT.):	0-5
SAMPLE NUMBER:	B-1	TECHNICIAN:	O. Figueroa
SAMPLE DESCRIPTION:	Very dark brown silty clay (CL-ML)	DATE COMPLETED:	8/9/2022

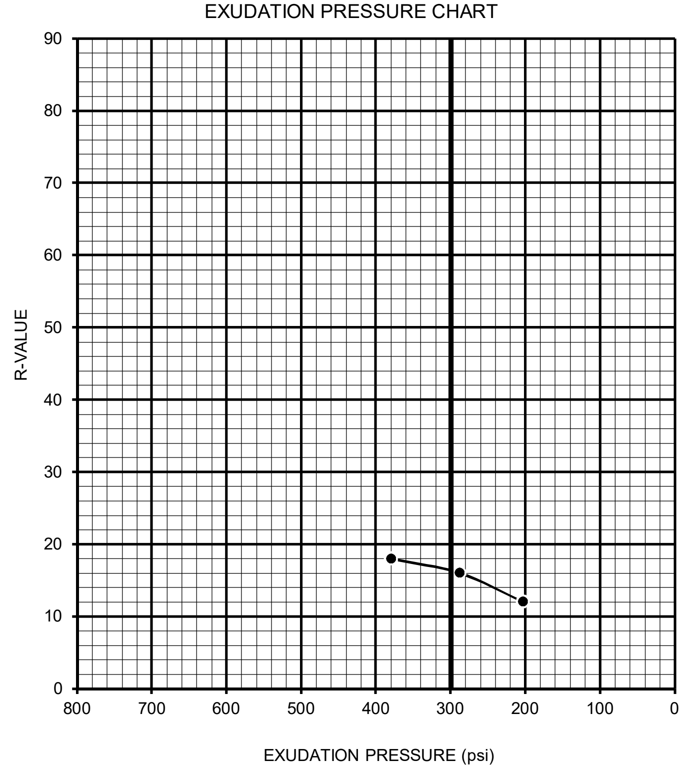
TEST SPECIMEN	a	b	c
MOISTURE AT COMPACTION %	17.1	18.0	19.3
HEIGHT OF SAMPLE, Inches	2.48	2.47	2.43
DRY DENSITY, pcf	113.6	112.9	109.5
COMPACTOR PRESSURE, psi	80	70	50
EXUDATION PRESSURE, psi	379	288	203
EXPANSION, Inches x 10exp-4	7	0	0
STABILITY Ph 2,000 lbs (160 psi)	119	124	132
TURNS DISPLACEMENT	3.80	3.95	4.03
R-VALUE UNCORRECTED	18	16	12
R-VALUE CORRECTED	18	16	12

DESIGN CALCULATION DATA	a	b	c
GRAVEL EQUIVALENT FACTOR	1.0	1.0	1.0
TRAFFIC INDEX	5.0	5.0	5.0
STABILOMETER THICKNESS, ft.	1.31	1.34	1.41
EXPANSION PRESSURE THICKNESS, ft.	0.23	0.00	0.00

EXPANSION PRESSURE CHART



EXUDATION PRESSURE CHART



R-VALUE BY EXPANSION:	25
R-VALUE BY EXUDATION:	16
EQUILIBRIUM R-VALUE:	16

Appendix D
Infiltration Test Data Sheets

Infiltration Test Data Sheet

LGC Geotechnical, Inc

131 Calle Iglesia Suite A, San Clemente, CA 92672 tel. (949) 369-6141

Project Name: SWA - Parnell Park
Project Number: 21291-01
Date: 7/22/2022
Location: I-1

Test hole dimensions (if circular)	
Boring Depth (feet)*:	7
Boring Diameter (inches):	8
Pipe Diameter (inches):	3

*measured at time of test

Test pit dimensions (if rectangular)	
Pit Depth (feet):	
Pit Length (feet):	
Pit Breadth (feet):	

Pre-Soak /Pre-Test

No.	Start Time (24:HR)	Stop Time (24:HR)	Time Interval (min)	Initial Depth to Water (feet)	Final Depth to Water (feet)	Total Change in Water Level (feet)	Comments
PS-1	7:20	7:50	30.0	1.00	1.04	0.04	
PS-2	7:50	8:20	30.0	1.04	1.09	0.05	
Pre-Test	8:20	8:50	30.0	1.09	1.13	0.04	

Main Test Data

Trial No.	Start Time (24:HR)	Stop Time (24:HR)	Time Interval, Δt (min)	Initial Depth to Water, D_o (feet)	Final Depth to Water, D_f (feet)	Change in Water Level, ΔD (feet)	Surface Area of Test Section (feet ^2)	Raw Percolation Rate (in/hr)
1	8:50	9:20	30.0	1.13	1.17	0.04	12.64	0.0
2	9:20	9:50	30.0	1.17	1.20	0.03	12.56	0.0
3	9:50	10:20	30.0	1.20	1.23	0.03	12.50	0.0
4	10:20	10:50	30.0	1.23	1.26	0.03	12.43	0.0
5	10:50	11:20	30.0	1.26	1.29	0.03	12.37	0.0
6	11:20	11:50	30.0	1.29	1.32	0.03	12.31	0.0
7								
8								
9								
10								
11								
12								

Measured Infiltration Rate	**0.0**
Feasibility Factor of Safety	2
Feasibility Infiltration Rate	**0.0**

Sketch:

Notes:

Based on Guidelines from: LA County dated 06/2017
Spreadsheet Revised on: 12/23/2019

LGC Geotechnical, Inc.

Appendix E
General Earthwork and Grading Specifications for Rough Grading

General Earthwork and Grading Specifications for Rough Grading

1.0 General

1.1 Intent

These General Earthwork and Grading Specifications are for the grading and earthwork shown on the approved grading plan(s) and/or indicated in the geotechnical report(s). These Specifications are a part of the recommendations contained in the geotechnical report(s). In case of conflict, the specific recommendations in the geotechnical report shall supersede these more general Specifications. Observations of the earthwork by the project Geotechnical Consultant during the course of grading may result in new or revised recommendations that could supersede these specifications or the recommendations in the geotechnical report(s).

1.2 The Geotechnical Consultant of Record

Prior to commencement of work, the owner shall employ a qualified Geotechnical Consultant of Record (Geotechnical Consultant). The Geotechnical Consultant shall be responsible for reviewing the approved geotechnical report(s) and accepting the adequacy of the preliminary geotechnical findings, conclusions, and recommendations prior to the commencement of the grading.

Prior to commencement of grading, the Geotechnical Consultant shall review the "work plan" prepared by the Earthwork Contractor (Contractor) and schedule sufficient personnel to perform the appropriate level of observation, mapping, and compaction testing.

During the grading and earthwork operations, the Geotechnical Consultant shall observe, map, and document the subsurface exposures to verify the geotechnical design assumptions. If the observed conditions are found to be significantly different than the interpreted assumptions during the design phase, the Geotechnical Consultant shall inform the owner, recommend appropriate changes in design to accommodate the observed conditions, and notify the review agency where required.

The Geotechnical Consultant shall observe the moisture-conditioning and processing of the subgrade and fill materials and perform relative compaction testing of fill to confirm that the attained level of compaction is being accomplished as specified. The Geotechnical Consultant shall provide the test results to the owner and the Contractor on a routine and frequent basis.

1.3 The Earthwork Contractor

The Earthwork Contractor (Contractor) shall be qualified, experienced, and knowledgeable in earthwork logistics, preparation and processing of ground to receive fill, moisture-conditioning and processing of fill, and compacting fill. The Contractor shall review and accept the plans, geotechnical report(s), and these Specifications prior to commencement of grading. The Contractor shall be solely responsible for performing the grading in accordance with the project plans and specifications. The Contractor shall prepare and submit to the owner and the Geotechnical Consultant a work plan that indicates the sequence of earthwork grading, the number of "equipment" of work and the estimated quantities of daily earthwork contemplated for the site prior to commencement of grading. The Contractor shall inform the owner and the Geotechnical Consultant of changes in work schedules and updates to the work plan at least 24 hours in advance of such changes so that appropriate personnel will be available for observation and testing. The Contractor shall not assume that the Geotechnical Consultant is aware of all grading operations.

