APPENDIX H
Hydrology and Water Quality

APPENDIX H1
Preliminary LID Report, The Residences at Duarte Station

Preliminary LID Report

The Residences at Duarte Station

Duarte, California

April 24, 2019

Prepared for

MW Investment Group, LLC

Prepared by

KHR ASSOCIATES

Consulting Engineers - Surveyors - Planners

ATTESTATION

This report has been prepared by, and under the direction of, the undersigned, a duly Registered Civil Engineer in the State of California. Except as noted, the undersigned attests to the technical information contained herein, and has judged to be acceptable the qualifications of any technical specialists providing engineering data for this report, upon which findings, conclusions, and recommendations are based.

James H. Kawamura, P.E.
Registered Civil Engineer No. C30560
Exp. 3/31/20

TABLE OF CONTENTS

APPENDIX

Section 1 Project Type and Regulations

Zone: SP-18 Metro RR – Duarte Station Specific Plan (Specific Plan Zone)

Project Area: 7.75 acres (337,769 S.F.)

Priority Project Category: Designated Project (Redevelopment Project where 50 percent or more of the impervious surface of a previously developed site is proposed to be altered and the previous development project was not subject to post-construction stormwater quality control measures, and which are developments that result in creation or addition of 5,000 square feet or more of impervious surface on a site that was previously developed as described in Section 2-1 of County of Los Angeles Department of Public Works of Low Impact Development Standards Manual dated February 2014.)

Assessor's Number: APN 8528-011-025

Rain Season: October 19th through April 23rd

Watershed: Los Angeles River Watershed

Regulations: National Pollution Discharge Elimination System (NPDES) Municipal Separate Storm Sewer System (MS4) Permit (CAS004001, Order No. R4-2012-0175); Los Angeles County Code Title 12, Chapter 84

Regulatory Agents: City of Duarte City Director of Public Works, their authorized deputy, agent, representative or inspector (including other county departments); U.S. Environmental Protection Agency; State Water Resources Control Board; Los Angeles County Flood Control District; and Los Angeles Regional Water Quality Control Board

Section 2 Property Description

1.1 Existing Conditions

The project site is located south of the Foothill Freeway (Interstate 210) and it is bounded by Business Center Drive to the north, Highland Avenue to the east, The Los Angeles County Metropolitan Transportation Authority (Metro) to the south, and a single-family residential neighborhood to the west. Appendix 1 illustrates the project vicinity and provides an aerial perspective of the project site and immediate surroundings.

1.2 Proposed Conditions

The proposed multifamily residential project, "*The Residences at Duarte Station*" entails the demolition of existing parking lots, industrial structures, and buildings. The project totals 7.75 acres and includes numerous multi-family residential units.

1.2 Feasibility of Infiltration

According to the information taken from the Geotechnical Investigation by LGC Geotechnical, Inc., dated December 20, 2018, subsurface materials shall have a design infiltration rate equal to or greater than 0.3 inchers per hour. The infiltration tests performed meet the minimum requirements of the County of Los Angeles testing guidelines therefore, the subsurface soils in the area is suitable for stormwater infiltration.

Stormwater infiltration has been determined to be feasible for the project site. Stormwater infiltration practices operate by capturing and temporarily storing stormwater, before allowing it to infiltrate into the underlying soil. A perforated Corrugated Metal Pipe (CMP) will be installed on the northwesterly and northeasterly corner of the site in order to store the stormwater mitigation volume captured within the project site for infiltration into the underlying soils. The stormwater will be collected throughout the site by a proposed private storm drainage system. For each subarea, the stormwater quality design flow (SWQDF) is diverted to an Aqua-Swirl Hydrodynamic Separator unit that will be used for pretreatment prior to infiltration.

Section 2 Hydrologic Setting

2.1 Watershed (Receiving Water)

The proposed project is located within the 834 square mile Los Angeles River Watershed. The receiving waters directly affected by the proposed development include Duarte Channel, Buena Vista Channel, Sawpit Creek, Rio Hondo Channel, Los Angeles River Reach 2, Los Angeles River Reach 1, Los Angeles River Estuary (Queensway Bay), and San Pedro Bay. The Final 2014/2016 California Integrated Report (Clean Water Act Section 303(d) list/305(b) Report) has the downstream receiving waters impaired for:

Sawpit Creek
303d/TMDLs
- Bis(2ethylhexyl)phthalate (DEHP)
- Indicator Bacteria

Peck Road Park Lake
303d/TMDLs
- Chlordane (tissue)
- DDT (tissue)
- Odor
- Organic Enrichment/Low Dissolved Oxygen
- Trash

Rio Honda Reach 3
303d/TMDL
- Indicator Bacteria
- Iron
- Oxygen, Dissolved

Rio Honda Reach 2
303d/TMDL
- Ammonia
- Coliform Bacteria
- Cyanide

Rio Honda Reach 1
303d/TMDL
- Copper
- Indicator Bacteria
- Lead
- pH (nitrogen)
- Toxicity
- Trash
- Zinc

Los Angeles River Reach 2
303d/TMDL

- Ammonia
- Copper
- Indicator Bacteria
- Lead
- Nutrients (algae)
- Oil
- Trash

Los Angeles River Reach 1
303d/TMDL

- Ammonia
- Cadmium
- Copper, Dissolved
- Cyanide
- Indicator Bacteria
- Lead
- Nutrients (algae)
- pH (nitrogen)
- Trash
- Zinc, Dissolved

Los Angeles River Estuary
303d/TMDL

- Chlordane
- DDT/Sediment
- PCBs (sediment from pesticides)
- Toxicity
- Trash

San Pedro Bay/Offshore Zone
303d/TMDL

- Chlordane
- PCBs
- Total DDT
- Toxicity

2.2 Existing Drainage Patterns

The site topography has an elevation change of approximately 15 feet from the northeast corner to the southwest corner. The runoff from the entire project site sheet generally flows south into the site and then travels westerly into one of the existing four inlets located near the southerly property line. The sheet flow will then travel along the storm drain line that is maintained by the Los Angeles County Flood Control District (LAFCD), that ultimately discharges into the Los Angeles River Watershed.

2.3 Proposed Drainage Patterns

On-site stormwater will be collected by a new private drainage system in order to accommodate the proposed residential buildings and improvements. The proposed site will be divided into two subareas A and B therefore, the on-site runoff for each area will be collected by the CMP infiltration tanks (Area A: 8' x 243'; Area B: 8' x 193'). If overflow occurs it will be diverted through the proposed 18-inch storm drain pipe towards the south side of the site where it will connect to the existing 30-42 inch storm drain County line.

Below is a table that summarizes the results of the calculations for sizing of the proposed infiltration tanks.

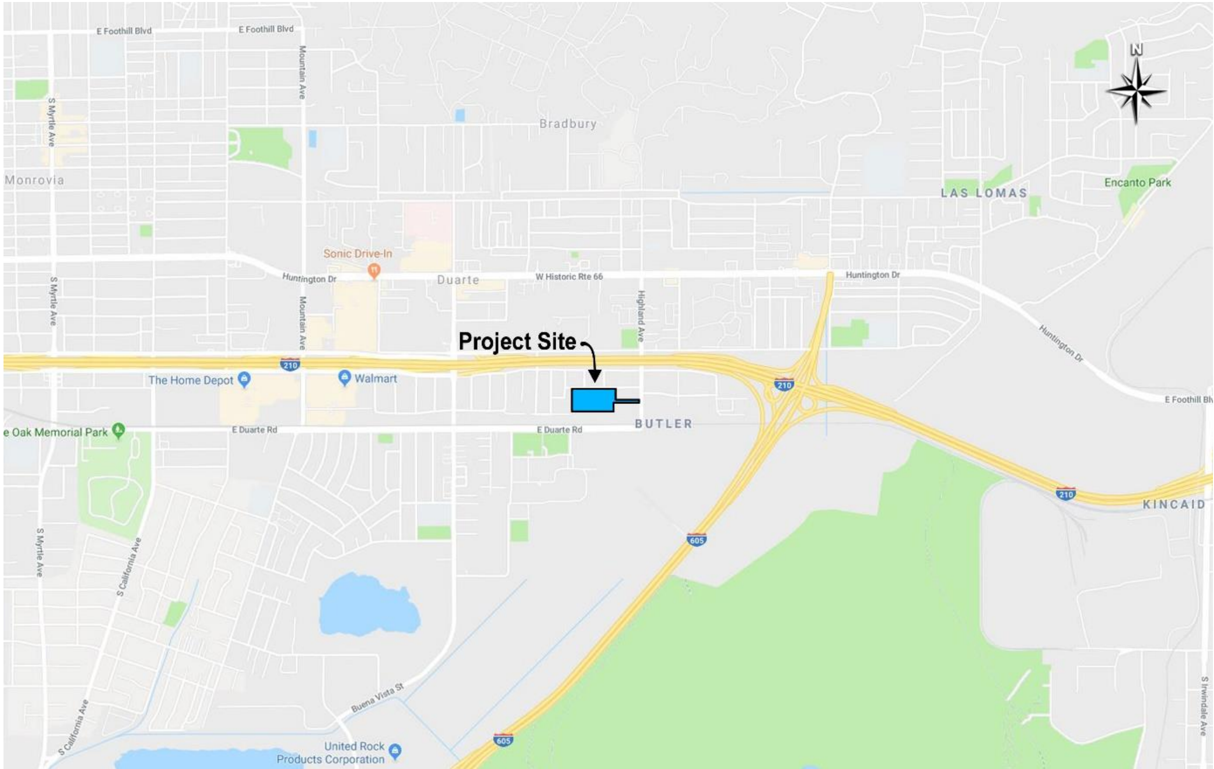
CMP Infiltration Tank Calculation Summary

Subarea	Design Volume	Tributary Area	CMP Dimensions (Trench Footprint)	Tank & Trench Volume	Drawdown Time
A	15,844	4.31	8 ft dia. x 243 lf	17,406	88.09
B	12,831	3.44	8 ft dia. x 193 lf	13,858	65.95

APPENDIX

Appendix 1

Area and Vicinity Map


Project Site
N
Bradbury
Monrovia
LAS LOMAS
Encanto Park
Sonic Drive-In
Duarte
Huntington Dr
W Historic Rte 66
The Home Depot
Walmart
BUTLER
E Duarte Rd
e Oak Memorial Park
KINCAID
Buena Vista St
United Rock Products Corporation


N
Evergreen Street
Fairdale Avenue
Glenford Avenue
Denning Avenue
Business Center Drive
Project Site
Highland Avenue
E. Duarte Road

Appendix 2

Preliminary LID Exhibit

LEGEND

SUBAREA BOUNDARY

X
X.XX AC
SUBAREA LABEL

SITE INFORMATION

AREA A

TOTAL AREA: 188,986 S.F. (4.34 ACRES)
MITIGATION VOLUME: 15,844 C.F. (118,521 GAL.)

AREA B

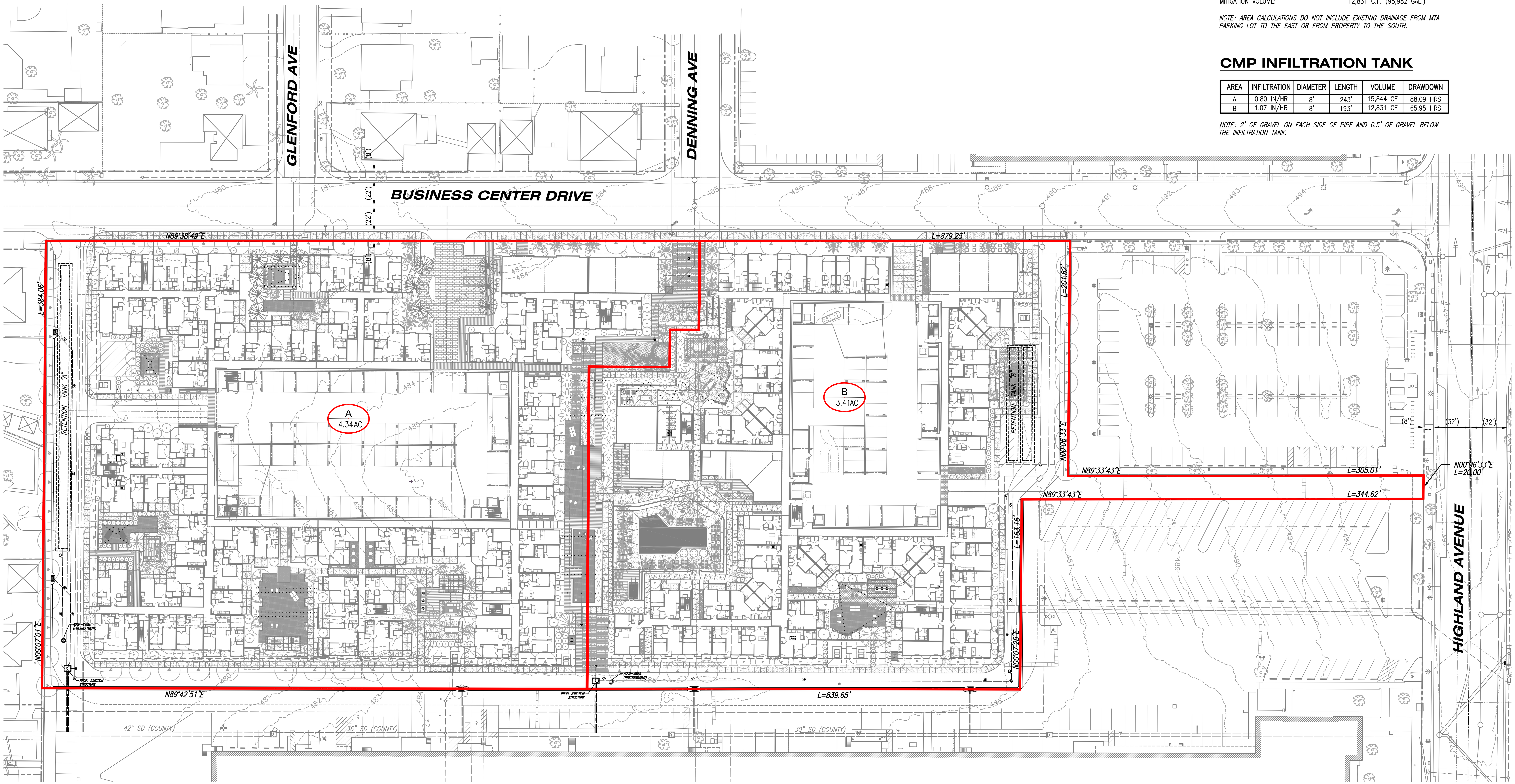
TOTAL AREA: 148,729 S.F. (3.41 ACRES)
MITIGATION VOLUME: 12,831 C.F. (95,982 GAL.)

NOTE: AREA CALCULATIONS DO NOT INCLUDE EXISTING DRAINAGE FROM MTA PARKING LOT TO THE EAST OR FROM PROPERTY TO THE SOUTH.

CMP INFILTRATION TANK

AREA	INFILTRATION	DIAMETER	LENGTH	VOLUME	DRAWDOWN
A	0.80 IN/HR	8'	243'	15,844 CF	88.09 HRS
B	1.07 IN/HR	8'	193'	12,831 CF	65.95 HRS

NOTE: 2' OF GRAVEL ON EACH SIDE OF PIPE AND 0.5' OF GRAVEL BELOW THE INFILTRATION TANK.



0 20 40 80 120

SCALE: 1"=40' DATE: 04/24/19

THE RESIDENCES AT DUARTE STATION

PRELIMINARY LID/STORM DRAIN PLAN

MW INVESTMENT GROUP, LLC

DUARTE, CALIFORNIA

KHR ASSOCIATES
CONSULTING ENGINEERS/SURVEYORS/PLANNERS
20411 SW Birch Street - Suite 310
Newport Beach, California 92660
Tel (949) 756-6440
Fax (949) 756-6444

Appendix 3

Peak Flow Hydraulic Analysis

85th Percentile Storm

Peak Flow Hydrologic Analysis

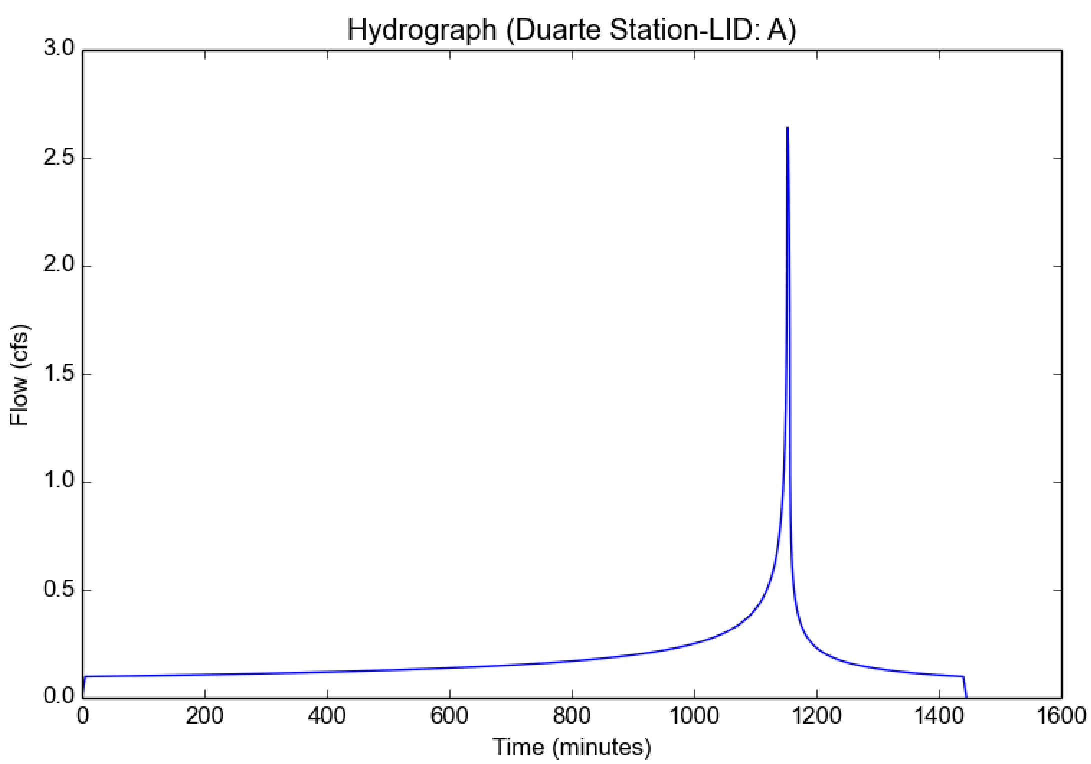
File location: R:/MW Investments/Highland-Duarte/Documents/LID/Calcs/19.03.29/MJWD-LID - A.pdf
Version: HydroCalc 0.3.1-beta

Input Parameters

Project Name	The Residences at Duarte Station
Subarea ID	A (LID)
Area (ac)	4.34
Flow Path Length (ft)	50.0
Flow Path Slope (vft/hft)	0.02
85th Percentile Rainfall Depth (in)	1.3
Percent Impervious	0.85
Soil Type	7
Design Storm Frequency	85th percentile storm
Fire Factor	0
LID	True

Output Results

Modeled (85th percentile storm) Rainfall Depth (in)	1.3
Peak Intensity (in/hr)	0.7756
Undeveloped Runoff Coefficient (Cu)	0.1282
Developed Runoff Coefficient (Cd)	0.7842
Time of Concentration (min)	5.0
Clear Peak Flow Rate (cfs)	2.6399
Burned Peak Flow Rate (cfs)	2.6399
24-Hr Clear Runoff Volume (ac-ft)	0.3637
24-Hr Clear Runoff Volume (cu-ft)	15844.222



