Appendix D Geotechnical Investigation and Results of Infiltration Testing

Appendix

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GEOTECHNICAL INVESTIGATION
PROPOSED WAREHOUSE BUILDING

4116 Azusa Canyon Road
Irwindale, California
for
Rexford Industrial

SOCALGEO
SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

February 14, 2020

Rexford Industrial
11620 Wilshire Boulevard, 10th Floor
Los Angeles, California 90025

Attention: Mr. Ricardo Rivas
Construction Manager

Project No.: **20G105-1**

Subject: **Geotechnical Investigation**
Proposed Warehouse
4416 Azusa Canyon Road
Irwindale, California

Gentlemen:

In accordance with your request, we have conducted a geotechnical investigation at the subject site. We are pleased to present this report summarizing the conclusions and recommendations developed from our investigation.

We sincerely appreciate the opportunity to be of service on this project. We look forward to providing additional consulting services during the course of the project. If we may be of further assistance in any manner, please contact our office.

Respectfully Submitted,

SOUTHERN CALIFORNIA GEOTECHNICAL, INC.

Daniel W. Nielsen, RCE 77915
Senior Engineer

Robert G. Trazo, GE 2655
Principal Engineer

Distribution: (1) Addressee

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1.0 EXECUTIVE SUMMARY

Presented below is a brief summary of the conclusions and recommendations of this investigation. Since this summary is not all inclusive, it should be read in complete context with the entire report.

Site Preparation

- Demolition of the existing structures and pavements will be necessary in order to facilitate the construction of the proposed development. Demolition should include all foundations, floor slabs, utilities and any other subsurface improvements that will not remain in place with the new development. Debris resultant from demolition should be disposed of offsite. Alternatively, concrete and asphalt debris may be pulverized to a maximum 2-inch particle size, well mixed with the on-site soils, and incorporated into new structural fills or it may be crushed and made into CMB, if desired.
- Initial site stripping should include the removal of any surficial vegetation. Based on conditions encountered at the time of the subsurface exploration, stripping of a few trees and some vegetation will be necessary along the perimeter of the site. Site stripping should remove any tree root masses in their entirety. These materials should be disposed of offsite.
- The near surface soils encountered at the trench locations generally consist of medium dense undocumented fill soils underlain by dense native alluvial soils consisting of silty sands and well-graded gravelly sands with significant cobble and boulder content. The undocumented fill soils extend to depths of 3 to 7½± feet at the trench locations.
- Remedial grading is recommended within the proposed building area, in order to provide uniform support conditions for the new foundations and the floor slab of the proposed structure and to remove undocumented fill soils and any soils disturbed during demolition. We recommend that the proposed building pad area be overexcavated to a depth of at least 3 feet below existing grade and to a depth of at least 3 feet below proposed pad grade. The overexcavation should also extend to a sufficient depth to remove all of the undocumented artificial fill materials within the building pad area. Overexcavation within the foundation areas is recommended to extend to a depth of at least 2 feet below proposed foundation bearing grade.
- As discussed above, the native alluvial soils possess significant amounts of oversized materials, including cobbles and boulders. Where grading will require excavation into these materials, consideration should be given to using selective grading techniques to remove the cobbles and/or boulders from these soils prior to reuse as fill. Recommendations regarding selective grading and handling of oversized materials are provided in Section 6.3 and Appendix D of this report.
- After overexcavation has been completed, the resulting subgrade soils should be evaluated by the geotechnical engineer to identify any additional soils that should be overexcavated. The resulting soils should be scarified and thoroughly flooded to achieve a moisture content of 0 to 4 percent above optimum moisture, to a depth of at least 24 inches. The overexcavation subgrade soils should then be recompacted under the observation of the geotechnical engineer. The previously excavated soils may then be replaced as structural fill, compacted to 90 percent of the ASTM D-1557 maximum dry density.

- The new parking area subgrade soils are recommended to be scarified to a depth of 12± inches, moisture conditioned to 0 to 4 percent above optimum, and recompacted to at least 90 percent of the ASTM D-1557 maximum dry density.

Building Foundations

- Spread footing foundations, supported in newly placed structural fill soils.
- Maximum, net allowable soil bearing pressure: 3,000 lbs/ft^2.
- Reinforcement consisting of at least two (2) No. 5 rebars (1 top and 1 bottom) in strip footings. Additional reinforcement may be necessary for structural considerations.

Building Floor Slabs

- Conventional Slab-on-Grade, at least 6 inches thick.
- Modulus of Subgrade Reaction: k = 200 psi/in.
- Reinforcement is not expected to be necessary for geotechnical considerations.
- The actual thickness and reinforcement of the floor slabs should be determined by the structural engineer.

Pavements

ASPHALT PAVEMENTS (R = 60)					
Materials	**Thickness (inches)**				
	Auto Parking and Auto Drive Lanes (TI = 4.0 to 5.0)	Truck Traffic			
		TI = 6.0	TI = 7.0	TI = 8.0	TI = 9.0
Asphalt Concrete	3	3½	4	5	5½
Aggregate Base	3	3	3	3	4
Compacted Subgrade	12	12	12	12	12

PORTLAND CEMENT CONCRETE PAVEMENTS				
Materials	**Thickness (inches)**			
	Autos and Light Truck Traffic (TI = 5.0 & 6.0)	Truck Traffic		
		TI = 7.0	TI = 8.0	TI = 9.0
PCC	5	5	6½	8
Compacted Subgrade (95% minimum compaction)	12	12	12	12

2.0 SCOPE OF SERVICES

The scope of services performed for this project was in accordance with our Proposal No. 19P370, dated September 25, 2019. The scope of services included a visual site reconnaissance, subsurface exploration, field and laboratory testing, and geotechnical engineering analysis to provide criteria for preparing the design of the building foundations, building floor slab, and parking lot pavements along with site preparation recommendations and construction considerations for the proposed development. The evaluation of the environmental aspects of this site was beyond the scope of services for this geotechnical investigation.

3.0 SITE AND PROJECT DESCRIPTION

3.1 Site Conditions

The subject site is located at the northeast corner of Azusa Canyon Road and Los Angeles Street in Irwindale, California. The site is bounded to the north by the Big Dalton Wash, to the west by Azusa Canyon Road, to the south by Los Angeles Street, and to the southeast and east by an existing railroad easement. The general location of the site is illustrated on the Site Location Map, included as Plate 1 of this report.

The site consists of an irregular-shaped parcel, 5.89± acres in size. The site is presently developed with one warehouse, 64,535± ft² in size, in the western half of the site. The warehouse is currently occupied by Pepsi Bottling Group. The building is a single-story structure of concrete tilt-up construction and is assumed to be supported on conventional shallow foundations with a concrete slab-on-grade floor. A loading dock is located along a portion of the northeast building wall. A modular building, about 1,000 ± ft² in size is present in the east-central portion of the site. This modular building appears to be supported directly on the pavements. The buildings are surrounded by asphaltic concrete pavements in the parking and drive areas, Portland cement concrete pavements in the loading dock areas, and concrete flatwork in limited areas throughout the site. The southeastern area of the site is vacant and undeveloped. The ground surface cover in this area consists of exposed soil with moderate to extensive native grass and weed growth.

Detailed topographic information was not available at the time of this report. Based on visual observations made at the time of the subsurface investigation and from elevation data obtained from Google Earth, the overall site topography generally slopes downward to the southwest at a gradient of 1 to 2± percent.

3.2 Proposed Development

A site plan, prepared by GAA Architects, has been provided to our office by the client. Based on this plan, a new warehouse, 130,540± ft² in size, will be constructed in the central area of the site. Dock-high doors will be constructed along a portion of the south building wall. The building will be surrounded by asphaltic concrete pavements in the parking and drive lanes, Portland cement concrete pavements in the loading dock areas, concrete flatwork and landscape planters throughout.

Detailed structural information has not been provided. It is assumed that the new building will be a single-story structure of tilt-up concrete construction, typically supported on conventional shallow foundations with a concrete slab-on-grade floor. Based on the assumed construction, maximum column and wall loads are expected to be on the order of 100 kips and 3 to 5 kips per linear foot, respectively.

Grading plans for the proposed development were not available at the time of this report. The proposed development is not expected to include any significant amounts of below-grade construction such as basements or crawl spaces. Based on the existing topography, and assuming a relatively balanced site, cuts and fills of 2 to 3± feet are expected to be necessary to achieve the proposed site grades.

4.0 SUBSURFACE EXPLORATION

4.1 Scope of Exploration/Sampling Methods

The subsurface exploration conducted for this project consisted of four (4) trenches excavated to depths of 6 to 9± feet below the existing site grades. All of the trenches were logged during excavation by a member of our staff.

The trenches were excavated using a backhoe with a 24-inch-wide bucket. Representative bulk and soil samples were taken during excavation. The bulk samples were collected in plastic bags to retain their original moisture content. The bulk samples were then sealed and transported to our laboratory.

The approximate locations of the trenches are indicated on the Trench Location Plan, included as Plate 2 in Appendix A of this report. The Trench Logs, which illustrate the conditions encountered at the trench locations, as well as the results of some of the laboratory testing, are included in Appendix B.

4.2 Geotechnical Conditions

Pavements

Asphaltic concrete pavements were encountered at the ground surface at all four of the trench locations. Trench No. T-1 encountered 4± inches of asphaltic concrete with no discernable layer of aggregate base. The pavement sections at Trench Nos. T-2 through T-4, inclusive, consist of 1½ to 3± inches of asphaltic concrete, underlain by 3 to 7± inches of aggregate base.

Artificial Fill

Artificial fill soils were encountered beneath the pavements at all of the trench locations, extending to depths of 3 to 7½± feet below the existing site grades. The artificial fill soils generally consist of medium dense fine sands, silty fine sands, and fine sands intermixed with silty fine sands. These soils contain trace amounts of medium to coarse sand, fine to coarse gravel, and occasional cobbles. At Trench Nos. T-1 and T-4 the fill soils contain occasional to some of clay nodules. The fill soils possess a disturbed appearance and occasional artificial debris content, such as glass fragments, resulting in their classification as artificial fill.

Alluvium

Native alluvium was encountered beneath the artificial fill soils at all of the trench locations, extending to at least the maximum depth explored of 9± feet below existing site grades. The alluvial soils generally consist of dense gravelly well-graded sands, with some cobbles and occasional boulders.

Groundwater

Groundwater was not encountered at any of the trenches. Based on the lack of any water within the trenches, and the moisture contents of the recovered soil samples, the static groundwater table is considered to have existed at a depth in excess of 9± feet below existing site grades, at the time of the subsurface investigation.

As part of our research, we reviewed available groundwater data in order to determine the historic high groundwater level for the site. The primary reference used to determine the historic groundwater depths in this area is CGS Open File Report 98-13, the Seismic Hazard Evaluation of the Baldwin Park Quadrangle which indicates that the historic high groundwater level for the site is greater than 130 feet below the ground surface. More recent water level data for a well located near the subject site was obtained from the California Department of Water Resources website, http://www.water.ca.gov/waterdatalibrary/. The nearest monitoring well in this database is located approximately 300 feet west of the site. Water level readings within this monitoring well indicate a groundwater level of 194± feet below the ground surface in January 2013.

5.0 LABORATORY TESTING

The soil samples recovered from the subsurface exploration were returned to our laboratory for further testing to determine selected physical and engineering properties of the soils. The tests are briefly discussed below. It should be noted that the test results are specific to the actual samples tested, and variations could be expected at other locations and depths.

Classification

All recovered soil samples were classified using the Unified Soil Classification System (USCS), in accordance with ASTM D-2488. Field identifications were then supplemented with additional visual classifications and/or by laboratory testing. The USCS classifications are shown on the Trench Logs and are periodically referenced throughout this report.

Moisture Content

The moisture content has been determined for selected representative samples. The moisture contents are determined in accordance with ASTM D-2216, and are expressed as a percentage of the dry weight. These test results are presented on the Trench Logs.

Maximum Dry Density and Optimum Moisture Content

One representative bulk sample was tested for its maximum dry density and optimum moisture content. The results have been obtained using the Modified Proctor procedure, per ASTM D-1557 and are presented on Plate C-1 in Appendix C of this report. These tests are generally used to compare the in-situ densities of undisturbed field samples, and for later compaction testing. Additional testing of other soil types or soil mixes may be necessary at a later date.

Soluble Sulfates

One representative samples of the near-surface soils was submitted to a subcontracted analytical laboratory for determination of soluble sulfate content. Sulfates are naturally present in soils, and if the concentration is high enough, can result in degradation of concrete which comes into contact with these soils. The results of the soluble sulfate testing are presented below and are discussed further in a subsequent section of this report.

Sample Identification	**Soluble Sulfates (%)**	**Severity**
T-3 @ 0 to 5 feet	<0.001	Not Applicable (S0)

Corrosivity Testing

A representative bulk sample of the near-surface soils was submitted to a subcontracted analytical laboratory for determination of electrical resistivity, pH, and chloride concentrations. The resistivity of the soils is a measure of their potential to attack buried metal improvements such as utility lines. The results of the resistivity and pH testing are presented below:

Sample Identification	**Resistivity** (ohm-cm)	**pH**	**Chlorides** (mg/kg)
T-3 @ 0 to 5 feet	14,800	8.0	0.6

6.0 CONCLUSIONS AND RECOMMENDATIONS

Based on the results of our review, field exploration, laboratory testing and geotechnical analysis, the proposed development is considered feasible from a geotechnical standpoint. The recommendations contained in this report should be taken into the design, construction, and grading considerations.

The recommendations are contingent upon all grading and foundation construction activities being monitored by the geotechnical engineer of record. The recommendations are provided with the assumption that an adequate program of client consultation, construction monitoring, and testing will be performed during the final design and construction phases to verify compliance with these recommendations. Maintaining Southern California Geotechnical, Inc., (SCG) as the geotechnical consultant from the beginning to the end of the project will provide continuity of services. The geotechnical engineering firm providing testing and observation services shall assume the responsibility of Geotechnical Engineer of Record.

The Grading Guide Specifications, included as Appendix D, should be considered part of this report, and should be incorporated into the project specifications. The contractor and/or owner of the development should bring to the attention of the geotechnical engineer any conditions that differ from those stated in this report, or which may be detrimental for the development.

6.1 Seismic Design Considerations

The subject site is located in an area which is subject to strong ground motions due to earthquakes. The performance of a site-specific seismic hazards analysis was beyond the scope of this investigation. However, numerous faults capable of producing significant ground motions are located near the subject site. Due to economic considerations, it is not generally considered reasonable to design a structure that is not susceptible to earthquake damage. Therefore, significant damage to structures may be unavoidable during large earthquakes. The proposed structures should, however, be designed to resist structural collapse and thereby provide reasonable protection from serious injury, catastrophic property damage and loss of life.

Faulting and Seismicity

Research of available maps indicates that the subject site is not located within an Alquist-Priolo Earthquake Fault Zone. Furthermore, SCG did not identify any evidence of faulting during the geotechnical investigation. Therefore, the possibility of significant fault rupture on the site is considered to be low.

Seismic Design Parameters

The 2019 California Building Code (CBC) provides procedures for earthquake resistant structural design that include considerations for on-site soil conditions, occupancy, and the configuration of the structure including the structural system and height. The seismic design parameters presented below are based on the soil profile and the proximity of known faults with respect to the subject site.

Based on standards in place at the time of this report, the proposed development is expected to be designed in accordance with the requirements of the 2019 edition of the California Building Code (CBC), which was adopted on January 1, 2020.

The 2019 CBC Seismic Design Parameters have been generated using the SEAOC/OSHPD Seismic Design Maps Tool, a web-based software application available at the website www.seismicmaps.org. This software application calculates seismic design parameters in accordance with several building code reference documents, including ASCE 7-16, upon which the 2019 CBC is based. The application utilizes a database of risk-targeted maximum considered earthquake (MCE_R) site accelerations at 0.01-degree intervals for each of the code documents. The tables below were created using data obtained from the application. The output generated from this program is included as Plate E-1 in Appendix E of this report.

The 2019 CBC requires that a site-specific ground motion study be performed in accordance with Section 11.4.8 of ASCE 7-16 for Site Class D sites with a mapped S_1 value greater than 0.2. However, Section 11.4.8 of ASCE 7-16 also indicates an exception to the requirement for a site-specific ground motion hazard analysis for certain structures on Site Class D sites. The commentary for Section 11 of ASCE 7-16 (Page 534 of Section C11 of ASCE 7-16) indicates that "In general, this exception effectively limits the requirements for site-specific hazard analysis to very tall and or flexible structures at Site Class D sites." **Based on our understanding of the proposed development, the seismic design parameters presented below were calculated assuming that the exception in Section 11.4.8 applies to the proposed structure at this site. However, the structural engineer should verify that this exception is applicable to the proposed structure.** Based on the exception, the spectral response accelerations presented below were calculated using the site coefficients (F_a and F_v) from Tables 1613.2.3(1) and 1613.2.3(2) presented in Section 16.4.4 of the 2019 CBC.

2019 CBC SEISMIC DESIGN PARAMETERS

Parameter		Value
Mapped Spectral Acceleration at 0.2 sec Period	S_S	1.659
Mapped Spectral Acceleration at 1.0 sec Period	S_1	0.615
Site Class	---	D
Site Modified Spectral Acceleration at 0.2 sec Period	S_{MS}	1.659
Site Modified Spectral Acceleration at 1.0 sec Period	S_{M1}	1.046
Design Spectral Acceleration at 0.2 sec Period	S_{DS}	1.106
Design Spectral Acceleration at 1.0 sec Period	S_{D1}	0.697

It should be noted that the site coefficient F_v and the parameters S_{M1} and S_{D1} were not included in the SEAOC/OSHPD Seismic Design Maps Tool output for the 2019 CBC. We calculated these parameters-based on Table 1613.2.3(2) in Section 16.4.4 of the 2019 CBC using the value of S_1

obtained from the Seismic Design Maps Tool, assuming that a site-specific ground motion hazards analysis is not required for the proposed buildings at this site.

Liquefaction

Liquefaction is the loss of strength in generally cohesionless, saturated soils when the pore-water pressure induced in the soil by a seismic event becomes equal to or exceeds the overburden pressure. The primary factors which influence the potential for liquefaction include groundwater table elevation, soil type and plasticity characteristics, relative density of the soil, initial confining pressure, and intensity and duration of ground shaking. The depth within which the occurrence of liquefaction may impact surface improvements is generally identified as the upper 50 feet below the existing ground surface. Liquefaction potential is greater in saturated, loose, poorly graded fine sands with a mean (d_{50}) grain size in the range of 0.075 to 0.2 mm (Seed and Idriss, 1971). Non-sensitive clayey (cohesive) soils which possess a plasticity index of at least 18 (Bray and Sancio, 2006) are generally not considered to be susceptible to liquefaction, nor are those soils which are above the historic static groundwater table.

The Seismic Hazards Map for the Baldwin Park, California 7.5 Minute Quadrangle, published by the California Geological Survey (CGS) indicates that the subject site is not located within a designated liquefaction hazard zone. In addition, the subsurface conditions encountered at the site are not considered to be conducive to liquefaction. Based on the mapping performed by CGS and the conditions encountered at the trench locations, liquefaction is not considered to be a significant design concern for this project.

6.2 Geotechnical Design Considerations

General

Artificial fill soils were encountered beneath the pavements at all of the trench locations, extending to depths of 3 to 7½± feet below the existing site grades. No documentation regarding the placement or compaction of these fill soils is known to exist. Based on these characteristics, the existing fill materials are considered to represent undocumented fill. The fill soils are generally underlain by dense well-graded sands and gravelly sands. The soils encountered at the trench locations generally possess significant over-sized material including extensive cobble content and occasional boulders throughout the depths explored. Some remedial grading is considered warranted within the proposed building area to provide more uniform support characteristics beneath the proposed slab and foundations, and to help facilitate construction activities by removing some of the over-sized materials.

Demolition of the existing pavements and structures is also expected to cause significant disturbance to the near surface soils. Any soils disturbed during demolition should also be removed prior to the placement of structural fill soils. The excavated soils may be moisture conditioned and recompacted as structural fill.

Most of the near-surface soils encountered at the trench locations possess occasional to extensive cobble content and occasional boulders.. Recommendations for the handling and placement of oversized materials are presented in Section 6.3 of this report.

Los Angeles County Section 111 Statement

Based on the results of our geotechnical analysis, the proposed development will be safe with regard to landslides, settlement and/or slippage. In addition, the proposed development will not adversely affect the geologic stability of the adjacent properties. This finding is in accordance with Section 111 of the Los Angeles County Building Code.

Settlement

The recommended remedial grading will remove the existing undocumented fill soils and a portion of the near-surface native alluvium soils and replace these materials as compacted structural fill. The native alluvium soils that will remain in place below the recommended depth of overexcavation will not be subject to significant stress increases from the foundations of the new structures. Therefore, following completion of the recommended grading, post-construction settlements are expected to be within tolerable limits.

Corrosion Potential

The results of the electrical resistivity and pH testing indicate that a sample of the on-site soils possesses a resistivity value of 14,800 ohm-cm, and pH values ranging from 8.0. These test results have been evaluated in accordance with guidelines published by the Ductile Iron Pipe Research Association (DIPRA). The DIPRA guidelines consist of a point system by which characteristics of the soils are used to quantify the corrosivity characteristics of the site. Resistivity and pH are two of the five factors that enter into the evaluation procedure. Redox potential, relative soil moisture content and sulfides are also included. Although sulfide testing was not part of the scope of services for this project, we have evaluated the corrosivity characteristics of the on-site soils using resistivity, pH and moisture content. Based on these factors, and utilizing the DIPRA procedure, the on-site soils are not considered to be corrosive to ductile iron pipe. Therefore, polyethylene protection is expected to be required for cast iron or ductile iron pipes. It should be noted that SCG does not practice in the field of corrosion engineering, and therefore, the client may also wish to contact a corrosion engineer to provide a more thorough evaluation.

Based on American Concrete Institute (ACI) Publication 318 Building Code Requirements for Structural Concrete and Commentary, reinforced concrete that is exposed to external sources of chlorides requires corrosion protection for the steel reinforcement contained within the concrete. ACI 318 defines concrete exposed to moisture and an external source of chlorides as "severe" or exposure category C2. ACI 318 does not clearly define a specific chloride concentration at which contact with the adjacent soil will constitute a "C2" or severe exposure. However, the Caltrans Memo to Designers 10-5, Protection of Reinforcement Against Corrosion Due to Chlorides, Acids and Sulfates, dated June 2010, indicates that soils possessing chloride concentrations greater than 500 mg/kg are considered to be corrosive to reinforced concrete. The results of the laboratory testing indicate chloride concentrations of less than 1 ppm. Although the soils contain trace chloride content, we do not expect that the chloride concentrations of the tested soils are high enough to constitute a "severe" or C2 chloride exposure, based on the Caltrans document referenced above. Therefore, a chloride exposure category of C1 is considered appropriate for this site. Since SCG does not practice in the area of corrosion engineering, the client may also wish to contact a corrosion engineer to provide a more thorough evaluation.