The Contractor shall have the sole responsibility to provide adequate equipment and methods to accomplish the earthwork in accordance with the applicable grading codes and agency ordinances, these Specifications, and the recommendations in the approved geotechnical report(s) and grading plan(s). If, in the opinion of the Geotechnical Consultant, unsatisfactory conditions, such as unsuitable soil, improper moisture condition, inadequate compaction, insufficient buttress key size, adverse weather, etc., are resulting in a quality of work less than required in these specifications, the Geotechnical Consultant shall reject the work and may recommend to the owner that construction be stopped until the conditions are rectified. It is the contractor's sole responsibility to provide proper fill compaction.

2.0 Preparation of Areas to be Filled

2.1 Clearing and Grubbing

Vegetation, such as brush, grass, roots, and other deleterious material shall be sufficiently removed and properly disposed of in a method acceptable to the owner, governing agencies, and the Geotechnical Consultant.

The Geotechnical Consultant shall evaluate the extent of these removals depending on specific site conditions. Earth fill material shall not contain more than 1 percent of organic materials (by volume). Nesting of the organic materials shall not be allowed.

If potentially hazardous materials are encountered, the Contractor shall stop work in the affected area, and a hazardous material specialist shall be informed immediately for proper evaluation and handling of these materials prior to continuing to work in that area.

As presently defined by the State of California, most refined petroleum products (gasoline, diesel fuel, motor oil, grease, coolant, etc.) have chemical constituents that are considered to be hazardous waste. As such, the indiscriminate dumping or spillage of these fluids onto the ground may constitute a misdemeanor, punishable by fines and/or imprisonment, and shall not be allowed. The contractor is responsible for all hazardous waste relating to his work. The Geotechnical Consultant does not have expertise in this area. If hazardous waste is a concern, then the Client should acquire the services of a qualified environmental assessor.

2.2 Processing

Existing ground that has been declared satisfactory for support of fill by the Geotechnical Consultant shall be scarified to a minimum depth of 6 inches. Existing ground that is not satisfactory shall be over-excavated as specified in the following section. Scarification shall continue until soils are broken down and free of oversize material and the working surface is reasonably uniform, flat, and free of uneven features that would inhibit uniform compaction.

2.3 Over-excavation

In addition to removals and over-excavations recommended in the approved geotechnical report(s) and the grading plan, soft, loose, dry, saturated, spongy, organic-rich, highly fractured or otherwise unsuitable ground shall be over-excavated to competent ground as evaluated by the Geotechnical Consultant during grading.

2.4 Benching

Where fills are to be placed on ground with slopes steeper than 5:1 (horizontal to vertical units), the ground shall be stepped or benched. Please see the Standard Details for a graphic illustration. The lowest bench or key shall be a minimum of 15 feet wide and at least 2 feet deep, into competent material as evaluated by the Geotechnical Consultant. Other benches shall be excavated a minimum height of 4 feet into competent material or as otherwise recommended by the Geotechnical Consultant. Fill placed on ground sloping flatter than 5:1 shall also be benched or otherwise over-excavated to provide a flat subgrade for the fill.

2.5 Evaluation/Acceptance of Fill Areas

All areas to receive fill, including removal and processed areas, key bottoms, and benches, shall be observed, mapped, elevations recorded, and/or tested prior to being accepted by the Geotechnical Consultant as suitable to receive fill. The Contractor shall obtain a written acceptance from the Geotechnical Consultant prior to fill placement. A licensed surveyor shall provide the survey control for determining elevations of processed areas, keys, and benches.

3.0 Fill Material

3.1 General

Material to be used as fill shall be essentially free of organic matter and other deleterious substances evaluated and accepted by the Geotechnical Consultant prior to placement. Soils of poor quality, such as those with unacceptable gradation, high expansion potential, or low strength shall be placed in areas acceptable to the Geotechnical Consultant or mixed with other soils to achieve satisfactory fill material.

3.2 Oversize

Oversize material defined as rock, or other irreducible material with a maximum dimension greater than 8 inches, shall not be buried or placed in fill unless location, materials, and placement methods are specifically accepted by the Geotechnical Consultant. Placement operations shall be such that nesting of oversized material does not occur and such that oversize material is completely surrounded by compacted or densified fill. Oversize material shall not be placed within 10 vertical feet of finish grade or within 2 feet of future utilities or underground construction.

3.3 Import

If importing of fill material is required for grading, proposed import material shall meet the requirements of the geotechnical consultant. The potential import source shall be given to the Geotechnical Consultant at least 48 hours (2 working days) before importing begins so that its suitability can be determined and appropriate tests performed.

4.0 Fill Placement and Compaction

4.1 Fill Layers

Approved fill material shall be placed in areas prepared to receive fill (per Section 3.0) in near-horizontal layers not exceeding 8 inches in loose thickness. The Geotechnical Consultant may accept thicker layers if testing indicates the grading procedures can adequately compact the thicker layers. Each layer shall be spread evenly and mixed thoroughly to attain relative uniformity of material and moisture throughout.

4.2 Fill Moisture Conditioning

Fill soils shall be watered, dried back, blended, and/or mixed, as necessary to attain a relatively uniform moisture content at or slightly over optimum. Maximum density and optimum soil moisture content tests shall be performed in accordance with the American Society of Testing and Materials (ASTM Test Method D1557).

4.3 Compaction of Fill

After each layer has been moisture-conditioned, mixed, and evenly spread, it shall be uniformly compacted to not less than 90 percent of maximum dry density (ASTM Test Method D1557). Compaction equipment shall be adequately sized and be either specifically designed for soil compaction or of proven reliability to efficiently achieve the specified level of compaction with uniformity.

4.4 Compaction of Fill Slopes

In addition to normal compaction procedures specified above, compaction of slopes shall be accomplished by backrolling of slopes with sheepsfoot rollers at increments of 3 to 4 feet in fill elevation, or by other methods producing satisfactory results acceptable to the Geotechnical Consultant. Upon completion of grading, relative compaction of the fill, out to the slope face, shall be at least 90 percent of maximum density per ASTM Test Method D1557.

4.5 Compaction Testing

Field tests for moisture content and relative compaction of the fill soils shall be performed by the Geotechnical Consultant. Location and frequency of tests shall be at the Consultant's discretion based on field conditions encountered. Compaction test locations will not necessarily be selected on a random basis. Test locations shall be selected to verify adequacy of compaction levels in areas that are judged to be prone to inadequate compaction (such as close to slope faces and at the fill/bedrock benches).

4.6 Frequency of Compaction Testing

Tests shall be taken at intervals not exceeding 2 feet in vertical rise and/or 1,000 cubic yards of compacted fill soils embankment. In addition, as a guideline, at least one test shall be taken on slope faces for each 5,000 square feet of slope face and/or each 10 feet of vertical height of slope. The Contractor shall assure that fill construction is such that the testing schedule can be accomplished by the Geotechnical Consultant. The Contractor shall stop or slow down the earthwork construction if these minimum standards are not met.

4.7 Compaction Test Locations

The Geotechnical Consultant shall document the approximate elevation and horizontal coordinates of each test location. The Contractor shall coordinate with the project surveyor to assure that sufficient grade stakes are established so that the Geotechnical Consultant can determine the test locations with sufficient accuracy. At a minimum, two grade stakes within a horizontal distance of 100 feet and vertically less than
5 feet apart from potential test locations shall be provided.

5.0 Subdrain Installation

Subdrain systems shall be installed in accordance with the approved geotechnical report(s), the grading plan, and the Standard Details. The Geotechnical Consultant may recommend additional subdrains and/or changes in subdrain extent, location, grade, or material depending on conditions encountered during grading. All subdrains shall be surveyed by a land surveyor/civil engineer for line and grade after installation and prior to burial. Sufficient time should be allowed by the Contractor for these surveys.

6.0 Excavation

Excavations, as well as over-excavation for remedial purposes, shall be evaluated by the Geotechnical Consultant during grading. Remedial removal depths shown on geotechnical plans are estimates only. The actual extent of removal shall be determined by the Geotechnical Consultant based on the field evaluation of exposed conditions during grading. Where fill-over-cut slopes are to be graded, the cut portion of the slope shall be made, evaluated, and accepted by the Geotechnical Consultant prior to placement of materials for construction of the fill portion of the slope, unless otherwise recommended by the Geotechnical Consultant.