Peak Flow Hydrologic Analysis

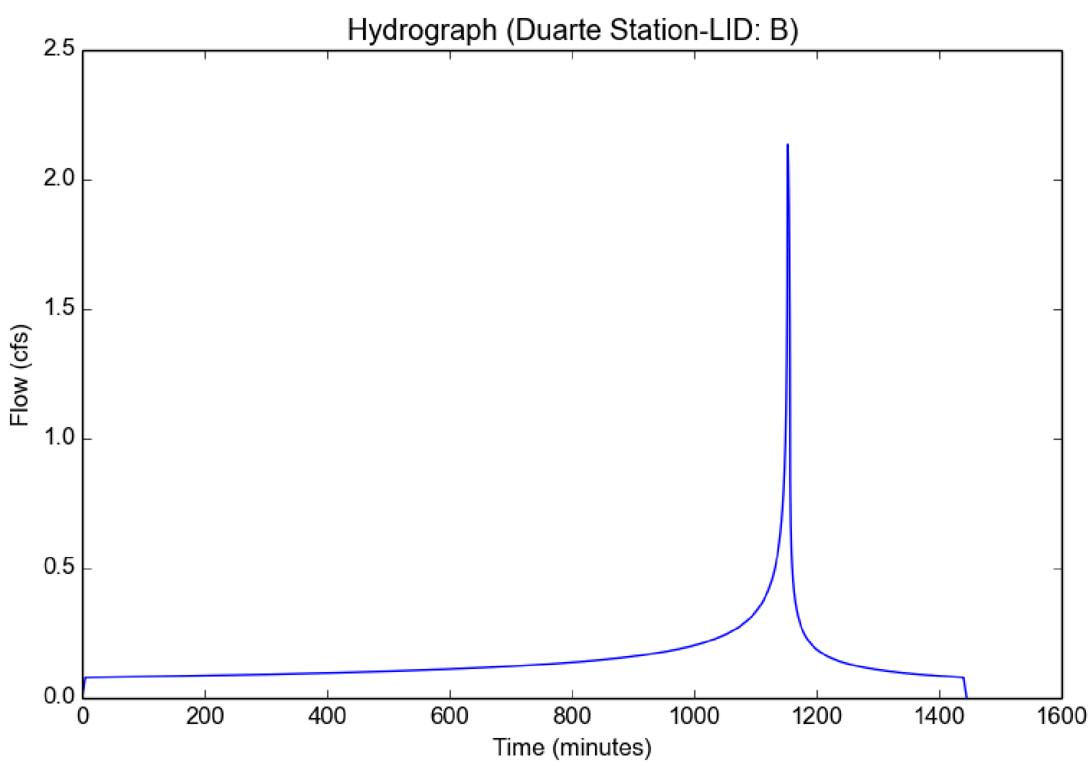
File location: R:/MW Investments/Highland-Duarte/Documents/LID/Calcs/19.03.29/MJWD-LID - B.pdf
Version: HydroCalc 0.3.1-beta

Input Parameters

Project Name	The Residences at Duarte Station
Subarea ID	B (LID)
Area (ac)	3.41
Flow Path Length (ft)	50.0
Flow Path Slope (vft/hft)	0.02
85th Percentile Rainfall Depth (in)	1.3
Percent Impervious	0.88
Soil Type	7
Design Storm Frequency	85th percentile storm
Fire Factor	0
LID	True

Output Results

Modeled (85th percentile storm) Rainfall Depth (in)	1.3
Peak Intensity (in/hr)	0.7756
Undeveloped Runoff Coefficient (Cu)	0.1282
Developed Runoff Coefficient (Cd)	0.8074
Time of Concentration (min)	5.0
Clear Peak Flow Rate (cfs)	2.1354
Burned Peak Flow Rate (cfs)	2.1354
24-Hr Clear Runoff Volume (ac-ft)	0.2946
24-Hr Clear Runoff Volume (cu-ft)	12831.8101



Appendix 4

CMP Infiltration Calculations

LID CALCULATIONS CMP INFILTRATION:

Area A

$K_{sat,measured}$:	1.20	in/hr
CMP Diameter:	8.00	feet
CMP_{Length}:	243	linear feet
G_{depth} (Porous Stone):	8.50	feet
G_{width} (Porous Stone):	12.00	feet
G_{length} (Porous Stone):	247	feet
T (Max. Drawdown Time):	96	hr
V_{design} (CF) :	From HydroCalc	
V_{design} (CF) :	15,844	C.F.
Factor of Safety(FS):	1.5	

Determine $K_{sat,design}$

$K_{sat,design} = K_{sat,measured} / FS$

$K_{sat,design} =$ 0.80 in/hr

Determine A_{min}

$A_{min} = (V_{design} \times 12 \text{ in/ft}) \div (T \times K_{sat,design})$

$A_{min} =$ 2,476 S.F.

Determine V_{CMP}

$V_{CMP} = (\pi r2) x CMP_{Length}$

$V_{CMP} =$ 12,215 C.F.

Determine V_{Stone}

$V_{stone} = ((G_{depth} \times G_{width} \times G_{length}) - V_{CMP}) \times 0.40$

$V_{stone} =$ 5,192 C.F.

Determine V_{Actual}

$V_{actual} = V_{CMP} + V_{stone}$

$V_{actual} =$ 17,406 C.F.

$V_{actuals} > = V_{design}$ TRUE

Determine A_{actual}

$A_{actual} = G_{width} \times G_{length}$

$A_{actual} =$ 2,964 S.F.

Determine T_{actual}

$T_{actual} = (V_{actual} \times 12 \text{ in/ft}) \div (A_{actual} \times K_{sat,design})$

$T_{actual} =$ 88.09 hr

$T_{actuals} < = T_{max}$ TRUE

LID CALCULATIONS CMP INFILTRATION:

Area B

$K_{sat,measured}$:	1.60	in/hr
CMP Diameter:	8.00	feet
CMP_{Length}:	193	linear feet
G_{depth} (Porous Stone):	8.50	feet
G_{width} (Porous Stone):	12.00	feet
G_{length} (Porous Stone):	197	feet
T (Max. Drawdown Time):	96	hr
V_{design} (CF) :	From HydroCalc	
V_{design} (CF) :	12,831	C.F.
Factor of Safety(FS):	1.5	

Determine $K_{sat,design}$

$K_{sat,design} = K_{sat,measured} / FS$

$K_{sat,design} =$ 1.07 in/hr

Determine A_{min}

$A_{min} = (V_{design} \times 12 \text{ in/ft}) \div (T \times K_{sat,design})$

$A_{min} =$ 1,504 S.F.

Determine V_{CMP}

$V_{CMP} = (\pi r2) x CMP_{Length}$

$V_{CMP} =$ 9,701 C.F.

Determine V_{Stone}

$V_{stone} = ((G_{depth} \times G_{width} \times G_{length}) - V_{CMP}) \times 0.40$

$V_{stone} =$ 4,157 C.F.

Determine V_{Actual}

$V_{actual} = V_{CMP} + V_{stone}$

$V_{actual} =$ 13,858 C.F.

$V_{actuals} >= V_{design}$ TRUE

Determine A_{actual}

$A_{actual} = G_{width} \times G_{length}$

$A_{actual} =$ 2,364 S.F.

Determine T_{actual}

$T_{actual} = (V_{actual} \times 12 \text{ in/ft}) \div (A_{actual} \times K_{sat,design})$

$T_{actual} =$ 65.95 hr

$T_{actuals} <= T_{max}$ TRUE

APPENDIX H2
Preliminary Geotechnical Subsurface Evaluation and Recommendations, The Residences at Duarte Station

LGC Geotechnical, Inc.

December 20, 2018

Project No. 18177-01

Mr. Matthew J. Waken
MW Investment Group, LLC
27702 Crown Valley Parkway, Suite D-4-197
Ladera Ranch, CA 92694

Subject: ***Preliminary Geotechnical Subsurface Evaluation and Recommendations, Proposed "Highland" Mixed-Use Development, Duarte, California***

In accordance with your request, LGC Geotechnical, Inc. (LGC Geotechnical) is providing a preliminary geotechnical report for the planned Highland Avenue mixed-use development located in the City of Duarte, California. This report presents the results of our limited subsurface explorations and geotechnical analysis and provides a summary of our conclusions and preliminary recommendations relative to the proposed site development.

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

Respectfully,

LGC Geotechnical, Inc.

REGISTERED PROFESSIONAL ENGINEER
RYAN L. DOUGLAS
No. 84840
CIVIL
STATE OF CALIFORNIA

Ryan Douglas, RCE C84840
Project Engineer

Dennis Boratynec, GE 2770
Vice President

REGISTERED PROFESSIONAL ENGINEER
DENNIS BORATYNEC
No. 2770
GEOTECHNICAL
STATE OF CALIFORNIA

RLD/DJB/CNJ/aca

Distribution: (4) Addressee (3 wet signed copies and 1 electronic version)
(1) KHR Associates (electronic version)
Attn: Mr. James Kawamura

TABLE OF CONTENTS

Section ***Page***

TABLE OF CONTENTS (Cont'd)

LIST OF TABLES, ILLUSTRATIONS, & APPENDICES

Tables

Figures

Appendices

1.0 INTRODUCTION

1.1 Purpose and Scope of Services

This preliminary geotechnical report is for the planned Highland Avenue mixed-use development located in the City of Duarte, California (see Site Location Map, Figure 1). The proposed "Highland" development, also known as the Duarte Station Apartments, will consist of two 5-story mixed-use residential/retail structures (referred to as Building A and Building B) and two respective 6.5-story parking structures. The purpose of our work was to evaluate site geotechnical conditions and to provide preliminary geotechnical recommendations with respect to the proposed development.

1.2 Project Description

Based on the provided information, the proposed development will consist of two separate mixed-use WRAP/parking structure buildings described as Building A and Building B. Building A (the western most building) is planned to consist of a 5-story mixed-use residential/retail structure, with 340 residential units, wrapping around a 6.5-story parking structure. Building B (the eastern most building) is planned to consist of a 5-story mixed-use residential/retail structure, with 265 residential units, wrapping around a 6.5-story parking structure. Each of the 6.5-story parking structures is planned to have half of a story/level constructed partially subterranean (estimated to be approximately 5 feet below existing grade). We anticipate finish grades will only vary slightly (± 2 feet) from current grade in the proposed residential/retail and parking structure areas. Two on-grade swimming pools/recreation areas are proposed in the courtyard areas, one near Building A and the other near Building B. One north to south central walkway corridor is planned between the two buildings. This walkway area is anticipated to include walkways, seating areas, amenities, landscaping, etc. The proposed development will also include residential amenities, retail space, driveways into the parking structures, etc. Presented in Table 1 is a summary of our estimated structural (dead plus live) loads for the proposed 5-story mixed-use residential/retail structures and the proposed 6.5-story parking structures. Please note that structural loads and a preliminary grading plan were not provided to us at the time of this report.

TABLE 1

Estimated Structural Loads

Planned Structure	**Column Loads (kips)**	**Wall Loads (kip/ft)**
5-Story Mixed-Use Structures	150	7
6.5-Story Parking Structures	750	25

The recommendations given in this report are based upon the proposed layout and estimated structural loading information above. We understand that the project plans are currently being developed at this time; LGC Geotechnical should be provided with updated project plans and the actual structural loads when they become available, in order to either confirm or modify the recommendations provided herein.

Geotechnical, Inc.

FIGURE 1
Site Location Map

PROJECT NAME	Highland - Duarte
PROJECT NO.	18177-01
ENG. / GEOL.	RLD
SCALE	Not to Scale
DATE	December 2018

1.3 Existing Conditions

The approximately 7.6-acre irregular shaped site is bound to the north by Business Center Drive, to the east by Highland Avenue and a Metro owned parking lot, to the south by existing an industrial building, and to the west by existing single-family residences. Existing improvements consist of a central industrial building with two parking lots, driveways, sidewalks, landscaping and other associated improvements.

The site has relatively minor relief with surface elevations ranging from approximately 477 to 493 feet above mean sea level (msl). Surface drainage generally flows from northeast to southwest.

1.4 Subsurface Exploration

A geotechnical field evaluation was performed by LGC Geotechnical in November of 2018. This program consisted of drilling and sampling ten small-diameter borings, two of which were for infiltration testing, and one Becker Hammer boring.

Borings HS-1 through HS-8 and I-1 through I-2 were drilled by 2R Drilling under subcontract to LGC Geotechnical, boring BD-1 was drilled by Great West Drilling under subcontract to LGC Geotechnical. The depth of the borings ranged from approximately 3 to 50 feet below existing grade. Auger refusal in all of the hollow-stem auger borings was encountered prior to the target depth of 50 feet. Borings I-1 and I-2 were used for shallow boring percolation testing. An LGC Geotechnical representative observed the drilling operations, logged the borings, and collected soil samples for laboratory testing. The borings were excavated using a CME-75 truck-mounted hollow-stem drill rig equipped with an 8-inch diameter hollow-stem auger. The Becker Hammer boring was excavated using an AP1000 Becker Hammer drill rig equipped with a 6.6-inch diameter double wall (open bottom) drive pipe in order to obtain samples between 25 and 50 feet below existing grade. Disturbed materials are vacuumed through the Becker Hammer drive pipe to the surface for sampling. Driven soil samples were collected by means of the Standard Penetration Test (SPT) and Modified California Drive (MCD) sampler generally obtained at 2.5 to 5-foot vertical increments in both the hollow-stem auger borings and Becker Hammer boring. The MCD is a split-barrel sampler with a tapered cutting tip and lined with a series of 1-inch-tall brass rings. The SPT sampler (1.4-inch ID) and MCD sampler (2.4-inch ID, 3.0-inch OD) were 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, the borings were backfilled with the native soil cuttings, tamped and the surface was replaced with asphalt cold-patch, where necessary. Some settlement of the backfill soils/asphalt cold-patch may occur over time.

Infiltration testing was performed within two of the borings (I-1 and I-2) to depths of 5 and 15 feet below existing grade. Infiltration testing was performed in general accordance with the County of Los Angeles testing guidelines. The locations were subsequently backfilled with native soils at the completion of testing.

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

1.5 Laboratory Testing

Representative driven and bulk samples were retained for laboratory testing during our field evaluation. Laboratory testing included in-situ unit weight and moisture content, fines content, consolidation, expansion index, direct shear, laboratory compaction and corrosion (sulfate, chloride, pH, and minimum resistivity).

The following is a summary of the laboratory test results.

- Dry density of the samples collected ranged from approximately 92 pounds per cubic foot (pcf) to 132 pcf, with an average of 117 pcf. Field moisture contents ranged from approximately 0 to 5 percent, with an average of approximately 2 percent.
- Five fines content tests indicated a fines content (percent passing No. 200 sieve) ranging from approximately 4 percent to 18 percent. Based on the Unified Soils Classification System (USCS), the three tested samples would be classified as "coarse-grained."
- One direct shear test was performed. The plot is provided in Appendix C.
- One consolidation test was performed. The sample was retrieved from a depth of approximately 20 feet below existing grade were inundated with water at a stress of 3 ksf. Percent of collapse was approximately 0.5 percent. The stress vs. deformation plot is provided in Appendix C.
- Two Expansion Index (EI) tests were performed. Results were EIs of 0 and 1, corresponding to "Very Low" expansion potential.
- Laboratory compaction (maximum dry density and optimum moisture content) test indicated a maximum dry density value of 134.0 pcf with optimum moisture content of 6.5 percent.
- Corrosion testing indicated soluble sulfate content less than 0.02 percent, chloride contents ranging from approximately of 31 to 61 parts per million (ppm), pH values of approximately 8.1, and minimum resistivity values of approximately 9,200 to 17,000 ohm-cm.

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

2.0 GEOTECHNICAL CONDITIONS

2.1 Regional Geology

The site is located within the northeastern boundary of the Peninsular Ranges Geomorphic Province. The San Gabriel Mountain Range rises north of the site and provides the sediment source for the alluvial fan deposits that underlie the area of the subject site. The site is located in the San Gabriel Valley, west of the San Gabriel River.

The subject area is geologically bounded at the northwest by the Raymond Fault and at the north by the Sierra Madre Fault Zone. The Raymond Fault is a left lateral fault that is generally assumed to be a part of the San Andreas Fault system; however, there is an indication of some reverse-slip motion along the fault. The Sierra Madre Fault Zone consists of reverse faults dipping to the north. The northeast trending Raymond Fault joins the east west trending Sierra Madre Fault Zone approximately two miles to the northwest of the site. The Sierra Madre is located less than a mile to the north of the site.

2.2 Generalized Subsurface Soils

Based on our subsurface evaluation (hollow-stem auger borings and Becker Hammer boring) and review of regional geologic maps and published geotechnical literature, the site contains up to approximately 5 feet of previously placed undocumented artificial fill over native young alluvial fan deposits. Older artificial fill soils encountered were primarily silty sands. Native alluvial fan deposits are primarily very dense sands and silty sands with gravel and cobble, including varying amounts of oversized materials, to a maximum explored depth of approximately 50 feet below exiting grade.

It should be noted that geotechnical explorations are only representative of the location where they are performed and varying subsurface conditions may exist outside of each 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 the boring logs provided in Appendix B.

2.3 Groundwater

Groundwater was not encountered in our borings to the maximum explored depth of approximately 50 feet below existing grade. Historic high groundwater is estimated to be about 150 feet or greater below existing grade (CDMG, 1998).

It should be noted that higher localized and seasonal perched groundwater conditions may accumulate below the surface, and should be expected throughout the design life of the proposed improvements. In general, groundwater conditions below any given site may vary over time depending on numerous factors including seasonal rainfall and local irrigation among others.

2.4 Field Infiltration Testing

Two shallow infiltration tests were performed in Borings I-1 and I-2 to approximate depths of 15 and 5 feet below existing grade, respectively. The approximate locations are shown on the Boring Location Map (Figure 2). The borings for the infiltration tests were excavated using a drill rig equipped with an 8-inch diameter hollow-stem auger. Estimation of infiltration rates was accomplished in general accordance with the guidelines set forth by the County of Los Angeles (2017). A 3-inch diameter perforated PVC pipe was placed in the borehole above a thin layer of gravel and the annulus was backfilled with gravel. The infiltration wells were pre-soaked 1 hour prior to testing. Initially the procedure for 30-minute reading intervals was followed for the borings (I-1 and I-2). During the 30-minute test, water was generally draining to the top of the gravel layer in less than 30 minutes; therefore, the procedure for the 10-minute reading interval test was subsequently followed. At the completion of infiltration testing, the pipe was removed and backfilled with cuttings and tamped. Some settlement of the backfill should be expected.

Based on the County of Los Angeles testing guidelines, the raw infiltration is calculated by dividing the volume of water discharged by the surface area of the test section (including sidewalls plus the bottom of the boring), in a given amount of time. The average of the stabilized infiltration rate over the last three consecutive readings is the measured infiltration rate. The measured infiltration rates are provided in Table 2 below. Please note that the values provided in Table 1 do not include reduction factors for the test procedure, site variability and long-term siltation plugging that are required for the design infiltration rate, refer to Table 8 in Section 4.10. Infiltration tests were performed using relatively clean water free of particulates, silt, etc. Refer to the infiltration test data provided in Appendix D.

TABLE 2

Summary of Field Infiltration Testing

Infiltration Test Location	Infiltration Test Depth (ft)	Measured Infiltration Rate* (inch/hr)
I-1	15	6.5
I-2	5	4.9

*Does Not Include Required Reduction Factors, refer to Table 8, Section 4.10.

2.5 Seismic Design Parameters

The site seismic characteristics were evaluated per the guidelines set forth in Chapter 16, Section 1613 of the 2016 CBC. Representative site coordinates of latitude 34.1337 degrees north and longitude -117.9692 degrees west were utilized in our analyses. The maximum considered earthquake (MCE) spectral response accelerations (S_{MS} and S_{M1}) and adjusted design spectral response acceleration parameters (S_{DS} and S_{D1}) for Site Class D are provided in Table 3 on the following page.

TABLE 3

Seismic Design Parameters

Selected Parameters from 2016 CBC, Section 1613 - Earthquake Loads	Seismic Design Values
Site Class per Chapter 20 of ASCE 7	D
Risk-Targeted Spectral Acceleration for Short Periods (S_S)*	2.257g
Risk-Targeted Spectral Accelerations for 1-Second Periods (S_1)*	0.914g
Site Coefficient F_a per Table 1613.3.3(1)	1.0
Site Coefficient F_v per Table 1613.3.3(2)	1.5
Site Modified Spectral Acceleration for Short Periods (S_{MS}) for Site Class D [Note: $S_{MS} = F_aS_S$]	2.257g
Site Modified Spectral Acceleration for 1-Second Periods (S_{M1}) for Site Class D [Note: $S_{M1} = F_vS_1$]	1.371g
Design Spectral Acceleration for Short Periods (S_{DS}) for Site Class D [Note: $S_{DS} = (^2/_3)S_{MS}$]	1.505g
Design Spectral Acceleration for 1-Second Periods (S_{D1}) for Site Class D [Note: $S_{D1} = (^2/_3)S_{M1}$]	0.914g
Mapped Risk Coefficient at 0.2 sec Spectral Response Period, C_{RS} (per ASCE 7)	0.976
Mapped Risk Coefficient at 1 sec Spectral Response Period, C_{R1} (per ASCE 7)	0.973

*From USGS, 2018

Section 1803.5.12 of the 2016 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 0.859g (USGS, 2018).

A deaggregation of the PGA based on a 2,475-year average return period indicates that an earthquake magnitude of 7.7 at a distance of approximately 7 km from the site would contribute the most to this ground motion (USGS, 2008).

2.6 Faulting

The subject site is not located within a State of California Earthquake Fault Zone (i.e., Alquist-Priolo Earthquake Fault Act Zone) and no active faults are known to cross the site (CGS, 2014). A fault is considered "active" if evidence of surface rupture in Holocene time (the last approximately 11,700 years) is present.

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 and 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. The nearby major active faults that could produce these secondary effects include the Raymond, Sierra Madre, Elsinore and San Andreas Faults, among others. A discussion of these secondary effects is provided in the following sections.