Expansion

The near-surface soils generally consist of silty sands and gravelly sands with only trace amounts of clay nodules present in the fill soils. These materials have been visually classified as very low to non-expansive. Therefore, no design considerations related to expansive soils are considered warranted for this site.

Soluble Sulfates

The results of the soluble sulfate testing indicate that the selected samples of the on-site soils to correspond to Class S0 with respect to the American Concrete Institute (ACI) Publication 318-14 Building Code Requirements for Structural Concrete and Commentary, Section 4.3. Therefore, specialized concrete mix designs are not considered to be necessary, with regard to sulfate protection purposes. It is, however, recommended that additional soluble sulfate testing be conducted at the completion of rough grading to verify the soluble sulfate concentrations of the soils which are present at pad grade within the building area.

Shrinkage/Subsidence

Removal and recompaction of the near surface fill soils is estimated to result in an average shrinkage of 4 to 8 percent. Recompaction of the native alluvium is expected to result in an average shrinkage of 0 to 5 percent. It should be noted that the potential shrinkage estimate is based on our experience with similar projects at nearby sites. It was not practical to obtain undisturbed samples based on the gravel, cobble, and boulder content of the onsite soils. Therefore, the actual amount of shrinkage could vary considerable from these estimates. If a more accurate and precise shrinkage estimate is desired, SCG can perform a shrinkage study involving several excavated test-pits where in-place densities are determined using in-situ testing methods. Please contact SCG for details and a cost estimate regarding a shrinkage study, if desired.

Minor ground subsidence is expected to occur in the soils below the zone of removal, due to settlement and machinery working. The subsidence is estimated to be 0.1± feet. This estimate may be used for grading in areas that are underlain by native alluvial soils.

These estimates are based on previous experience and the subsurface conditions encountered at the trench locations. The actual amount of subsidence is expected to be variable and will be dependent on the type of machinery used, repetitions of use, and dynamic effects, all of which are difficult to assess precisely.

Grading and Foundation Plan Review

No grading or foundation plans were available at the time of this report. It is therefore recommended that we be provided with copies of the preliminary plans, when they become available, for review with regard to the conclusions, recommendations, and assumptions contained within this report.

6.3 Site Grading Recommendations

The grading recommendations presented below are based on the subsurface conditions encountered at the trench locations and our understanding of the proposed development. We recommend that all grading activities be completed in accordance with the Grading Guide Specifications included as Appendix D of this report, unless superseded by site-specific recommendations presented below.

Site Stripping and Demolition

Demolition of the existing structures and pavements will be necessary in order to facilitate the construction of the proposed development. Demolition should include all foundations, floor slabs, utilities and any other subsurface improvements that will not remain in place with the new development. Debris resultant from demolition should be disposed of offsite. Alternatively, concrete and asphalt debris may be pulverized to a maximum 2-inch particle size, well mixed with the on-site soils, and incorporated into new structural fills or it may be crushed and made into CMB, if desired.

Initial site stripping should include removal of any surficial vegetation. Based on conditions encountered at the time of the subsurface exploration, stripping of some trees will be necessary along the perimeter of the site. Site stripping should remove any root masses in their entirety. The actual extent of site stripping should be determined in the field by the geotechnical engineer, based on the organic content and stability of the materials encountered.

Treatment of Existing Soils: Building Pad

Remedial grading should be performed within the proposed building pad area in order to remove the existing undocumented fill soils, a portion of the near-surface alluvium, and all soils disturbed during demolition. Based on conditions encountered at the trench locations, the existing soils within the proposed building area are recommended to be overexcavated to a depth of at least 3 feet below existing grade and to a depth of at least 3 feet below proposed building pad subgrade elevations, whichever is greater. However, overexcavation to greater depths will be required to remove the undocumented fill soils, which extend to depths of 3 to 7½± at the trench locations. Additional overexcavation should also be performed within the influence zones of the new foundations, to provide for a new layer of compacted structural fill extending to a depth of at least 2 foot below proposed bearing grade.

The overexcavation areas should extend at least 5 feet beyond the building perimeter, and to an extent equal to the depth of fill below the new foundations. If the proposed structure will incorporate any exterior columns (such as for a canopy or overhang) the area of overexcavation should also encompass these areas.

Following completion of the overexcavation, the subgrade soils within the building area should be evaluated by the geotechnical engineer to verify their suitability to serve as the structural fill subgrade, as well as to support the foundation loads of the new structure. This evaluation should include proofrolling and probing to identify any soft, loose, or otherwise unstable soils that must be removed. Some localized areas of deeper excavation may be required if loose, porous, or low density native soils are encountered at the base of the overexcavation.

After a suitable overexcavation subgrade has been achieved, the exposed soils should be scarified to a depth of at least 12 inches, and thoroughly flooded to raise the moisture content of the underlying soils to at least 0 to 4 percent above optimum moisture content, extending to a depth of at least 24 inches. The moisture conditioning of the overexcavation subgrade soils should be verified by the geotechnical engineer. The subgrade soils should then be recompacted to at least 90 percent of the ASTM D-1557 maximum dry density. The previously excavated soils may then be replaced as compacted structural fill.

<u>Treatment of Existing Soils: Parking Areas</u>

Based on economic considerations, overexcavation of the existing soils in the new parking and drive areas is not considered warranted, with the exception of areas where lower strength, or unstable soils are identified by the geotechnical engineer during grading. Subgrade preparation in the new parking and drive areas should initially consist of removal of all soils disturbed during stripping and demolition operations.

The geotechnical engineer should then evaluate the subgrade to identify any areas of additional unsuitable soils. Any such materials should be removed to a level of firm and unyielding soil. The exposed subgrade soils should then be scarified to a depth of 12± inches, moisture conditioned to 0 to 4 percent above optimum, and recompacted to at least 90 percent of the ASTM D-1557 maximum dry density. Based on the presence of variable strength surficial soils throughout the site, it is expected that some isolated areas of additional overexcavation may be required to remove zones of lower strength, unsuitable soils.

The grading recommendations presented above for the proposed parking area assume that the owner and/or developer can tolerate minor amounts of settlement within the proposed parking areas. The grading recommendations presented above do not completely mitigate the extent of the existing fill soils in the parking areas. As such, settlement and associated pavement distress could occur. Typically, repair of such distressed areas involves significantly lower costs than completely mitigating these soils at the time of construction. If the owner cannot tolerate the risk of such settlements, the parking and drive areas should be overexcavated to a depth of 2 feet below proposed pavement subgrade elevation, with the removed soils replaced as compacted structural fill.

<u>Treatment of Existing Soils: Retaining Walls and Site Walls</u>

The existing soils within the areas of any proposed retaining and site walls should be overexcavated to a depth of 2 feet below foundation bearing grade and replaced as compacted structural fill as discussed above for the proposed building pad. Any undocumented fill soils within any of these foundation areas should be removed in their entirety. Erection pads for concrete tilt-up walls are considered part of the foundation system, and the recommended overexcavation should also be performed beneath erection pads. The overexcavation subgrade soils should be evaluated by the geotechnical engineer prior to scarifying, moisture conditioning, and recompacting the upper 12 inches of exposed subgrade soils, as discussed for the building area. The previously excavated soils may then be replaced as compacted structural fill.

Fill Placement

- Fill soils should be placed in thin (6± inches), near-horizontal lifts, moisture conditioned to 0 to 4 percent above the optimum moisture content, and compacted.
- On-site soils may be used for fill provided they are cleaned of any debris to the satisfaction of the geotechnical engineer.
- All grading and fill placement activities should be completed in accordance with the requirements of the 2019 CBC and the grading code of the city of Irwindale and county of Los Angeles.
- All fill soils should be compacted to at least 90 percent of the ASTM D-1557 maximum dry density. Fill soils should be well mixed.
- Compaction tests should be performed periodically by the geotechnical engineer as random verification of compaction and moisture content. These tests are intended to aid the contractor. Since the tests are taken at discrete locations and depths, they may not be indicative of the entire fill and therefore should not relieve the contractor of his responsibility to meet the job specifications.

Selective Grading and Oversized Material Placement

The native alluvial soils possess significant cobble and boulder content. It is expected that large scrapers (Caterpillar 657 or equivalent) will be adequate to move the cobble containing soils as well the soils containing smaller boulders. It may be necessary to move larger boulders individually, and place them as oversized materials in accordance with the Grading Guide Specifications, in Appendix D of this report.

Since the proposed grading will require excavation of cobble and boulder containing soils, it may be desirable to selectively grade the proposed building pad area. The presence of particles greater than 3 inches in diameter within the upper 1 to 3 feet of the building pad subgrade will impact the utility and foundation excavations. Depending on the depths of fills required within the proposed parking areas, it may be feasible to sort the on-site soils, placing the materials greater than 3 inches in diameter within the lower depths of the fills, and limiting the upper 1 to 3 feet of soils to materials less than 3 inches in size. Oversized materials could also be placed within the lower depths of the recommended overexcavations. In order to achieve this grading, it would likely be necessary to use rock buckets and/or rock sieves to separate the oversized materials from the remaining soil. Although such selective grading will facilitate further construction activities, it is not considered mandatory and a suitable subgrade could be achieved without such extensive sorting. However, in any case, it is recommended that all materials greater than 6 inches in size be excluded from the upper 1 foot of the surface of any compacted fills.

The placement of any oversized materials should be performed in accordance with the Grading Guide Specifications included in Appendix D of this report. If disposal of oversized materials is required, rock blankets or windrows should be used and such areas should be observed during construction and placement by a representative of the geotechnical engineer.

Imported Structural Fill

All imported structural fill should consist of very low expansive (EI < 20), well graded soils possessing at least 10 percent fines (that portion of the sample passing the No. 200 sieve). Additional specifications for structural fill are presented in the Grading Guide Specifications, included as Appendix D.

Utility Trench Backfill

In general, all utility trench backfill should be compacted to at least 90 percent of the ASTM D-1557 maximum dry density. It is recommended that materials in excess of 6 inches in size not be used for utility trench backfill. Compacted trench backfill should conform to the requirements of the local grading code, and more restrictive requirements may be indicated by the city of Irwindale and the county of Los Angeles. All utility trench backfills should be witnessed by the geotechnical engineer. The trench backfill soils should be compaction tested where possible; probed and visually evaluated elsewhere.

Utility trenches which parallel a footing, and extending below a 1h:1v plane projected from the outside edge of the footing should be backfilled with structural fill soils, compacted to at least 90 percent of the ASTM D-1557 standard. Pea gravel backfill should not be used for these trenches.

6.4 Construction Considerations

Excavation Considerations

The near-surface soils at this site generally consist of well-graded silty sands and gravely sands. These materials may be subject to caving within shallow excavations. Where caving occurs within shallow excavations, flattened excavation slopes may be sufficient to provide excavation stability. Deeper excavations may require some form of external stabilization such as shoring or bracing. Maintaining adequate moisture content within the near-surface soils will improve excavation stability. Temporary excavation slopes should be no steeper than 2h:1v. All excavation activities on this site should be conducted in accordance with Cal-OSHA regulations.

Groundwater

Based on our research, the historic high ground water level is considered to be greater than 130± feet below the ground surface. Therefore, groundwater is not expected to impact grading or foundation construction activities.

6.5 Foundation Design and Construction

Based on the preceding grading recommendations, it is assumed that the new building pad will be underlain by new structural fill soils used to replace existing undocumented fill and a portion of the near-surface aluvium. These structural fill soils are expected to extend to depths of at least 2 feet below proposed foundation bearing grade, underlain by 1± foot of additional soil that has been densified and moisture conditioned in place. Based on this subsurface profile, the proposed structure may be supported on conventional shallow foundations.

Foundation Design Parameters

New square and rectangular footings may be designed as follows:

- Maximum, net allowable soil bearing pressure: 3,000 lbs/ft^2.

- Minimum wall/column footing width: 14 inches/24 inches.

- Minimum longitudinal steel reinforcement within strip footings: Two (2) No. 5 rebars (1 top and 1 bottom).

- Minimum foundation embedment: 12 inches into suitable structural fill soils, and at least 18 inches below adjacent exterior grade. Interior column footings may be placed immediately beneath the floor slab.

- It is recommended that the perimeter building foundations be continuous across all exterior doorways. Any flatwork adjacent to the exterior doors should be doweled into the perimeter foundations in a manner determined by the structural engineer.

The allowable bearing pressure presented above may be increased by one-third when considering short duration wind or seismic loads. The minimum steel reinforcement recommended above is based on geotechnical considerations; additional reinforcement may be necessary for structural considerations. The actual design of the foundations should be determined by the structural engineer.

Foundation Construction

The foundation subgrade soils should be evaluated at the time of overexcavation, as discussed in Section 6.3 of this report. It is further recommended that the foundation subgrade soils be evaluated by the geotechnical engineer immediately prior to steel or concrete placement. Soils suitable for direct foundation support should consist of newly placed structural fill, compacted to at least 90 percent of the ASTM D-1557 maximum dry density. Any unsuitable materials should be removed to a depth of suitable bearing compacted structural fill, with the resulting excavations backfilled with compacted fill soils. As an alternative, lean concrete slurry (500 to 1,500 psi) may be used to backfill such isolated overexcavations.

The foundation subgrade soils should also be properly moisture conditioned to 0 to 4 percent above the Modified Proctor optimum, to a depth of at least 12 inches below bearing grade. Since it is typically not feasible to increase the moisture content of the floor slab and foundation subgrade soils once rough grading has been completed, care should be taken to maintain the moisture content of the building pad subgrade soils throughout the construction process.

Estimated Foundation Settlements

Post-construction total and differential settlements of shallow foundations designed and constructed in accordance with the previously presented recommendations are estimated to be less than 1.0 and 0.5 inches, respectively. Differential movements are expected to occur over a 30-foot span, thereby resulting in an angular distortion of less than 0.002 inches per inch.

Lateral Load Resistance

Lateral load resistance will be developed by a combination of friction acting at the base of foundations and slabs and the passive earth pressure developed by footings below grade. The following friction and passive pressure may be used to resist lateral forces:

- Passive Earth Pressure: 300 lbs/ft^3
- Friction Coefficient: 0.32

These are allowable values, and include a factor of safety. When combining friction and passive resistance, the passive pressure component should be reduced by one-third. These values assume that footings will be poured directly against compacted structural fill. The maximum allowable passive pressure is 3,000 lbs/ft^2.

6.6 Floor Slab Design and Construction

Subgrades which will support new floor slabs should be prepared in accordance with the recommendations contained in the ***Site Grading Recommendations*** section of this report. Based on the anticipated grading which will occur at this site, the floor of the new building may be constructed as a conventional slab-on-grade supported on newly placed structural fill soils, extending to a depth of at least 3 feet below the proposed pad grade. Based on geotechnical considerations, the floor slabs may be designed as follows:

- Minimum slab thickness: 6 inches.

- Modulus of Subgrade Reaction: k = 200 psi/in

- Minimum slab reinforcement: Reinforcement is not expected to be required for geotechnical conditions. The actual floor slab reinforcement should be determined by the structural engineer, based upon the imposed loading.

- Slab underlayment: If moisture sensitive floor coverings will be used the minimum slab underlayment should consist of a moisture vapor barrier constructed below the entire area of the slab that such moisture sensitive floor coverings are anticipated. The moisture vapor barrier should meet or exceed the Class A rating as defined by ASTM E 1745-97 and have a permeance rating less than 0.01 perms as described in ASTM E 96-95 and ASTM E 154-88. A polyolefin material such as Stego® Wrap Vapor Barrier or equivalent will meet these specifications. The moisture vapor barrier should be properly constructed in accordance with all applicable manufacturer specifications. Given that a rock free subgrade is anticipated and that a capillary break is not required, sand below the barrier is not required. The need for sand and/or the amount of sand above the moisture vapor barrier should be specified by the structural engineer or concrete contractor. The selection of sand above the barrier is not a geotechnical engineering issue and hence outside our purview.

- Moisture condition the floor slab subgrade soils to 0 to 4 percent above the Modified Proctor optimum moisture content, to a depth of 12 inches. The moisture content of the floor slab subgrade soils should be verified by the geotechnical engineer within 24 hours prior to concrete placement.

- Proper concrete curing techniques should be utilized to reduce the potential for slab curling or the formation of excessive shrinkage cracks.

The actual design of the floor slab should be completed by the structural engineer to verify adequate thickness and reinforcement.

6.7 Retaining Wall Design and Construction

Although not indicated on the site plan, some retaining walls may be required to facilitate the new site grades. The parameters recommended for use in the design of these walls are presented below.

Retaining Wall Design Parameters

Based on the conditions encountered at the trench locations, the following parameters may be used in the design of new retaining walls for this site. We have provided parameters assuming the use of on-site soils for retaining wall backfill. The near surface soils generally consist of silty sands and gravelly sands. Based on their classifications, the near surface soils are expected to possess a friction angle of at least 32 degrees when compacted to 90 percent of the ASTM-1557 maximum dry density.

If desired, SCG could provide design parameters for an alternative select backfill material behind the retaining walls. The use of select backfill material could result in lower lateral earth pressures. In order to use the design parameters for the imported select fill, this material must be placed within the entire active failure wedge. This wedge is defined as extending from the heel of the retaining wall upwards at an angle of approximately 60° from horizontal. If select backfill material behind the retaining wall is desired, SCG should be contacted for supplementary recommendations.

RETAINING WALL DESIGN PARAMETERS

Design Parameter		Soil Type On-Site Soils
Internal Friction Angle (ϕ)		32°
Unit Weight		130 lbs/ft^3
Equivalent Fluid Pressure:	Active Condition (level backfill)	40 lbs/ft^3
	Active Condition (2h:1v backfill)	61 lbs/ft^3
	At-Rest Condition (level backfill)	61 lbs/ft^3

The walls should be designed using a soil-footing coefficient of friction of 0.32 and an equivalent passive pressure of 300 lbs/ft^3. The structural engineer should incorporate appropriate factors of safety in the design of the retaining walls.

The active earth pressure may be used for the design of retaining walls that do not directly support structures or support soils that in turn support structures and which will be allowed to deflect. The at-rest earth pressure should be used for walls that will not be allowed to deflect such as those which will support foundation bearing soils, or which will support foundation loads directly.

Where the soils on the toe side of the retaining wall are not covered by a "hard" surface such as a structure or pavement, the upper 1 foot of soil should be neglected when calculating passive resistance due to the potential for the material to become disturbed or degraded during the life of the structure.

Seismic Lateral Earth Pressures

In accordance with the 2019 CBC, any retaining walls more than 6 feet in height must be designed for seismic lateral earth pressures. If walls 6 feet or more are required for this site, the geotechnical engineer should be contacted for supplementary seismic lateral earth pressure recommendations.

Retaining Wall Foundation Design

The retaining wall foundations should be supported within newly placed structural fill. Foundations to support new retaining walls should be designed in accordance with the general Foundation Design Parameters presented in a previous section of this report.

Backfill Material

On-site soils may be used to backfill the retaining walls. However, all backfill material placed within 3 feet of the back-wall face should have a particle size no greater than 3 inches. The retaining wall backfill materials should be well graded.

It is recommended that a properly installed prefabricated drainage composite such as the MiraDRAIN 6000XL (or approved equivalent), which is specifically designed for use behind retaining walls be used. If the drainage composite material is not covered by an impermeable surface, such as a structure or pavement, a 12-inch thick layer of a low permeability soil should be placed over the backfill to reduce surface water migration to the underlying soils. The drainage composite should be separated from the backfill soils by a suitable geotextile, approved by the geotechnical engineer.

All retaining wall backfill should be placed and compacted under engineering-controlled conditions in the necessary layer thicknesses to ensure an in-place density between 90 and 93 percent of the maximum dry density as determined by the Modified Proctor test (ASTM D1557-91). Care should be taken to avoid over-compaction of the soils behind the retaining walls, and the use of heavy compaction equipment should be avoided.

Subsurface Drainage

As previously indicated, the retaining wall design parameters are based upon drained backfill conditions. Consequently, some form of permanent drainage system will be necessary in conjunction with the appropriate backfill material. Subsurface drainage may consist of either:

- A weep hole drainage system typically consisting of a series of 4-inch diameter holes in the wall situated slightly above the ground surface elevation on the exposed side of the wall and at an approximate 8-foot on-center spacing. The weep holes should include a 2 cubic foot pocket of open graded gravel, surrounded by an approved geotextile fabric, at each weep hole location.

- A 4-inch diameter perforated pipe surrounded by 2 cubic feet of gravel per linear foot of drain placed behind the wall, above the retaining wall footing. The gravel layer should be wrapped in a suitable geotextile fabric to reduce the potential for migration of fines. The footing drain should be extended to daylight or tied into a storm drainage system.

6.8 Pavement Design Parameters

Site preparation in the pavement area should be completed as previously recommended in the ***Site Grading Recommendations*** section of this report. The subsequent pavement recommendations assume proper drainage and construction monitoring, and are based on either PCA or CALTRANS design parameters for a twenty (20) year design period. However, these designs also assume a routine pavement maintenance program to obtain the anticipated 20-year pavement service life.

Pavement Subgrades

It is anticipated that the new pavements will be primarily supported on a layer of compacted structural fill, consisting of scarified, thoroughly moisture conditioned and recompacted existing soils. The on-site soils generally consist of well graded sands and sandy gravels. Based on their classification, these materials are expected to possess good to excellent pavement support characteristics, with R-values in the range of 60 to 70. Since R-value testing was not included in the scope of services for this project, the subsequent pavement design is based upon an assumed R-value of 60. Any fill material imported to the site should have support characteristics equal to or greater than that of the on-site soils and be placed and compacted under engineering controlled conditions. It is recommended that R-value testing be performed after completion of rough grading. Depending upon the results of the R-value testing, it may be feasible to use thinner pavement sections in some areas of the site.

Asphaltic Concrete

Presented below are the recommended thicknesses for new flexible pavement structures consisting of asphaltic concrete over a granular base. The pavement designs are based on the traffic indices (TI's) indicated. The client and/or civil engineer should verify that these TI's are representative of the anticipated traffic volumes. If the client and/or civil engineer determine that the expected traffic volume will exceed the applicable traffic index, we should be contacted for supplementary recommendations. The design traffic indices equate to the following approximate daily traffic volumes over a 20 year design life, assuming six operational traffic days per week.