7.0 Trench Backfills

7.1 The Contractor shall follow all OHSA and Cal/OSHA requirements for safety of trench excavations.

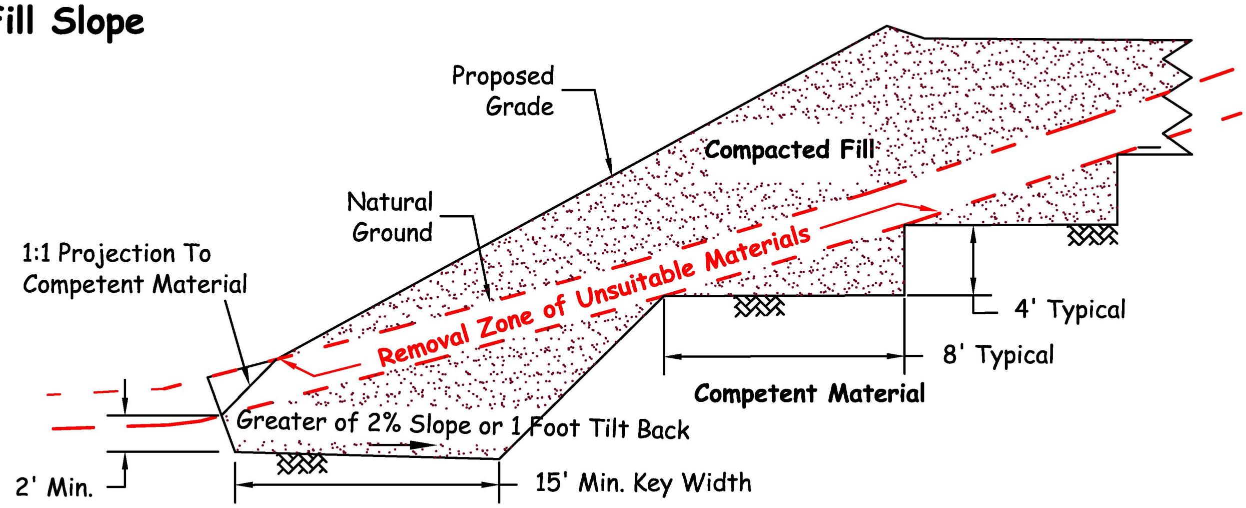
7.2 All bedding and backfill of utility trenches shall be done in accordance with the applicable provisions of Standard Specifications of Public Works Construction. Bedding material shall have a Sand Equivalent greater than 30 (SE>30). The bedding shall be placed to 1 foot over the top of the conduit and densified by jetting. Backfill shall be placed and densified to a minimum of 90 percent of maximum from 1 foot above the top of the conduit to the surface.

7.3 The jetting of the bedding around the conduits shall be observed by the Geotechnical Consultant.

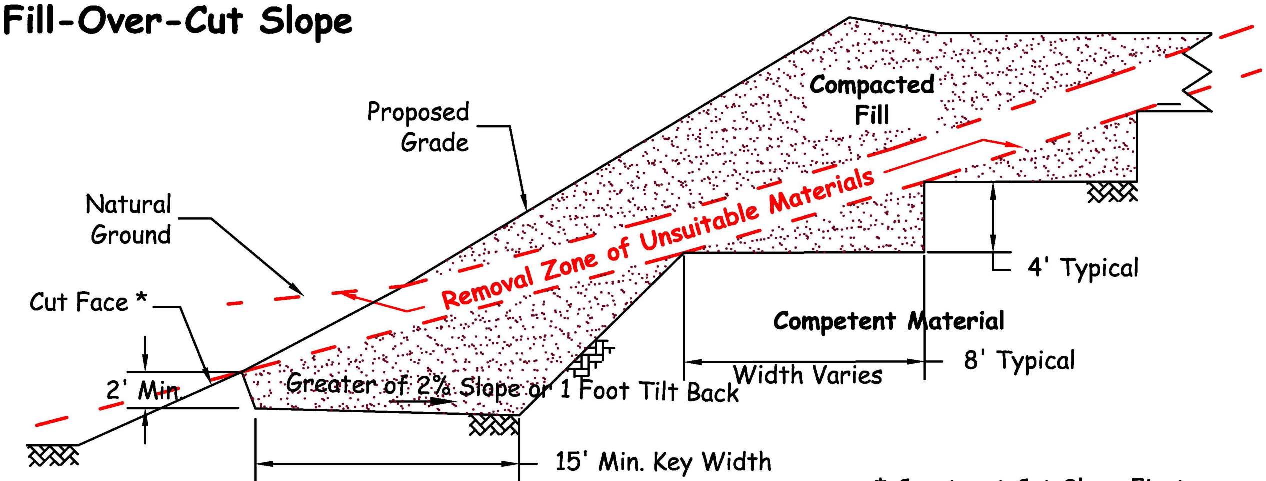
7.4 The Geotechnical Consultant shall test the trench backfill for relative compaction. At least one test should be made for every 300 feet of trench and 2 feet of fill.

7.5 Lift thickness of trench backfill shall not exceed those allowed in the Standard Specifications of Public Works Construction unless the Contractor can demonstrate to the Geotechnical Consultant that the fill lift can be compacted to the minimum relative compaction by his alternative equipment and method.

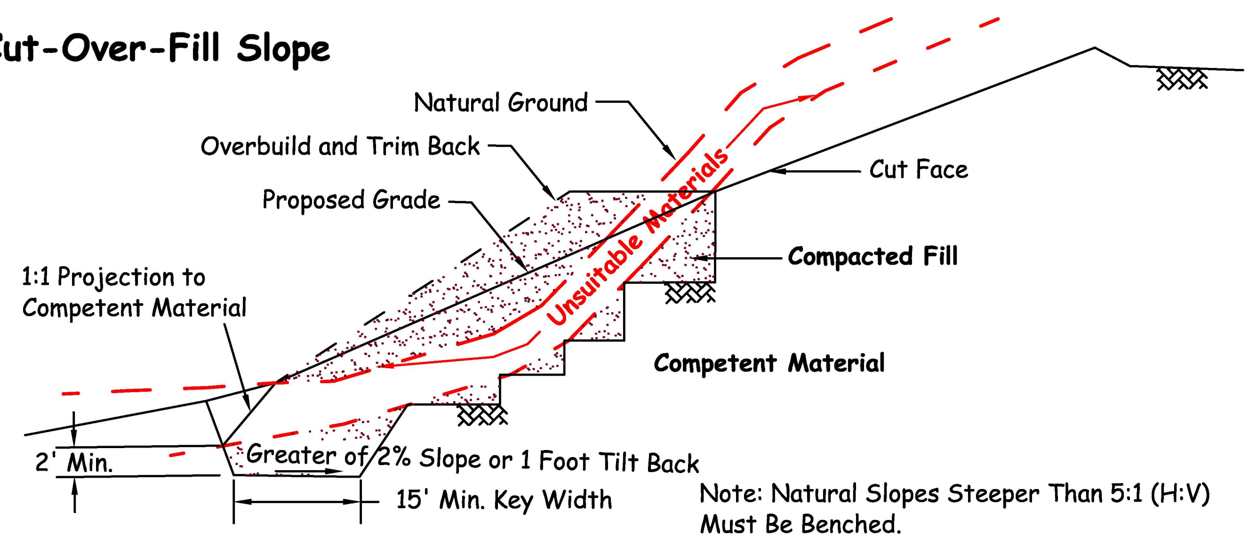
Fill Slope



Fill-Over-Cut Slope



Cut-Over-Fill Slope



Note: Natural Slopes Steeper Than 5:1 (H:V) Must Be Benched.