2.6.1 Liquefaction and Dynamic Settlement

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 loose, saturated, 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 (Bray & Sancio, 2006). Effects of liquefaction on level ground include settlement, sand boils, and bearing capacity failures below structures. Dynamic settlement of dry sands can occur as the sand particles tend to settle and densify as a result of a seismic event.

The site is not located within a State of California Seismic Hazard Zone for liquefaction potential (CGS, 1999). Site soils are not generally susceptible to liquefaction due to a lack of groundwater in the upper 50 feet and generally very dense sands and gravels. The potential for liquefaction and seismic settlement is considered low.

2.6.2 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 very low potential for liquefaction, the potential for lateral spreading is considered low.

2.7 Expansion Potential

Based on the results of previous laboratory testing by others and our recent laboratory testing, site soils have a “Very Low” expansion potential. Final expansion potential of site soils should be determined at the completion of grading. Results of expansion testing at finish grades will be utilized to confirm final foundation design.

2.8 ***Static Settlement and Hydro-Consolidation***

After the completion of grading, the subsurface conditions will consist of engineered fill placed over alluvial fan deposits. The underlying alluvial fan deposits were generally found to be very dense sands and gravelly sands. Static settlements will be induced by subjecting the finish grades to building loads. Static settlement should occur relatively quickly. Anticipated static settlements for the proposed 6.5-story parking structures and 5-story mixed use buildings are presented in Section 4.2, "Allowable Bearing Pressures and Passive Resistance".

In addition to static settlement, our recent exploratory drilling and laboratory testing indicate the presence of potentially collapsible soils. The collapse potential (or hydro-collapse) of the tested sample was found to be approximately 0.5 percent which is considered to be slightly susceptible to hydro-collapse.

3.0 CONCLUSIONS

Based on the results of our subsurface evaluation and understanding of the proposed development, it is our opinion that the proposed development is feasible from a geotechnical standpoint. A summary of our conclusions are as follows:

- Based on our subsurface evaluation (hollow-stem auger borings and Becker Hammer boring) the site is estimated to contain up to approximately 5 feet of previously placed undocumented artificial fill over native alluvial soils. Older artificial fill soils encountered were primarily silty sands. Native alluvial fan deposits are primarily very dense sands and silty sands with gravel and cobble, including varying amounts of oversized materials, to a maximum explored depth of approximately 50 feet below exiting grade.
- The near-surface soils are generally loose, dry and slightly collapsible and are not suitable for the planned improvements in their present condition (refer to Section 4.1); removal and recompaction will be required.
- Groundwater was not encountered during our recent subsurface evaluation to the maximum explored depth of approximately 50 feet below existing ground surface. Historic high groundwater for the site is about 150 feet or greater below existing ground surface (CDMG, 1998).
- The site is not located within a State of California Seismic Hazard Zone for liquefaction potential (CGS, 1999). The potential for liquefaction and seismic settlement is considered low due to the lack of a groundwater table in the upper 50 feet and generally very dense sandy and gravelly soils.
- The proposed development will likely be subjected to strong seismic ground shaking during its design life. The site is not located within a State of California Earthquake Fault Zone (i.e., Alquist-Priolo Earthquake Fault Act Zone) and no active faults are known to cross the site (CGS, 2014).
- Due to the proximity of the proposed improvements to the property line in portions of the site, temporary shoring or "A-B-C" slot cuts may be required to achieve required earthwork removal and recompaction.
- Provided our earthwork removal and recompction recommendations are implemented (refer to Section 4.1), the proposed 6.5-story parking structures and 5-story mixed use buildings may be supported on shallow foundation systems. Preliminary long-term static settlement estimates based on the estimated building loads are presented in Section 4.2.
- Based on the results of preliminary laboratory testing, site soils are anticipated to have "Very Low" expansion potential. Final design expansion potential must be determined at the completion of grading.
- Based on the corrosion test results, soils are not considered corrosive per the Caltrans criteria (Caltrans, 2015).
- Excavations into the existing site soils should be feasible with heavy construction equipment in good working order. We anticipate that the sands with silt, gravel and cobble generated from the excavations will be generally suitable for re-use as compacted fill, provided they are relatively free of rocks larger than 8 inches in dimension, construction debris, and significant organic material. Oversized materials should be anticipated.

4.0 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 the owner with sufficient information to develop the site in general accordance with the 2016 California Building Code (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 structures 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, soil settlement, groundwater seepage, etc. It should be understood, however, that our recommendations are intended to maintain the structural integrity of the proposed development and structures given the site geotechnical conditions, but 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 will generally consist of demolition of existing improvements, required removal and recompaction, subgrade preparation, precise grading and construction of the proposed new improvements including the parking and residential structures, site amenities, courtyards, subsurface utilities, driveways, etc.

We recommend that earthwork onsite be performed in accordance with the following recommendations, future geotechnical reports, the 2016 CBC/City of Duarte requirements and the General Earthwork and Grading Specifications 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 based upon future evaluation and our review of updated project plans and/or the field conditions exposed during site grading/construction.

4.1.1 Clearing and Grubbing

Prior to earthwork of areas to receive structural fill, engineered structures or improvements, the areas should be cleared of existing vegetation, building structures, pavement, utilities, surface obstructions, existing debris and potentially compressible or otherwise unsuitable material. Vegetation and debris should be removed and properly disposed of off-site. Holes resulting from the removal of buried obstructions, which extend below proposed removal bottoms, should be replaced with properly compacted fill material. Any abandoned sewer, storm drain or utility lines should be completely removed and replaced with properly placed compacted fill. Deeper demolition may be required in order to remove existing foundations or utilities. We recommend the trenches associated with demolition which extend below the remedial grading depth be backfilled and properly compacted prior to the demolition contractor leaving the site.

If cesspools or septic systems are encountered during earthwork, they should be removed in their entirety. The resulting excavation should be backfilled with properly compacted fill soils. As an alternative, cesspools can be backfilled with lean sand-cement slurry. At the conclusion of the clearing operations, a representative of LGC Geotechnical should observe and accept the site prior to further earthwork.

4.1.2 Removal and Recompaction Depths and Limits

In order to provide a relatively uniform bearing condition for the planned building structures, parking structures and improvements, we recommend the site soils be removed and recompacted. Existing onsite artificial fill shall be fully removed to suitable, competent native materials prior to placement of fill to design grades. Subsurface site soils should be removed and recompacted according to the criteria outlined below. Updated recommendations may be required based on additional field evaluations, changes to building layouts and actual structural loads.

6.5-Story Parking Structures: We recommend that soils within the proposed parking structure footprint areas be removed and recompacted to a minimum depth of 7 feet below existing grade or 3 feet beneath the base of the parking structure foundations, whichever is deeper. Localized deeper removal and recompaction may be required.

5-Story Mixed-Use Buildings: We recommend that soils within the proposed mixed-use residential/retail building footprint areas be removed and recompacted to a minimum depth of 5 feet below existing grade or 3 feet beneath the base of foundations, whichever is deeper. Localized deeper removal and recompaction may be required.

The base of removal bottoms should extend laterally a minimum distance equal to the depth of removal and recompaction below finish grade. Specifically, soils located within a 1:1 (horizontal to vertical) projection of the bottom of footings must be engineered compacted fill or competent natural ground. Building lines may be defined as the perimeter of the building proper, plus attached or adjacent foundation supported features, including canopies, elevators, walls, etc.

For minor site structures, such as free-standing, minor retaining walls, etc., removal and recompaction should extend at least 3 feet beneath existing grade or 2 feet beneath the base of foundations, whichever is deeper. In general, the envelope for removal and recompaction should extend laterally a minimum distance of 3 feet beyond the edges of the proposed improvements mentioned above.

Within non-structural areas (i.e., areas designed to receive concrete/asphalt paving, pavers, flatwork, etc.), the soils should be removed and replaced as properly compacted fill to a minimum depth of 2 feet below existing grade or 1-foot below the proposed finished subgrade, whichever is deeper. In general, the envelope for removal and recompaction should extend laterally a minimum distance of 2 feet beyond the edges of the proposed improvements mentioned above.

Local conditions may be encountered which could require additional removal and recompaction beyond the above-noted minimum to obtain an acceptable subgrade. The actual depths and lateral extents of removal and recompaction should be determined by the geotechnical consultant based on the subsurface conditions encountered during grading. Removal and recompaction areas and areas should be accurately staked in the field by the Project Surveyor.

4.1.3 Excavations

Excavations up to approximately 10 feet are anticipated for the recommended earthwork removal and recompaction. 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 investigation, the majority of site soils are anticipated to be OSHA Type "C" soils (refer to the attached boring logs). Sandy soils are present and should be considered susceptible to caving. Raveling of the sandy soils should be anticipated for temporary slopes. Flatter slope inclinations should be considered if raveling cannot be tolerated. The exposed slope surface may be kept surficially moist (but not saturated) during construction to reduce (not eliminate) potential sloughing. 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. 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.

The potential for impacting the existing improvements may be reduced by the installation of temporary shoring or performing narrow "A-B-C" slot cuts while performing earthwork removal and recompaction for the proposed perimeter parking structure and mixed-use buildings. Slot cuts should be backfilled immediately with properly compacted fill to finish grade prior to excavation of the adjacent two slots. Please note sands susceptible to caving are present at the site. Recommendations for slot cuts should be evaluated based on the proposed grading plan and during grading. Protection of the existing offsite improvements during grading is the responsibility of the contractor.

Surcharge loads (vehicular traffic, soil stockpiles, construction equipment, etc.) should be set back from the perimeter of excavations a minimum distance equivalent to a 1:1 projection from the bottom of the excavation or 5 feet, whichever is greater, unless the cut is properly shored and designed for the applicable surcharge load. 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. Temporary shoring parameters are provided in Section 4.6.

4.1.4 Removal Bottoms and Subgrade Preparation

In general, removal bottoms and areas to receive compacted fill should be scarified to a minimum depth of 6 inches, brought to a near-optimum moisture condition (generally within optimum and 2 percent above optimum moisture content), 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. Soil subgrade for planned footings and improvements (e.g., slabs, etc.) should be firm and competent.

4.1.5 Material for Fill

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

From a geotechnical viewpoint, any required import soils for general fill (i.e., non-retaining wall backfill) should consist of soils of granular soils of "Very Low" expansion potential (expansion index 20 or less based on American Society for Testing and Materials [ASTM] D 4829), and free of organic materials, construction debris and any material greater than 3 inches in maximum dimension. Import for any required retaining wall backfill should meet the criteria outlined in the following paragraph. Source samples should be provided to the geotechnical consultant for laboratory testing a minimum of four working days prior to any planned importation.

Retaining wall backfill should consist of onsite clean granular (sandy) soils with a maximum of 35 percent fines (passing the No. 200 sieve) per American Society for Testing and Materials (ASTM) D1140 (or ASTM D6913/ASTM D422) and a "Very Low" expansion potential (EI of 20 or less per ASTM D4829). Soils should also be screened of organic materials, construction debris, and any material greater than 3 inches in maximum dimension. The majority of the onsite soils should be suitable for retaining wall backfill once screened of material greater than 3 inches in maximum dimension. Therefore, select grading, screening and stockpiling of onsite soils meeting the criteria outlined above will be required by the contractor for obtaining suitable retaining wall backfill soil. These preliminary findings should be confirmed during grading.

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 ("Green Book") for untreated base materials (except processed miscellaneous base), Caltrans Class 2 aggregate base or the City of Duarte requirements.

The placement of demolition materials in compacted fill is acceptable from a geotechnical viewpoint provided the demolition material is broken up into pieces not larger than typically used for aggregate base (approximately 1-inch in maximum dimension) and well blended into fill soils with essentially no resulting voids. Demolition material placed in fills must be free of construction debris (wood, brick, etc.) and reinforcing steel. If asphalt concrete fragments will be incorporated into the demolition materials, approval from an environmental viewpoint may be required and is not the purview of the geotechnical consultant. From our previous experience, we recommend that asphalt concrete fragments be limited to fill areas within planned parking structure footprint (i.e., not within building pad areas).

4.1.6 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 (adding water) of site soils will be required in order to achieve adequate compaction prior to reusing the materials in compacted fills. In general, the site soils will require additional moisture in order to achieve the required compaction.

The optimum lift thickness to produce a uniformly compacted fill will depend on the type and size of compaction equipment used. In general, fill should be placed in uniform lifts not exceeding 8 inches in compacted 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 the local grading ordinances 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 a minimum of 95 percent relative compaction at or slightly above optimum moisture content per ASTM D1557. Subgrade below aggregate base should be compacted to a minimum of 90 percent relative compaction per ASTM D1557 at near-optimum moisture content (generally within optimum and 2 percent above optimum moisture content).

If gap-graded ¾-inch rock is used for backfill (around storm drain storage chambers, retaining wall backfill, etc.) it will require compaction. Rock shall be placed in thin lifts (typically not exceeding 6 inches) and mechanically compacted with observation by geotechnical consultant. Backfill rock shall meet the requirements of ASTM D2321. Gap-graded rock is required to be wrapped in filter fabric to prevent the migration of fines into the rock backfill.

4.1.7 Trench and Retaining Wall Backfill and Compaction

The onsite soils may generally be suitable as trench backfill, provided the soils are screened of rocks, construction debris, other material greater than 6 inches in diameter and significant 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 Caltrans Test Method [CTM] 217) may be used to bed and shade the pipes within the bedding zone. Based on our field evaluation, onsite soils may not meet this sand equivalent requirement. 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 lifts (as outlined above in section "Material for Fill") by mechanical means to at least 90 percent relative compaction (per ASTM D1557).

Utility trenches running parallel to footings should not be excavated within a 1:1 (horizontal to vertical) downward projection from adjacent footings ("footing influence zone") to avoid potential undermining. Depending on the utility line and structural loading of the footing, utility trenches running perpendicular to footings may require special provisions such as sand-cement slurry backfill of the utility trench in this zone or flexible sleeves through the footings. These conditions should be evaluated on a case-by-case basis.

Retaining wall backfill should consist of sandy soils as defined in the above section "Material for Fill." The limits of select sandy backfill should extend at minimum ½ the height of the retaining wall or the width of the heel (if applicable), whichever is greater (see Figure 3). Retaining wall backfill soils should be compacted in relatively uniform thin lifts to at least 90 percent relative compaction (per ASTM D1557). Jetting or flooding of retaining wall backfill materials should not be permitted.

In backfill areas where mechanical compaction of soil backfill is impractical due to space constraints, typically sand-cement slurry may be substituted for compacted backfill. The slurry should contain about one sack of cement per cubic yard. When set, such a mix typically has the consistency of compacted soil. Sand cement slurry placed near the surface within landscape areas should be evaluated for potential impacts on planned improvements.

A representative from LGC Geotechnical should observe, probe, and test backfill to verify compliance with the project recommendations.

4.1.8 Shrinkage and Subsidence

Allowance in the earthwork volumes budget should be made for an estimated 0 to 10 percent reduction in volume of the upper approximate 5 feet of site soils. It should be stressed that these values are only estimates and that an actual shrinkage factor would be extremely difficult to predetermine. Subsidence due to earthwork equipment is expected to be on the order of 0.1 feet. These values are estimates only and exclude losses due to removal of vegetation or debris. The effective shrinkage of onsite soils will depend primarily on the type of compaction equipment, method of compaction used onsite by the contractor and accuracy of the topographic survey.

4.2 Allowable Bearing Pressures and Passive Resistance

Provided our earthwork removal and recompaction recommendations are implemented, the proposed 6.5-story parking structures and 5-story mixed-use buildings may be supported on shallow foundation systems. The following minimum footing widths and embedments are recommended for the corresponding allowable bearing pressures for both continuous wall and column spread footings.

TABLE 4

Allowable Soil Bearing Pressures

Allowable Static Bearing Pressure (psf)	Minimum Footing Width (feet)	Minimum Footing Embedment* (feet)
5,000	6	3
4,000	4	2
3,000	2	2
2,500	1.5	1.5

* Refers to minimum depth measured below lowest adjacent grade.

These net bearing pressures (exclusive of the weight of the footings) are for dead plus live loads and may be increased one-third for short-term, transient, wind and seismic loading. The allowable bearing pressures are applicable for level (ground slope equal to or flatter than 5H:1V) conditions only. The maximum edge pressures induced by eccentric loading or overturning moments should not be allowed to exceed these recommended values. For any bearing pressures, less than 2,500 psf, a minimum footing width of 18 inches and depth of 18 inches below lowest adjacent grade should be used.

Soil settlement is a function of footing dimensions and applied soil bearing pressure. In utilizing the above-mentioned allowable bearing capacity and recommended earthwork removals, foundation settlement due to structural loads for the 6.5-story parking structures and 5-story mixed-use residential buildings are on the order of 1-inch. Foundation settlement due to structural loads for the 5-story apartment buildings is on the order of ½-inch. Differential settlement should be anticipated between nearby columns or walls where a large differential loading condition exists and may be taken as half of the static settlement over a horizontal distance of 40 feet (i.e., ½-inch over a horizontal span of 40 feet). Static settlement is anticipated to occur relatively quickly after construction. Final settlement estimates should be evaluated by LGC Geotechnical when actual building loads and foundation plans are made available.

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. An allowable passive lateral earth pressure of 280 pcf to a maximum of 2,800 pcf may be used for lateral resistance for properly compacted fill and suitable dense native soils. This allowable passive pressure may be increased to 380 pcf to a maximum of

3,800 pcf for short-duration seismic loading. This passive pressure is 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 pressure includes a static and seismic factor of safety of 1.5 and 1.1, respectively.

4.3 ***Building Slabs***

Concrete building slabs should be supported on compacted and moisture-conditioned site sandy soils with Very Low expansion potential (EI of 20 or less per ASTM D4829) as outlined in the "Site Earthwork" section of this report. The foundation designer may use a modulus of vertical subgrade reaction (k) of 200 pounds per cubic inch (pounds per square inch per inch of deflection). Structural design of the slabs (thickness, reinforcement, etc.) should be performed by the structural engineer.

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 foundation. The foundation engineer/architect 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.

4.4 ***Lateral Earth Pressures for Retaining Wall Design***

The following may be used for design of site retaining walls. A portion of the proposed parking structure will be partially subterranean (estimated at approximately 5 feet below existing grade). Retaining walls are anticipated within portions of the proposed parking structure. Lateral earth pressures are provided as equivalent fluid unit weights, in pound per square foot (psf) per foot of depth or pcf. A soil unit weight of 125 pcf may be assumed for calculating the actual weight of soil over the wall footing.

The following lateral earth pressures are presented on Table 5 for approved select granular soils with a maximum of 35 percent fines (passing the No. 200 sieve per ASTM D-421/422), a "Very Low" expansion potential (EI of 20 or less per ASTM D4829) and maximum material size of 3 inches in maximum dimension. The wall designer should clearly indicate on the retaining wall plans the required sandy soil backfill criteria. The majority of the onsite soils should be suitable for retaining wall backfill once screened of material greater than 3 inches in maximum dimension. Therefore, select grading, screening and stockpiling of onsite soils meeting the criteria outlined above will be required by the contractor for obtaining suitable retaining wall backfill soil. These preliminary findings should be confirmed during grading.

TABLE 5

Lateral Earth Pressures – On-site or Imported Select Sandy Backfill

Conditions	**Equivalent Fluid Unit Weight (pcf)**	**Equivalent Fluid Unit Weight (pcf)**
	Level Backfill	**2:1 Sloped Backfill**
	Approved Soils	**Approved Soils**
Active	35	55
At-Rest	55	70

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. Retaining wall structures should be provided with appropriate drainage and appropriately waterproofed. Typical conventional retaining wall drainage is shown on Figure 3. Please note that waterproofing and outlet systems are not the purview of the geotechnical consultant. 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 basement/retaining wall footing will surcharge the proposed retaining structure. In addition to the recommended earth pressure, retaining walls adjacent to streets should be designed to resist vehicle traffic if applicable. Typical vehicle traffic may be estimated as equivalent to 2 feet of compacted fill, a vertical pressure of 240 psf. A uniform lateral pressure for typical vehicle traffic of 80 psf and 120 psf may be used for the active and at-rest conditions, respectively. Uniform lateral surcharges may be estimated using the applicable coefficient of lateral earth pressure using a rectangular distribution. A factor of 0.45 and 0.3 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 15 pcf for level backfill conditions. This increment should be applied in addition to the provided 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 2016 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. The provided seismic lateral earth pressure should not be used for retaining walls exceeding 10 feet in height. This seismic lateral earth pressure is estimated using the procedure outlined by the Structural Engineers Association of California (Lew, et al, 2010).

Soil bearing and lateral resistance (friction coefficient and passive resistance) are provided in Section 4.2. Earthwork considerations (temporary backcuts, backfill, compaction, etc.) for retaining walls are provided in Section 4.1 (Site Earthwork) and the subsequent earthwork related sub-sections.