Traffic Index	No. of Heavy Trucks per Day
4.0	0
5.0	1
6.0	3
7.0	11
8.0	35
9.0	93

For the purpose of the traffic volumes indicated above, a truck is defined as a 5-axle tractor trailer unit with one 8-kip axle and two 32-kip tandem axles. All of the traffic indices allow for 1,000 automobiles per day.

ASPHALT PAVEMENTS (R=60)					
Materials	Thickness (inches)				
	Auto Parking and Auto Drive Lanes (TI = 4.0 to 5.0)	Truck Traffic			
		TI = 6.0	TI = 7.0	TI = 8.0	TI = 9.0
Asphalt Concrete	3	3½	4	5	5½
Aggregate Base	3	3	3	3	4
Compacted Subgrade	12	12	12	12	12

The aggregate base course should be compacted to at least 95 percent of the ASTM D-1557 maximum dry density. The asphaltic concrete should be compacted to at least 95 percent of the Marshall maximum density, as determined by ASTM D-2726. The aggregate base course may consist of crushed aggregate base (CAB) or crushed miscellaneous base (CMB), which is a recycled gravel, asphalt and concrete material. The gradation, R-Value, Sand Equivalent, and

Percentage Wear of the CAB or CMB should comply with appropriate specifications contained in the current edition of the "Greenbook" Standard Specifications for Public Works Construction.

Portland Cement Concrete

The preparation of the subgrade soils within Portland cement concrete pavement areas should be performed as previously described for proposed asphalt pavement areas. The minimum recommended thicknesses for the Portland Cement Concrete pavement sections are as follows:

PORTLAND CEMENT CONCRETE PAVEMENTS				
Materials	**Thickness (inches)**			
	Autos and Light Truck Traffic (TI = 5.0 & 6.0)	Truck Traffic		
		TI = 7.0	TI = 8.0	TI = 9.0
PCC	5	5	6½	8
Compacted Subgrade (95% minimum compaction)	12	12	12	12

The concrete should have a 28-day compressive strength of at least 3,000 psi. Reinforcing within all pavements should be designed by the structural engineer. The maximum joint spacing within all of the PCC pavements is recommended to be equal to or less than 30 times the pavement thickness. The actual joint spacing and reinforcing of the Portland cement concrete pavements should be determined by the structural engineer.

7.0 GENERAL COMMENTS

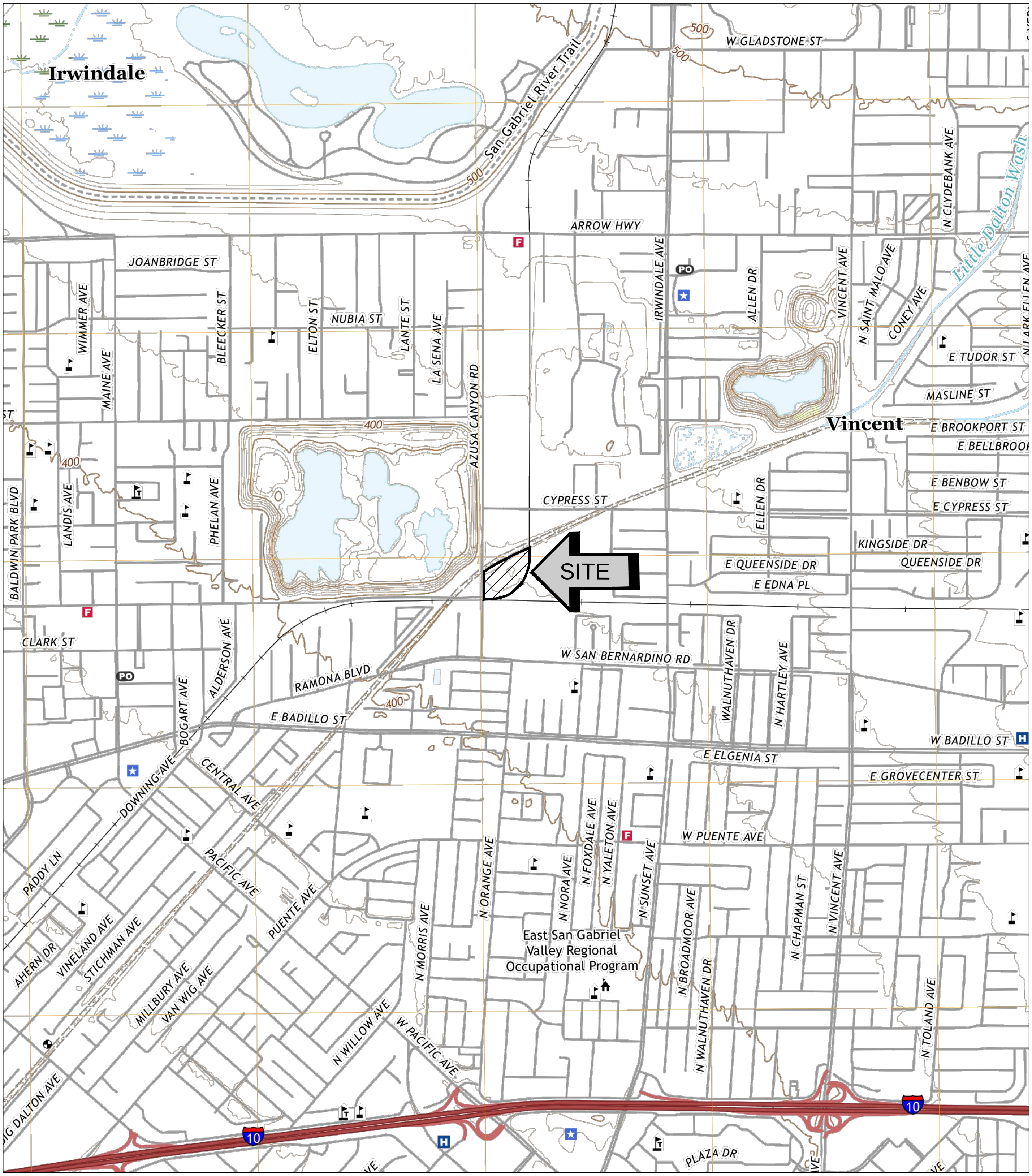
This report has been prepared as an instrument of service for use by the client, in order to aid in the evaluation of this property and to assist the architects and engineers in the design and preparation of the project plans and specifications. This report may be provided to the contractor(s) and other design consultants to disclose information relative to the project. However, this report is not intended to be utilized as a specification in and of itself, without appropriate interpretation by the project architect, civil engineer, and/or structural engineer. The reproduction and distribution of this report must be authorized by the client and Southern California Geotechnical, Inc. Furthermore, any reliance on this report by an unauthorized third party is at such party's sole risk, and we accept no responsibility for damage or loss which may occur. The client(s)' reliance upon this report is subject to the Engineering Services Agreement, incorporated into our proposal for this project.

The analysis of this site was based on a subsurface profile interpolated from limited discrete soil samples. While the materials encountered in the project area are considered to be representative of the total area, some variations should be expected between trench locations and sample depths. If the conditions encountered during construction vary significantly from those detailed herein, we should be contacted immediately to determine if the conditions alter the recommendations contained herein.

This report has been based on assumed or provided characteristics of the proposed development. It is recommended that the owner, client, architect, structural engineer, and civil engineer carefully review these assumptions to ensure that they are consistent with the characteristics of the proposed development. If discrepancies exist, they should be brought to our attention to verify that they do not affect the conclusions and recommendations contained herein. We also recommend that the project plans and specifications be submitted to our office for review to verify that our recommendations have been correctly interpreted.

The analysis, conclusions, and recommendations contained within this report have been promulgated in accordance with generally accepted professional geotechnical engineering practice. No other warranty is implied or expressed.

APPENDIX A



SOURCE: USGS TOPOGRAPHIC MAP OF THE BALDWIN PARK QUADRANGLE, LOS ANGELES COUNTY, CALIFORNIA, 2018.

SITE LOCATION MAP

PROPOSED WAREHOUSE

IRWINDALE, CALIFORNIA

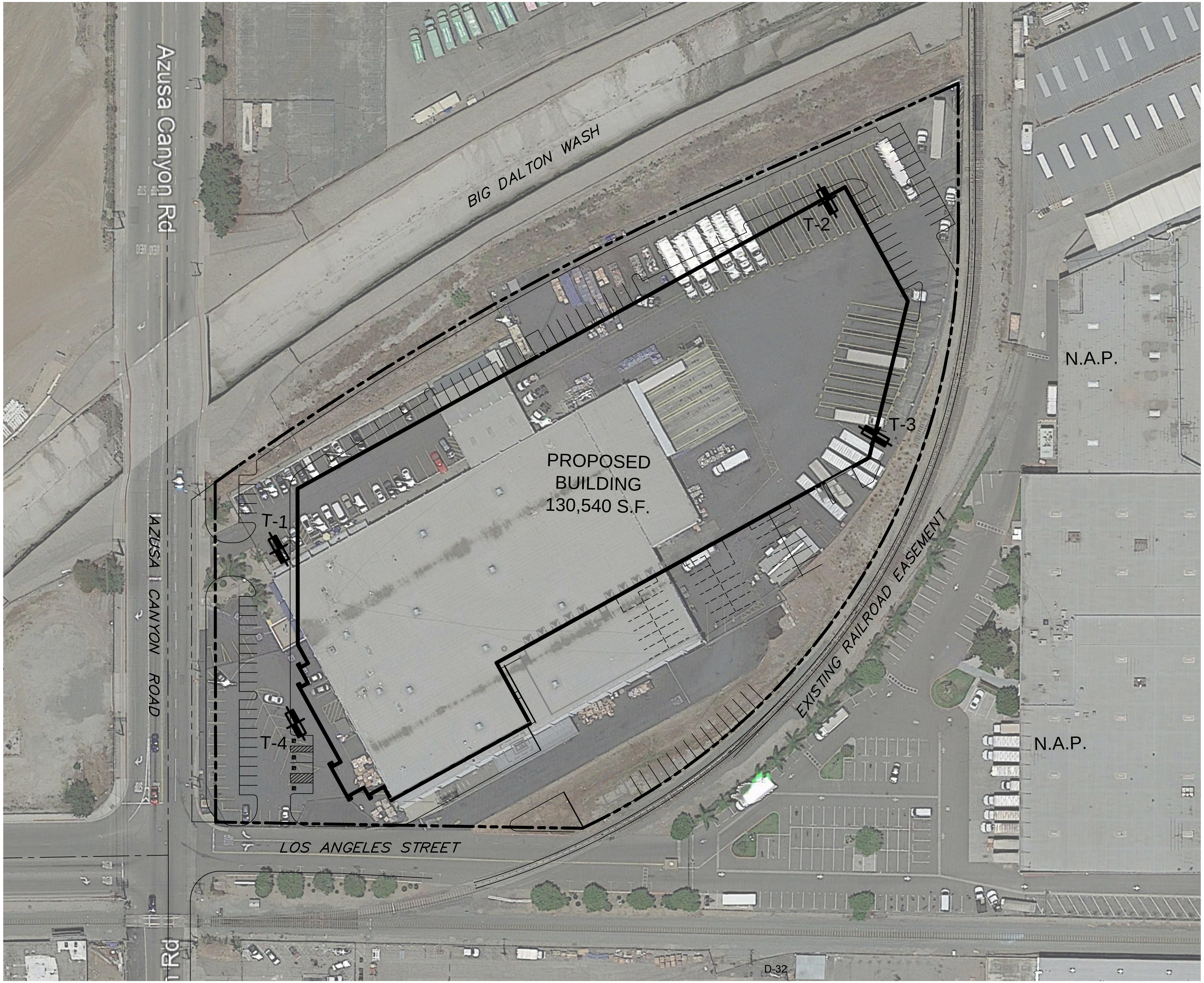
SCALE: 1" = 2000'

DRAWN: JAH
CHKD: RGT

SCG PROJECT
20G105-1

PLATE 1

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL



GEOTECHNICAL LEGEND

APPROXIMATE TRENCH LOCATION

NOTE: CONCEPTUAL SITE PLAN PREPARED BY GAA ARCHITECTS.

TRENCH LOCATION PLAN

PROPOSED WAREHOUSE

IRWINDALE, CALIFORNIA

SCALE: 1" = 80'

DRAWN: JAH
CHKD: RGT

SCG PROJECT
20G105-1

PLATE 2

SOUTHERN CALIFORNIA GEOTECHNICAL

APPENDIX B

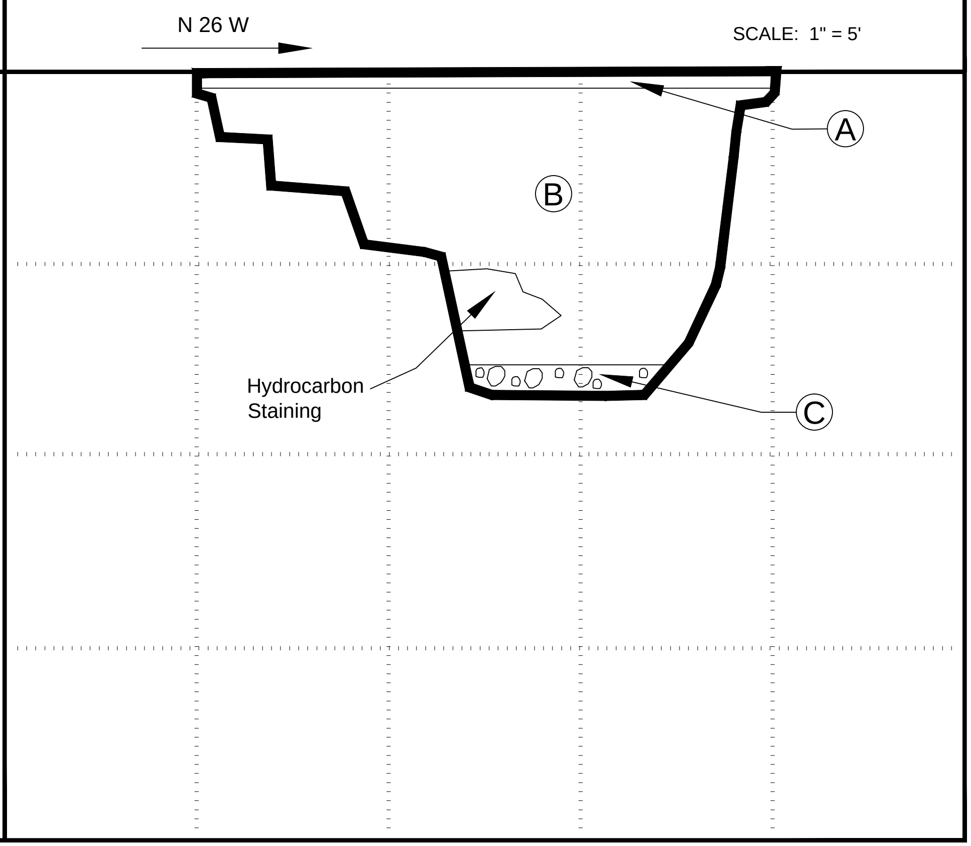
SOUTHERN CALIFORNIA GEOTECHNICAL

TRENCH NO. T-1

JOB NO.: 20G105-1	EQUIPMENT USED: Backhoe	WATER DEPTH: Dry
PROJECT: Prop Warehouse	LOGGED BY: Jamie Hayward	SEEPAGE DEPTH: Dry
LOCATION: Irwindale, CA	ORIENTATION: N 26 W	READINGS TAKEN: At Completion
DATE: 1-31-2020	ELEVATION: ---	

DEPTH	SAMPLE	DRY DENSITY (PCF)	MOISTURE (%)	EARTH MATERIALS DESCRIPTION
	b		7	A: 4 inches Asphaltic Concrete, No Discernable Aggregate Base B: FILL: Gray Brown fine Sand, trace medium to coarse Sand, trace fine Gravel, occasional Clay nodules, medium dense-damp to very moist
5				@ 6 to 8 feet, some hydrocarbon staining
	b		16	C: ALLUVIUM: Gray Brown Gravelly fine to coarse Sand, some Cobbles, occasional Boulders, dense-dry
	b		1	Trench Terminated @ 8.5 feet
10				
15				

GRAPHIC REPRESENTATION



KEY TO SAMPLE TYPES:
B - BULK SAMPLE (DISTURBED)
R - RING SAMPLE 2-1/2" DIAMETER
(RELATIVELY UNDISTURBED)

TRENCH LOG

PLATE B-1

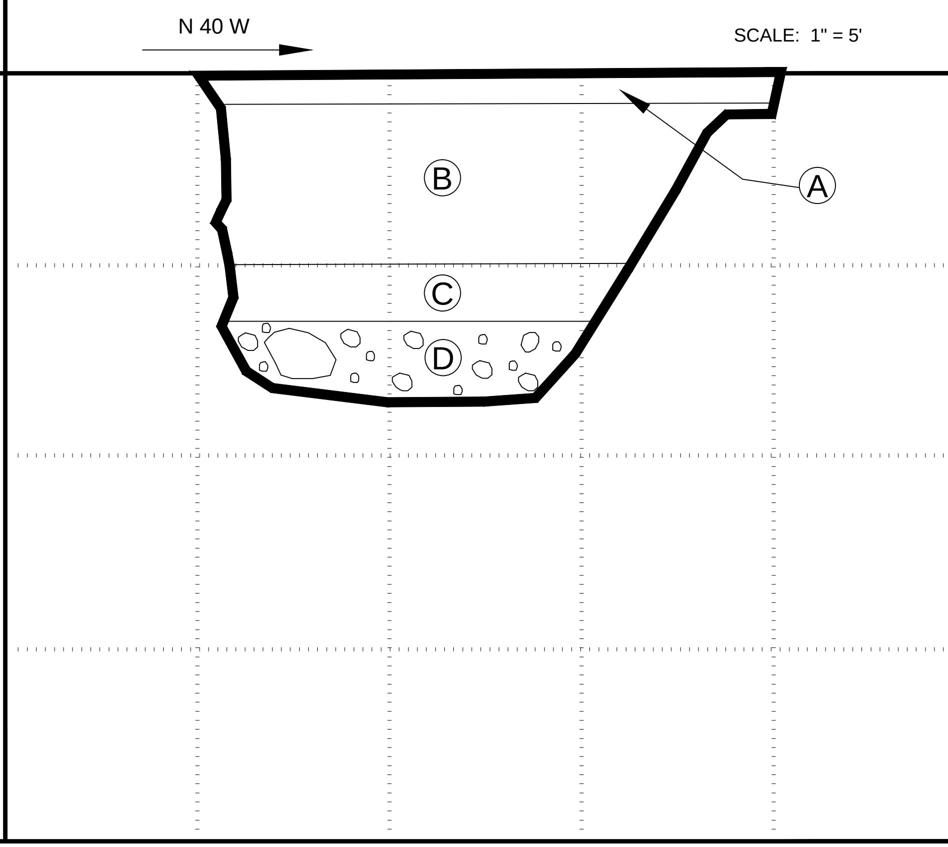
SOUTHERN CALIFORNIA GEOTECHNICAL

TRENCH NO. T-2

JOB NO.: 20G105-1	EQUIPMENT USED: Backhoe	WATER DEPTH: Dry
PROJECT: Prop Warehouse	LOGGED BY: Jamie Hayward	SEEPAGE DEPTH: Dry
LOCATION: Irwindale, CA	ORIENTATION: N 40 W	READINGS TAKEN: At Completion
DATE: 01-31-2020	ELEVATION: ---	

DEPTH	SAMPLE	DRY DENSITY (PCF)	MOISTURE (%)	EARTH MATERIALS DESCRIPTION
				A: 1.5 inches Asphaltic Concrete, 7 inches Aggregate Base
	b		9	B: FILL: Brown Silty fine Sand intermixed with gravelly fine to coarse Sand, trace glass fragments, some Cobbles, medium dense-moist
	b		12	
5				C: FILL: Gray Brown fine Sand, trace medium to coarse Sand, little fine Gravel, loose-moist
	b		9	D: ALLUVIUM: Gray Brown Gravelly fine to coarse Sand, some cobbles, occasional Boulders, dense-damp
	b		2	
	b		3	
				Trench Terminated @ 8.5 feet
10				
15				

GRAPHIC REPRESENTATION



KEY TO SAMPLE TYPES:
B - BULK SAMPLE (DISTURBED)
R - RING SAMPLE 2-1/2" DIAMETER
(RELATIVELY UNDISTURBED)

TRENCH LOG

PLATE B-2

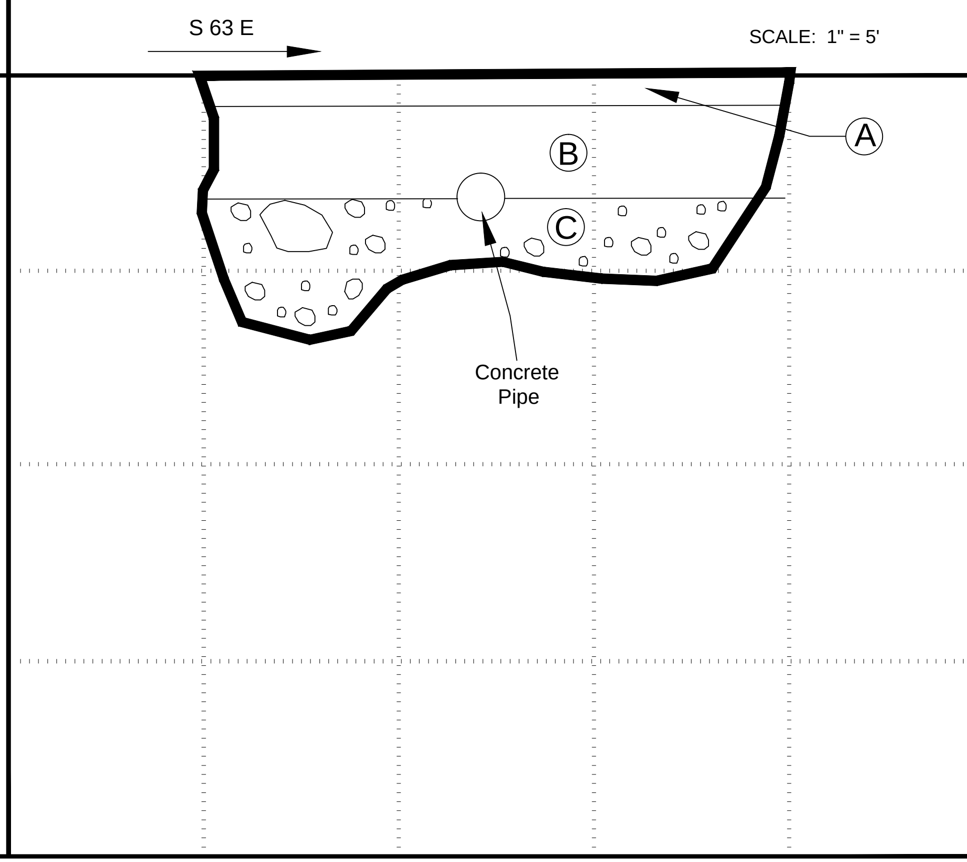
SOUTHERN CALIFORNIA GEOTECHNICAL

TRENCH NO. T-3

JOB NO.: 20G105-1	EQUIPMENT USED: Backhoe	WATER DEPTH: Dry
PROJECT: Prop Warehouse	LOGGED BY: Jamie Hayward	SEEPAGE DEPTH: Dry
LOCATION: Irwindale, CA	ORIENTATION: S 63 E	READINGS TAKEN: At Completion
DATE: 01-28-2020	ELEVATION: ---	