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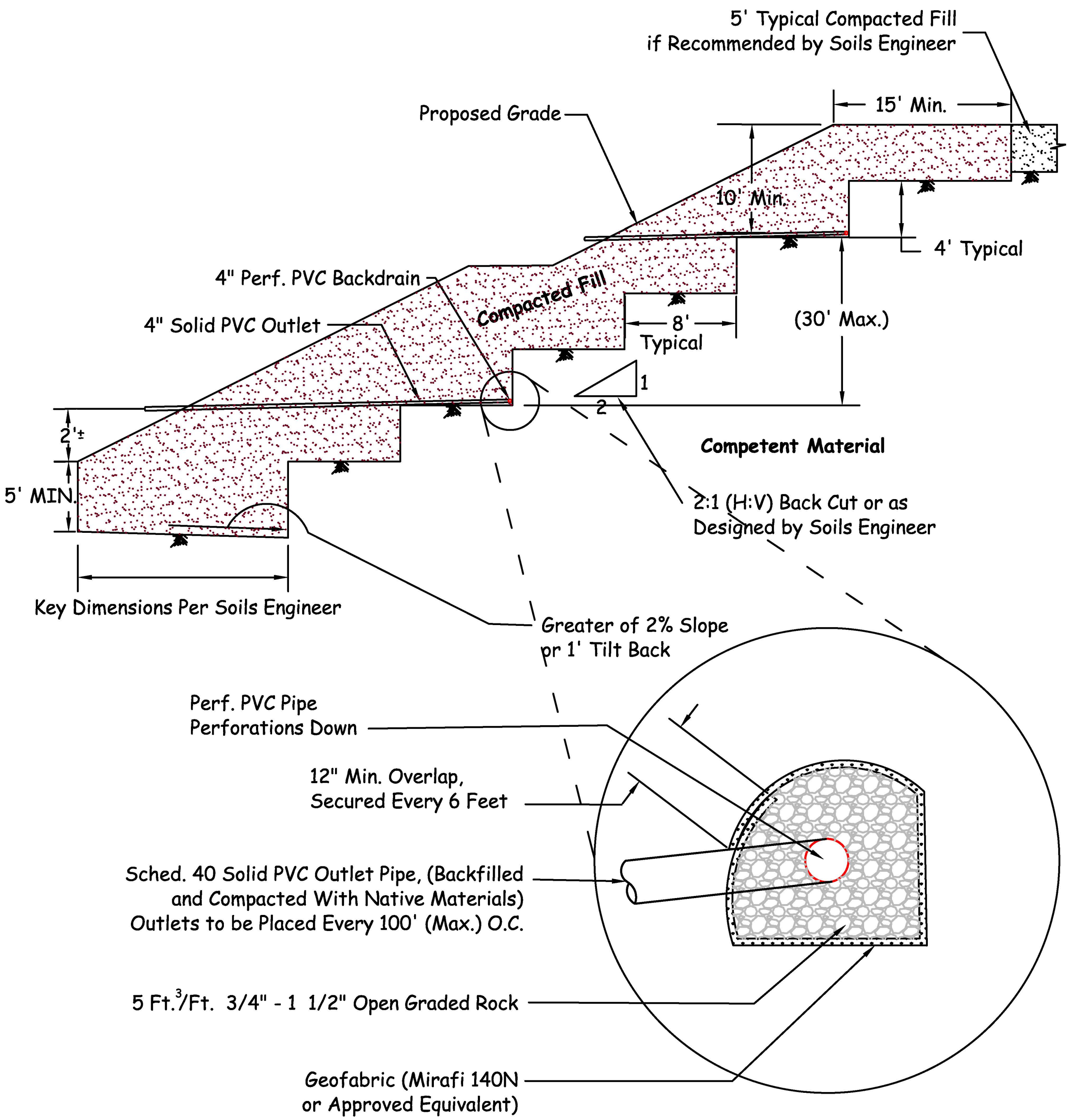
KEYING AND BENCHING



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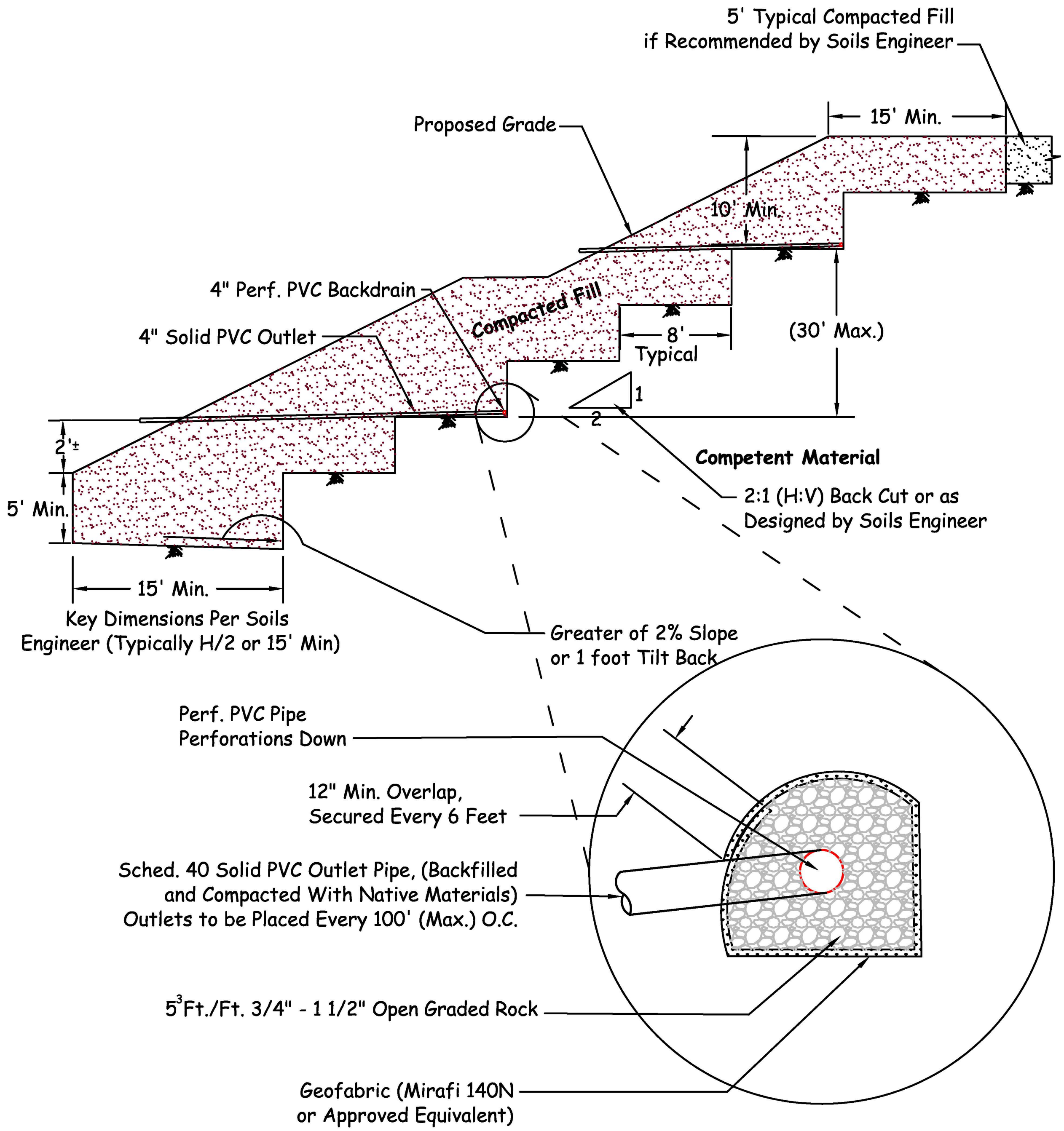
TYPICAL BUTTRESS DETAIL