4.5 Temporary Shoring

Typical cantilever temporary shoring, where deflection of the shoring will not impact the performance of adjacent structures, with a level backfill may be designed using the active equivalent fluid pressures of 35 pounds per square foot (psf) per foot of depth (or pcf). Braced shoring may be used in areas where the shoring will be located close to existing structures in order to limit shoring defections or required due to the proposed depth of excavation. Braced shoring with a level backfill and without hydrostatic pressures (above groundwater), may be designed using a uniform soil pressure of 24H in pounds per square foot (psf), where H is equal to the depth in feet of the excavation being shored. Any slopes above temporary shoring will increase the above-noted lateral earth pressures and can be provided on a case-by-case basis. Any building, equipment or traffic loads located within a 1:1 (horizontal to vertical) projection from the base of the shoring should be added to the applicable lateral earth pressure. Refer to the surcharge loading effects discussion provided in above Section 4.4. The shoring designer should contact the geotechnical engineer for any required geotechnical input in estimating any applicable lateral surcharge loads.

Continuous lagging should be provided between the soldier piles. Lagging should be placed in a timely manner during excavation in order to minimize potential spalling and sloughing. Careful installation of the lagging will be necessary to achieve bearing against the retained earth. The backfill of the lagging should consist of one sack sand-cement slurry or compacted moistened granular soil. It should be noted that backfill of the lagging with compacted granular soil may result in continuation of caving as the excavation depth progresses. Means and methods are per the contractor in order to ultimately ensure full bearing of retained earth to the lagging. The soldier piles should be designed for the full anticipated lateral pressure, however, the pressure on the lagging will be less due to soil arching between the piles. We recommend that the lagging be designed for the recommended earth pressure, but may be limited to a maximum value of 400 psf if surcharge loads are not present. Lagging placed behind the solider piles will negate the soil arching effect.

It is difficult to accurately predict the amount of deflection of the shored mass. It should be realized, however, that some deflection will occur. The shoring should be designed to limit deflection to within tolerable limits. If greater deflection occurs during construction, additional bracing may be necessary. In areas where less deflection is desired, such as adjacent to existing settlement sensitive improvements, the shoring should be designed for higher lateral earth pressures.

For piles spaced a minimum of 2.5 pile diameters on-center, an allowable passive pressure of 560 pcf may be used for passive resistance. The provided passive pressure is based on an arching factor of 2 (e.g., 280 pcf x 2) and should be limited to a maximum of 10 times the value provided above (e.g., 560 pcf to a maximum of 5,600 psf). The passive pressure is only applicable for level (5 horizontal feet to 1-foot vertical or flatter) soil conditions. To develop the full lateral value, provisions should be made to assure firm contact between the soldier piles and the undisturbed soils. The concrete placed in the soldier pile excavation below the excavated level should be of adequate strength to transfer the imposed loads to the surrounding soils. Structural designer should incorporate appropriate factor of safety and/or load factor in design. The provided allowable passive pressure is based on a factor of safety of 1.5.

It should be noted very difficult drilling conditions should be anticipated due to very dense sandy and gravelly soils and the presence of oversized material such as cobbles and boulders. Frequent auger refusal may occur. The contractor should evaluate the potential drilling conditions when planning the installation methods, refer to Section 4.11.

4.6 <u>Soil Corrosivity</u>

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.

Corrosion testing indicated soluble sulfate content less than 0.02 percent, chloride contents ranging from approximately 31 to 61 parts per million (ppm), pH values of approximately 8.1, and minimum resistivities ranging from approximately 9,200 to 17,000 ohm-cm. Based on Caltrans Corrosion Guidelines (2015), soils are considered corrosive if the pH is 5.5 or less, or the chloride concentration is 500 ppm or greater, or the sulfate concentration is 2,000 ppm (0.2 percent) or greater. Based on the test results, soils are not considered corrosive using Caltrans criteria.

Based on laboratory sulfate test results, the near surface soils have a severity categorization of "Not Applicable" and 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.

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.

4.7 <u>Preliminary Pavement Recommendations</u>

The following preliminary minimum asphalt concrete (AC) pavement sections provided in Table 6 are based on an assumed R-value of 40 for Traffic Indices of 5.0, 6.0 and 6.5. These recommendations must be confirmed with R-value testing of representative near-surface soils at the completion of earthwork 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. If requested, additional sections may be provided based on other traffic index values. It is our understanding that the County of Los Angeles follows the Caltrans Highway Design Manual which requires a minimum pavement section consisting of 4.2 inches of asphalt concrete over 4.2 inches of aggregate base (AB). Should the City of Duarte have more stringent requirements, updated pavement recommendations can be provided.

TABLE 6

Preliminary Asphalt Concrete Paving Section Options

Assumed Traffic Index	≤ 6.0	6.5
R -Value Subgrade	40	40
AC Thickness	4.2 inches	4.2 inches
Aggregate Base Thickness	4.2 inches	5.5 inches

The pavement section thicknesses provided above are considered 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 pavement 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.

4.8 Nonstructural Concrete Flatwork

Nonstructural concrete flatwork (such as sidewalks, patios/entryways 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 7 on the following page. 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.

TABLE 7

Preliminary Geotechnical Parameters for Nonstructural Concrete Flatwork Placed on Very Low to Low Expansion Potential Subgrade

	Sidewalks	**Patios/Entryways**	**City Sidewalk Curb and Gutters**
Minimum Thickness (in.)	4 (nominal)	4 (full)	City/Agency Standard
Presoaking	Wet down prior to placing	Wet down prior to placing	City/Agency Standard
Reinforcement	—	No. 3 at 24 inches on centers	City/Agency Standard
Thickened Edge (in.)	—	—	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	City/Agency Standard
Maximum Joint Spacing	5 feet	6 feet	City/Agency Standard
Aggregate Base Thickness (in.)	—	—	City/Agency Standard

To reduce the potential for flatwork to separate from the building foundations, the owner may elect to install dowels to tie these two elements together.

4.9 Control of Surface Water and Drainage Control

From a geotechnical perspective, we recommend that compacted finished grade soils adjacent to proposed structures be sloped away from the proposed structure and towards an approved drainage device or unobstructed swale. Drainage swales, wherever feasible, should not be constructed within 5 feet of buildings. Where lot and building, geometry necessitates that the drainage swales be routed closer than 5 feet to structural foundations, we recommend the use of area drains together with drainage swales. Drainage swales used in conjunction with area drains should be designed by the project civil engineer so that a properly constructed and maintained system will prevent ponding within 5 feet of the foundation. Code compliance of grades is not the purview of the geotechnical consultant.

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.

4.10 Subsurface Water Infiltration

Recent regulatory changes in some jurisdictions have recommended that low flow runoff be infiltrated rather than discharged via conventional storm drainage systems. In general, the vast majority of geotechnical distress issues are directly related to improper drainage. In general, distress in the form of movement of improvements could occur as a result of soil saturation and loss of soil support, expansion, internal soil erosion, collapse and/or settlement. Infiltrated water may enter underground utility pipe zones and migrate along the pipe backfill, potentially impacting other improvements located far away from the point of infiltration. From a geotechnical perspective, we typically do not recommend that water be intentionally infiltrated. Impacts from infiltration could result in additional foundation settlement beyond the amount estimated due to structural loads. This additional settlement could impact structural foundations as well as existing and planned improvements. If it is determined that water must be infiltrated due to regulatory requirements, the infiltration system designer may use the information provided below.

Per the Los Angeles County testing guidelines (2017), the design infiltration rate is determined by dividing the measured infiltration rate by a series of reduction factors including; test procedure (RF_t), site variability (RF_v) and long-term siltation plugging and maintenance (RF_s). The reduction factor for long-term siltation plugging and maintenance (RF_s) is the purview of the infiltration system designer per the Los Angeles County testing guidelines (2017). The test procedure reduction factor and recommended site variability reduction factor applied to the measured infiltration rate are provided below in Table 8. The design infiltration rate is the measured percolation rate divided by the total reduction factor (RF_t x RF_v x RF_s).

TABLE 8

Reduction Factors Applied to Measured Infiltration Rate

Consideration	Reduction Factor
Test procedure, boring percolation, RF_t	2
Site variability, number of tests, etc., RF_v	2
Long-term siltation plugging and maintenance, RF_s	Per Infiltration Designer
Total Reduction Factor, RF = RF_t x RF_v x RF_s	**TBD**

Per the requirements of the Los Angeles County testing guidelines (2017), subsurface materials shall have a design infiltration rate equal to or greater than 0.3 inches per hour. Based on the minimum Reduction Factor of 4 calculated above (not including long-term siltation plugging and maintenance), the infiltration tests performed meet the minimum requirements of the County of Los Angeles testing guidelines. Therefore, considering the results of the infiltration testing, if required, stormwater may be infiltrated into the subsurface soils at the depths tested below existing grade, using the values presented in Table 2 and Reduction Factors presented above in Table 8. Results of field infiltration testing are provided in Appendix D.

The following should be considered for design of any required infiltration system:

- Water discharge from any infiltration systems should not occur within the zone of influence of foundation footings (column and load bearing wall locations). For preliminary purposes we recommend a minimum setback of 15 feet from the structural improvements.
- An adequate setback distance between any infiltration facility and adjacent private property should be maintained.
- It may be prudent to provide an overflow system directly connected to the storm drain system in order to prevent failure of the infiltration system, either as a result of lower than anticipated infiltration and/or very high flow volumes. It should be noted that if pretreatment of runoff to remove debris, soil particles, etc., cannot be performed, design infiltration rates may need to be further reduced. Over time, siltation and plugging may reduce the infiltration rate and subsequent effectiveness of the infiltration system.
- Dry wells are not recommended due to the subsurface soil conditions. Please refer to Section 4.11 for information on anticipated drilling conditions
- Any designed infiltration system will require routine periodic maintenance.
- As with any systems that are designed to concentrate the surface flow and direct the water into the subsurface soils, some type of nuisance water and/or other water-related issues should be expected.

LGC Geotechnical should be provided with details for any planned required infiltration system early in the design process for geotechnical input.

4.11 <u>Pier Shafts and Drilling Conditions</u>

Soldier pile boreholes for temporary shoring should be plumb and free of loose or softened material. Extreme care in drilling, placement of reinforcement steel, and the pouring of concrete will be essential to avoid excessive disturbance of borehole walls. The soldier pile steel section should be installed and the concrete pumped immediately after drilling is completed. If applicable, concrete placement by pumping or tremie tube to the bottom of CIDH excavations is recommended. No soldier pile borehole should be left open overnight. We recommend that pile borings not be drilled immediately adjacent to another pile until the concrete in the other pile has attained its initial set. A representative from LGC Geotechnical should be onsite during the drilling of soldier pile boreholes to verify the assumptions made during the design stages.

The contractor should carefully evaluate the onsite geotechnical conditions as it relates to selecting an appropriate drilling method/construction technique (including, but not limited to, auger type, need for casing, construction sequence, etc.). It should be noted that auger refusal was encountered in all the hollow-stem auger borings HS-1 through HS-8 at depths ranging from approximately 3 to 24 feet presumably due to the presence of very dense sands, gravels, cobbles or boulders. The contractor should anticipate difficult drilling conditions (including the presence of cobbles, boulders, etc.). The drilling contractor would likely encounter frequent auger refusal and should plan accordingly. Sandy soils are present at the site and these materials are generally susceptible to caving. Some caving of drilled holes should be anticipated. The contractor should anticipate that any borehole left open for any extended period of time will likely experience additional caving and possible seepage typically from local irrigation. A geotechnical representative should be onsite during the drilling of any temporary shoring piers or dry wells. Refer to the boring logs in Appendix B. If caving occurs during construction of soldier pile boreholes a temporary casing may be required.

4.12 Geotechnical Plan Review

Project plans (grading, foundation, etc.) should be reviewed by this office prior to construction to verify that our geotechnical recommendations have been incorporated. Additional field work and/or modified geotechnical recommendations may be necessary.

4.13 Pre-Construction Monitoring

It is recommended that a program of pre-construction documentation and monitoring be devised and put into practice before the onset of any groundwork.

The monitoring program should include, but not necessarily be limited to, detailed documentation of the existing improvements, buildings and utilities around the site, with particular attention to any distress that is already present prior to the start of work.

4.14 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 is required per Section 1705 of the 2016 CBC.

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

- During grading (removal bottoms, fill placement, etc.);
- During drilling of pier shafts for temporary shoring;
- During retaining wall backfill and compaction;
- During utility trench backfill and compaction;
- After presoaking building pads and other concrete-flatwork subgrades, and prior to placement of aggregate base or concrete;
- After footing excavation and prior to placing concrete and/or reinforcement;
- Preparation of pavement subgrade and placement of aggregate base; and
- When any unusual soil conditions are encountered during any construction operation subsequent to issuance of this report.

5.0 LIMITATIONS

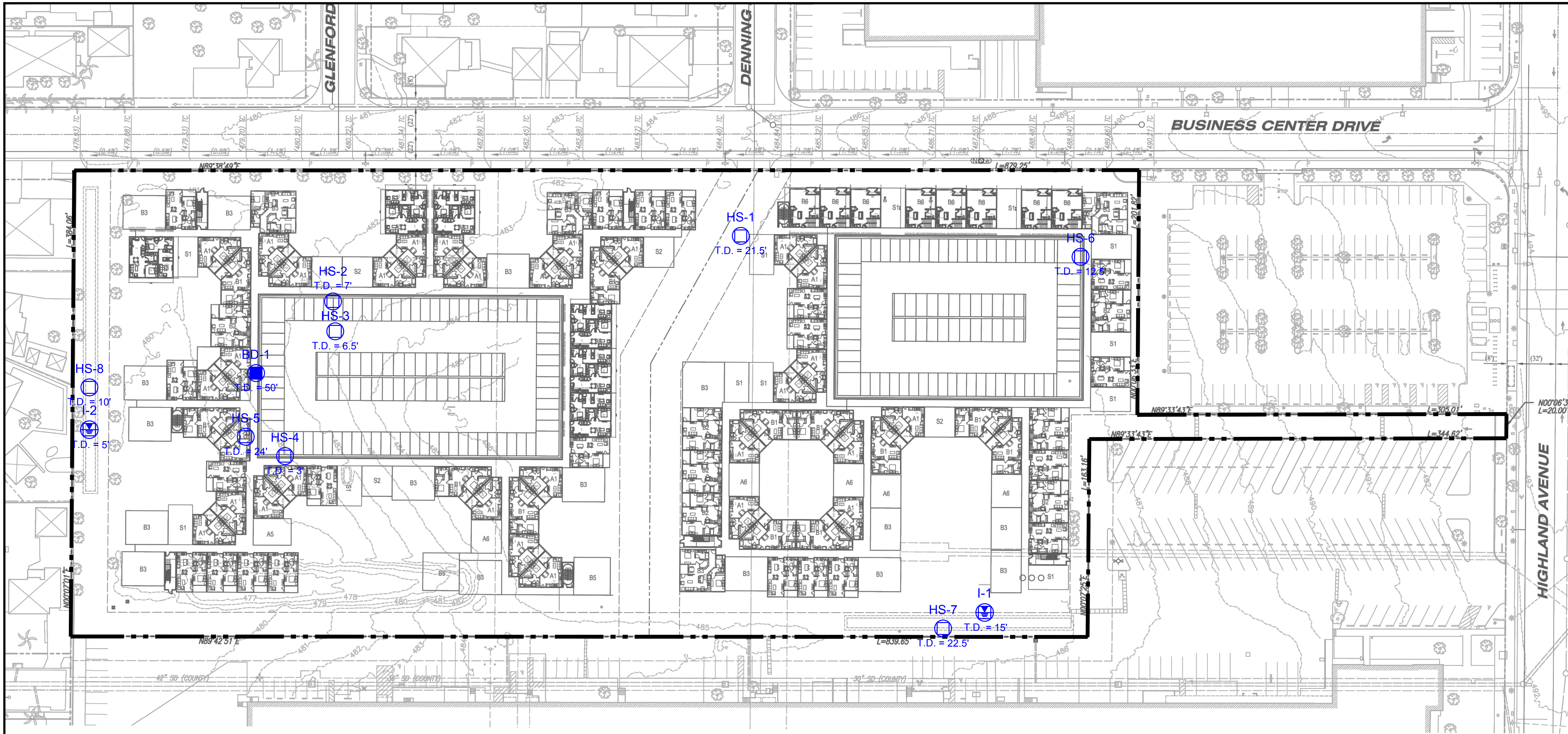
Our services were performed using the degree of care and skill ordinarily exercised, under similar circumstances, by reputable soils engineers and geologists practicing in this or similar localities. No other warranty, expressed or implied, is made as to the conclusions and professional advice included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

This report is issued with the understanding that it is the responsibility of the owner, or of their representative, to ensure that the information and recommendations contained herein are brought to the attention of the other consultants and incorporated into the 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-8
T.D. = 10'

Approximate Location of Hollow Stem Boring by LGC Geotechnical, with Approximate Total Depth in Feet

I-2
T.D. = 5'

Approximate Location of Hollow Stem Auger Infiltration Boring by LGC Geotechnical, with Approximate Total Depth in Feet

BD-1
T.D. = 50'

Approximate Location of Becker Hammer Drill Boring by LGC Geotechnical, with Approximate Total Depth in Feet

Approximate Limits of This Project

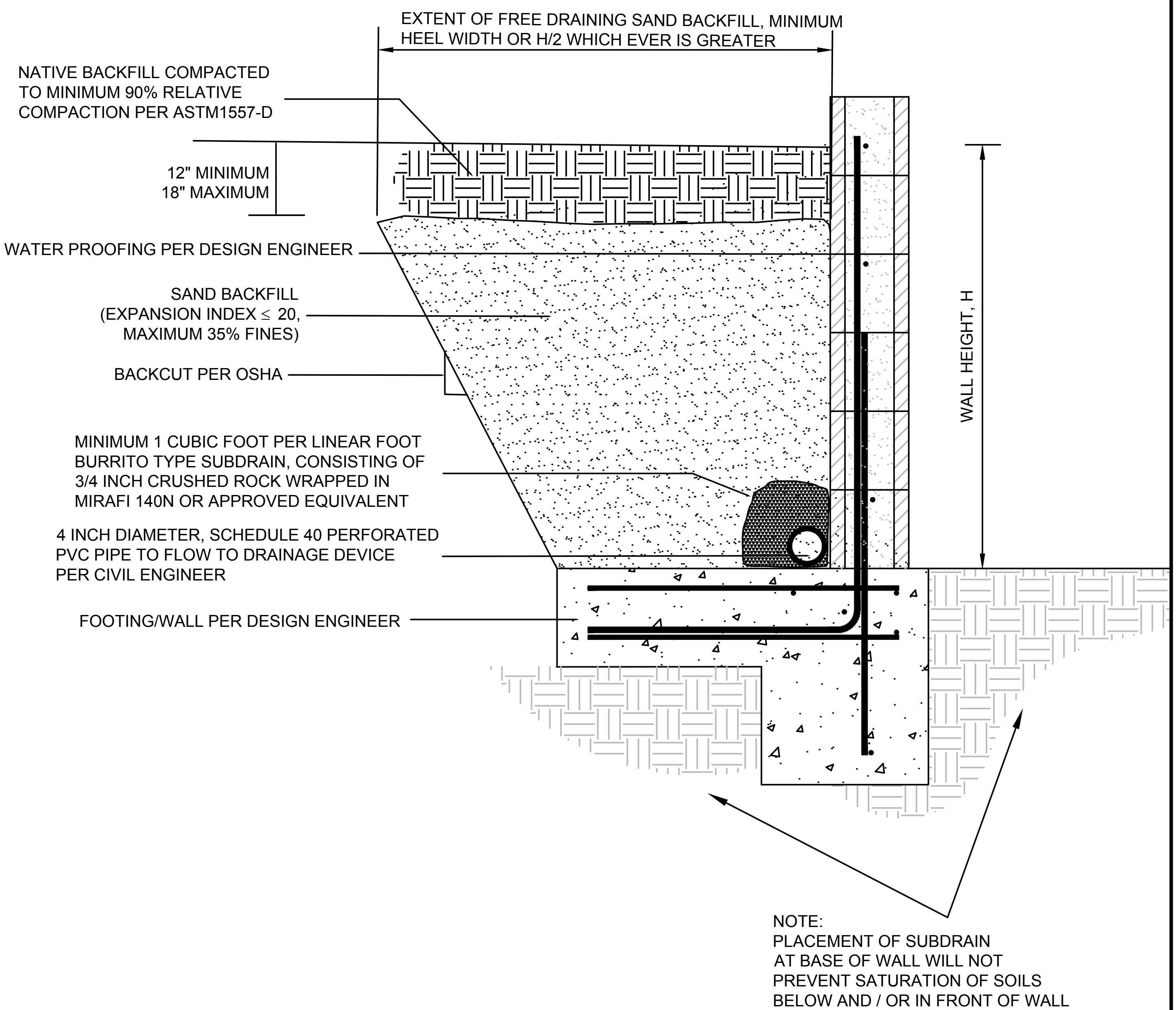
SCALE: 1"=80'

80' 0 80' 160'

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	MWIG - Highland, Duarte
PROJECT NO.	18177-01
ENG. / GEOL.	RLD
SCALE	1" = 80'
DATE	December 2018



LGC Geotechnical, Inc.