DEPTH	SAMPLE	DRY DENSITY (PCF)	MOISTURE (%)	EARTH MATERIALS DESCRIPTION
				A: 3 inches Asphaltic Concrete, 5 inches Aggregate Base
	b		12	B: FILL: Brown Silty fine Sand, little fine to coarse Gravel, medium dense-moist
	b		3	C: ALLUVIUM: Gray Brown Gravelly fine to coarse Sand, some Cobbles, occasional Boulders, dense-damp
5				
				Trench Terminated @ 6 feet due to damaged pipe
10				
15				

GRAPHIC REPRESENTATION



KEY TO SAMPLE TYPES:
B - BULK SAMPLE (DISTURBED)
R - RING SAMPLE 2-1/2" DIAMETER
(RELATIVELY UNDISTURBED)

TRENCH LOG

PLATE B-3

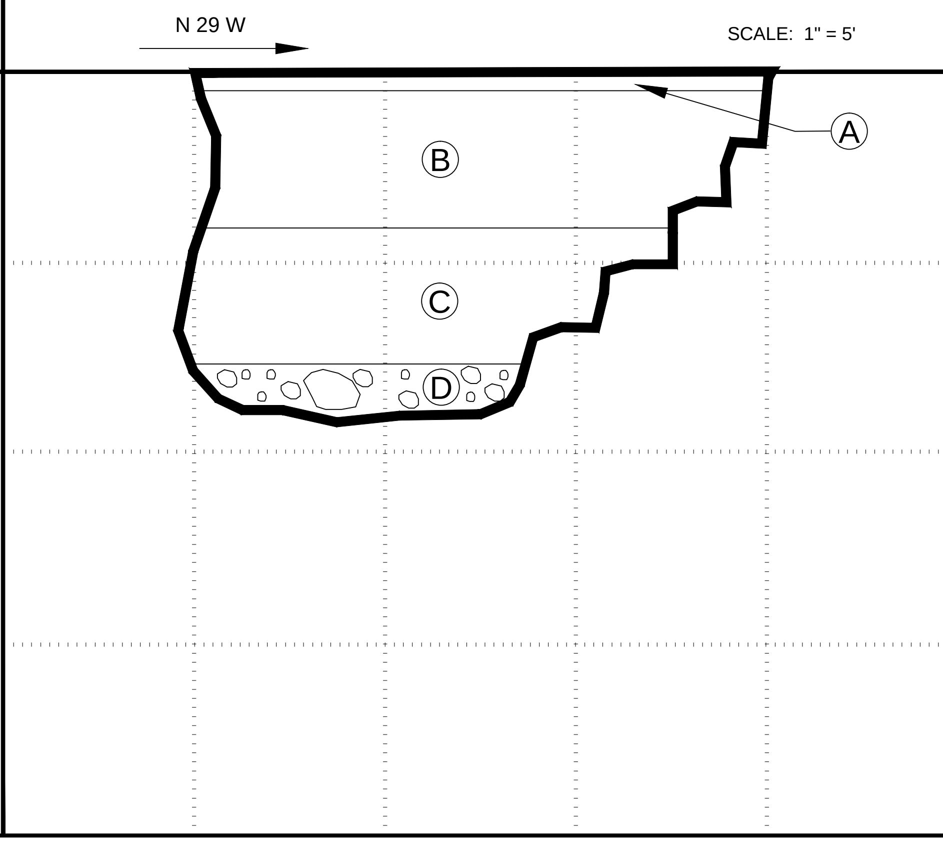
SOUTHERN CALIFORNIA GEOTECHNICAL

TRENCH NO. T-4

JOB NO.: 20G105-1	EQUIPMENT USED: Backhoe	WATER DEPTH: Dry
PROJECT: Prop Warehouse	LOGGED BY: Jamie Hayward	SEEPAGE DEPTH: Dry
LOCATION: Irwindale, CA	ORIENTATION: N 29 W	READINGS TAKEN: At Completion
DATE: 01-31-2020	ELEVATION: ---	

DEPTH	SAMPLE	DRY DENSITY (PCF)	MOISTURE (%)	EARTH MATERIALS DESCRIPTION
				A: 2 inches Asphaltic concrete, 3 inches Aggregate Base
	b		10	B: FILL: Gray Brown fine Sand, little medium to coarse Sand, trace fine Gravel, occasional Clay nodules, trace Silt, medium dense-moist
5	b		11	C: FILL: Brown Silty fine Sand intermixed with fine Sand, trace Clay nodules, little Iron oxide staining, medium dense-moist to very moist
	b		14	
				D: ALLUVIUM: Gray Brown Gravelly fine to coarse Sand, some Cobbles, occasional Boulders, dense-damp
	b		3	
				Trench Terminated @ 9 feet
10				
15				

GRAPHIC REPRESENTATION

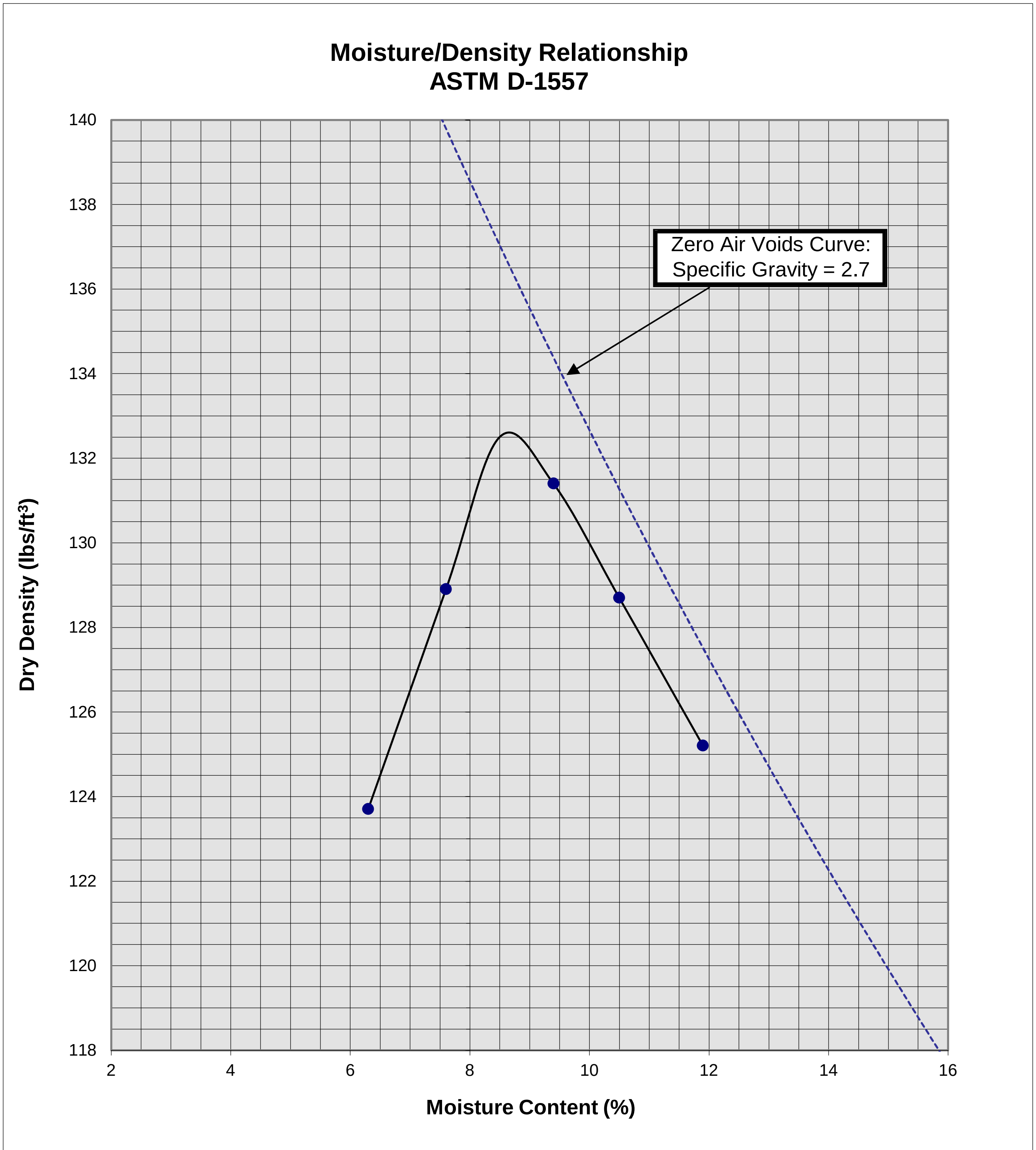


KEY TO SAMPLE TYPES:
B - BULK SAMPLE (DISTURBED)
R - RING SAMPLE 2-1/2" DIAMETER
(RELATIVELY UNDISTURBED)

TRENCH LOG

PLATE B-4

APPENDIX C



Soil ID Number	T-2 @ 0-5'
Optimum Moisture (%)	8.5
Maximum Dry Density (pcf)	132.5
Soil Classification	Brown Silty fine to coarse Sand, little fine to coarse Gravel, some Cobbles

Proposed Warehouse
Irwindale, California
Project No. 20G105

PLATE C-1

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

APPENDIX D

GRADING GUIDE SPECIFICATIONS

These grading guide specifications are intended to provide typical procedures for grading operations. They are intended to supplement the recommendations contained in the geotechnical investigation report for this project. Should the recommendations in the geotechnical investigation report conflict with the grading guide specifications, the more site specific recommendations in the geotechnical investigation report will govern.

General

- The Earthwork Contractor is responsible for the satisfactory completion of all earthwork in accordance with the plans and geotechnical reports, and in accordance with city, county, and applicable building codes.
- The Geotechnical Engineer is the representative of the Owner/Builder for the purpose of implementing the report recommendations and guidelines. These duties are not intended to relieve the Earthwork Contractor of any responsibility to perform in a workman-like manner, nor is the Geotechnical Engineer to direct the grading equipment or personnel employed by the Contractor.
- The Earthwork Contractor is required to notify the Geotechnical Engineer of the anticipated work and schedule so that testing and inspections can be provided. If necessary, work may be stopped and redone if personnel have not been scheduled in advance.
- The Earthwork Contractor is required to have suitable and sufficient equipment on the job-site to process, moisture condition, mix and compact the amount of fill being placed to the approved compaction. In addition, suitable support equipment should be available to conform with recommendations and guidelines in this report.
- Canyon cleanouts, overexcavation areas, processed ground to receive fill, key excavations, subdrains and benches should be observed by the Geotechnical Engineer prior to placement of any fill. It is the Earthwork Contractor's responsibility to notify the Geotechnical Engineer of areas that are ready for inspection.
- Excavation, filling, and subgrade preparation should be performed in a manner and sequence that will provide drainage at all times and proper control of erosion. Precipitation, springs, and seepage water encountered shall be pumped or drained to provide a suitable working surface. The Geotechnical Engineer must be informed of springs or water seepage encountered during grading or foundation construction for possible revision to the recommended construction procedures and/or installation of subdrains.

Site Preparation

- The Earthwork Contractor is responsible for all clearing, grubbing, stripping and site preparation for the project in accordance with the recommendations of the Geotechnical Engineer.
- If any materials or areas are encountered by the Earthwork Contractor which are suspected of having toxic or environmentally sensitive contamination, the Geotechnical Engineer and Owner/Builder should be notified immediately.

- Major vegetation should be stripped and disposed of off-site. This includes trees, brush, heavy grasses and any materials considered unsuitable by the Geotechnical Engineer.

- Underground structures such as basements, cesspools or septic disposal systems, mining shafts, tunnels, wells and pipelines should be removed under the inspection of the Geotechnical Engineer and recommendations provided by the Geotechnical Engineer and/or city, county or state agencies. If such structures are known or found, the Geotechnical Engineer should be notified as soon as possible so that recommendations can be formulated.

- Any topsoil, slopewash, colluvium, alluvium and rock materials which are considered unsuitable by the Geotechnical Engineer should be removed prior to fill placement.

- Remaining voids created during site clearing caused by removal of trees, foundations basements, irrigation facilities, etc., should be excavated and filled with compacted fill.

- Subsequent to clearing and removals, areas to receive fill should be scarified to a depth of 10 to 12 inches, moisture conditioned and compacted

- The moisture condition of the processed ground should be at or slightly above the optimum moisture content as determined by the Geotechnical Engineer. Depending upon field conditions, this may require air drying or watering together with mixing and/or discing.

<u>Compacted Fills</u>

- Soil materials imported to or excavated on the property may be utilized in the fill, provided each material has been determined to be suitable in the opinion of the Geotechnical Engineer. Unless otherwise approved by the Geotechnical Engineer, all fill materials shall be free of deleterious, organic, or frozen matter, shall contain no chemicals that may result in the material being classified as "contaminated," and shall be very low to non-expansive with a maximum expansion index (EI) of 50. The top 12 inches of the compacted fill should have a maximum particle size of 3 inches, and all underlying compacted fill material a maximum 6-inch particle size, except as noted below.

- All soils should be evaluated and tested by the Geotechnical Engineer. Materials with high expansion potential, low strength, poor gradation or containing organic materials may require removal from the site or selective placement and/or mixing to the satisfaction of the Geotechnical Engineer.

- Rock fragments or rocks less than 6 inches in their largest dimensions, or as otherwise determined by the Geotechnical Engineer, may be used in compacted fill, provided the distribution and placement is satisfactory in the opinion of the Geotechnical Engineer.

- Rock fragments or rocks greater than 12 inches should be taken off-site or placed in accordance with recommendations and in areas designated as suitable by the Geotechnical Engineer. These materials should be placed in accordance with Plate D-8 of these Grading Guide Specifications and in accordance with the following recommendations:

  - Rocks 12 inches or more in diameter should be placed in rows at least 15 feet apart, 15 feet from the edge of the fill, and 10 feet or more below subgrade. Spaces should be left between each rock fragment to provide for placement and compaction of soil around the fragments.

  - Fill materials consisting of soil meeting the minimum moisture content requirements and free of oversize material should be placed between and over the rows of rock or concrete. Ample water and compactive effort should be applied to the fill materials as they are placed in order that all of the voids between each of the fragments are filled and compacted to the specified density.

  - Subsequent rows of rocks should be placed such that they are not directly above a row placed in the previous lift of fill. A minimum 5-foot offset between rows is recommended.

  - To facilitate future trenching, oversized material should not be placed within the range of foundation excavations, future utilities or other underground construction unless specifically approved by the soil engineer and the developer/owner representative.

- Fill materials approved by the Geotechnical Engineer should be placed in areas previously prepared to receive fill and in evenly placed, near horizontal layers at about 6 to 8 inches in loose thickness, or as otherwise determined by the Geotechnical Engineer for the project.

- Each layer should be moisture conditioned to optimum moisture content, or slightly above, as directed by the Geotechnical Engineer. After proper mixing and/or drying, to evenly distribute the moisture, the layers should be compacted to at least 90 percent of the maximum dry density in compliance with ASTM D-1557-78 unless otherwise indicated.

- Density and moisture content testing should be performed by the Geotechnical Engineer at random intervals and locations as determined by the Geotechnical Engineer. These tests are intended as an aid to the Earthwork Contractor, so he can evaluate his workmanship, equipment effectiveness and site conditions. The Earthwork Contractor is responsible for compaction as required by the Geotechnical Report(s) and governmental agencies.

- Fill areas unused for a period of time may require moisture conditioning, processing and recompaction prior to the start of additional filling. The Earthwork Contractor should notify the Geotechnical Engineer of his intent so that an evaluation can be made.

- Fill placed on ground sloping at a 5-to-1 inclination (horizontal-to-vertical) or steeper should be benched into bedrock or other suitable materials, as directed by the Geotechnical Engineer. Typical details of benching are illustrated on Plates D-2, D-4, and D-5.

- Cut/fill transition lots should have the cut portion overexcavated to a depth of at least 3 feet and rebuilt with fill (see Plate D-1), as determined by the Geotechnical Engineer.

- All cut lots should be inspected by the Geotechnical Engineer for fracturing and other bedrock conditions. If necessary, the pads should be overexcavated to a depth of 3 feet and rebuilt with a uniform, more cohesive soil type to impede moisture penetration.

- Cut portions of pad areas above buttresses or stabilizations should be overexcavated to a depth of 3 feet and rebuilt with uniform, more cohesive compacted fill to impede moisture penetration.

- Non-structural fill adjacent to structural fill should typically be placed in unison to provide lateral support. Backfill along walls must be placed and compacted with care to ensure that excessive unbalanced lateral pressures do not develop. The type of fill material placed adjacent to below grade walls must be properly tested and approved by the Geotechnical Engineer with consideration of the lateral earth pressure used in the design.

Foundations

- The foundation influence zone is defined as extending one foot horizontally from the outside edge of a footing, and proceeding downward at a ½ horizontal to 1 vertical (0.5:1) inclination.

- Where overexcavation beneath a footing subgrade is necessary, it should be conducted so as to encompass the entire foundation influence zone, as described above.

- Compacted fill adjacent to exterior footings should extend at least 12 inches above foundation bearing grade. Compacted fill within the interior of structures should extend to the floor subgrade elevation.

Fill Slopes

- The placement and compaction of fill described above applies to all fill slopes. Slope compaction should be accomplished by overfilling the slope, adequately compacting the fill in even layers, including the overfilled zone and cutting the slope back to expose the compacted core

- Slope compaction may also be achieved by backrolling the slope adequately every 2 to 4 vertical feet during the filling process as well as requiring the earth moving and compaction equipment to work close to the top of the slope. Upon completion of slope construction, the slope face should be compacted with a sheepsfoot connected to a sideboom and then grid rolled. This method of slope compaction should only be used if approved by the Geotechnical Engineer.

- Sandy soils lacking in adequate cohesion may be unstable for a finished slope condition and therefore should not be placed within 15 horizontal feet of the slope face.

- All fill slopes should be keyed into bedrock or other suitable material. Fill keys should be at least 15 feet wide and inclined at 2 percent into the slope. For slopes higher than 30 feet, the fill key width should be equal to one-half the height of the slope (see Plate D-5).

- All fill keys should be cleared of loose slough material prior to geotechnical inspection and should be approved by the Geotechnical Engineer and governmental agencies prior to filling.

- The cut portion of fill over cut slopes should be made first and inspected by the Geotechnical Engineer for possible stabilization requirements. The fill portion should be adequately keyed through all surficial soils and into bedrock or suitable material. Soils should be removed from the transition zone between the cut and fill portions (see Plate D-2).

Cut Slopes

- All cut slopes should be inspected by the Geotechnical Engineer to determine the need for stabilization. The Earthwork Contractor should notify the Geotechnical Engineer when slope cutting is in progress at intervals of 10 vertical feet. Failure to notify may result in a delay in recommendations.

- Cut slopes exposing loose, cohesionless sands should be reported to the Geotechnical Engineer for possible stabilization recommendations.

- All stabilization excavations should be cleared of loose slough material prior to geotechnical inspection. Stakes should be provided by the Civil Engineer to verify the location and dimensions of the key. A typical stabilization fill detail is shown on Plate D-5.

- Stabilization key excavations should be provided with subdrains. Typical subdrain details are shown on Plates D-6.

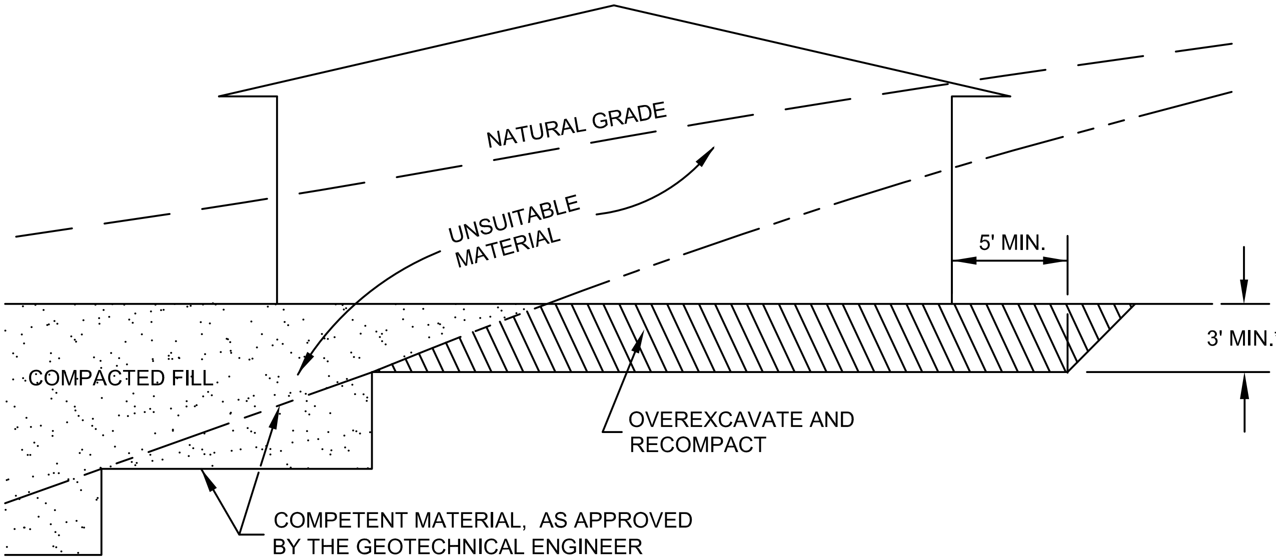
Subdrains

- Subdrains may be required in canyons and swales where fill placement is proposed. Typical subdrain details for canyons are shown on Plate D-3. Subdrains should be installed after approval of removals and before filling, as determined by the Soils Engineer.