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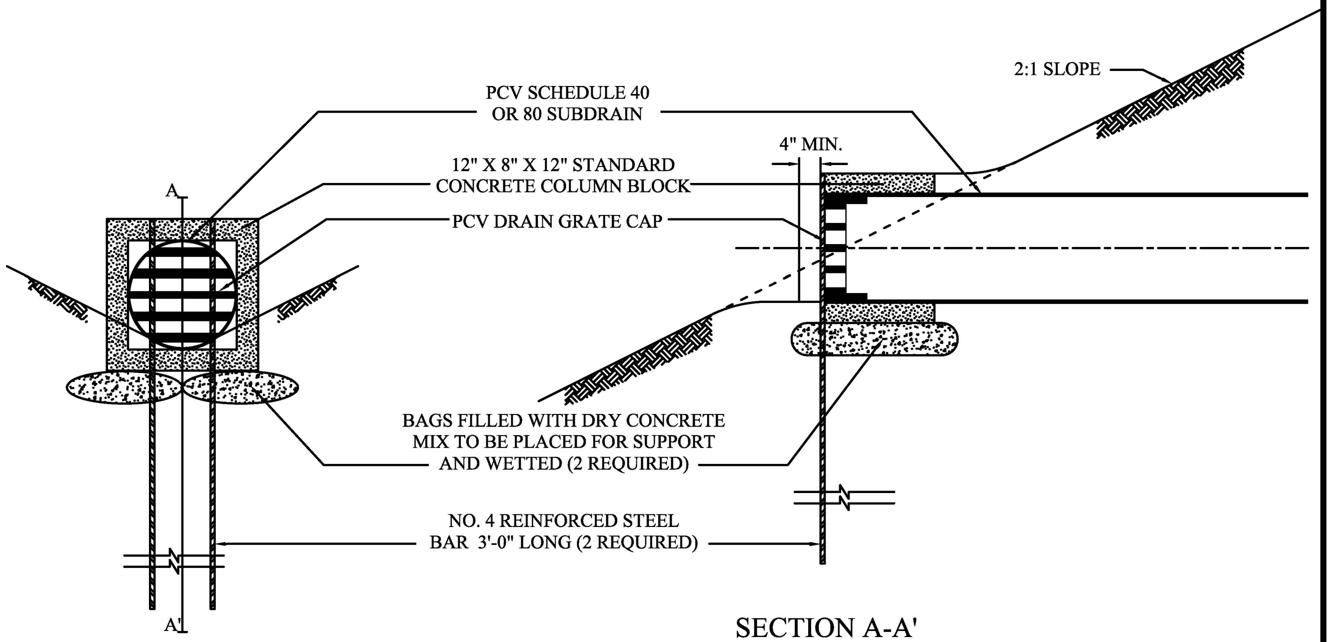
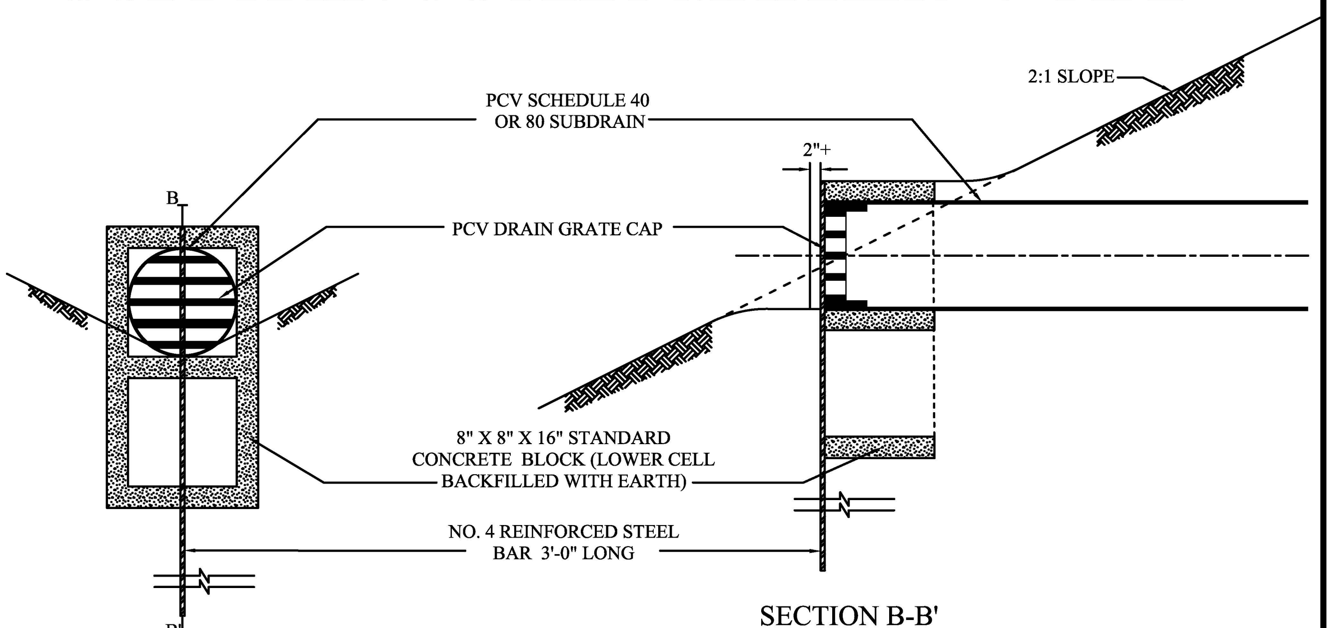
TYPICAL STABILIZATION FILL DETAIL

SUBDRAIN OUTLET MARKER -6" & 8" PIPE



SECTION A-A'

SUBDRAIN OUTLET MARKER -4" PIPE



SECTION B-B'

NOT TO SCALE

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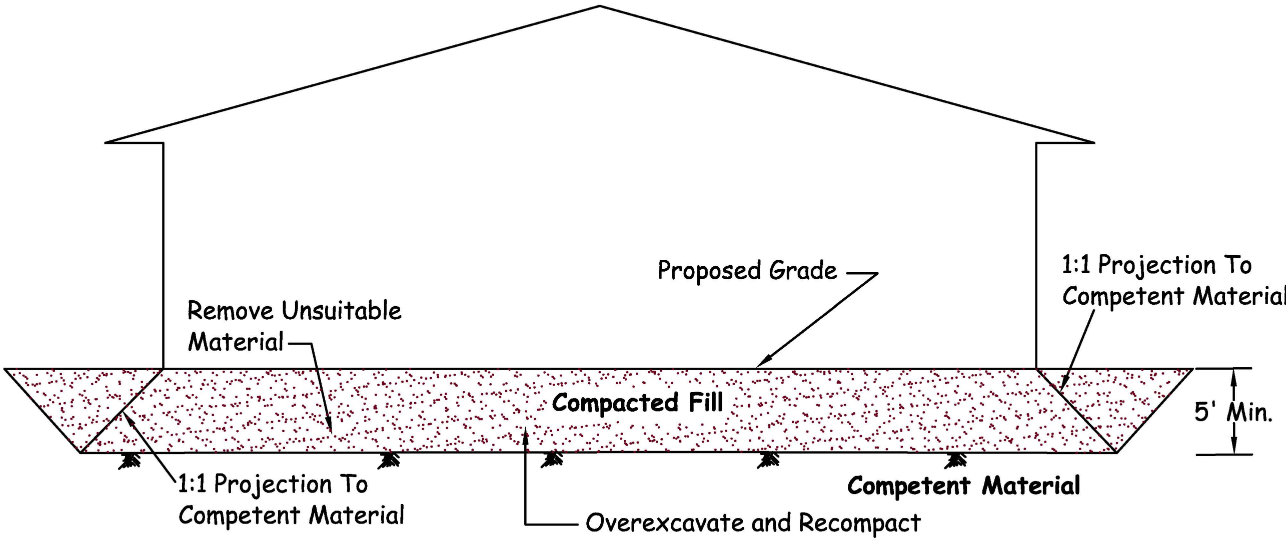
SUBDRAIN OUTLET MARKER DETAIL