FIGURE 3
Retaining Wall Backfill Detail

PROJECT NAME	Highland - Duarte
PROJECT NO.	18177-01
ENG.	RLD
SCALE	Not to Scale
DATE	December 2018

Appendix A
References

APPENDIX A

References

American Society of Civil Engineers (ASCE), 2013, Minimum Design Loads for Buildings and Other Structures, ASCE/SEI 7-10, Third Printing, 2013.

American Society for Testing and Materials (ASTM), Volume 04.08 Soil and Rock (I): D420 – D5876.

Architects Orange, 2018, Duarte Station Apartments Site Plan, Duarte California, dated December 10, 2018.

California Building Standards Commission, 2016, California Building Code, California Code of Regulations Title 24, Volumes 1 and 2, dated July 2016.

California Department of Conservation, Division of Mines and Geology (CDMG), 1998, Seismic Hazard Zone Report for the Azusa 7.5-Minute Quadrangles, Los Angeles County, California, Seismic Hazard Zone Report, Seismic Hazard Zone Report 98-12.

California Department of Transportation (Caltrans), 2015, Corrosion Guidelines, Version 2.1, dated January 2015.

California Geological Survey (CGS), (Previously California Division of Mines and Geology [CDMG]), 1999, State of California Seismic Hazard Zones, Azusa Quadrangle, Official Map Released: March 25, 1999.

_______, 2003, Preliminary Geologic Map of the San Bernardino 30’ x 60’ Quadrangle, California, Open-File Report 03-293.

_______, 2008, Guidelines for Evaluating and Mitigating Seismic Hazards in California, Special Publication 117A, Revision dated September 11, 2008.

______, 2014, Earthquake Zones of Required Investigation, Azusa Quadrangle, Official Map Released: November 6, 2014.

Caltrans, 2015, Corrosion Guidelines, Version 2.1, dated January 2015.

County of Los Angeles, 2017, Guidelines for Geotechnical Investigation and Reporting Low Impact Development Stormwater Infiltration, Department of Public Works Geotechnical and Materials Engineering Division, GS200.2, dated June 30, 2017.

Lew, et al, 2010, Seismic Earth Pressures on Deep Basements, Structural Engineers Association of California (SEAOC) Convention Proceedings.

Morton, D.M., et al, 2003, Preliminary Geologic Map of the San Bernardino 30’ by 60’ Quadrangle, Southern California, Scale 1:100,000, Version 1.0, compiled for California Geologic Survey, Open File Report 03-293, dated 2003.

References

NCEER, 1997, "Proceeding of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils", T. L. Youd and I. M. Idriss Editors, Technical Report NCEER-97-0022, NCEER, Buffalo, NY.

Pradel, Daniel, 1998, Procedure to evaluate earthquake-induced settlement in dry sandy soils, *Journal of Geotechnical and Geoenvironmental Engineering*, Volume 124(4), pp. 364-368, dated April and October 1998.

Southern California Earthquake Center (SCEC), 1999, "Recommended Procedure for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigation Liquefaction Hazards in California", Edited by Martin, G.R., and Lew, M., dated March 1999.

Tokimatsu, K., and Seed, H. B., 1987, "Evaluation of Settlements in Sands Due to Earthquake Shaking", Journal of Geotechnical Engineering, ASCE, Vol. 113, No. 8, pp. 861-878.

United States Geological Survey (USGS), 2008, Unified Hazard Tool, Dynamic: Conterminous U.S. 2008 (v3.3.1), Retrieved November 21, 2018, from: https://earthquake.usgs.gov/hazards/interactive/

______, 2018, U.S. Seismic Design Maps, Retrieved November 21, 2018, from: http://geohazards.usgs.gov/designmaps/us/batch.php#csv

Appendix B
Boring Logs

Geotechnical Boring Log Borehole HS-1

Date: 11/12/2018
Drilling Company: 2R
Project Name: Highland - Duarte
Type of Rig: CME-75
Project Number: 18177-01
Drop: 30" **Hole Diameter:** 8"
Elevation of Top of Hole: ~485' MSL
Drive Weight: 140 pounds
Hole Location: See Geotechnical Map

Logged By ARN
Sampled By ARN
Checked By RLD

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	0	B-1						@0' Approximately 5" asphalt concrete over 5" base	
								@0.8' to T.D. Quaternary Young Alluvial Fan Deposits (Qyf)	
							SP	@0.8'-T.D. SAND with Gravel: brown, dry, very dense	
480	5			50/3"				@5' SAND with Gravel: brown, dry, very dense	
			R-2	50/5"				@7.5' SAND with Gravel: brown, dry, very dense	
475	10		R-3	45 50/4"	121.6	2.8		@10' SAND with Gravel: brown, dry, very dense	
470	15		SPT-1	25 27 27		2.5		@15' SAND with Gravel: brown, dry, very dense	
								@20' SAND with Gravel: brown, dry, very dense	
465	20		R-4	50/2"					
								@21.5' Auger refusal	
460	25							Total Depth = 21.5' Groundwater Not Encountered Backfilled with Cuttings and Capped with AC cold patch on 11/12/2018	
	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: 12/7/2018

Geotechnical Boring Log Borehole HS-2

Date: 11/12/2018	**Drilling Company:** 2R
Project Name: Highland - Duarte	**Type of Rig:** CME-75
Project Number: 18177-01	**Drop:** 30" **Hole Diameter:** 8"
Elevation of Top of Hole: ~482' MSL	**Drive Weight:** 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 1

Logged By ARN
Sampled By ARN
Checked By RLD

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	0							@0' Approximately 5" asphalt concrete over 5" base	
480								**@0.8' to T.D Quaternary Young Alluvial Fan Deposits (Qyf)**	
							SP	@0.8' SAND with Gravel: brown, dry, very dense	
	5		R-1	50/4"				@5' SAND with Gravel and Cobbles: brownish gray, dry, very dense	
								@7' Auger Refusal	
475								Total Depth = 7' Groundwater Not Encountered Backfilled with Cuttings and Capped with AC cold patch on 11/12/2018	
	10								
470									
	15								
465									
	20								
460									
	25								
455									
	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: 12/7/2018

Geotechnical Boring Log Borehole HS-3

Date: 11/12/2018	**Drilling Company:** 2R
Project Name: Highland - Duarte	**Type of Rig:** CME-75
Project Number: 18177-01	**Drop:** 30" **Hole Diameter:** 8"
Elevation of Top of Hole: ~483' MSL	**Drive Weight:** 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 1

Logged By ARN
Sampled By ARN
Checked By RLD

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	0							@0' Approximately 4" asphalt concrete over 5" base	
								@0.75' to T.D. Quaternary Young Alluvial Fan Deposits (Qyf)	
480							SP	@0.75' SAND with Gravel: brown, dry, very dense	
	5		R-1	50/4"				@5' SAND with Gravel and Cobbles: brown, dry, very dense	
			R-2	50/2"				@6.5' No Recovery, Auger Refusal	
475								Total Depth = 6.5' Groundwater Not Encountered Backfilled with Cuttings and Capped with AC cold patch inches on 11/12/2018	
	10								
470									
	15								
465									
	20								
460									
	25								
455									
	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: 12/7/2018

Geotechnical Boring Log Borehole HS-4

Date: 11/12/2018	**Drilling Company:** 2R
Project Name: Highland - Duarte	**Type of Rig:** CME-75
Project Number: 18177-01	**Drop:** 30" **Hole Diameter:** 8"
Elevation of Top of Hole: ~481' MSL	**Drive Weight:** 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 1

Logged By ARN
Sampled By ARN
Checked By RLD

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
480	0						SP	@0' Approximately 5" asphalt concrete over 5" base **@0.8' to T.D. Quaternary Young Alluvial Fan Deposits (Qyf)** @0.8' SAND with Gravel: brown, dry, very dense @3' Auger Refusal	
475	5							Total Depth = 3' Groundwater Not Encountered Backfilled with Cuttings and Capped with AC cold patch on 11/12/2018	
470	10								
465	15								
460	20								
455	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: 12/7/2018

Geotechnical Boring Log Borehole HS-5

Date: 11/12/2018	**Drilling Company:** 2R
Project Name: Highland - Duarte	**Type of Rig:** CME-75
Project Number: 18177-01	**Drop:** 30" **Hole Diameter:** 8"
Elevation of Top of Hole: ~481' MSL	**Drive Weight:** 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 1

Logged By ARN
Sampled By ARN
Checked By RLD

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
480	0	B-1						@0' Approximately 5" asphalt concrete over 5" base	EI
								@0.8' to T.D. Quaternary Young Alluvial Fan Deposits (Qyf)	CR
							SP	@0.8' SAND with Gravel: brown, dry, very dense;	
475	5		R-1	44 50/3"	114.3	0.5		@5' SAND with Gravel: brown, dry, very dense	
			R-2	50/3"				@7.5' SAND with Gravel: brown, dry, very dense	
470	10		R-3	25 50/5"			GP	@10' GRAVEL with Sand: brown, dry, very dense	CN -#200
465	15		SPT-1	39 50/5.5"		0.8	SP	@15' SAND with Gravel: brownish gray, dry, very dense	
460	20		R-4	35 50/2.5"				@20' SAND with Gravel: brown, dry, very dense; coarse sand	CN -#200
								@24' Auger Refusal	
455	25							Total Depth = 24' Groundwater Not Encountered Backfilled with Cuttings and Capped with AC cold patch on 11/12/2018	
	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: 12/7/2018

Geotechnical Boring Log Borehole HS-6

Date: 11/12/2018	**Drilling Company:** 2R
Project Name: Highland - Duarte	**Type of Rig:** CME-75
Project Number: 18177-01	**Drop:** 30" **Hole Diameter:** 8"
Elevation of Top of Hole: ~488' MSL	**Drive Weight:** 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 1

Logged By ARN
Sampled By ARN
Checked By RLD

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	0						SP	@0' Approximately 4" asphalt concrete over 5.5" base **@0.8' to T.D. Quaternary Young Alluvial Fan Deposits (Qyf)** @0.8' SAND with Gravel and Cobbles: brown, dry, very dense	
485									
	5		R-1	43 50/3"	116.7	1.6		@5' SAND with Gravel: brown to blackish brown, dry, very dense	
480			R-2	21 40 50/4"	131.7	1.8		@7.5' SAND with Gravel: brown, dry, very dense	
	10		R-3	50/2"				@10' SAND with Gravel: brown, dry, very dense @12.5' Auger Refusal	
475								Total Depth = 12.5' Groundwater Not Encountered Backfilled with Cuttings and Capped with AC cold patch on 11/12/2018	
	15								
470									
	20								
465									
	25								
460									
	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: 12/7/2018

Geotechnical Boring Log Borehole HS-7

Date: 11/13/2018
Project Name: Highland - Duarte
Project Number: 18177-01
Elevation of Top of Hole: ~486' MSL
Hole Location: See Geotechnical Map

Drilling Company: 2R
Type of Rig: CME-75
Drop: 30" **Hole Diameter:** 8"
Drive Weight: 140 pounds

Logged By ARN
Sampled By ARN
Checked By RLD

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
485	0	B-1						@0' Approximately 4" asphalt concrete over 5" base	DS MD SA EI CR
								@0.75' to 5' Artificial Fill (Af)	
							SM	@0.75' Silty SAND: brown, dry, very dense	
								@5' to T.D. Quaternary Young Alluvial Fan Deposits (Qyf)	
480	5		R-1	19 50/2"	109.2	1.3	SM-SP	@5' Silty SAND to SAND with Gravel: brown, dry, very dense	
			R-2	45 50/3"	119.3	2.1	SP	@7.5' SAND with Gravel: brown and grey, dry, very dense	
475	10		R-3	50/4"	91.5	1.3	SM	@10' Silty SAND with Gravel: olive brown, dry, very dense	
470	15		SPT-1	25 31 28		1.8	SP	@15' SAND with Gravel: brown, dry, very dense	
465	20		R-4	50/2"	109.6	1.4	SM	@20' Silty SAND with Gravel: brown, dry, very dense	
								@22.5' Auger refusal	
460	25							Total Depth = 22.5' Groundwater Not Encountered Backfilled with Cuttings and Capped with AC cold patch on 11/13/2018	
	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

Geotechnical Boring Log Borehole HS-8

Date: 11/13/2018	**Drilling Company:** 2R
Project Name: Highland - Duarte	**Type of Rig:** CME-75
Project Number: 18177-01	**Drop:** 30" **Hole Diameter:** 8"
Elevation of Top of Hole: ~480' MSL	**Drive Weight:** 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 1

Logged By ARN
Sampled By ARN
Checked By RLD

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	0							@0' Approximately 5" asphalt concrete over 5" base	
							SM	**@0.8' to T.D. Quaternary Young Alluvial Fan Deposits (Qyf)**	
								@0.8' Silty SAND with Gravel: brown, dry, very dense	
475	5		R-1	37 50/4"		0.2		@5' Silty SAND with Gravel: grayish brown, dry, very dense	
			R-2	50/5.5"				@7.5' Silty SAND with Gravel: grayish brown, dry, very dense	
470	10		R-3	50/5.5"	124.6	1.7		@10' Silty SAND with Gravel: brownish gray, dry, very dense	
								Total Depth = 10' Groundwater Not Encountered Backfilled with Cuttings and Capped with AC cold patch on 11/13/2018	
465	15								
460	20								
455	25								
450	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: 12/7/2018

Geotechnical Boring Log Borehole I-1

Date: 11/13/2018	**Drilling Company:** 2R
Project Name: Highland - Duarte	**Type of Rig:** CME-75
Project Number: 18177-01	**Drop:** 30" **Hole Diameter:** 8"
Elevation of Top of Hole: ~486' MSL	**Drive Weight:** 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 1

Logged By ARN
Sampled By ARN
Checked By RLD

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
485	0						SM	@0' Approximately 4" asphalt concrete over 5.5" base **@0.8' to T.D. Quaternary Young Alluvial Fan Deposits (Qyf)** @0.8' Silty SAND with Gravel: brown, slightly moist to dry, very dense	
480	5		R-1	50/2"	121.8	4.6		@5' Silty SAND with Gravel: greyish brown, slightly moist, very dense	
			R-2	50/1.5"				@7.5' Silty SAND with Gravel: brown, dry, very dense	
475	10		R-3	50/.5"				@10' Silty SAND with Gravel: brown, dry, very dense	
								@15' Silty SAND with Gravel: brown, dry, very dense	
470	15							Total Depth = 15' Groundwater Not Encountered Backfilled with Cuttings and Capped with AC cold patch on 11/14/2018	
465	20								
460	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: 12/7/2018

Geotechnical Boring Log Borehole I-2

Date: 11/13/2018	**Drilling Company:** 2R
Project Name: Highland - Duarte	**Type of Rig:** CME-75
Project Number: 18177-01	**Drop:** 30" **Hole Diameter:** 8"
Elevation of Top of Hole: ~480' MSL	**Drive Weight:** 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 1

Logged By ARN
Sampled By ARN
Checked By RLD

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	0						SM	@0' Approximately 5" asphalt concrete over 5" base **@0.8' to T.D. Quaternary Young Alluvial Fan Deposits (Qyf)** @0.8' Silty SAND with Gravel: brown, dry, very dense @5' Silty SAND with Gravel: grayish brown, dry, very dense	
475	5							Total Depth = 5.1' Groundwater Not Encountered Backfilled with Cuttings and Capped with AC cold patch on 11/14/2018	
470	10								
465	15								
460	20								
455	25								
450	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

Geotechnical Boring Log Borehole BD-1

Date: 11/28/2018	**Drilling Company:** Great West Drilling
Project Name: Highland - Duarte	**Type of Rig:** AP 1000 Becker Hammer
Project Number: 18177-01	**Drop:** 30" **Hole Diameter:** 6.6"
Elevation of Top of Hole: ~481' MSL	**Drive Weight:** 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 2

Logged By ARN
Sampled By ARN
Checked By RLD

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
480	0							@0' Approximately 3" asphalt concrete over 5" base	
								@0.7' to T.D. Quaternary Young Alluvial Fan Deposits (Qyf)	
							SM	@0.7' Silty SAND with Gravel: light brown-gray, dry	
475	5						SP	@2.5' SAND: medium brown, dry to slightly moist; medium fine sand	
470	10	GB-1					SP	@10'-20' SAND with Gravel: brown, slightly moist	
465	15								
460	20	GB-2							
455	25		R-1	50/4"			SM	@25' No recovery: Silty SAND with Gravel and Cobbles	
		GB-3					SP	@27' Sand	
	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: 12/7/2018

Geotechnical Boring Log Borehole BD-1

Date: 11/28/2018	**Drilling Company:** Great West Drilling
Project Name: Highland - Duarte	**Type of Rig:** AP 1000 Becker Hammer
Project Number: 18177-01	**Drop:** 30" **Hole Diameter:** 6.6"
Elevation of Top of Hole: ~481' MSL	**Drive Weight:** 140 pounds
Hole Location: See Geotechnical Map	Page 2 of 2

Logged By ARN
Sampled By ARN
Checked By RLD

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
450	30		SPT-1	35 52 54		1.9	SP-SM	@30' SAND with Silt and Gravel: light brown/olive brown, dry, very dense	-#200
445	35	GB-4, GB-5	R-2	50/4"				@35' No recovery	
440	40	GB-6	SPT-2	50/5"		1.9		@40' SAND with Silt and Gravel: gray-brown, dry, very dense	-#200
435	45	GB-7	SPT-3	50/3"				@45' No recovery	
							SP	@48' SAND with Gravel: olive brown, dry, very dense; moderate to poor sorting	
			SPT-4	50/2"					
430	50							Total Depth = 50' Groundwater Not Encountered Backfilled with Cuttings and Capped with AC cold patch on 11/28/2018	
425	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

Appendix C
Laboratory Test Results

APPENDIX C

Laboratory Test Results

The laboratory testing program was directed towards providing quantitative data relating to the relevant engineering properties of the soils. 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 driven samples obtained from the test borings. The results of these tests are presented on the boring logs in Appendix B. Where applicable, only moisture content was determined from SPT or disturbed samples.

Grain Size Distribution/Fines Content: 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 (ASTM D1140). Where applicable, 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 D6913 (sieve) or ASTM D422 (sieve and hydrometer).

Sample Location	Description	% Passing # 200 Sieve
HS-5 @ 10 ft	Gravel with Sand	4
HS-5 @ 20 ft	Sand with Gravel	4
HS-7 @ 0-5 ft	Silty Sand	18
BD-1 @ 30 ft	Sand with Silt and Gravel	6
BD-1 @ 40 ft	Sand with Silt and Gravel	5

Direct Shear: One direct shear test was performed on a relatively undisturbed driven sample. The ring samples were soaked for a minimum of 24 hours prior to testing. The samples were tested under various normal loads using a motor-driven, strain-controlled, direct-shear testing apparatus (ASTM D3080). The plot is provided in this Appendix.

Consolidation: One consolidation test was performed per ASTM D2435. A sample (2.4 inches in diameter and 1 inch in height) was placed in a consolidometer and increasing loads were applied. The sample was allowed to consolidate under “double drainage” and total deformation for each loading step was recorded. The percent consolidation for each load step was recorded as the ratio of the amount of vertical compression to the original sample height. The consolidation pressure curve is provided in this Appendix.

Laboratory Test Results

Expansion Index: The expansion potential of a selected representative samples was evaluated by the Expansion Index Test per ASTM D4829.

Sample Location	Expansion Index	Expansion Potential*
HS-5 @ 0-5 ft	0	Very Low
HS-7 @ 0-5 ft	2	Very Low

* Per ASTM D4829

Laboratory Compaction: The maximum dry density and optimum moisture content of typical materials were determined in accordance with ASTM D1557. The results of this tests are presented in the table below.

Sample Location	Sample Description	Maximum Dry Density (pcf)	Optimum Moisture Content (%)
HS-7 @ 0-5 ft	Brown silty sand	134.0	6.5

Soluble Sulfates: The soluble sulfate contents of a selected samples was determined by standard geochemical methods (CTM 417). The test results are presented in the table below.