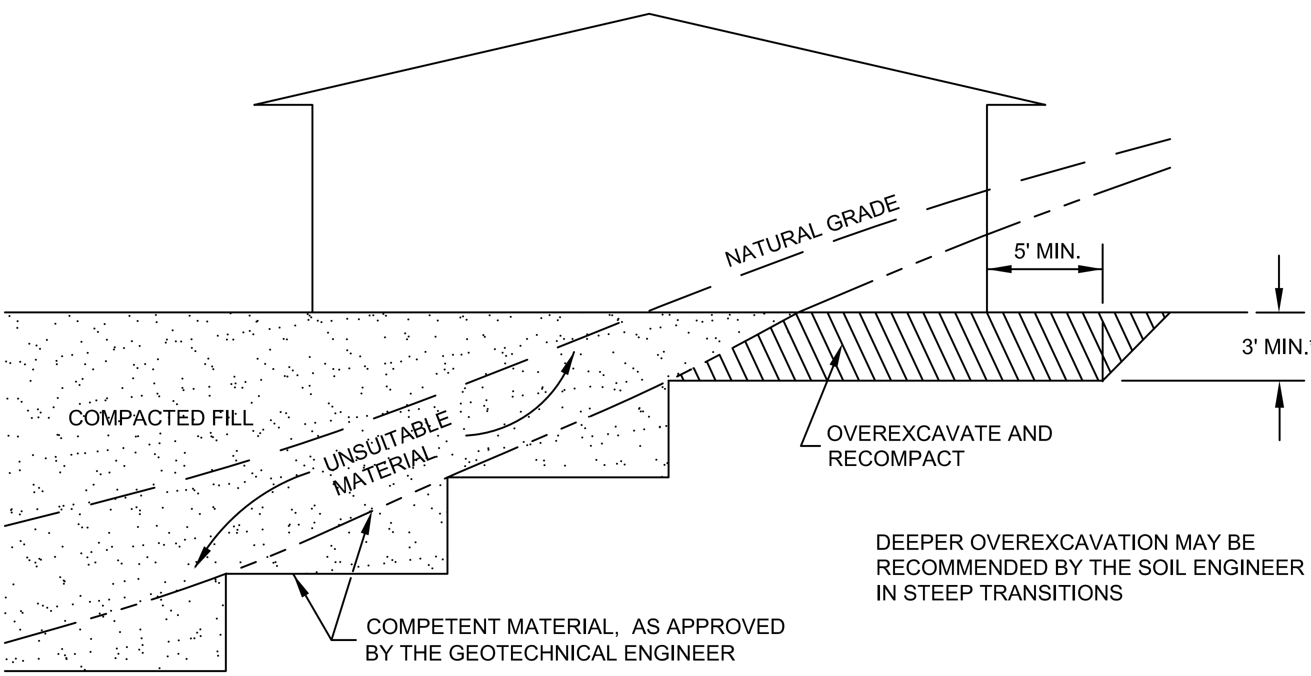
- Plastic pipe may be used for subdrains provided it is Schedule 40 or SDR 35 or equivalent. Pipe should be protected against breakage, typically by placement in a square-cut (backhoe) trench or as recommended by the manufacturer.

- Filter material for subdrains should conform to CALTRANS Specification 68-1.025 or as approved by the Geotechnical Engineer for the specific site conditions. Clean ¾-inch crushed rock may be used provided it is wrapped in an acceptable filter cloth and approved by the Geotechnical Engineer. Pipe diameters should be 6 inches for runs up to 500 feet and 8 inches for the downstream continuations of longer runs. Four-inch diameter pipe may be used in buttress and stabilization fills.

CUT LOT



CUT/FILL LOT (TRANSITION)



DEEPER OVEREXCAVATION MAY BE RECOMMENDED BY THE SOIL ENGINEER IN STEEP TRANSITIONS

*SEE TEXT OF REPORT FOR SPECIFIC RECOMMENDATION.
ACTUAL DEPTH OF OVEREXCAVATION MAY BE GREATER.

TRANSITION LOT DETAIL

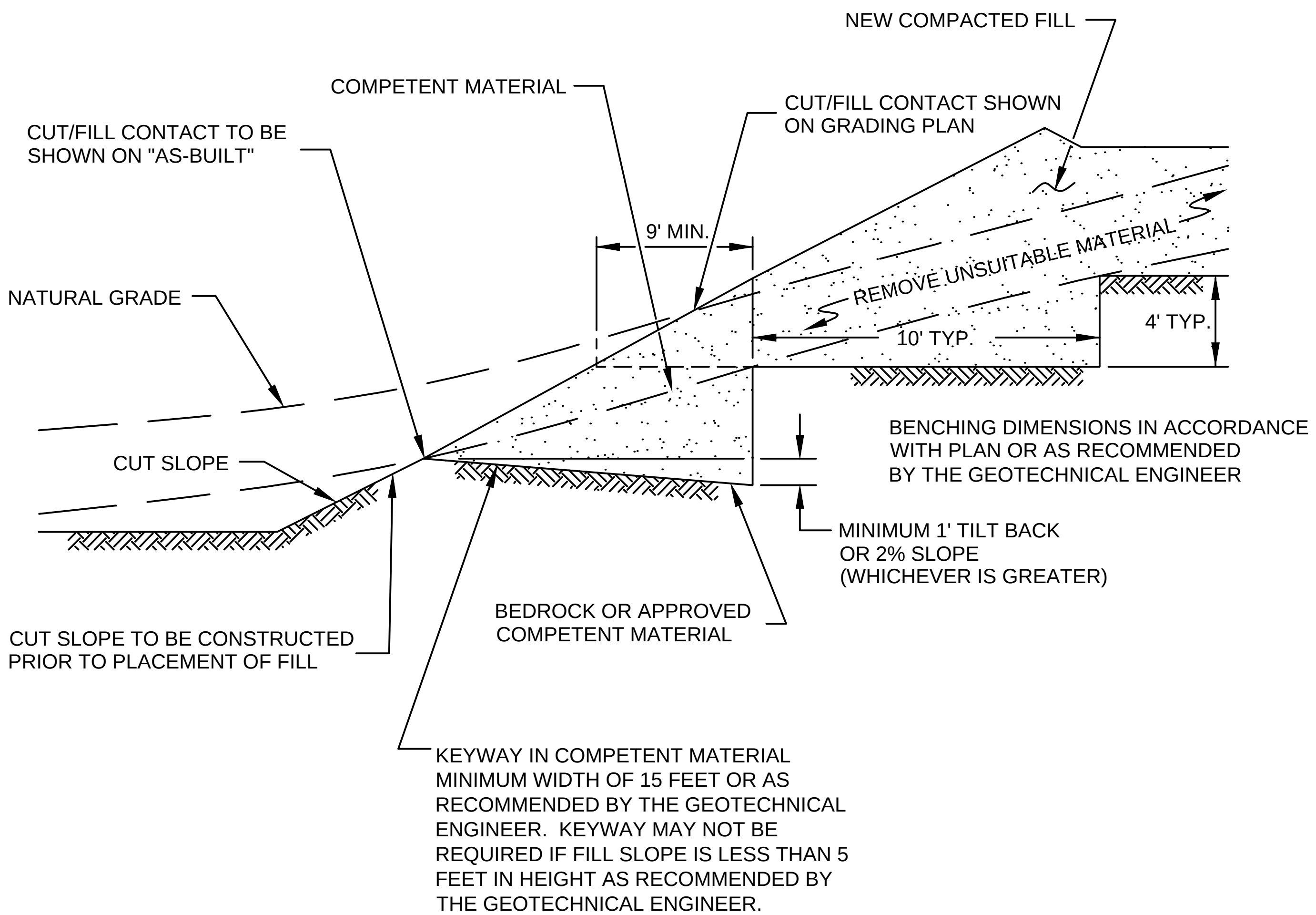
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-1

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL



FILL ABOVE CUT SLOPE DETAIL

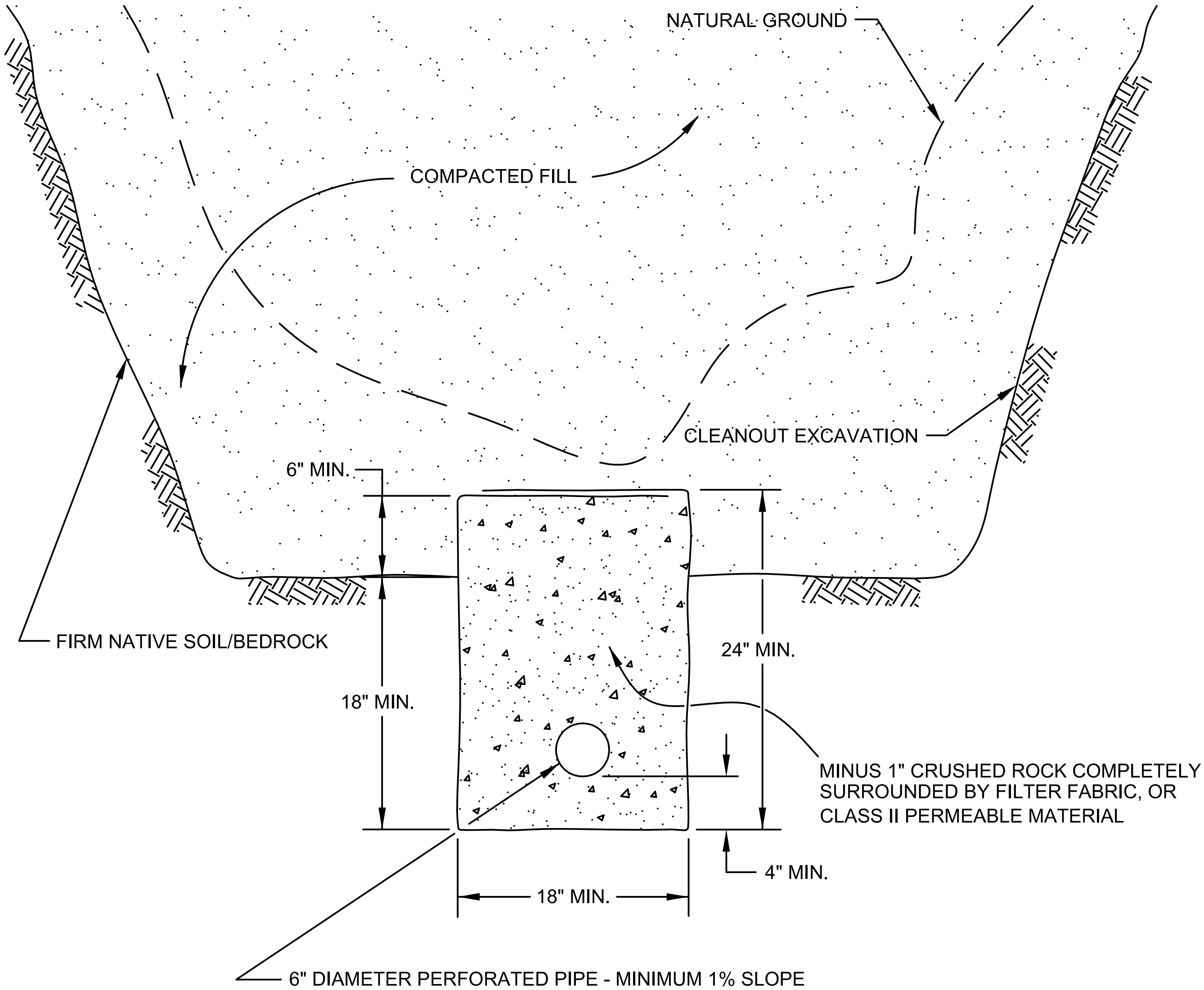
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-2

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL



PIPE MATERIAL	DEPTH OF FILL OVER SUBDRAIN
ADS (CORRUGATED POLETHYLENE)	8
TRANSITE UNDERDRAIN	20
PVC OR ABS: SDR 35	35
SDR 21	100

SCHEMATIC ONLY
NOT TO SCALE

CANYON SUBDRAIN DETAIL

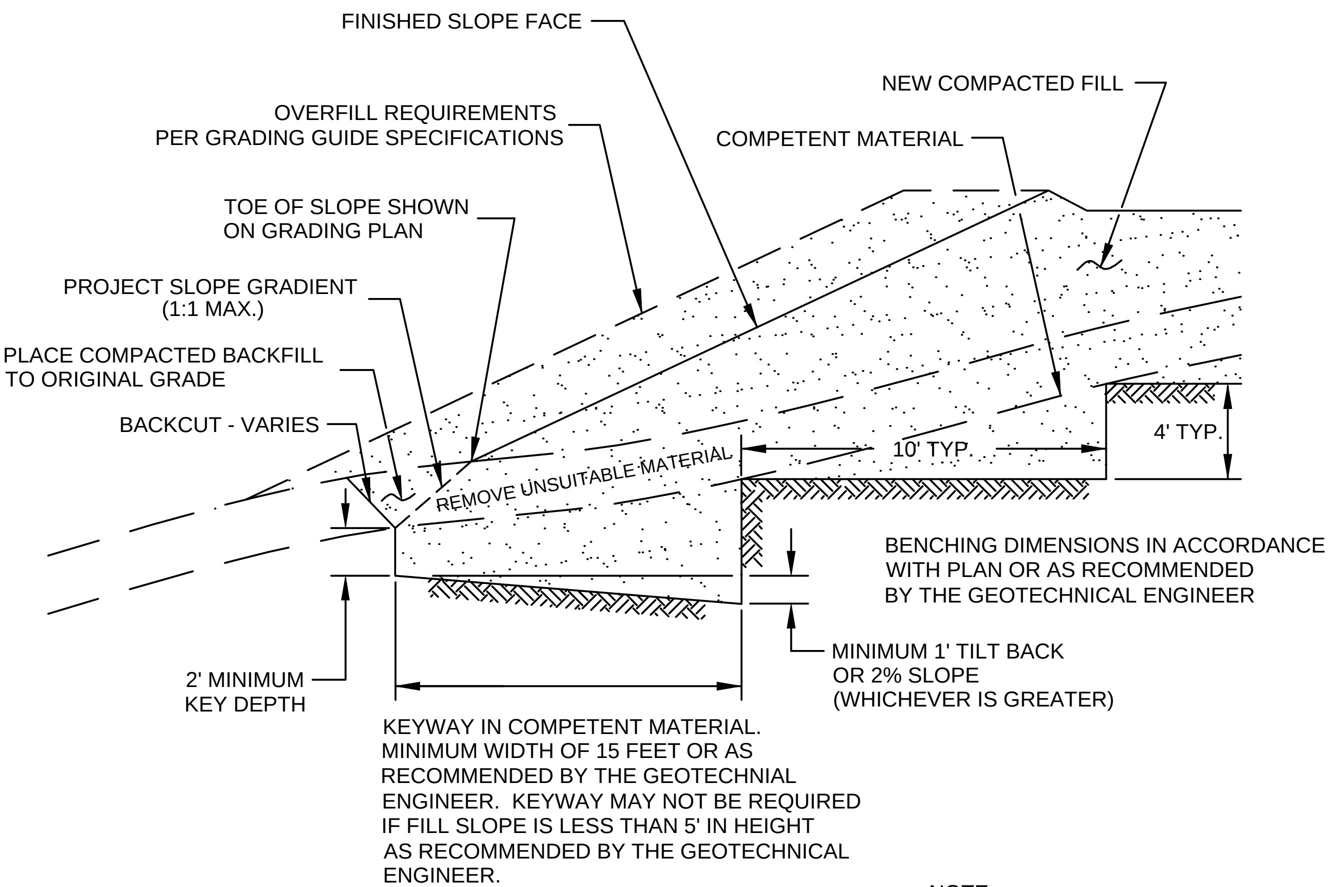
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-3

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL



NOTE:
BENCHING SHALL BE REQUIRED WHEN NATURAL SLOPES ARE EQUAL TO OR STEEPER THAN 5:1 OR WHEN RECOMMENDED BY THE GEOTECHNICAL ENGINEER.

FILL ABOVE NATURAL SLOPE DETAIL

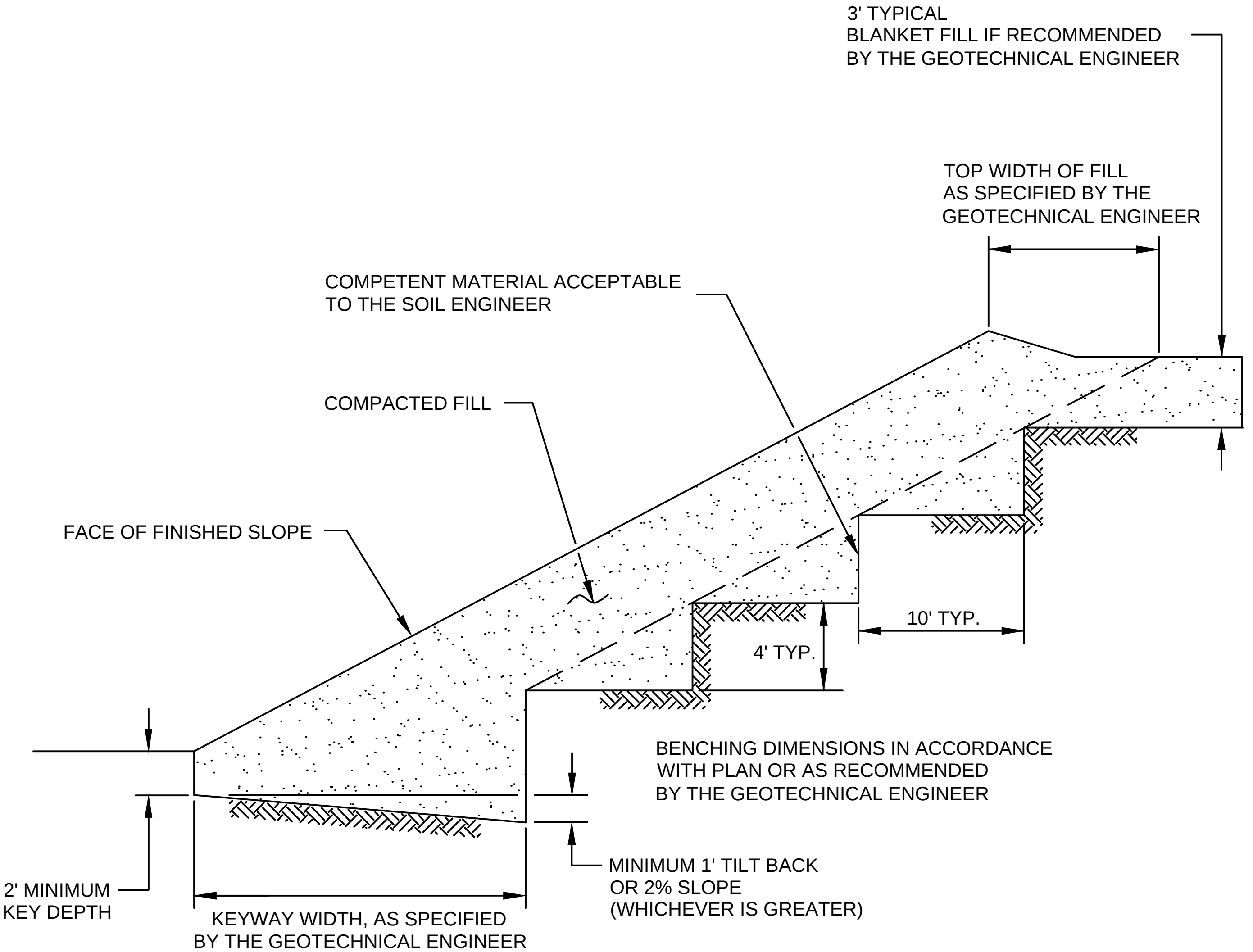
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

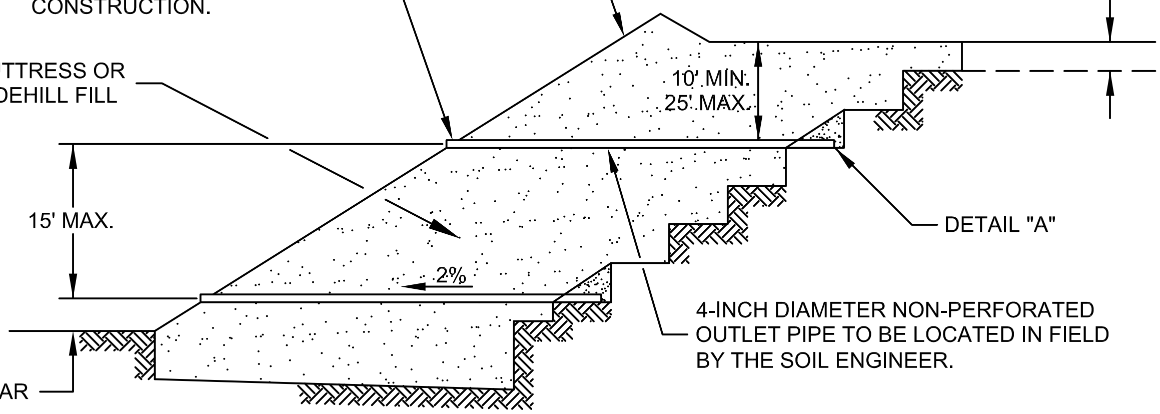
DRAWN: JAS
CHKD: GKM

PLATE D-4

SOUTHERN CALIFORNIA GEOTECHNICAL



STABILIZATION FILL DETAIL	
GRADING GUIDE SPECIFICATIONS	
NOT TO SCALE	SOUTHERN CALIFORNIA GEOTECHNICAL
DRAWN: JAS CHKD: GKM	
PLATE D-5	



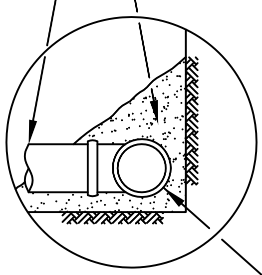
"FILTER MATERIAL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT: (CONFORMS TO EMA STD. PLAN 323)

SIEVE SIZE	PERCENTAGE PASSING
1"	100
3/4"	90-100
3/8"	40-100
NO. 4	25-40
NO. 8	18-33
NO. 30	5-15
NO. 50	0-7
NO. 200	0-3

"GRAVEL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT:

SIEVE SIZE	MAXIMUM PERCENTAGE PASSING
1 1/2"	100
NO. 4	50
NO. 200	8

SAND EQUIVALENT = MINIMUM OF 50



DETAIL "A"

NOTES:
1. TRENCH FOR OUTLET PIPES TO BE BACKFILLED WITH ON-SITE SOIL.