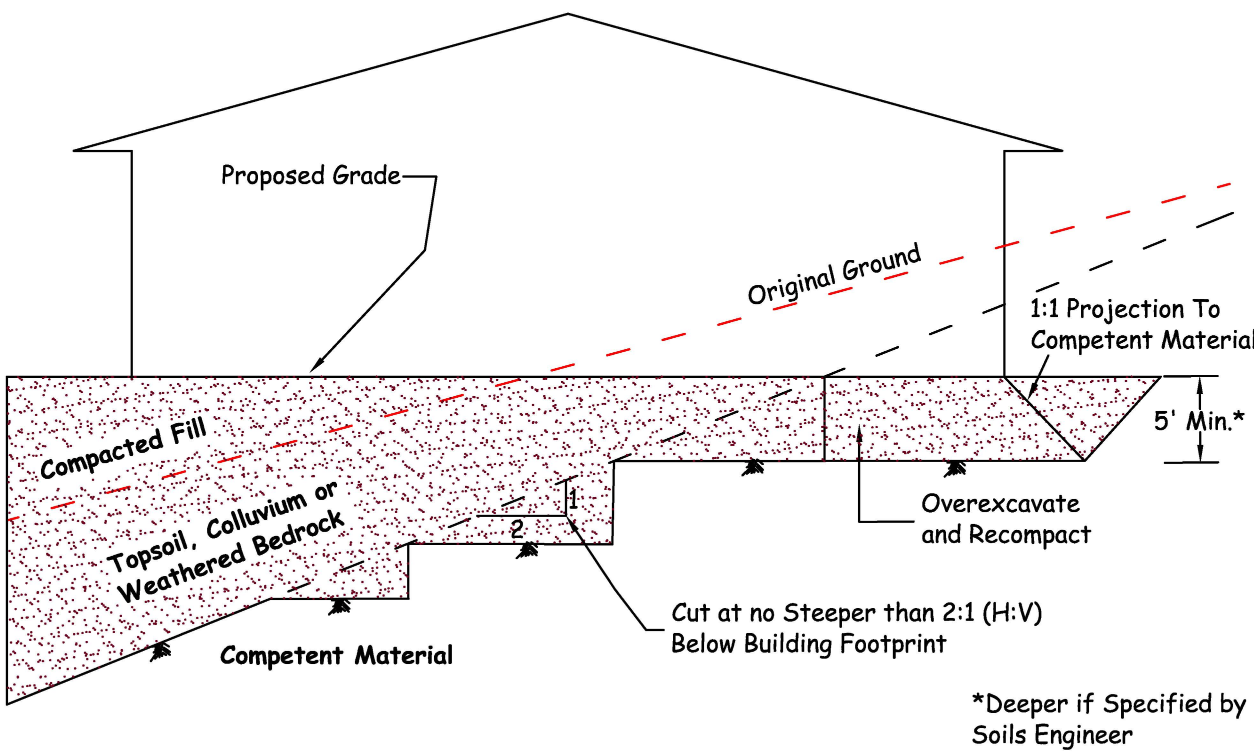
Cut Lot
(Exposing Unsuitable Soils at Design Grade)



Note 1: Removal Bottom Should be Graded With Minimum 2% Fall Towards Street or Other Suitable Area (as Determined by Soils Engineer) to Avoid Ponding Below Building

Note 2: Where Design Cut Lots are Excavated Entirely Into Competent Material, Overexcavation May Still be Required for Hard-Rock Conditions or for Materials With Variable Expansion Characteristics.

Cut/Fill Transition Lot



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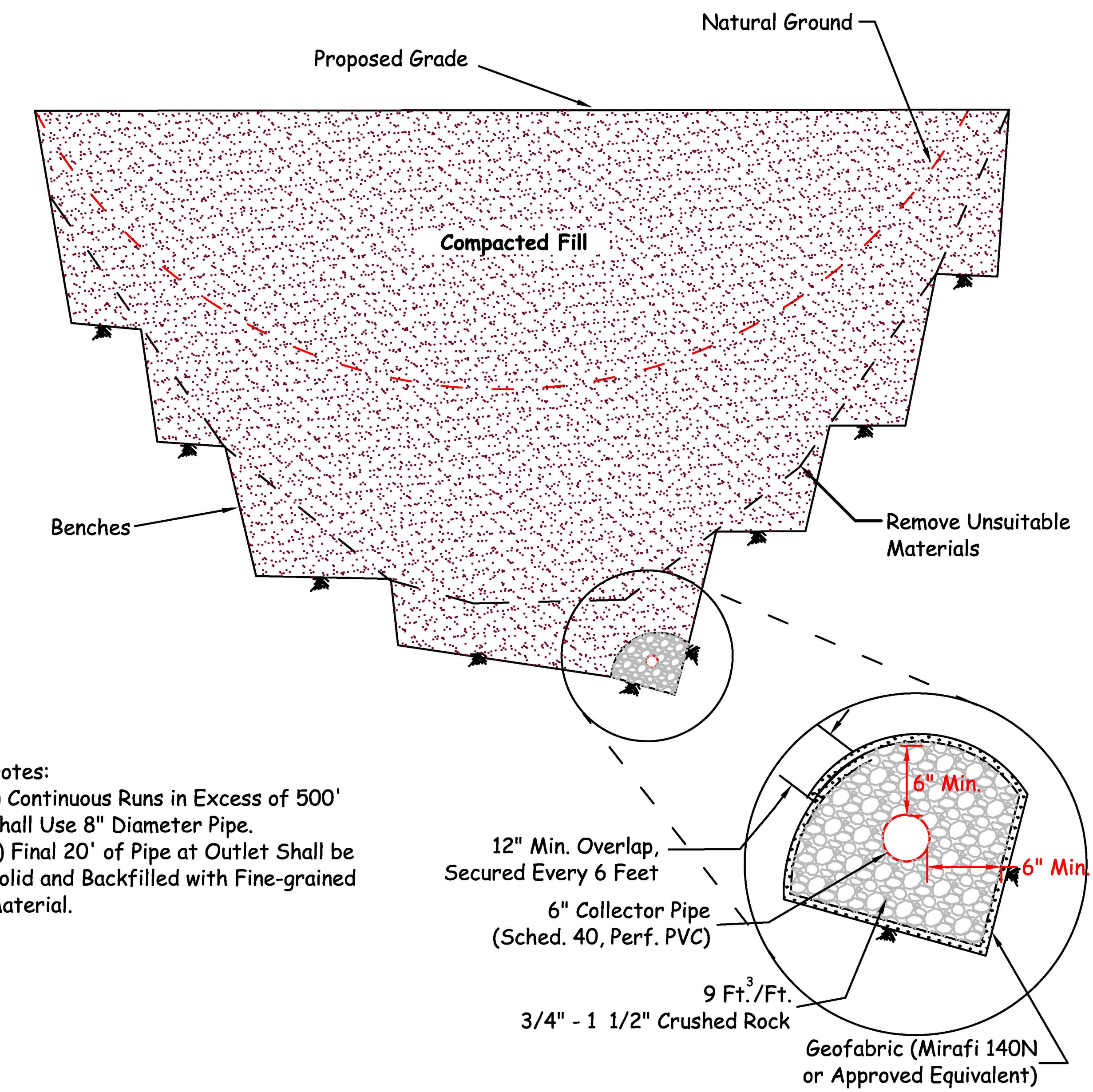
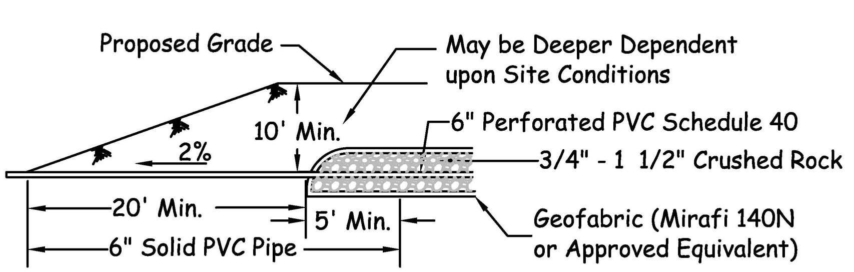
CUT AND TRANSITION LOT OVEREXCAVATION DETAIL



Notes:
1) Continuous Runs in Excess of 500' Shall Use 8" Diameter Pipe.
2) Final 20' of Pipe at Outlet Shall be Solid and Backfilled with Fine-grained Material.