Sample Location	Sulfate Content (%)
HS-5 @ 0-5 ft	< 0.02
HS-7 @ 0-5 ft	< 0.02

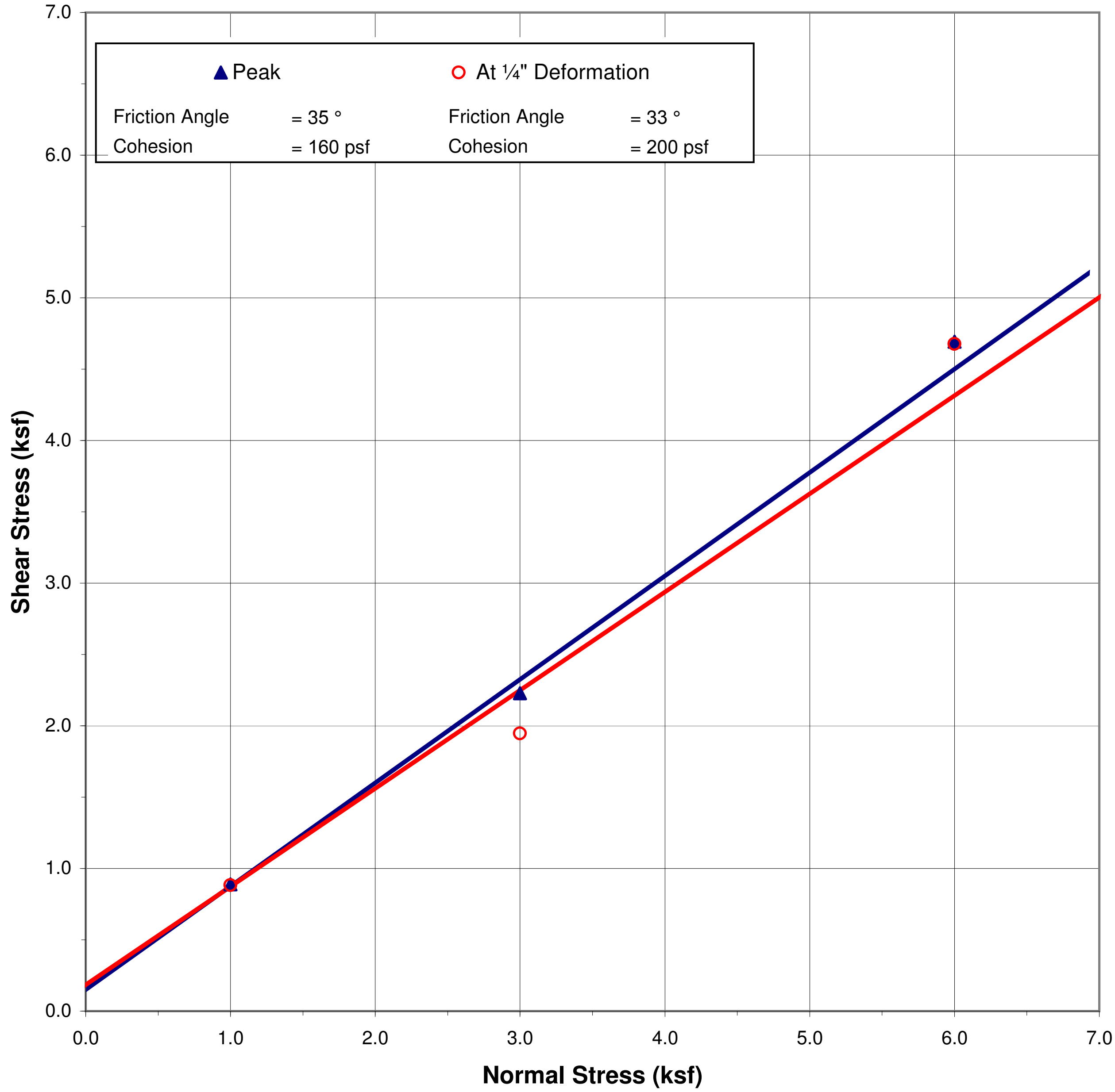
Chloride Content: Chloride content was tested per CTM 422. The results are presented below.

Sample Location	Chloride Content (ppm)
HS-5 @ 0-5 ft	31
HS-7 @ 0-5 ft	61

Laboratory Test Results

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-5 @ 0-5 ft	8.1	9,200
HS-7 @ 0-5 ft	8.1	17,000



Location:	Sample No.:	Depth (ft)	Sample Type	Shear Rate (inch/min)	Dry Density (pcf)	Initial Moisture Content (%)	Final Moisture Content (%)
HS-7	**B-1**	**0-5 ft**	**Remolded**	**0.004**	**110.0**	**6.5**	**14.0**

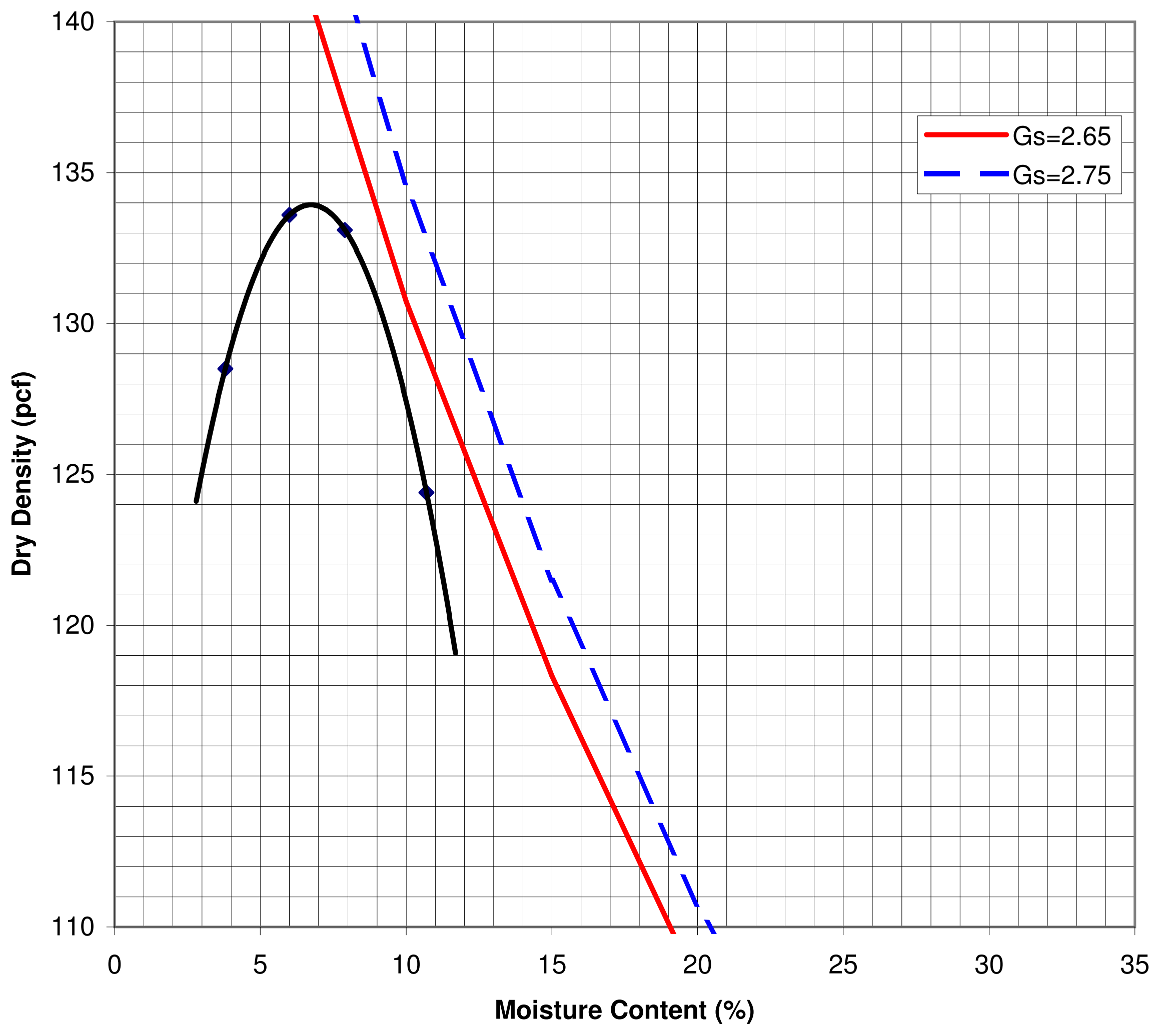
Sample Description: Silty sand (SM)

LGC Geotechnical, Inc.

DIRECT SHEAR PLOT

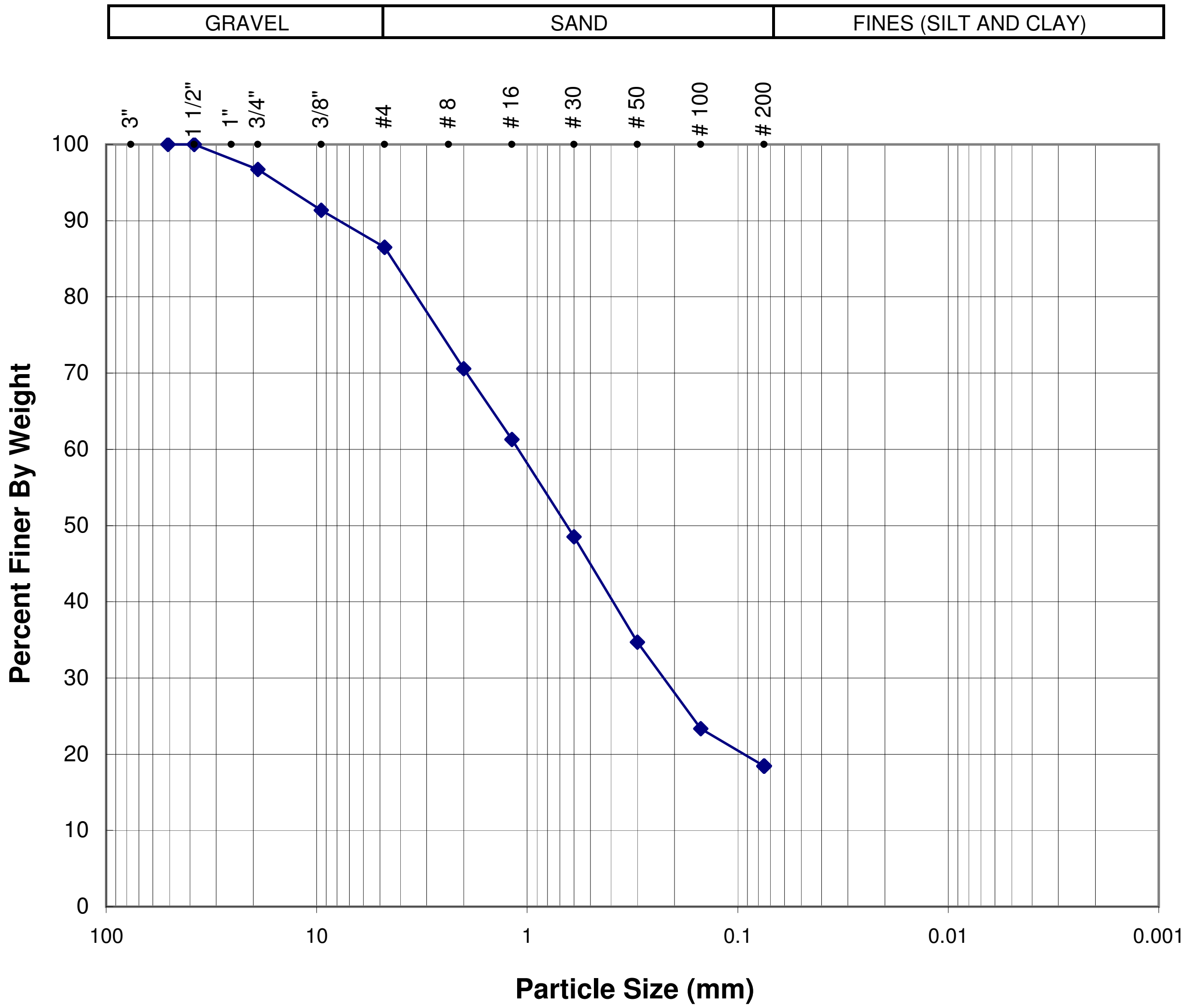
Project Number: 18177-01
Date: Nov-18

Highland - Duarte



Location:	Sample No.:	Depth (ft)	Sample Description	Maximum Dry Density (pcf)	Optimum Moisture Content (%)
HS-7	**HS-7 B-1**	**0-5'**	**Brown silty sand**	**134.0**	**6.5**

LGC Geotechnical, Inc.	**LABORATORY COMPACTION** (ASTM D 1557)	**Project Number:** 18177-01 **Date:** Nov-18 **Highland - Duarte**



Location:	Sample No.:	Depth (ft.)	Soil Type	Gravel (%)	Sand (%)	Fines (%)
HS-7	**HS-7 B-1**	**0-5'**	**SM**	**13**	**68**	**18**

Sample Description: Silty sand

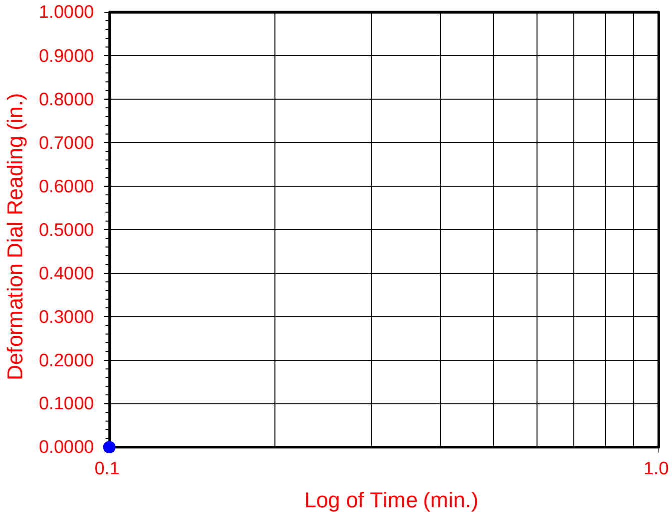
LGC Geotechnical, Inc.

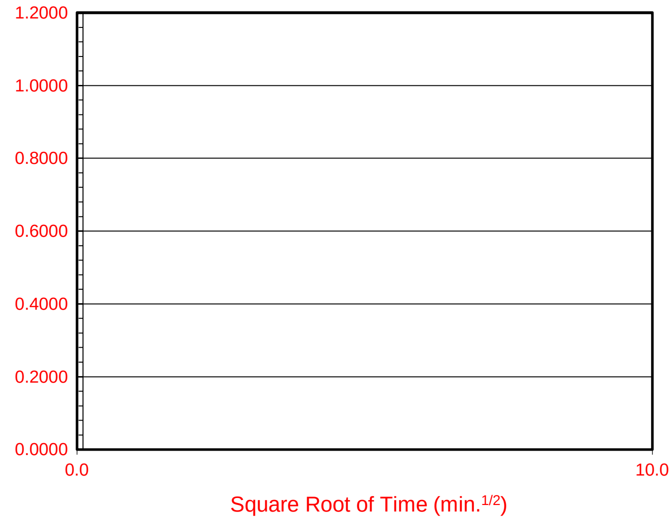
PARTICLE SIZE ANALYSIS
(ASTM D 422)

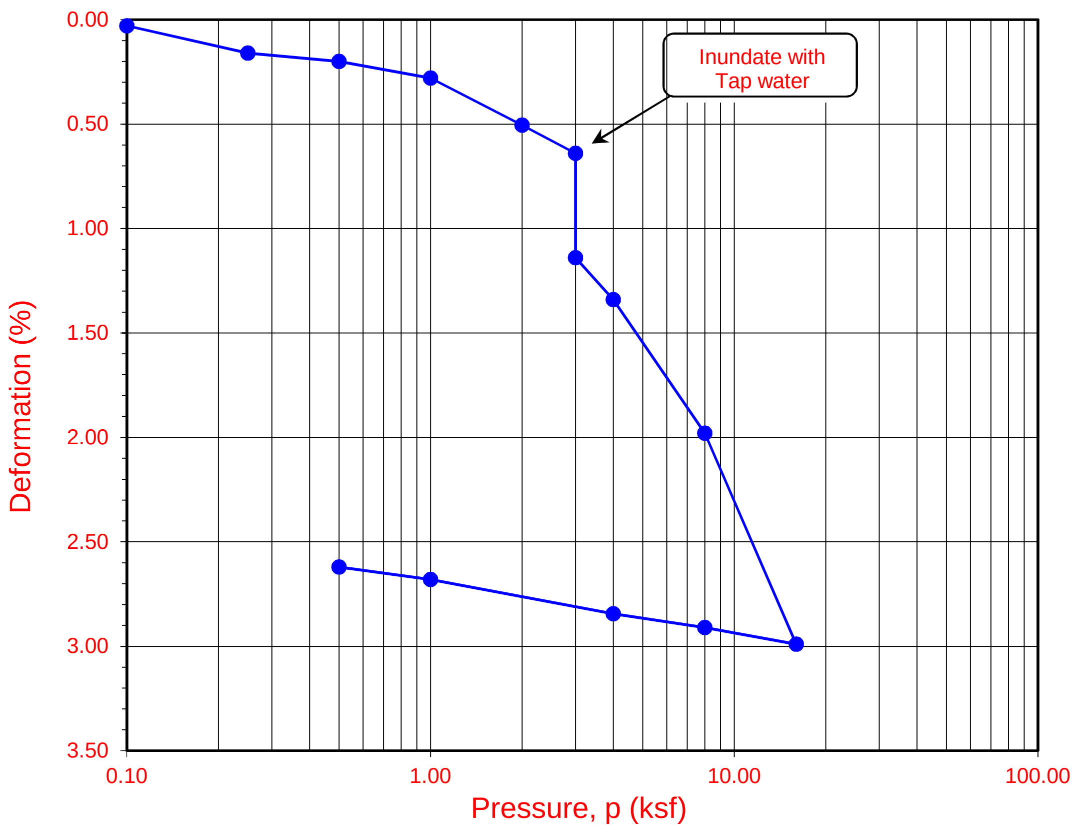
Project Number: 18177-01
Date: Nov-18

Highland - Duarte

Time Readings







Boring No.	Sample No.	Depth (ft.)	Moisture Content (%)		Dry Density (pcf)		Void Ratio		Degree of Saturation (%)	
			Initial	Final	Initial	Final	Initial	Final	Initial	Final
HS-5	**R-4**	**20.0**	**2.6**	**18.7**	**103.0**	**104.9**	**0.637**	**0.594**	**11**	**83**

Soil Identification: Brown sand with gravel (SP)

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS
ASTM D 2435

Project No.: 18177-01

Highland - Duarte

12-18

Appendix D
Infiltration Test Data

Infiltration Test Data Sheet

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

Project Name: Highland - Duarte
Project Number: 18177-01
Date: 11/15/2018
Location: I-1

Test hole dimensions (if circular)	
Boring Depth (feet)*:	15.2
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							11/13/2018
Pre-Test	8:21	8:51	30.0	13.9	14.81	0.91	
Pre-Test	8:52	9:27	35.0	14.21	14.83	0.62	

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	9:30	10:00	30.0	12.21	14.77	2.56	3.93	5.5
2	10:02	10:32	30.0	12.28	14.77	2.49	3.86	5.4
3	10:34	11:04	30.0	12.20	14.77	2.57	3.94	5.5
4	11:04	11:14	10.0	12.10	13.82	1.72	5.04	8.6
5	11:14	11:28	14.0	13.82	14.67	0.85	2.35	6.5
6	11:30	11:44	14.0	12.44	14.17	1.73	4.32	7.2
7	11:46	11:56	10.0	10.91	13.52	2.61	6.60	9.9
8	11:59	12:09	10.0	9.31	12.23	2.92	9.63	7.6
9	12:09	12:19	10.0	12.23	13.75	1.52	4.98	7.7
10	12:19	12:29	10.0	13.75	14.44	0.69	2.66	6.5
11	12:32	12:42	10.0	9.74	12.49	2.75	8.90	7.8
12	12:42	12:55	13.0	12.49	13.99	1.50	4.45	6.5

Measured Infiltration Rate	**6.5**
Minimum Reduction Factor	4
Max. Design Infiltration Rate	**1.6**

Sketch:

Notes:

Based on Guidelines from: LA County dated 06/2017
Spreadsheet Revised on: 11/29/2017

LGC Geotechnical, Inc.

Infiltration Test Data Sheet

LGC Geotechnical, Inc

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

Project Name: Highland - Duarte
Project Number: 18177-01
Date: 11/15/2018
Location: I-2

Test hole dimensions (if circular)	
Boring Depth (feet)*:	5.1
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							11/13/2018
Pre-Test	8:12	8:42	30.0	3.57	4.7	1.13	
Pre-Test	8:44	9:19	35.0	3.87	4.71	0.84	

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	9:16	9:46	30.0	3.40	4.66	1.26	2.59	4.1
2	9:48	10:18	30.0	3.38	4.60	1.22	2.67	3.8
3	10:20	10:50	30.0	3.10	4.58	1.48	2.99	4.1
4	10:52	11:22	30.0	2.89	4.58	1.69	3.21	4.4
5	11:23	11:53	30.0	2.88	4.49	1.61	3.31	4.1
6	11:53	12:03	10.0	2.65	3.64	0.99	4.44	5.6
7	12:03	12:13	10.0	3.64	4.23	0.59	2.79	5.3
8	12:15	12:25	10.0	2.72	3.59	0.87	4.42	4.9
9	12:25	12:35	10.0	3.59	4.28	0.69	2.79	6.2
10	12:37	12:47	10.0	2.65	3.49	0.84	4.60	4.6
11	12:47	12:59	12.0	3.49	4.24	0.75	2.94	5.4
12	13:00	13:10	10.0	2.97	3.8	0.83	3.94	5.3

Measured Infiltration Rate	**4.9**
Minimum Reduction Factor	4
Max. Design Infiltration Rate	**1.2**

Sketch:

Notes:

Based on Guidelines from: LA County dated 06/2017

Spreadsheet Revised on: 11/29/2017

LGC Geotechnical, Inc.