SLOPE FILL SUBDRAINS

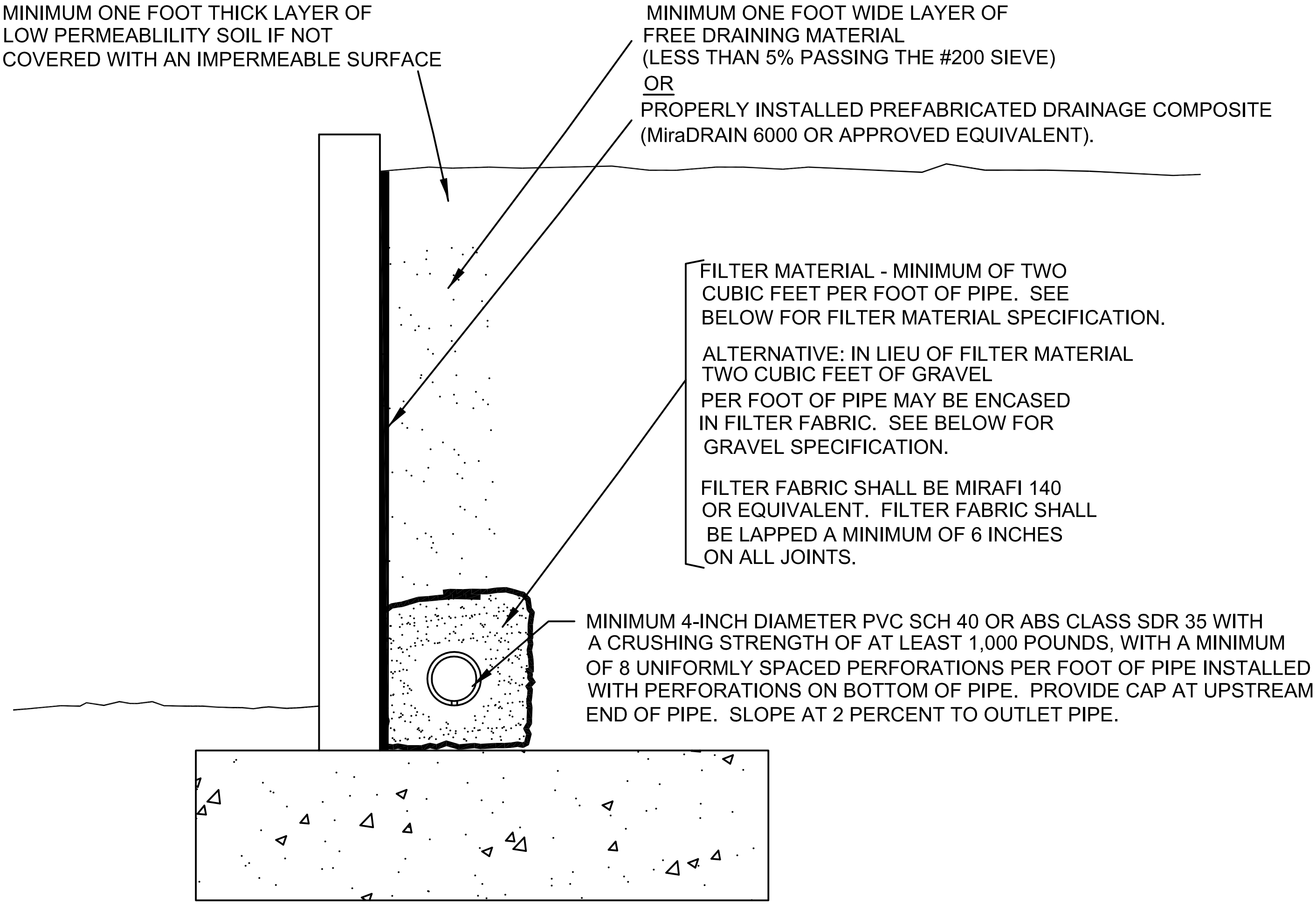
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-6

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL



"FILTER MATERIAL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT: (CONFORMS TO EMA STD. PLAN 323)

SIEVE SIZE	PERCENTAGE PASSING
1"	100
3/4"	90-100
3/8"	40-100
NO. 4	25-40
NO. 8	18-33
NO. 30	5-15
NO. 50	0-7
NO. 200	0-3

"GRAVEL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT:

SIEVE SIZE	MAXIMUM PERCENTAGE PASSING
1 1/2"	100
NO. 4	50
NO. 200	8

SAND EQUIVALENT = MINIMUM OF 50

RETAINING WALL BACKDRAINS

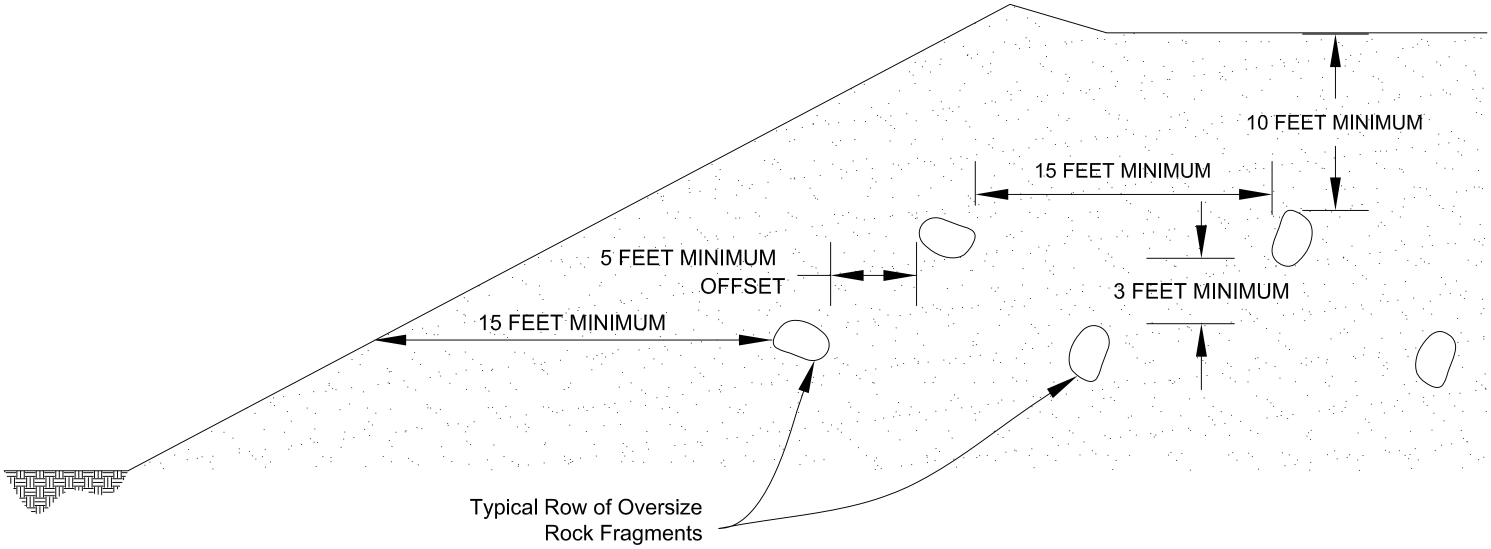
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

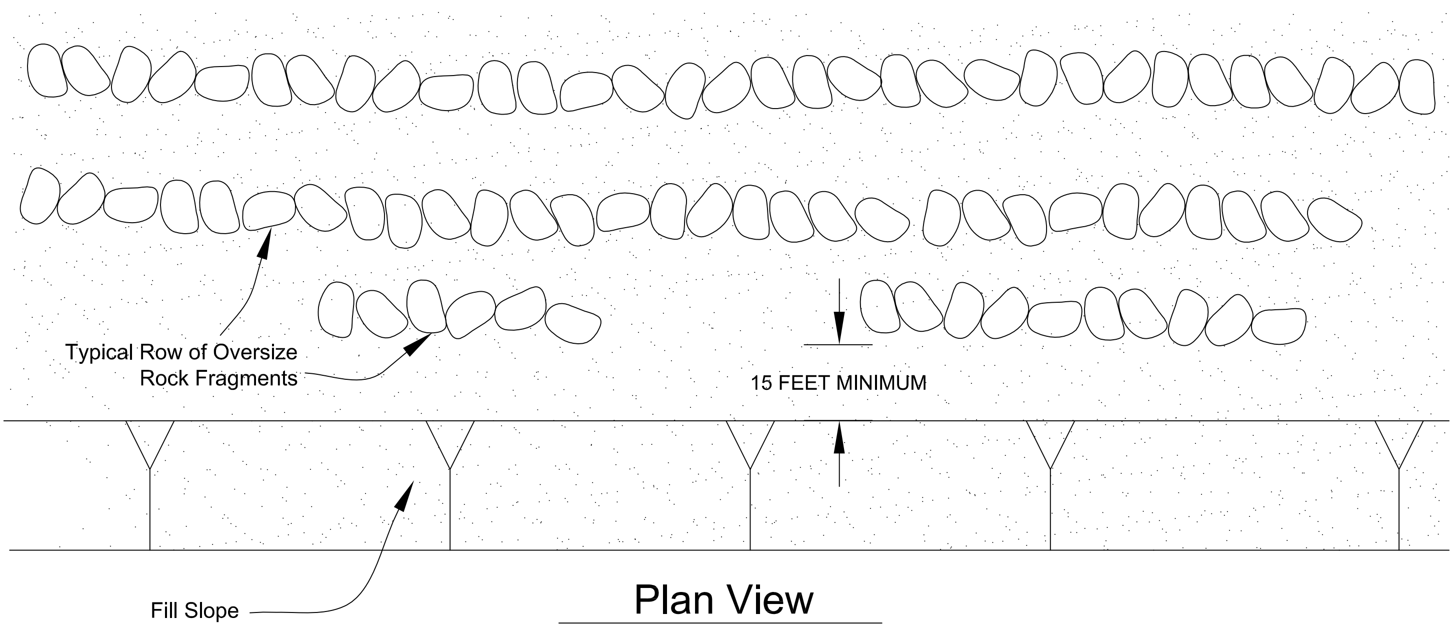
DRAWN: JAS
CHKD: GKM

PLATE D-7

SOUTHERN CALIFORNIA GEOTECHNICAL



Section View



Plan View

PLACEMENT OF OVERSIZED MATERIAL	
GRADING GUIDE SPECIFICATIONS	
NOT TO SCALE	SOUTHERN CALIFORNIA GEOTECHNICAL
DRAWN: PM CHKD: GKM	
PLATE D-8	

APPENDIX E

Latitude, Longitude: 34.093405, -117.941965

Date	2/4/2020, 1:54:27 PM
Design Code Reference Document	ASCE7-16
Risk Category	III
Site Class	D - Stiff Soil

Type	Value	Description
S_S	1.659	MCE_R ground motion. (for 0.2 second period)
S_1	0.615	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.659	Site-modified spectral acceleration value
S_{M1}	null -See Section 11.4.8	Site-modified spectral acceleration value
S_{DS}	1.106	Numeric seismic design value at 0.2 second SA
S_{D1}	null -See Section 11.4.8	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	null -See Section 11.4.8	Seismic design category
F_a	1	Site amplification factor at 0.2 second
F_v	null -See Section 11.4.8	Site amplification factor at 1.0 second
PGA	0.703	MCE_G peak ground acceleration
F_{PGA}	1.1	Site amplification factor at PGA
PGA_M	0.774	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
SsRT	1.659	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	1.814	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	1.981	Factored deterministic acceleration value. (0.2 second)
S1RT	0.615	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.68	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.621	Factored deterministic acceleration value. (1.0 second)
PGAd	0.794	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.914	Mapped value of the risk coefficient at short periods
C_{R1}	0.905	Mapped value of the risk coefficient at a period of 1 s

SOURCE: SEAOC/OSHPD Seismic Design Maps Tool
<https://seismicmaps.org/>

SEISMIC DESIGN PARAMETERS - 2019 CBC
PROPOSED WAREHOUSE
IRWINDALE, CALIFORNIA

DRAWN: JAH
CHKD: RGT
SCG PROJECT
20G105-1
PLATE E-1

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL

February 13, 2020

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

Rexford Industrial
11620 Wilshire Boulevard, 10th floor
Los Angeles, California 90025

Attention: Mr. Ricardo Rivas
Construction Manager

Project No.: **20G105-2**

Subject: **Results of Infiltration Testing**
Proposed Warehouse - Infiltration
4416 Azusa Canyon Road
Irwindale, California

Reference: Geotechnical Investigation and Infiltration Testing, Proposed Warehouse, 4416 Azusa Canyon Road, Irwindale, California, prepared for Rexford Industrial, by Southern California Geotechnical, Inc. (SCG), SCG Project No. 20G105-1.

Dear Mr. Rivas:

In accordance with your request, we have conducted infiltration testing at the subject site. We are pleased to present this report summarizing the results of the infiltration testing and our design recommendations.

Scope of Services

The scope of services performed for this project was in general accordance with our Proposal No. 19P370, dated September 25, 2019. The scope of services included site reconnaissance, subsurface exploration, obtaining representative soil samples, laboratory testing, review of relevant geological literature, analysis to determine the infiltration rates of the onsite soils, and preparation of a geotechnical report documenting our findings. The infiltration testing was performed in general accordance with ASTM Test Method D-3385-03, Standard Test Method for Infiltration Rate of Soils in Field Using Double Ring Infiltrometer.

Site and Project Description

The subject site is located at the northeast corner of Azusa Canyon Road and Los Angeles Street in Irwindale, California. The site is bounded to the north by the Big Dalton Wash, to the west by Azusa Canyon Road, to the south by Los Angeles Street, and to the southeast and east by an existing railroad easement. The general location of the site is illustrated on the Site Location Map, included as Plate 1 of this report.

The site consists of an irregular-shaped parcel, 5.89± acres in size. The site is presently developed with one warehouse, 64,535± ft² in size, in the western half of the site. The warehouse is currently occupied by Pepsi Bottling Group. The building is a single-story structure of concrete tilt-up construction and is assumed to be supported on conventional shallow foundations with a concrete slab-on-grade floor. A loading dock is located along a portion of the northeast building wall. A modular building, about 1,000 ± ft² in size is present in the east-central portion of the site. This modular building appears to be supported directly on the pavements. The buildings are surrounded by asphaltic concrete pavements in the parking and drive areas, Portland cement concrete pavements in the loading dock areas, and concrete flatwork in limited areas throughout the site. The southeastern area of the site is vacant and undeveloped. The ground surface cover in this area consists of exposed soil with moderate to extensive native grass and weed growth.

Detailed topographic information was not available at the time of this report. Based on visual observations made at the time of the subsurface investigation and from elevation data obtained from Google Earth, the overall site topography generally slopes downward to the southwest at a gradient of 1 to 2± percent.

Proposed Development

A site plan, prepared by GAA Architects, has been provided to our office by the client. Based on this plan, a new warehouse, 130,540± ft² in size, will be constructed in the central area of the site. Dock-high doors will be constructed along a portion of the south building wall. The building will be surrounded by asphaltic concrete pavements in the parking and drive lanes, Portland cement concrete pavements in the loading dock areas, concrete flatwork and landscape planters throughout.

Detailed structural information has not been provided. It is assumed that the new building will be a single-story structure of tilt-up concrete construction, typically supported on conventional shallow foundations with a concrete slab-on-grade floor. Based on the assumed construction, maximum column and wall loads are expected to be on the order of 100 kips and 3 to 5 kips per linear foot, respectively.

Grading plans for the proposed development were not available at the time of this report. The proposed development is not expected to include any significant amounts of below-grade construction such as basements or crawl spaces. Based on the existing topography, and assuming a relatively balanced site, cuts and fills of 2 to 3± feet are expected to be necessary to achieve the proposed site grades.

Concurrent Study

Southern California Geotechnical, Inc. (SCG) concurrently conducted a geotechnical investigation at the subject site. As part of this study, four (4) trenches were excavated to depths of 6 to 9± feet below the existing site grades. Artificial fill soils were encountered beneath the pavements at all trench locations, extending to depths of 3 to 7½± feet below the existing site grades. At Trench Nos. T-1 and T-4, the fill soils contain occasional clay nodules. The artificial fill soils generally consist of medium dense fine sands, silty sands with varying fine to coarse gravel content and occasional Cobbles. The fill soils possess a disturbed appearance and some samples contain debris, such as glass fragments, resulting in their classification as artificial fill. Native alluvial soils were encountered at all of the trench locations, extending to at least the maximum depth explored of 9± feet below the existing site grades. The native alluvial soils generally consist of gravelly well-graded sands, with some cobbles and occasional boulders.

Groundwater

Free water was not encountered during the drilling of any of the trench locations. Based on the lack of any water within the trenches, and the moisture contents of the recovered soil samples, the static groundwater is considered to have existed at a depth in excess of 10± feet at the time of the subsurface exploration.

As part of our research, we reviewed available groundwater data in order to determine the historic high groundwater level for the site. The primary reference used to determine the groundwater depths in this area is the California Department of Water Resources website, http://www.water.ca.gov/waterdatalibrary/. The nearest monitoring well in this database is located approximately 300 feet West of the site. Water level readings within this monitoring well indicate groundwater levels of 194± feet below the ground surface in April 2017.

Subsurface Exploration

Scope of Exploration

The subsurface exploration conducted for this project consisted of two (2) backhoe-excavated infiltration trenches to depths of 9 to 10± feet below existing site grades. The trenches were logged during excavation by a member of our staff. The approximate locations of the infiltration trenches (identified as I-1 and I-2) are included in this report as Plate 2.

Geotechnical Conditions

Artificial fill soils were encountered at the two (2) infiltration test locations and extend to depths of 4 to 6± feet. At Infiltration No. I-1, the fill extends to 6 feet below the existing site grades. The fill soils at this location consist of loose and dry silty fine sands with trace to occasional medium to coarse sand and trace gravel. These soils are underlain by a 1-foot-thick soft and damp silty clay layer between 3 and 4 feet below ground surface. At Infiltration No. I-2, the fill consists of loose and damp silty fine sand with trace gravel. At 1½± feet, little to some soft and damp clay was encountered to the maximum fill depth of 4± feet. The artificial fill soils possess a disturbed appearance and metal fragments were observed within the fill at Infiltration No. I-1.

Native alluvium was encountered beneath the artificial fill soils at all of the infiltration locations, extending to at least the maximum depth explored of 10± feet below existing site grades. The alluvial soils beneath the artificial fill consist of loose and damp gravelly fine to coarse sand with some cobble content at both infiltration test locations. At Infiltration Trench No. I-2, cobble content varies within the alluvium, with extensive cobble content between 5 and 7 feet, and occasional cobbles between 9½ and 10± feet. The Trench Logs, which illustrate the conditions encountered at the infiltration test locations, are presented on plates B-1 and B-2 of this report.

Infiltration Testing

We understand that the results of the testing will be used to prepare a preliminary design for the storm water infiltration systems that will be used at the subject site. As previously mentioned, the infiltration testing was performed in general accordance with ASTM Test Method D-3385-03, Standard Test Method for Infiltration Rate of Soils in Field Using Double Ring Infiltrometer.

Two stainless steel infiltration rings were used for the infiltration testing. The outer infiltration ring is 2 feet in diameter and 20 inches in height. The inner infiltration ring is 1 foot in diameter and 20 inches in height. At each test location, a trench was excavated to the proposed depth of the infiltration system and the outer ring was driven 3± inches into the soil at the base of each trench. The inner ring was centered inside the outer ring and subsequently driven 3± inches into the soil at the base of the trench. The rings were driven into the soil using a ten-pound sledge hammer. The soil surrounding the wall of the infiltration rings was only slightly disturbed during the driving process.

Infiltration Testing Procedure

Infiltration testing was performed at both of the infiltration trench locations. The infiltration testing consisted of filling the inner ring and the annular space (the space between the inner and outer rings) with water, approximately 3 to 4 inches above the soil. To prevent the flow of water from one ring to the other, the water level in both the inner ring and the annular space between the rings was maintained using constant-head float valves. The volume of water that was added to maintain a constant head in the inner ring and the annular space during each time interval was determined and recorded. A cap was placed over the rings to minimize the evaporation of water during the tests.

The schedule for readings was determined based on the observed soil type at the base of each backhoe-excavated trench. Based on the existing soils at the trench locations, the volumetric measurements were made at 1-minute increments at I-1, and 4-minute increments at I-2. The water volume measurements are presented on the spreadsheets enclosed with this report. The infiltration rates for each of the timed intervals are also tabulated on these spreadsheets.

The infiltration rates for the infiltration tests are calculated in centimeters per hour and then converted to inches per hour. The rates are summarized below:

Infiltration Test No.	**Depth (feet)**	**Soil Description**	**Infiltration Rate (inches/hour)**
I-1	9	Light Gray Sandy fine to coarse Gravel, some Cobble content, trace Silt	19.4
I-2	10	Light Gray Gravelly fine to coarse Sand, occasional Cobble content, trace Silt	10.5

Laboratory Testing

Moisture Content

The moisture contents for selected soil samples within the trenches were determined in accordance with ASTM D-2216 and are expressed as a percentage of the dry weight. These test results are presented on the Trench Logs.

Grain Size Analysis

The grain size distribution of selected soils collected from the base of each infiltration test trench has been determined using a range of wire mesh screens. These tests were performed in general accordance with ASTM D-422 and/or ASTM D-1140. The weight of the portion of the sample retained on each screen is recorded and the percentage finer or coarser of the total weight is calculated. The results of the grainsize analysis are presented on Plates C-1 and C-2 of this report.

Design Recommendations

Two (2) infiltration tests were performed at the subject site. As noted above, the calculated infiltration rates at the infiltration test locations are 19.1 and 10.5 inches per hour. **Based on the results of Infiltration Test Nos. I-1 and I-2, we recommend an infiltration rate of 10 inches per hour be used for the design of the proposed below-grade chamber system located in the east-central region and for the proposed chamber system located in the southwestern region of the site.**

We recommend that a representative from the geotechnical engineer be on-site during the construction of the proposed infiltration systems to identify the soil classification at the base of each chamber system. It should be confirmed that the soils at the base of the proposed infiltration systems correspond with those presented in this report to ensure that the performance of the systems will be consistent with the rates reported herein.