Proposed Outlet Detail



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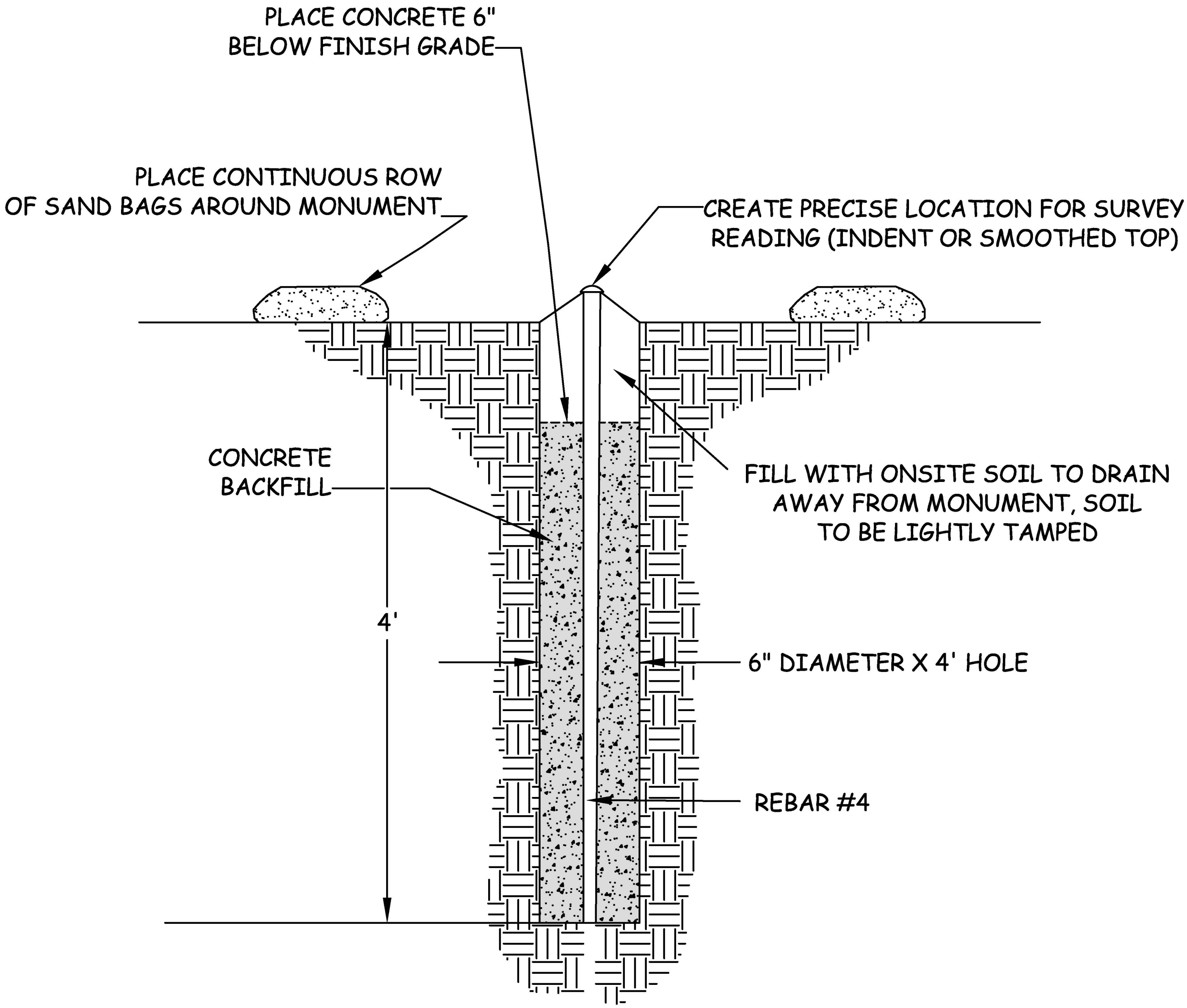
CANYON SUBDRAINS

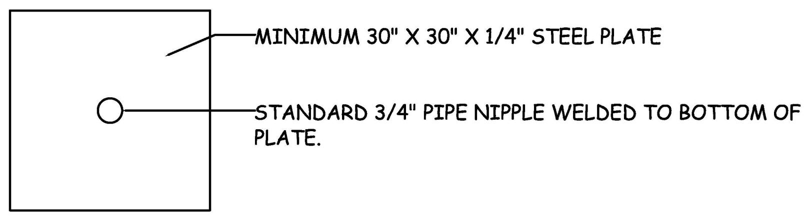


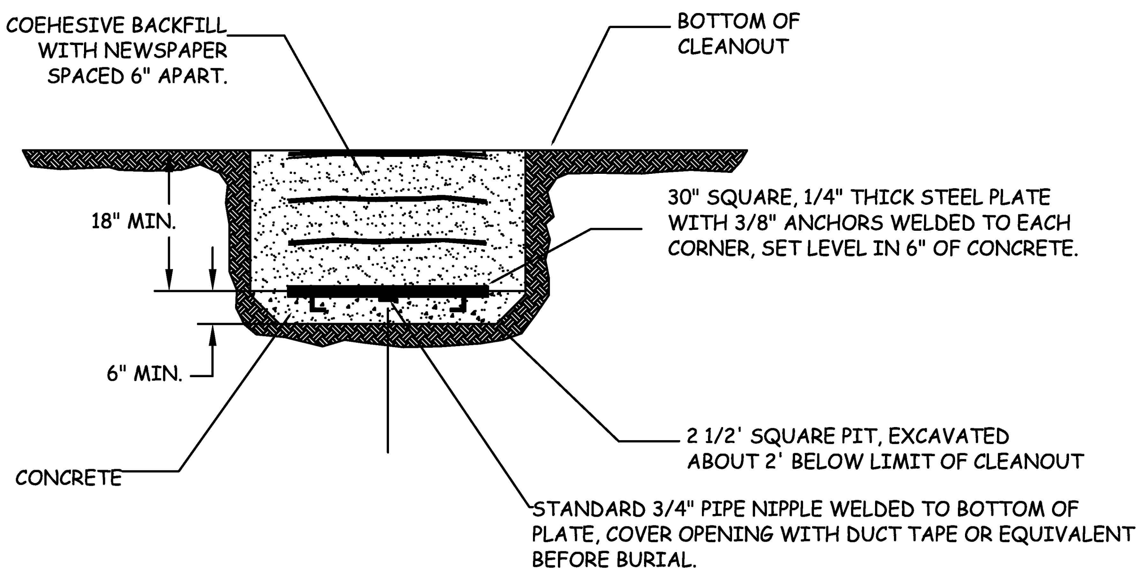
NO CONSTRUCTION EQUIPMENT WITHIN 25 FEET
OF ANY INSTALLED SETTLEMENT MONUMENTS

LGC Geotechnical, Inc.

TYPICAL SURFACE SETTLEMENT MONUMENT

TOP VIEW





1. SURVEY FOR HORIZONTAL AND VERTICAL LOCATION TO NEAREST .01 INCH PRIOR TO BACKFILL USING KNOW LOCATIONS THAT WILL REMAIN INTACT DURING THE DURATION OF THE MONITORING PROGRAM. KNOW POINTS EXPLICITELY NOT ALLOWED ARE THOSE LOCATED ON FILL OR THAT WILL BE DESTROYED DURING GRADING.
2. IN THE EVENT OF DAMAGE TO SETTLEMENT PLATE DURING GRADING, CONTRACTOR SHALL IMMEDIATELY NOTIFY THE GEOTECHNICAL ENGINEER AND SHALL BE RESPONSIBLE FOR RESTORING THE SETTLEMENT PLATES TO WORKING ORDER.
3. DRILL TO RECOVER AND ATTACH RISER PIPE.