Appendix E
General Earthwork and Grading Specifications

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.

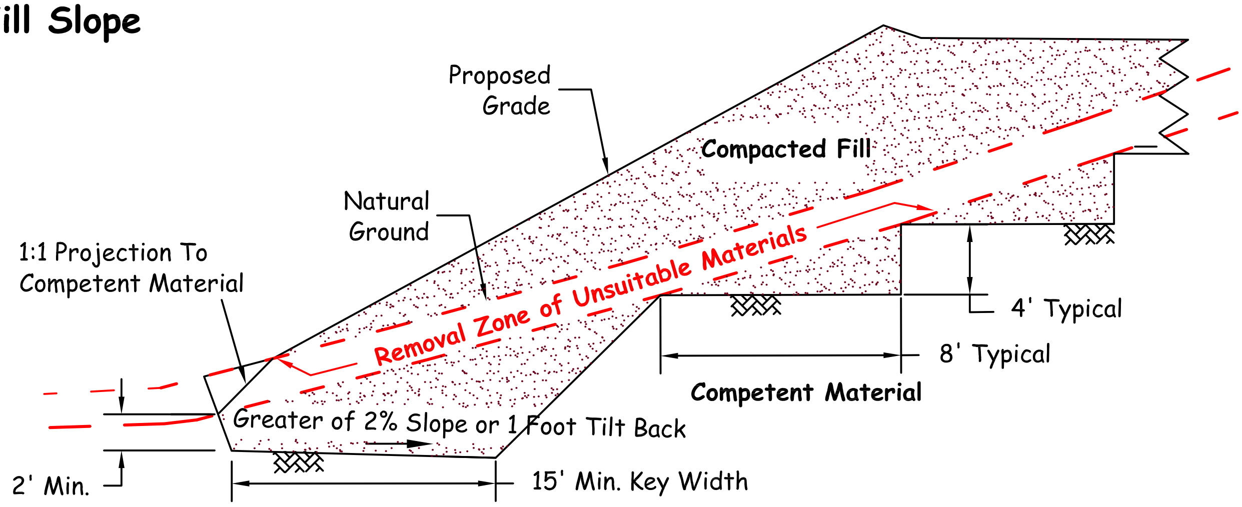
7.2 All bedding and backfill of utility trenches shall be done in accordance with the applicable provisions of Standard Specifications of Public Works Construction. Bedding material shall have a Sand Equivalent greater than 30 (SE>30). The bedding shall be placed to 1 foot over the top of the conduit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

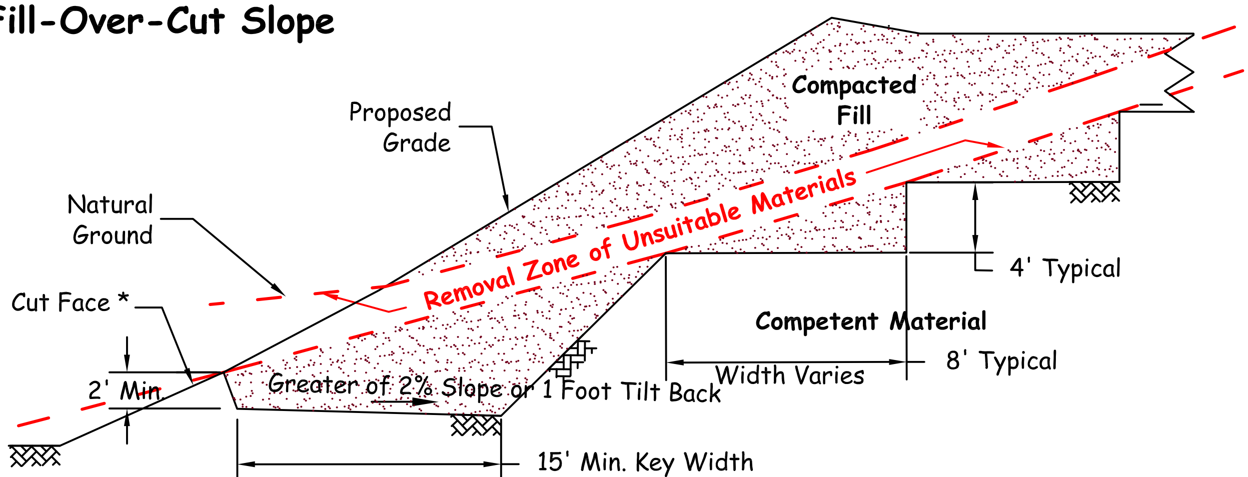
7.4 The Geotechnical Consultant shall test the trench backfill for relative compaction. At least one test should be made for every 300 feet of trench and 2 feet of fill.

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.

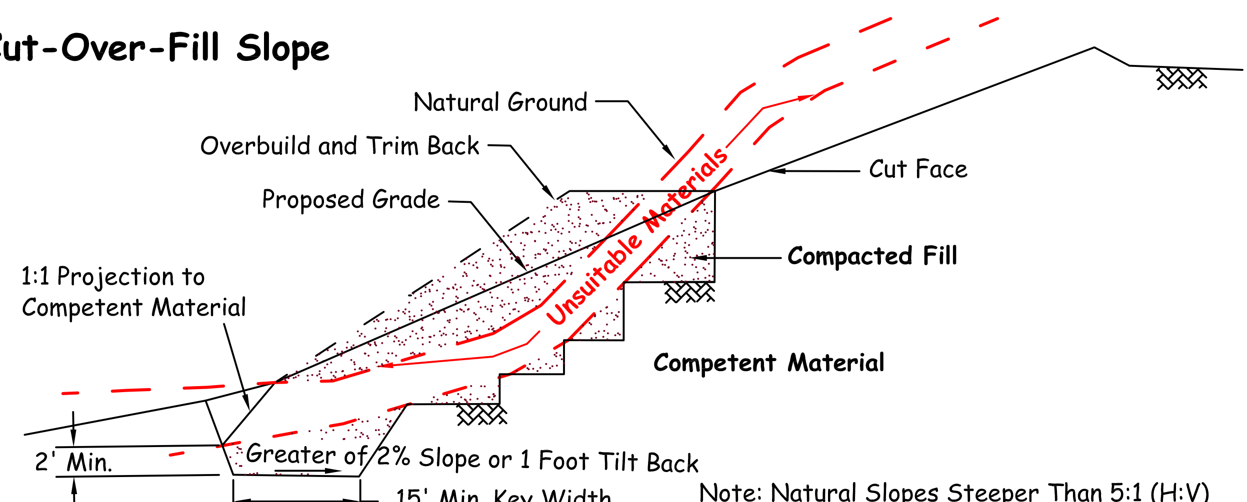
Fill Slope



Fill-Over-Cut Slope

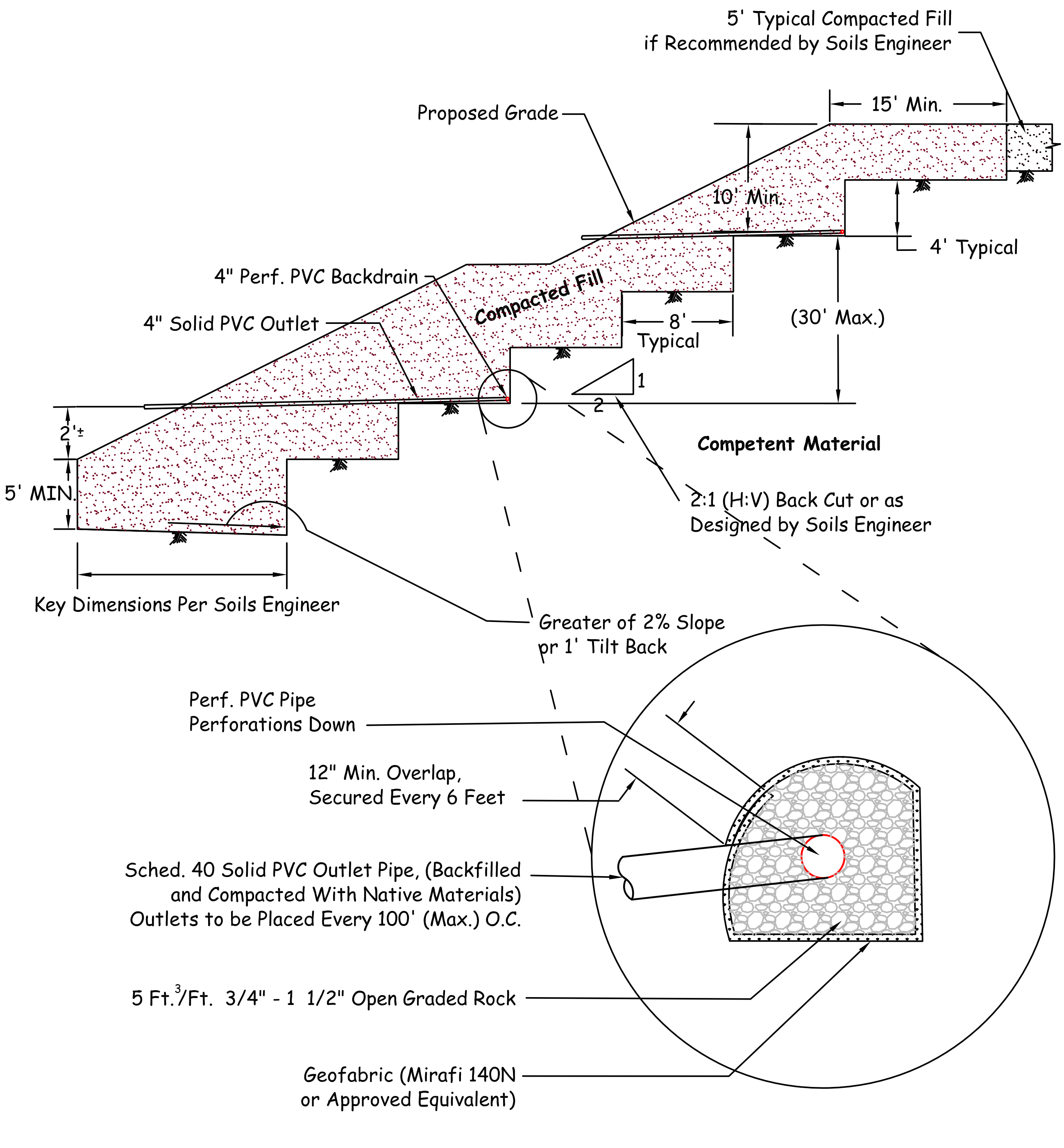


Cut-Over-Fill Slope



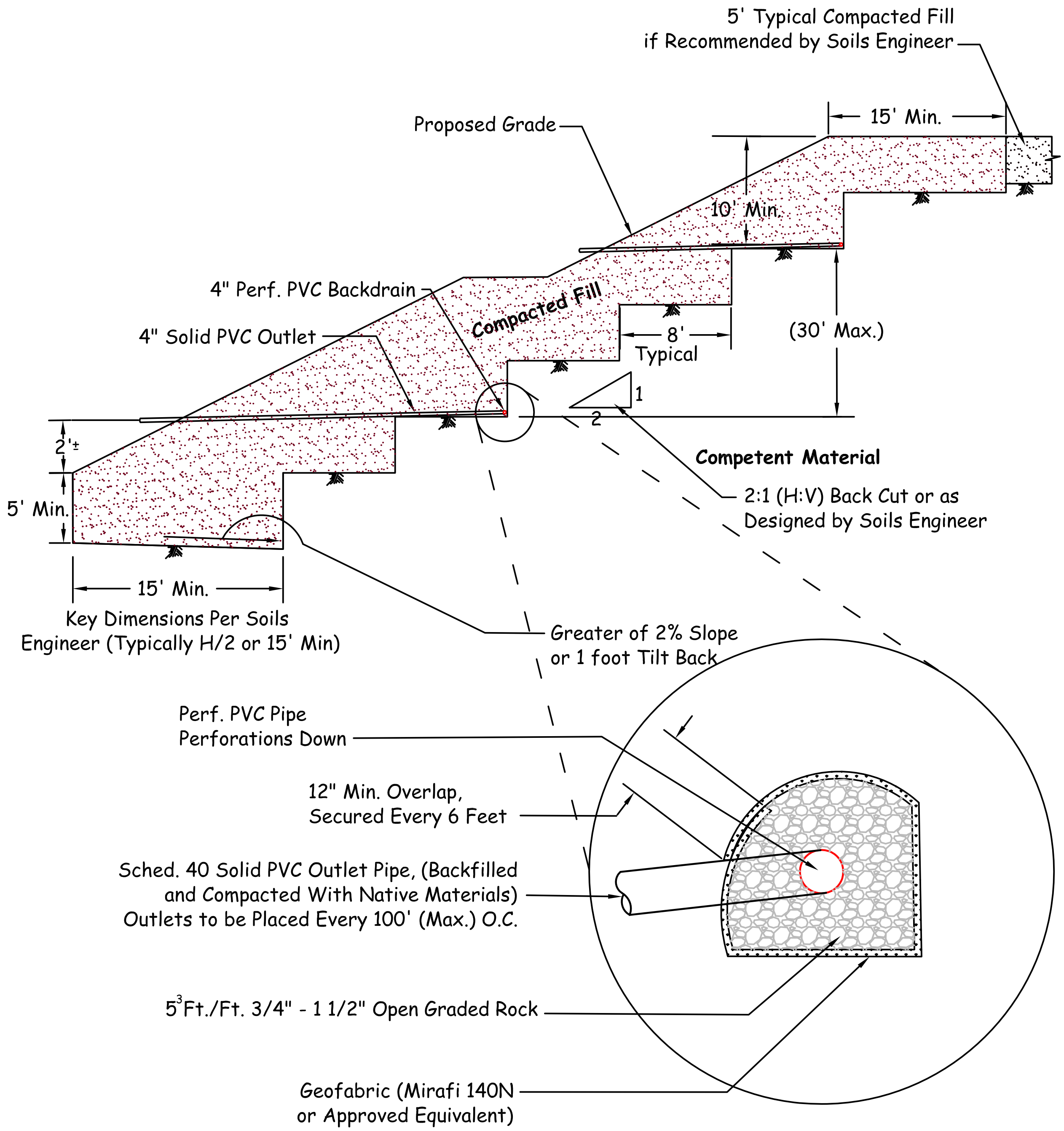
LGC Geotechnical, Inc.

KEYING AND BENCHING



LGC Geotechnical, Inc.

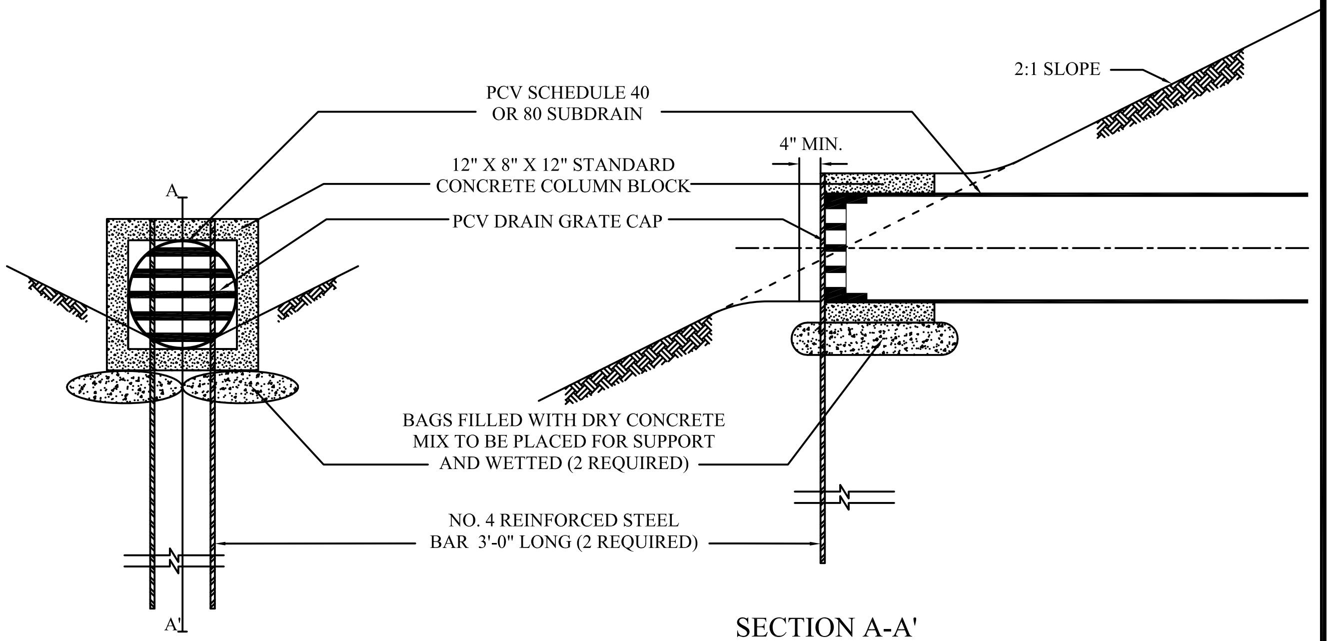
TYPICAL BUTTRESS DETAIL



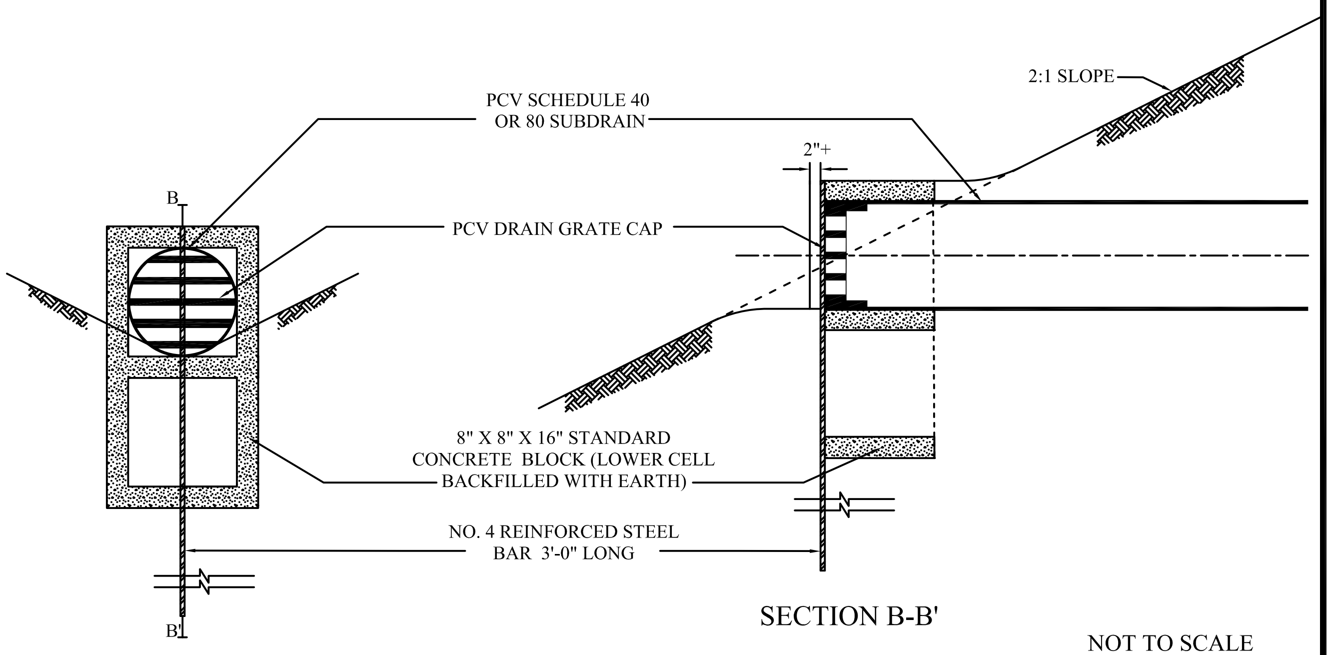
LGC Geotechnical, Inc.