The design of the proposed storm water infiltration systems should be performed by the project civil engineer, in accordance with the City of Irwindale and/or County of Los Angeles guidelines. However, it is recommended that the systems be constructed so as to facilitate removal of silt and clay, or other deleterious materials from any water that may enter the system. The presence of such materials would decrease the effective infiltration rates. **It is recommended that the project civil engineer apply an appropriate factor of safety. The infiltration rates recommended above are based on the assumption that only clean water will be introduced to the subsurface profile. Any fines, debris, or organic materials could significantly impact the infiltration rates.** It should be noted that the recommended infiltration rates are based on infiltration testing at two (2) discrete locations, and the overall infiltration rates of the storm water infiltration systems could vary considerably.

Construction Considerations

The infiltration rates presented in this report are specific to the tested locations and tested depths. Infiltration rates can be significantly reduced if the soils are exposed to excessive disturbance or compaction during construction. Therefore, the subgrade soils within proposed infiltration system areas should not be over-excavated, undercut or compacted in any significant manner. **It is recommended that a note to this effect be added to the project plans and/or specifications.**

Infiltration versus Permeability

Infiltration rates are based on unsaturated flow. As water is introduced into soils by infiltration, the soils become saturated and the wetting front advances from the unsaturated zone to the saturated zone. Once the soils become saturated, infiltration rates become zero, and water can only move through soils by hydraulic conductivity at a rate determined by pressure head and soil permeability. The infiltration rates presented herein were determined in accordance with the ASTM Test Method D-3385-03 standard and are considered valid for the time and place of the actual test. Changes in soil moisture content will affect these infiltration rates. Infiltration rates should be expected to decrease until the soils become saturated. Soil permeability values will then govern groundwater movement. Permeability values may be on the order of 10 to 20 times less than infiltration rates. The system designer should incorporate adequate factors of safety and allow for overflow design into appropriate traditional storm drain systems, which would transport storm water off-site.

Location of Infiltration Systems

The use of on-site storm water infiltration systems carries a risk of creating adverse geotechnical conditions. Increasing the moisture content of the soil can cause the soil to lose internal shear strength and increase its compressibility, resulting in a change in the designed engineering properties. Overlying structures and pavements in the infiltration areas could potentially be damaged due to saturation of subgrade soils. **The proposed infiltration systems for this site should be located at least 25 feet away from any structures, including retaining walls.** Even with this provision of locating the infiltration systems at least 25 feet from the buildings, it is possible that infiltrating water into the subsurface soils could have an adverse effect on the proposed or existing structures. It should also be noted that utility trenches which happen to collect storm water can also serve as conduits to transmit storm water toward the structure, depending on the slope of the utility trench. Therefore, consideration should also be given to the proposed locations of underground utilities which may pass near the proposed infiltration system.

General Comments

This report has been prepared as an instrument of service for use by the client in order to aid in the evaluation of this property and to assist the architects and engineers in the design and preparation of the project plans and specifications. This report may be provided to the contractor(s) and other design consultants to disclose information relative to the project. However, this report is not intended to be utilized as a specification in and of itself, without appropriate interpretation by the project architect, structural engineer, and/or civil engineer. The design of the infiltration system is the responsibility of the civil engineer. The role of the geotechnical engineer is limited to determination of infiltration rate only. By using the design infiltration rates contained herein, the civil engineer agrees to indemnify, defend, and hold harmless the geotechnical engineer for all aspects of the design and performance of the infiltration system. The reproduction and distribution of this report must be authorized by the client and Southern California Geotechnical, Inc. Furthermore, any reliance on this report by an unauthorized third party is at such party's sole risk, and we accept no responsibility for damage or loss which may occur. The analysis of this site was based on a subsurface profile interpolated from limited discrete soil samples. While the materials encountered in the project area are considered to be representative of the total area, some variations should be expected between trench locations and testing depths. If the conditions encountered during construction vary significantly from those detailed herein, we should be contacted immediately to determine if the conditions alter the recommendations contained herein.

This report has been based on assumed or provided characteristics of the proposed development. It is recommended that the owner, client, architect, structural engineer, and civil engineer carefully review these assumptions to ensure that they are consistent with the characteristics of the proposed development. If discrepancies exist, they should be brought to our attention to verify that they do not affect the conclusions and recommendations contained herein. We also recommend that the project plans and specifications be submitted to our office for review to verify that our recommendations have been correctly interpreted. The analysis, conclusions, and recommendations contained within this report have been promulgated in accordance with generally accepted professional geotechnical engineering practice. No other warranty is implied or expressed.

Closure

We sincerely appreciate the opportunity to be of service on this project. We look forward to providing additional consulting services during the course of the project. If we may be of further assistance in any manner, please contact our office.

Respectfully Submitted,

SOUTHERN CALIFORNIA GEOTECHNICAL, INC.

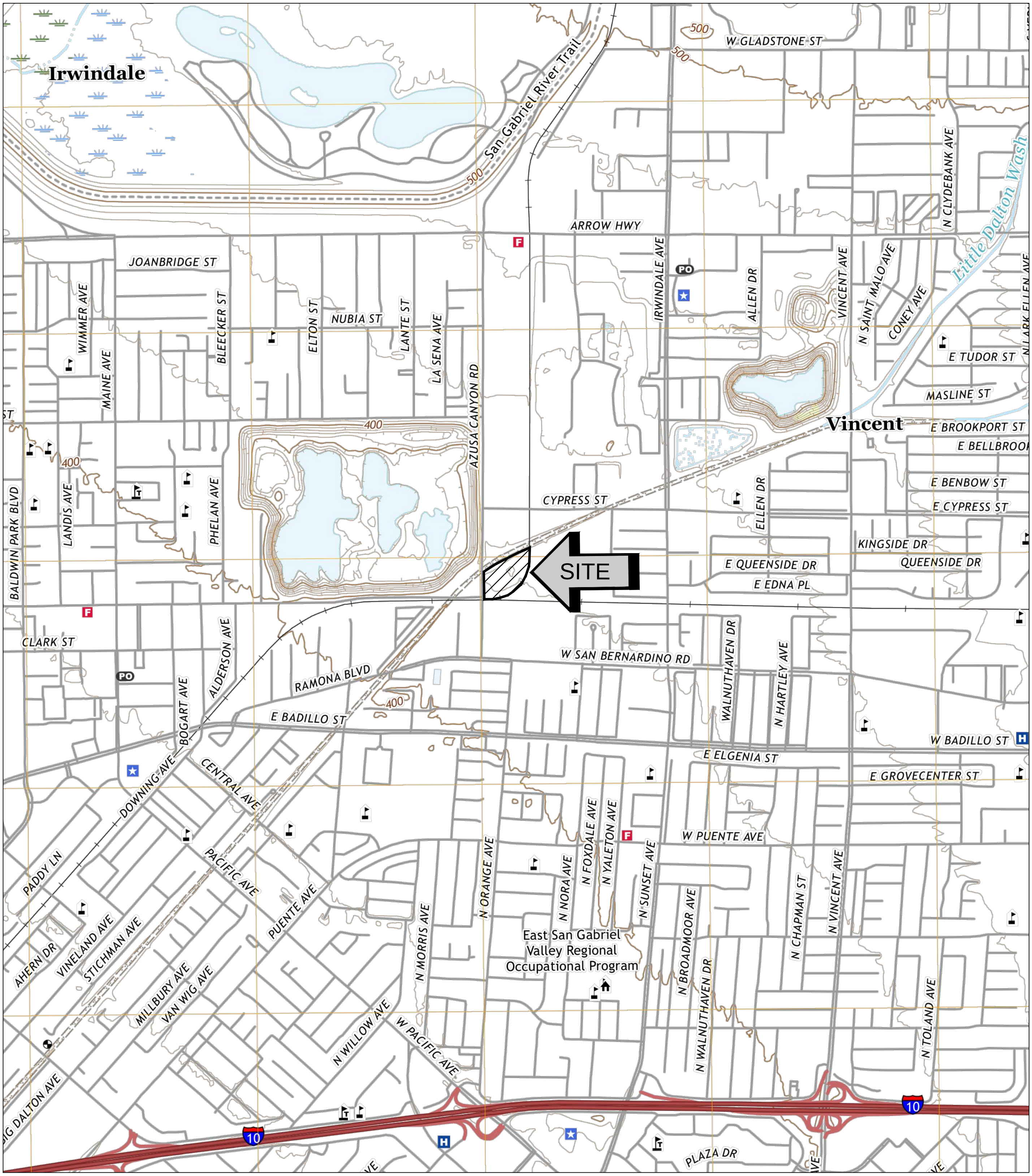
Ryan Bremer
Staff Geologist

REGISTERED PROFESSIONAL ENGINEER ROBERT TRAZO No. 2655 GEOTECHNICAL STATE OF CALIFORNIA

Robert G. Trazo, GE 2655
Principal Engineer

Distribution: (1) Addressee

Enclosures: Plate 1 - Site Location Map
Plate 2 - Infiltration Test Location Plan
Trench Logs (2 pages)
Infiltration Test Results Spreadsheets (2 pages)
Grain Size Distribution Graphs (2 pages)



SOURCE: USGS TOPOGRAPHIC MAP OF THE BALDWIN PARK QUADRANGLE, LOS ANGELES COUNTY, CALIFORNIA, 2018.

SITE LOCATION MAP

PROPOSED WAREHOUSE

IRWINDALE, CALIFORNIA

SCALE: 1" = 2000'

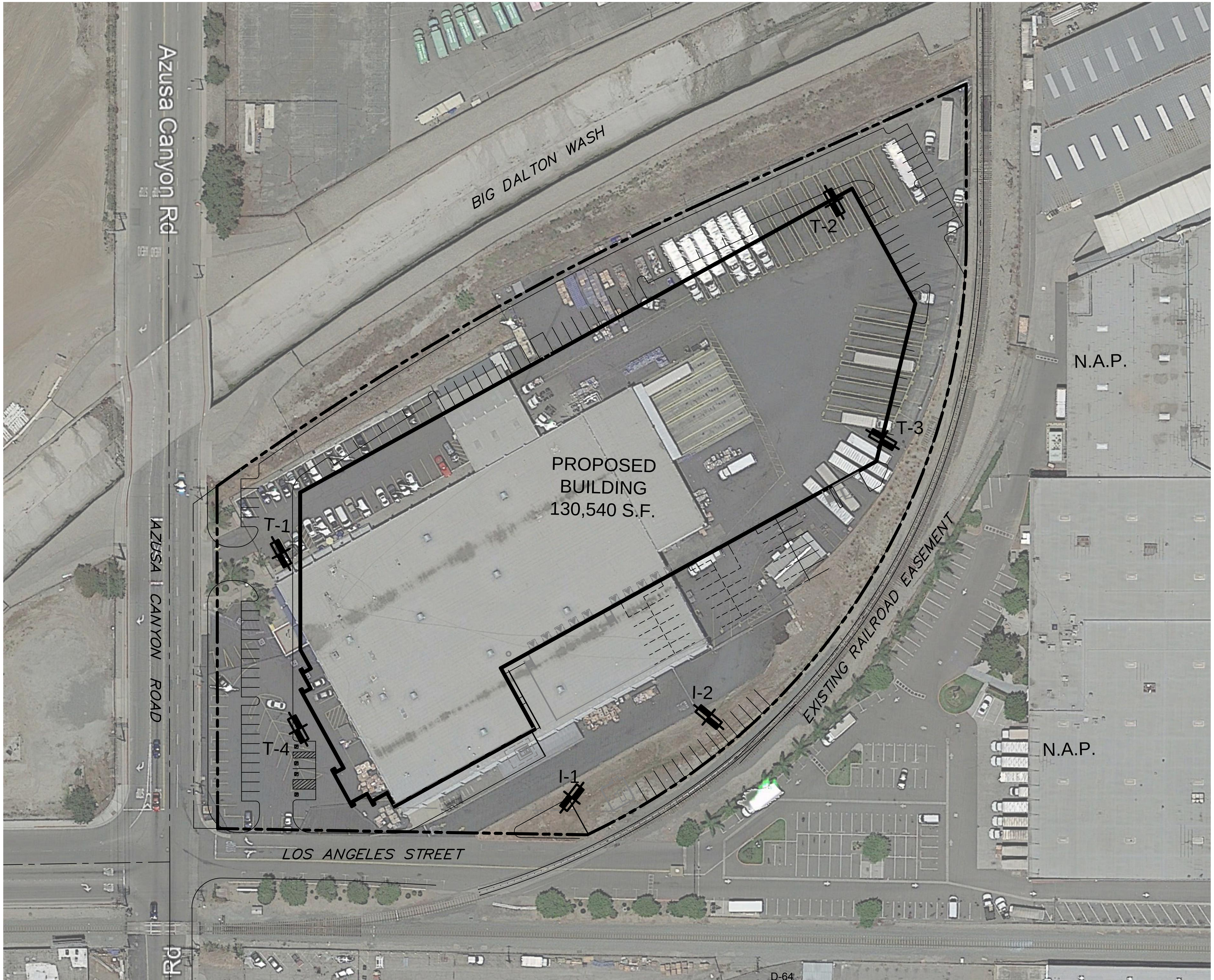
DRAWN: RB
CHKD: RGT

SCG PROJECT
20G105-2

PLATE 1

SoCalGeo

SOUTHERN CALIFORNIA GEOTECHNICAL



NORTH
SoCalGeo

GEOTECHNICAL LEGEND

APPROXIMATE TRENCH LOCATION

NOTE: CONCEPTUAL SITE PLAN PREPARED BY GAA ARCHITECTS.

TRENCH AND INFILTRATION LOCATION PLAN

PROPOSED WAREHOUSE

IRWINDALE, CALIFORNIA

SCALE: 1" = 80'

DRAWN: RB
CHKD: RGT

SCG PROJECT
20G105-2

PLATE 2

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL

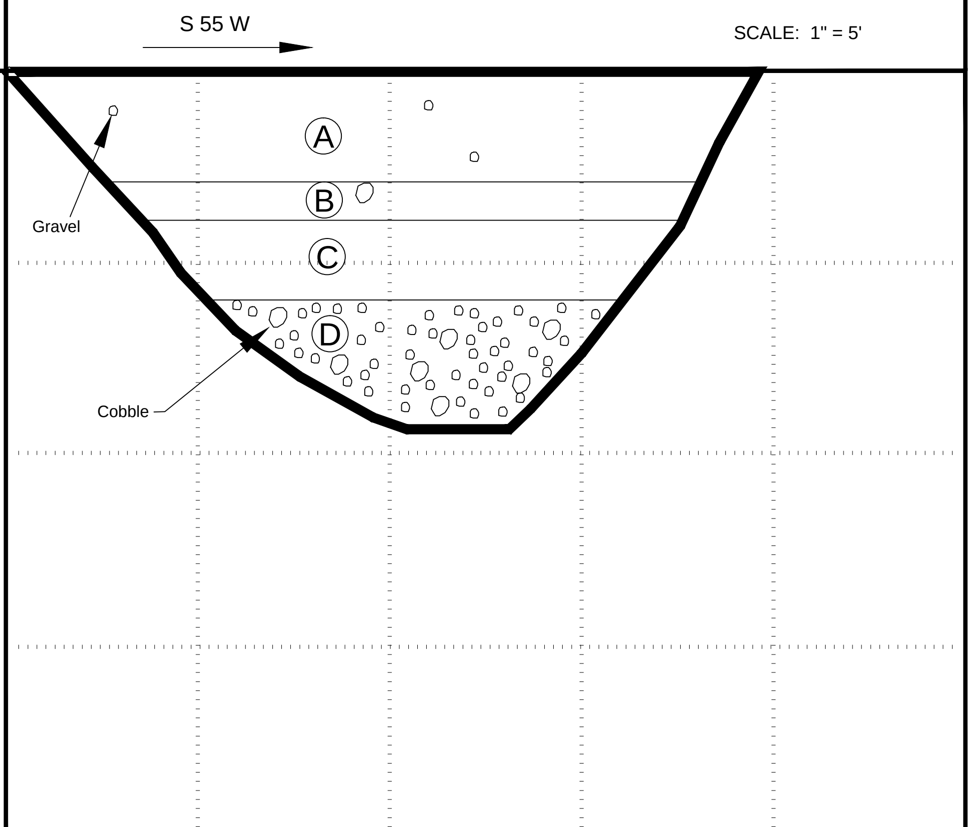
SOUTHERN CALIFORNIA GEOTECHNICAL

TRENCH NO.
I-1

JOB NO.: 20G105-2	EQUIPMENT USED: Backhoe	WATER DEPTH: Dry
PROJECT: Proposed Warehouse	LOGGED BY: Ryan Bremer	SEEPAGE DEPTH: Dry
LOCATION: Irwindale, CA	ORIENTATION: S 55 W	READINGS TAKEN: At Completion
DATE: 1/31/20		

DEPTH	SAMPLE	DRY DENSITY (PCF)	MOISTURE (%)	EARTH MATERIALS DESCRIPTION
				A: FILL: Brown Silty fine Sand, trace medium to coarse Sand, trace fine to coarse Gravel, abundant fine root fibers, mottled, loose-dry
				B: FILL: Dark brown Silty Clay, occasional fine Sand, trace fine root fibers, soft-damp
5				C: FILL: Brown Silty fine Sand, some medium to coarse Sand, some fine root fibers, loose-damp
				D: ALLUVIUM: Light gray Sandy fine to coarse Gravel, some Cobble content, trace Silt, loose-damp
	b		3	
				Trench Terminated @ 9 feet
10				
15				

GRAPHIC REPRESENTATION



KEY TO SAMPLE TYPES:
B - BULK SAMPLE (DISTURBED)
R - RING SAMPLE 2-1/2" DIAMETER
(RELATIVELY UNDISTURBED)

TRENCH LOG

PLATE B-1

SOUTHERN CALIFORNIA GEOTECHNICAL

TRENCH NO. I-2

JOB NO.: 20G105-2
PROJECT: Proposed Warehouse
LOCATION: Irwindale, CA
DATE: 1/28/20

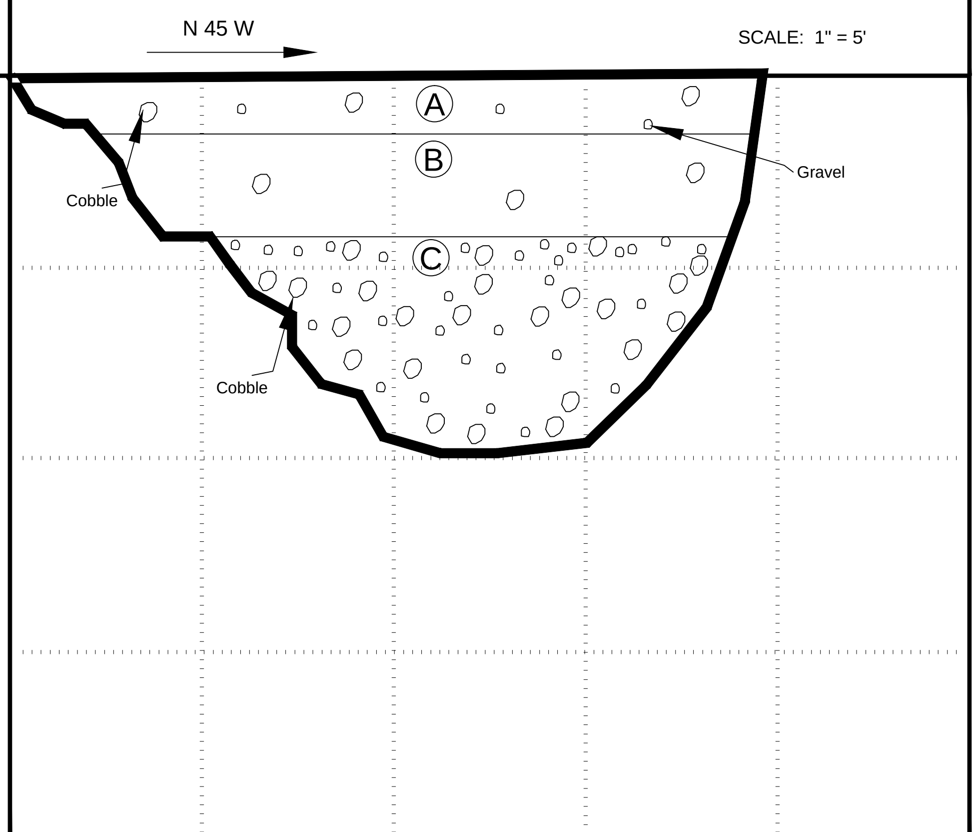
EQUIPMENT USED: Backhoe
LOGGED BY: Ryan Bremer
ORIENTATION: N 45 W

WATER DEPTH: Dry
SEEPAGE DEPTH: Dry
READINGS TAKEN: At Completion

DEPTH	SAMPLE	DRY DENSITY (PCF)	MOISTURE (%)	EARTH MATERIALS DESCRIPTION
				A: FILL: Brown Silty fine Sand, trace fine to coarse Gravel, trace Cobble content, some fine root fragments, trace metal, loose-damp B: FILL: @ 1.5 feet little to some Clay
5				C: ALLUVIUM: Light gray Gravelly fine to coarse Sand, some Cobble content, trace Silt, loose-damp @ 5 to 7 feet Cobbly fine to coarse Sand, some fine to coarse Gravel
	b		5	
	b		4	@ 9.5 feet occasional Cobble content
10				Trench Terminated @ 10 feet
15				

GRAPHIC REPRESENTATION

Metal



KEY TO SAMPLE TYPES:
B - BULK SAMPLE (DISTURBED)
R - RING SAMPLE 2-1/2" DIAMETER
(RELATIVELY UNDISTURBED)

TRENCH LOG

PLATE B-2

INFILTRATION CALCULATIONS

Project Name	Proposed Warehouse
Project Location	Irwindale, CA
Project Number	20G105-2
Engineer	Ryan Bremer

Infiltration Test No	I-1

Constants			
	Diameter (ft)	Area (ft^2)	Area (cm^2)
Inner	1	0.79	730
Anlr. Spac	2	2.36	2189