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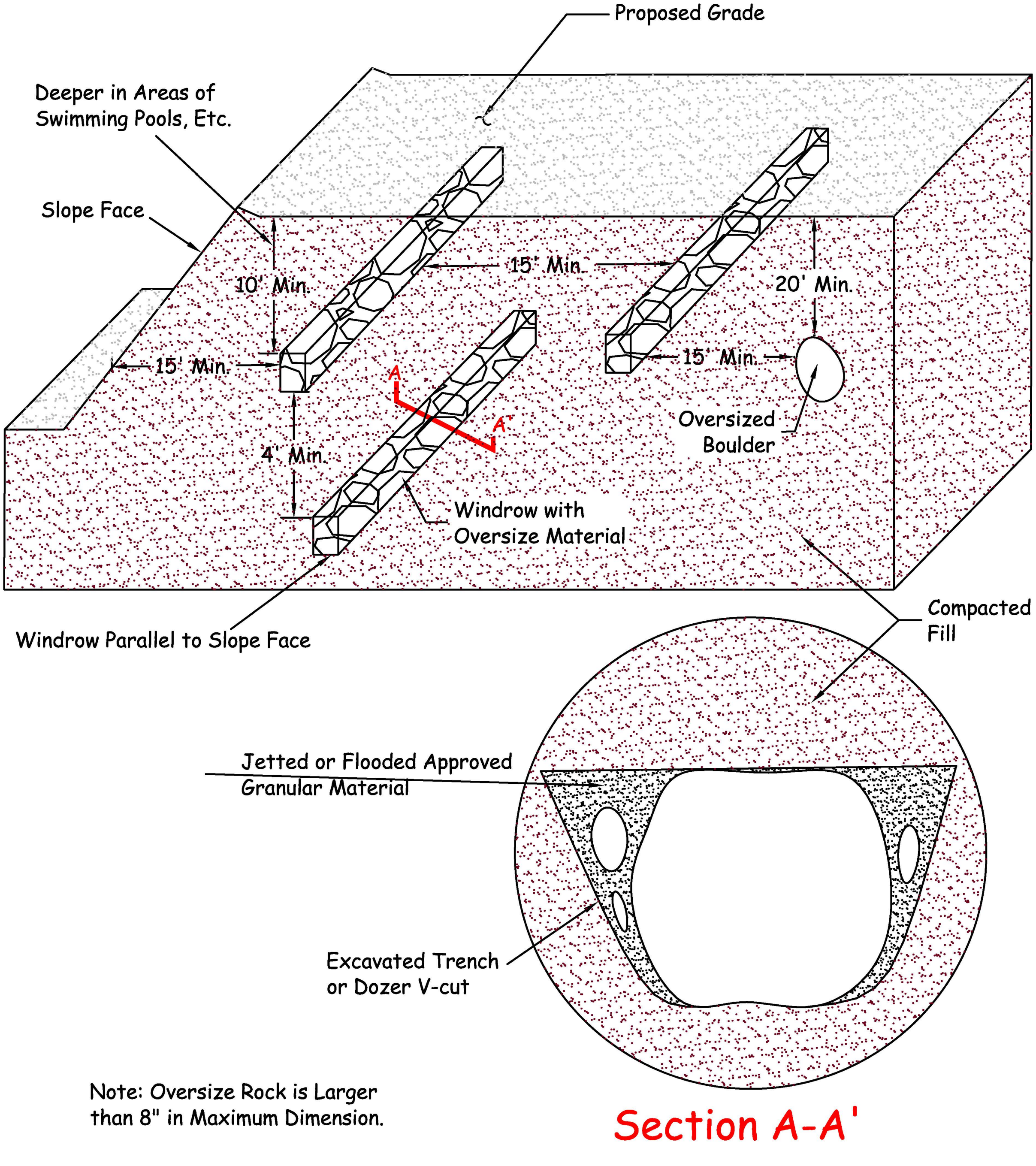
TYPICAL SETTLEMENT PLATE AND RISER



Note: Oversize Rock is Larger than 8" in Maximum Dimension.

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OVERSIZE ROCK DISPOSAL DETAIL

Appendix F
Liquefaction Evaluation

LIQUEFACTION EVALUATION

Based on *Proceeding of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils* , Technical Report NCEER-97-0022, December 31, 1997
and *Evaluation of Settlments in Sand due to Earthquake Shaking* , Tokimatsu and Seed, 1987

Seismic Event		**Profile Constants**		**Depth to GWT**		**Project Name**	SWA - Parnell Park
Moment Magnitude	**6.8**	Total Unit Weight (lb/ft^3)	120	During Investigation (ft)	21	**Project Number**	21291-01
Peak Ground Acceleration	**0.835** g	Unit Weight of Water (lbs/ft^3	62.4	During Design Event (ft)	15	**Boring**	HS- 2

Determination of Cyclic Resitance Ratio

Sampling Data					During Investigation			Sampling Correction Factors												
		Blow Count		Thickness	Total Stress	Pore Pressure	Effective	Sampler	SPT	Overburden	Energy	Borehole	Rod Length	Sampler Type		Fines				
Depth (ft)	Depth (m)	SPT	Rings	(ft)	Stress (psf)	Pressure (psf)	Stress (psf)	Diameter	N_m	C_N	C_E	C_B	C_R	C_S	$(N_1)_{60}$	Content	**$(N_1)_{60cs}$**	K_σ	$CRR_{7.5}$	Depth
2.5	0.8		62	2.5	420	0	420	0.62	38.44	1.70	1.25	1.00	0.75	1.00	61.26	15	**66.71**	1.000	SPT >30 NF	2.5
5	1.5		64	2.5	720	0	720	0.62	39.68	1.70	1.25	1.00	0.75	1.00	63.35	15	**68.89**	1.000	SPT >30 NF	5
7.5	2.3		92	2.5	1020	0	1020	0.62	57.04	1.43	1.25	1.00	0.75	1.00	76.51	50	**96.81**	1.000	SPT >30 NF	7.5
10	3.0		50	5	1320	0	1320	0.62	31.00	1.26	1.25	1.00	0.75	1.00	36.55	50	**48.86**	1.000	SPT >30 NF	10
15	4.6	26		5	1920	0	1920	1.00	26.00	1.04	1.25	1.00	0.85	1.10	31.69	50	**43.03**	1.000	SPT >30 NF	15
20	6.1		50	5	2520	0	2520	0.62	31.00	0.91	1.25	1.00	0.95	1.00	33.51	82	**45.21**	0.964	SPT >30 NF	20
25	7.6	29		5	3120	249.6	2870.4	1.00	29.00	0.85	1.25	1.00	0.95	1.10	32.31	70	**43.77**	0.940	SPT >30 NF	25
30	9.1		33	5	3720	561.6	3158.4	0.62	20.46	0.81	1.25	1.00	0.95	1.00	19.75	61	**28.71**	0.922	0.336	30
35	10.7	29		5	4320	873.6	3446.4	1.00	29.00	0.78	1.58	1.00	1.00	1.10	39.23	50	**52.08**	0.905	SPT >30 NF	35
40	12.2		32	5	4920	1185.6	3734.4	0.62	19.84	0.75	1.58	1.00	1.00	1.00	23.44	50	**33.13**	0.888	SPT >30 NF	40
45	13.7	23		5	5520	1497.6	4022.4	1.00	23.00	0.72	1.58	1.00	1.00	1.10	28.80	85	**39.56**	0.873	SPT >30 NF	45
50	15.2		38	1.5	6120	1809.6	4310.4	0.62	23.56	0.70	1.58	1.00	1.00	1.00	25.91	50	**36.09**	0.858	SPT >30 NF	50
51.5	15.7																			

Determination of Cyclic Stress Ratio

Sampling Data					During Design Event						
		Blow Count			Total Stress	Pore Pressure	Effective				
Depth (ft)	Depth (m)	SPT	Rings	Thickness	Stress (psf)	Pressure (psf)	Stress (psf)	r_d	CSR	MSF	FS
2.5	0.76		62	2.5	300	0	300	0.99615	0.54066	1.285	Above GWT
5	1.52		64	2.5	600	0	600	0.99024	0.537451	1.285	Above GWT
7.5	2.29		92	2.5	900	0	900	0.98456	0.53437	1.285	Above GWT
10	3.05		50	5	1200	0	1200	0.97914	0.531429	1.285	Above GWT
15	4.57	26		5	1800	0	1800	0.96856	0.525685	1.285	Corr. SPT>30
20	6.10		50	5	2400	312	2088	0.9569	0.59696	1.285	Corr. SPT>30
25	7.62	29		5	3000	624	2376	0.94183	0.64543	1.285	Corr. SPT>30
30	9.14		33	5	3600	936	2664	0.92058	0.675194	1.285	Clay-Bray
35	10.67	29		5	4200	1248	2952	0.89062	0.68774	1.285	Corr. SPT>30
40	12.19		32	5	4800	1560	3240	0.85103	0.684295	1.285	Corr. SPT>30
45	13.72	23		5	5400	1872	3528	0.80363	0.667607	1.285	Corr. SPT>30
50	15.24		38	1.5	6000	2184	3816	0.75271	0.642352	1.285	Corr. SPT>30
51.5	15.70										