TYPICAL STABILIZATION FILL DETAIL

SUBDRAIN OUTLET MARKER -6" & 8" PIPE



SUBDRAIN OUTLET MARKER -4" PIPE

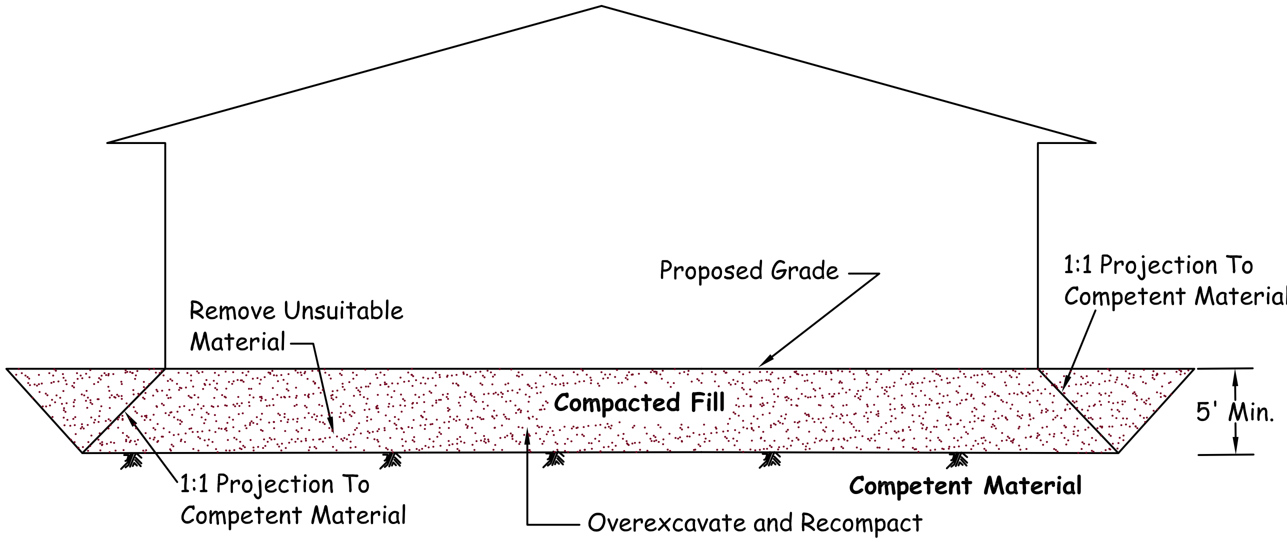


NOT TO SCALE

LGC Geotechnical, Inc.

SUBDRAIN OUTLET MARKER DETAIL

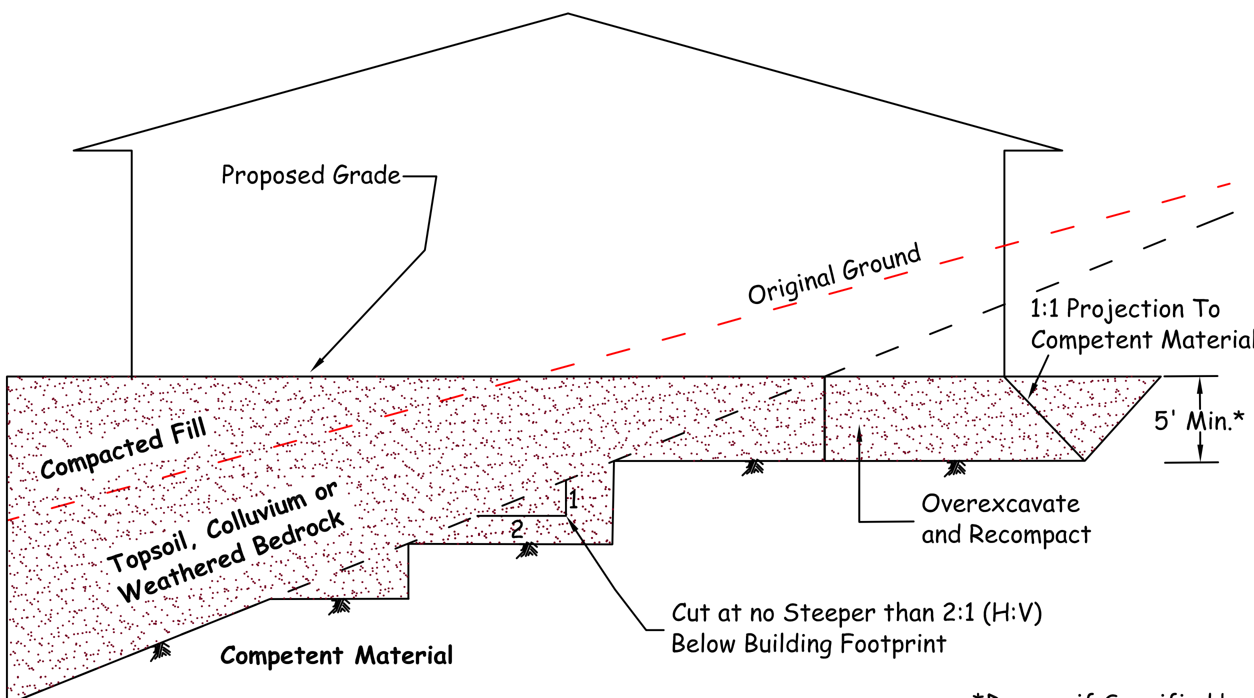
Cut Lot
(Exposing Unsuitable Soils at Design Grade)



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.

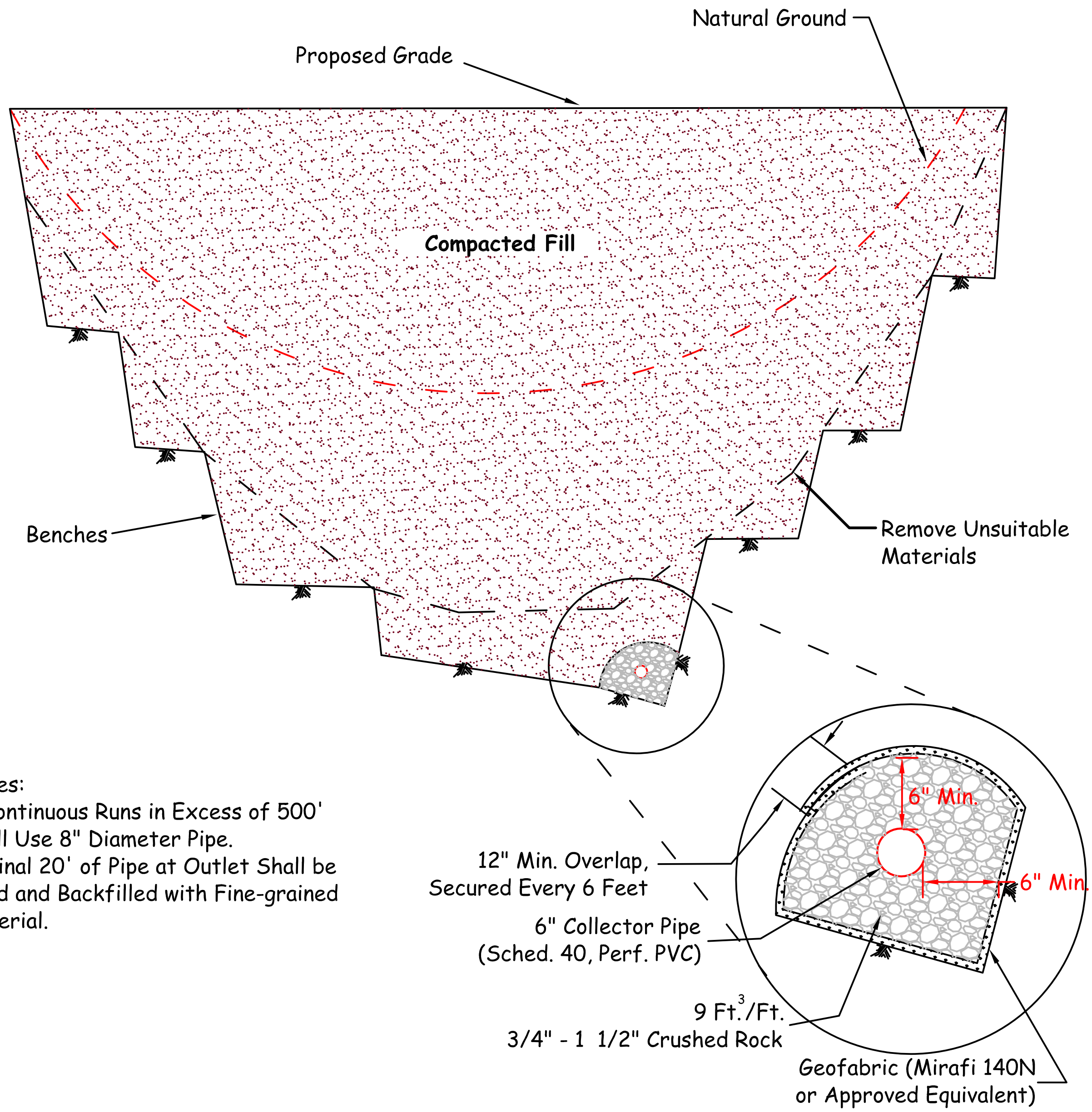
Cut/Fill Transition Lot



*Deeper if Specified by Soils Engineer

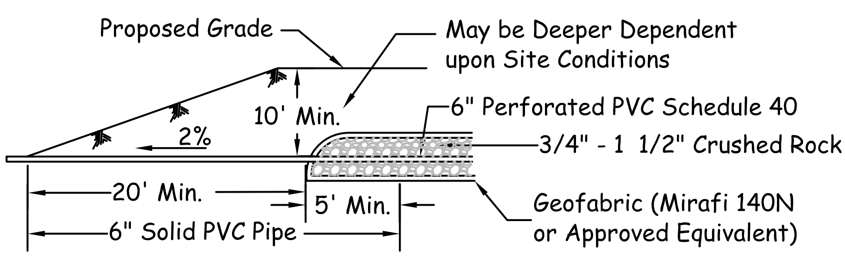
LGC Geotechnical, Inc.

CUT AND TRANSITION LOT OVEREXCAVATION DETAIL



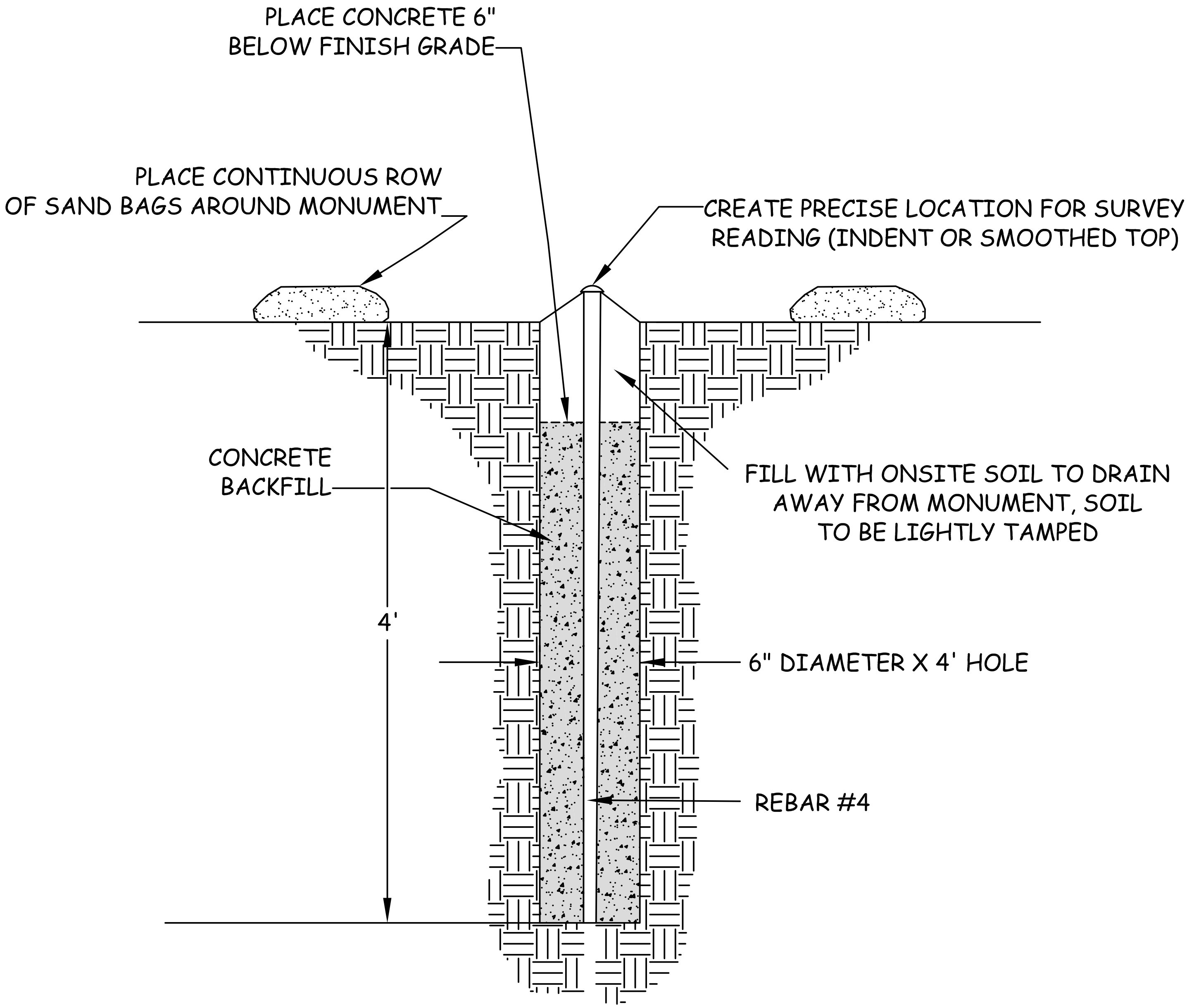
Notes:
1) Continuous Runs in Excess of 500' Shall Use 8" Diameter Pipe.
2) Final 20' of Pipe at Outlet Shall be Solid and Backfilled with Fine-grained Material.

Proposed Outlet Detail



LGC Geotechnical, Inc.

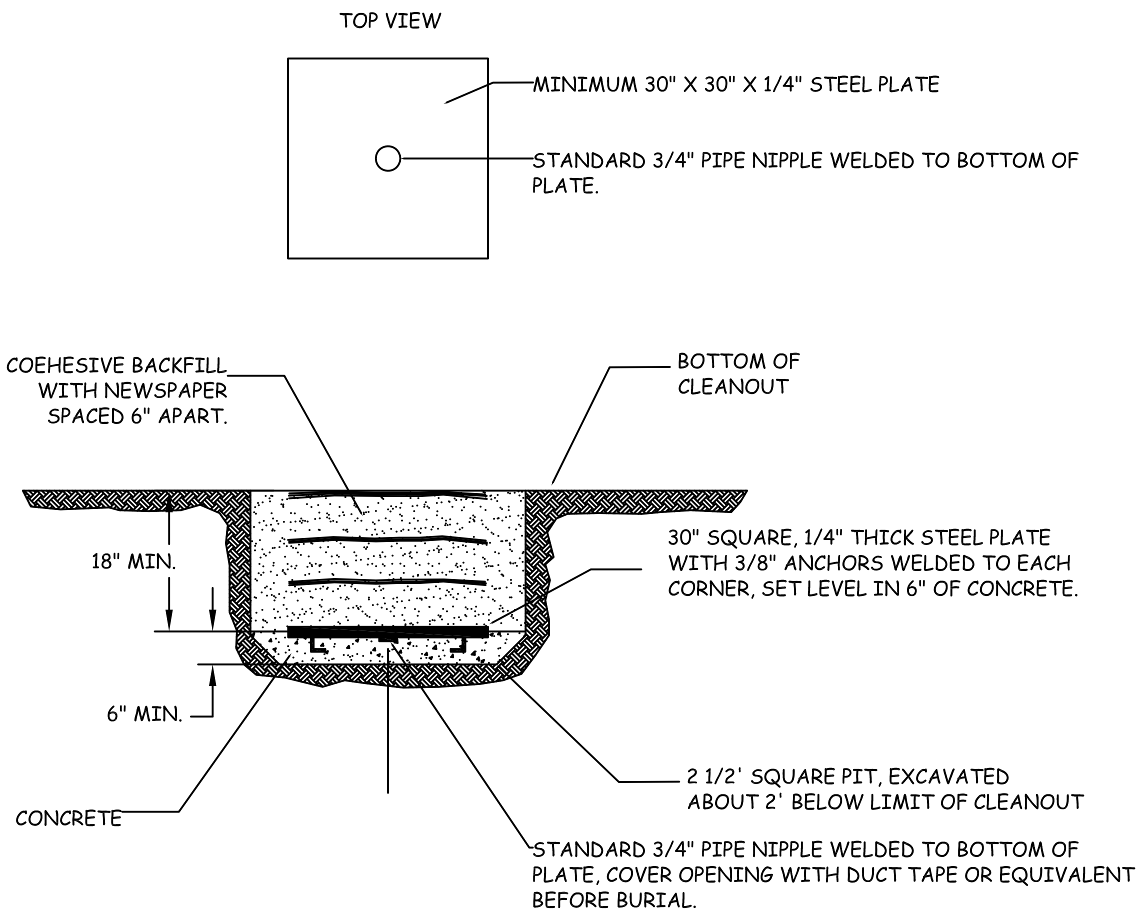
CANYON SUBDRAINS



NO CONSTRUCTION EQUIPMENT WITHIN 25 FEET
OF ANY INSTALLED SETTLEMENT MONUMENTS

LGC Geotechnical, Inc.

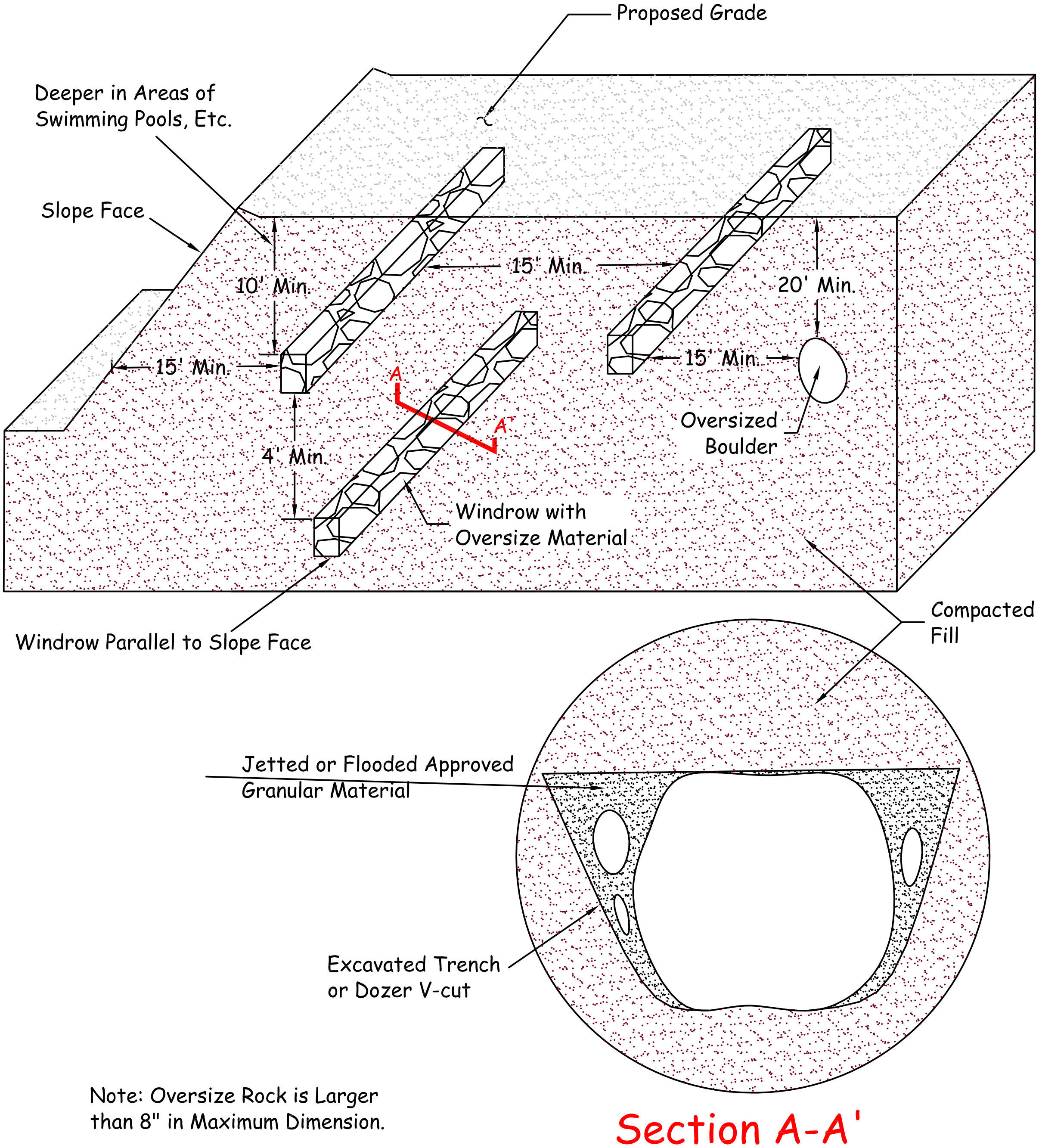
TYPICAL SURFACE SETTLEMENT MONUMENT



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.

LGC Geotechnical, Inc.

TYPICAL SETTLEMENT PLATE AND RISER



LGC Geotechnical, Inc.

OVERSIZE ROCK DISPOSAL DETAIL