*Note: The infiltration rate was calculated based on current time interval

				Flow Readings				Infiltration Rates			
Test Interval		Time (hr)	Interval **Elapsed** (min)	Inner Ring (ml)	Ring Flow (cm^3)	Annular Ring (ml)	Space Flow (cm^3)	Inner Ring* (cm/hr)	Annular Space* (cm/hr)	Inner Ring* (in/hr)	Annular Space* (in/hr)
1	Initial	9:30 AM	0	0	700	0	5500	140.02	366.71	55.13	144.38
	Final	9:31 AM	**1**	700		5500					
2	Initial	9:33 AM	1	0	600	0	5400	49.34	148.02	19.43	58.28
	Final	9:34 AM	**4**	600		5400					
3	Initial	9:37 AM	1	0	600	0	4400	49.34	120.61	19.43	47.48
	Final	9:38 AM	**8**	600		4400					
4	Initial	9:40 AM	1	0	600	0	4600	49.34	126.09	19.43	49.64
	Final	9:41 AM	**11**	600		4600					
5	Initial	9:43 AM	1	0	550	0	4500	45.23	123.35	17.81	48.56
	Final	9:44 AM	**14**	550		4500					
6	Initial	9:46 AM	1	0	600	0	4400	49.34	120.61	19.43	47.48
	Final	9:47 AM	**17**	600		4400					
7	Initial	9:49 AM	1	0	600	0	4500	49.34	123.35	19.43	48.56
	Final	9:50 AM	**20**	600		4500					
8	Initial	9:51 AM	1	0	600	0	4600	49.34	126.09	19.43	49.64
	Final	9:52 AM	**22**	600		4600					
9	Initial	9:54 AM	1	0	550	0	4600	45.23	126.09	17.81	49.64
	Final	9:55 AM	**25**	550		4600					
10	Initial	9:56 AM	1	0	600	0	4400	49.34	120.61	19.43	47.48
	Final	9:57 AM	**27**	600		4400					

INFILTRATION CALCULATIONS

Project Name	Proposed Warehouse
Project Location	Irwindale, CA
Project Number	20G105-2
Engineer	Ryan Bremer

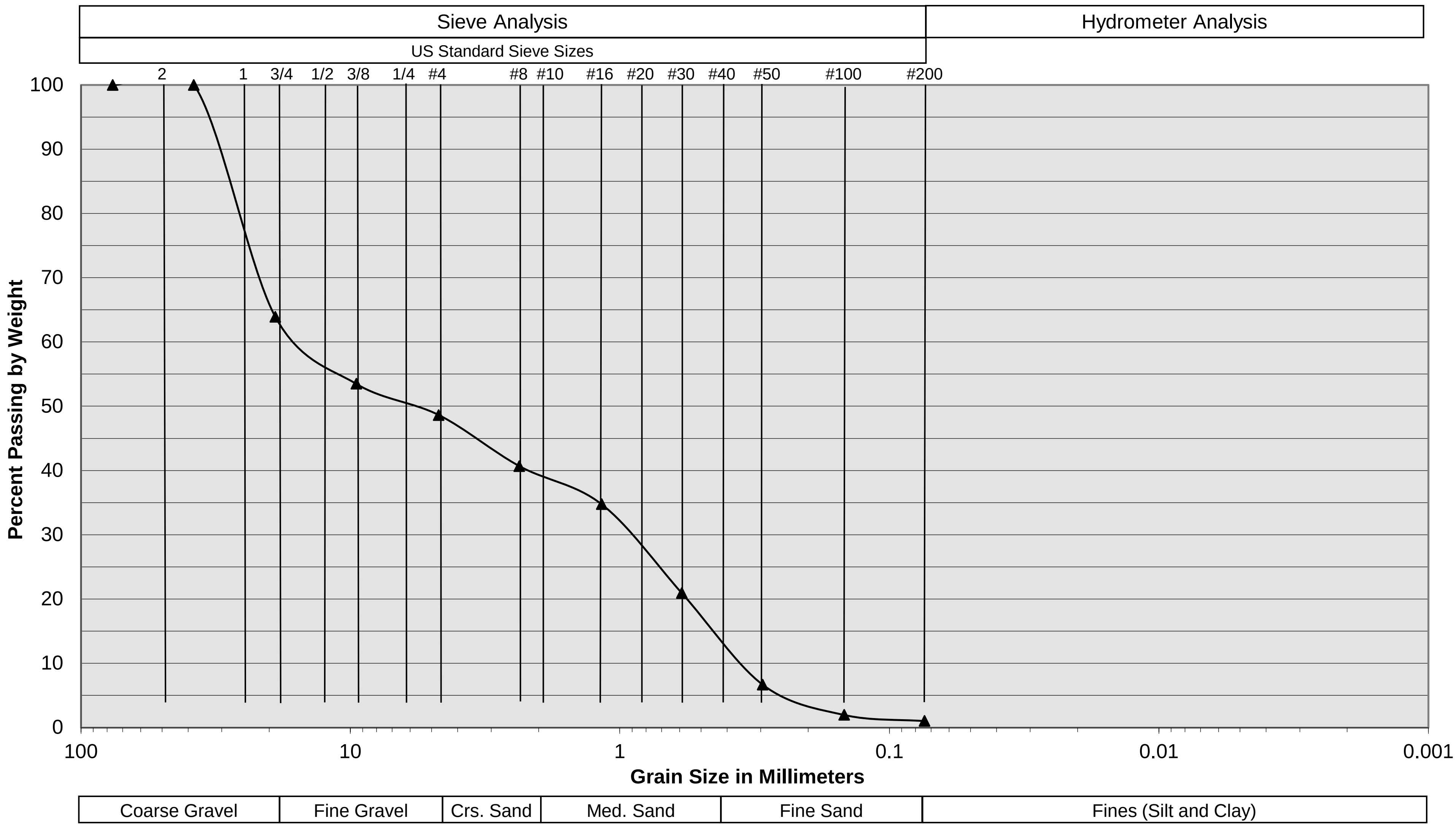
Infiltration Test No	I-2

Constants	Diameter (ft)	Area (ft^2)	Area (cm^2)
Inner	1	0.79	730
Anlr. Spac	2	2.36	2189

*Note: The infiltration rate was calculated based on current time interval

Test Interval		Time (hr)	Interval **Elapsed** (min)	Flow Readings: Inner Ring (ml)	Ring Flow (cm^3)	Annular Ring (ml)	Space Flow (cm^3)	Infiltration Rates: Inner Ring* (cm/hr)	Annular Space* (cm/hr)	Inner Ring* (in/hr)	Annular Space* (in/hr)
1	Initial	9:52 AM	0	0	1050	0	8000	210.03	533.40	82.69	210.00
	Final	9:56 AM	**4**	1050		8000					
2	Initial	9:57 AM	4	0	1200	0	6500	24.67	44.54	9.71	17.54
	Final	10:01 AM	**9**	1200		6500					
3	Initial	10:02 AM	4	0	1300	0	6300	26.73	43.17	10.52	17.00
	Final	10:06 AM	**14**	1300		6300					
4	Initial	10:07 AM	4	0	1300	0	6400	26.73	43.86	10.52	17.27
	Final	10:11 AM	**19**	1300		6400					
5	Initial	10:12 AM	4	0	1300	0	6300	26.73	43.17	10.52	17.00
	Final	10:16 AM	**24**	1300		6300					
6	Initial	10:17 AM	4	0	1300	0	6500	26.73	44.54	10.52	17.54
	Final	10:21 AM	**29**	1300		6500					
7	Initial	10:22 AM	4	0	1300	0	6500	26.73	44.54	10.52	17.54
	Final	10:26 AM	**34**	1300		6500					
8	Initial	10:28 AM	4	0	1300	0	6400	26.73	43.86	10.52	17.27
	Final	10:32 AM	**40**	1300		6400					
9	Initial	10:33 AM	4	0	1350	0	6400	27.75	43.86	10.93	17.27
	Final	10:37 AM	**45**	1350		6400					
10	Initial	10:39 AM	4	0	1300	0	6500	26.73	44.54	10.52	17.54
	Final	10:43 AM	**51**	1300		6500					

Grain Size Distribution

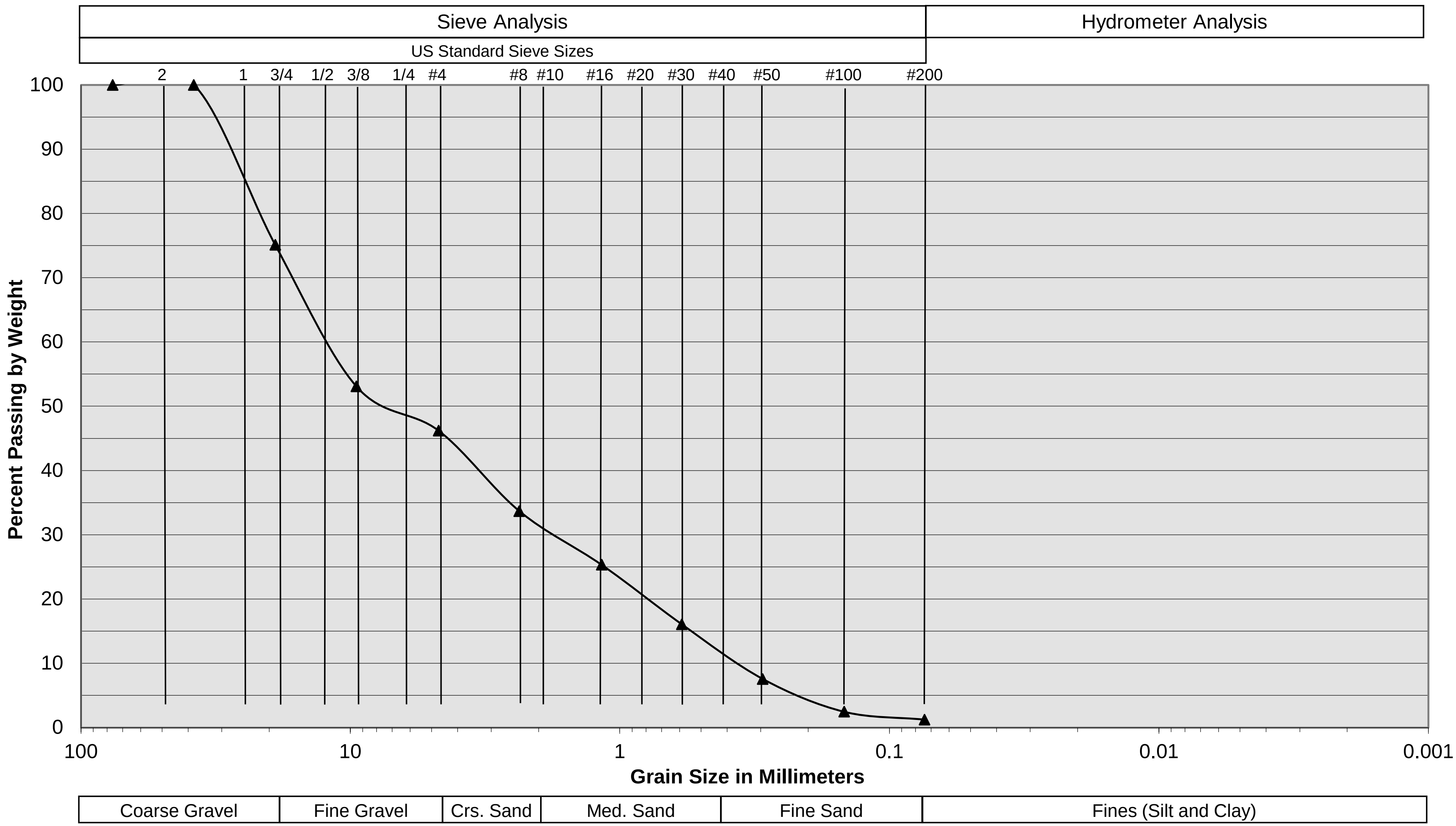


Sample Description	I-1 @ 9 feet
Soil Classification	Light Gray Sandy fine to coarse Gravel, occasional coarse Sand, trace Silt

Proposed Warehouse
Irwindale, California
Project No. 20G105-2
PLATE C-1

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

Grain Size Distribution



Sample Description	I-2 @ 9.5 feet
Soil Classification	Light Gray Sandy fine to coarse Gravel, occasional fine Sand

Proposed Warehouse
Irwindale, California
Project No. 20G105-2
PLATE C-2

D-70

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL A California Corporation

August 30, 2021

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

Rexford Industrial Reality, Inc.
333 City Boulevard West, Suite 705
Orange, California 92868

Attention: Mr. Nick Kreuter, MBA
Project Manager

Project No.: **20G105-3**

Subject: **Response to Third-Party Geotechnical Review**
Proposed Warehouse
4416 Azusa Canyon Road
Irwindale, California

References: Geotechnical Investigation, Proposed Warehouse, 4416 Azusa Canyon Road, Irwindale, California, prepared for Rexford Industrial, by Southern California Geotechnical, Inc. (SCG), SCG Project No. 20G105-1.

Results of Infiltration Testing, Proposed Warehouse, 4416 Azusa Canyon Road, Irwindale, California, prepared for Rexford Industrial, prepared by SCG, SCG Project No. 20G105-2, dated February 13, 2020.

Third-Praty Review of Southern California Geoptechnical Reports Dates February 13 and February 14, 2020, Proposed Industrial Development, 4116 Azusa Canyon Road, Irwindale, California, prepared by LGC, Geotechnical, Inc., LGC Project No. 20249-01, dated May 11, 2021.

Mr. Kreuter:

In accordance with your request, this letter presents our response to the above referenced third-party geotechnical review prepared by LGC Geotechnical, Inc. (LGC). This review was performed for the geotechnical report and infiltration testing report that we have previously prepared for the proposed development and the subject site (Reference Nos. 1 and 2, respectively). We have duplicated each of the comments by LGC below, for the geotechnical and infiltration testing reports followed by our response.

Third Party Review Comments for Geotechnical Investigation Report

LGC No. 1: *Subsurface explorations for the geotechnical evaluation and infiltration evaluation were performed via open trench excavations to maximum depths of approximately 9 feet and 10 feet, respectively. It is reasonable to assume the influence of building foundations (e.g., loading dock wall footings, column footings, etc.) will influence/load native soils deeper than 10 feet below existing grade and that the infiltration of stormwater will percolate into soils deeper than 10 feet. The lack of subsurface data to depths to/beyond the anticipated influence of the proposed building foundations should be justified. Additional field work (i.e., deeper borings)* *and laboratory testing are suggested to confirm the preliminary subsurface assumptions and geotechnical recommendations.*

SCG: The near surface soils at this site consist of gravelly well-graded sands with occasional to extensive cobble and Boulder content. Based on our previous experience with other projects near the subject site, it is not feasible to drill borings using a conventional drilling rig equipped with standard hollow-stem augers in these soils due to the gravel, cobble, and boulder content. For other projects where similar cobble and boulder containing soils were anticipated (for which liquefaction evaluation was required, and/or for larger structures) we have explored to greater depths using an air rotary drill rig equipped with a Becker hammer. However, we did not think that the expense of such a rig was warranted for a job of this size and scope. Based on our knowledge of nearby sites, we understand that similar soils consisting of well-graded gravelly sands with cobbles and boulders are present to depths extending much deeper than the depth of the anticipated foundation influence zones.

Based on our knowledge of nearby sites, we expect that the native alluvium directly beneath the depths explored during our investigation is very similar to the native soils that were explored in our backhoe-excavated test pits. Based on this expectation and the type of structure proposed for this project, we do not plan to perform additional subsurface exploration for this project.

LGC No. 2: *The potential geotechnical restraints or hazards, if any, associated with proximity of the subject site to the deep excavation known as the adjacent Olive Pit mine should be addressed.*

SCG: The proposed structure at this site will consist of a new warehouse building of concrete tilt-up construction, supported on conventional shallow foundations. Based on the anticipated foundation loads, the lateral extent of foundation influence will not significantly extend beyond the area of the proposed structure. Because the proposed structure will be located more than 100 feet from the mine site, we do not anticipate any significant hazard related to the Olive Pit mine.

LGC No. 3: *The potential impacts to adjacent (offsite) properties, structures and improvements as a result of site grading and construction and additional recommendations to protect these offsite improvements should be addressed, as necessary.*

SCG: Grading and foundation plans for the proposed development have not yet been provided to our office. At the time of our grading plan review, we typically provide additional recommendations for new screen walls located along property lines. Where the full lateral extent of the recommended remedial grading cannot be performed, we typically recommend that new screen walls be redesigned for a reduced allowable foundation bearing pressure. Depending on the extent of the remedial grading that can be performed, we would typically recommend that screen wall foundations be redesigned for a maximum allowable bearing pressure of 1,500 to 2,000 pounds per square foot. At the time of our grading and foundation plan reviews for this project, we will review new screen wall plans and determine if a reduced allowable bearing pressure is warranted.

At the time of our grading plan review, we will also review any available utility and foundation plans to determine if any additional grading or construction recommendations are necessary. Excavations near property lines may require the use of shoring or specialized grading techniques.

LGC No. 4: *The potential for hydro collapse of the dry to damp granular alluvial soils should be addressed.*

SCG: For virtually all new building projects, we attempt to obtain relatively undisturbed samples for consolidation/collapse testing. However, the near-surface soils at this site contain large particles such as coarse gravel, cobbles, and boulders. Therefore, it was not practical to obtain any relatively undisturbed samples of the near-surface native alluvium for collapse testing.

Based on the presence of fine to coarse gravel, cobbles, and boulders, we expect that the near-surface native alluvial soils at this site were deposited during a high energy flow, and possess high relative densities and low void ratios. Furthermore, during subsurface exploration, we did not identify any apparent porosity in the native alluvium encountered in our test pits. Based on these considerations, we expect that the collapse potential of the near surface native alluvium at this site is relatively low.

Any variable density artificial fill materials which may possess some potential for collapse will be removed from the proposed building area and replaced as compacted structural fill.

LGC No. 5: *Expansion potential laboratory testing should be performed at the completion of grading to verify the preliminary assumptions.*

SCG: We concur. However, we do not anticipate that the near surface soils at this site possess significant potential for expansion, as only "trace" or "occasional" clay content was observed in the artificial fill materials at the trenches. The near surface alluvium at this site consists of well graded granular soils which are considered to be non-expansive based on their lack of any appreciable clay content.

<u>Third Party Review Comments for Geotechnical Investigation Report</u>

LGC No. 1: *Subsurface explorations for the geotechnical evaluation and infiltration evaluation were performed via open trench excavations to maximum depths of approximately 9 feet and 10 feet, respectively. It is reasonable to assume the influence of building foundations (e.g., loading dock wall footings, column footings, etc.) will influence/load native soils deeper than 10 feet below existing grade and that the infiltration of stormwater will percolate into soils deeper than 10 feet. The lack of subsurface data to depths beyond the proposed bottom of the infiltration system should be justified. Additional field work (i.e., deeper borings) and laboratory* *testing are suggested to confirm the preliminary subsurface assumptions and geotechnical recommendations.*

SCG: Please see our response to Item No. 1, above.

Additionally, for most projects we would typically drill borings to depths greater than the proposed invert elevation of the stormwater disposal system in order to determine if the conditions below the proposed system are similar to the soils being tested at the system bottom. Such comparison is useful to determine if we can expect the infiltration characteristics to be similar to those soils at the test depths. Deeper borings are also typically performed in order to rule out the presence of groundwater within 10 feet of the bottom of the system. However, as discussed above, it was not feasible to drill borings at this site using conventional drilling equipment, and it was not feasible to excavate deeper trenches at this site without significant benching and the use of larger equipment.

As discussed above, based on our knowledge of other sites in the area, we expect that the native alluvial soils located directly beneath the proposed infiltration system will be very similar to the native alluvial soils encountered in our backhoe-excavated test pits. We expect that the soils located directly below the system bottom will consist of well-graded granular soils with relatively high infiltration rates. Based on our research of historic high groundwater levels (discussed in Section 4.2 of the referenced geotechnical report), we expect that the groundwater table is located more than 100 feet below the bottom of the proposed system.

Closure

We sincerely appreciate the opportunity to be of service on this project. We look forward to providing additional consulting services during the course of the project. If we may be of further assistance in any manner, please contact our office.

Respectfully Submitted,

SOUTHERN CALIFORNIA GEOTECHNICAL, INC.

Daniel W. Nielsen, GE 3166
Senior Engineer

REGISTERED PROFESSIONAL ENGINEER
DANIEL W. NIELSEN
No. 3166
GEOTECHNICAL
STATE OF CALIFORNIA

Distribution: (1) Addressee

Enclosures: Geotechnical Review Sheet (4 pages)

May 11, 2021

Project No. 20249-01

Ms. Dina El Chammas
Placeworks
3 MacArthur Place, Suite 1100
Santa Ana, CA 92707

Subject: ***Third-Party Review of Southern California Geotechnical Reports Dated February 13, 2020 and February 14, 2020, Proposed Industrial Development, 4116 Azusa Canyon Road, Irwindale, California***

Introduction

In accordance with your request, LGC Geotechnical, Inc. has prepared this third-party review of the referenced geotechnical reports for the proposed industrial development located at 4116 Azusa Canyon Road, Irwindale, California. As part of our review, we conducted a site reconnaissance visit on April 15, 2021.

Project Overview

The subject site is currently occupied by an existing industrial building, loading docks, parking areas, driveways, landscaping, miscellaneous improvements and a perimeter fence. It is our understanding the site is no longer operational. Vegetation was found throughout the site growing within cracks in the asphalt concrete pavement and unpaved areas. Extensive cracking of the asphalt concrete was observed. The building is a single level structure constructed with masonry block and tilt-up panel walls. The exterior loading dock is partially covered with a roof. Minor to significant cracking of the interior floor slabs was observed.

To the north of the site is a concrete lined open channel known as Big Dalton Wash, to the east of the site are existing railroad tracks, to the south of the site is Los Angeles Street and to the west of the site is Azusa Canyon Road. A few hundred feet east of the site, beyond Azusa Canyon Road and Big Dalton Wash, is the Olive Pit mining quarry. The Olive Pit mining quarry began operations in 1925 and ceased operations in the mid 1970's. The bottom of the Olive Pit excavation is approximately 200 feet below the adjacent street grades and contains side slopes with inclinations ranging from approximately 2:1 (horizontal to vertical) up to approximately 1:1.

Subsurface conditions consist of varying thicknesses of older artificial fill (not documented) overlying native alluvial materials. Artificial fill was found to depths up to 7.5 feet below existing grade. Groundwater is not anticipated to impact development of the site. Historic high groundwater is estimated to be greater than approximately 130 feet below ground surface (CDMG, 1998). No active

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faults are mapped as crossing through or nearby to the site. The site is not located in a state mapped liquefaction hazard zone (CDMG, 1999).

Based on the preliminary conceptual grading plans (G4, 2020), site development will consist of one approximately 129,000 square foot industrial building, a loading dock, on-grade parking areas and a water quality system. The proposed industrial building is anticipated to be at-grade concrete tilt-up structure.

Third Party Review of Southern California Geotechnical Results of Infiltration Testing, Proposed Warehouse, 4416 Azusa Canyon Road, Irwindale, California, Project No. 20G105-2, dated February 13, 2020.

Comment No. 1 – Subsurface explorations for the geotechnical evaluation and infiltration evaluation were performed via open trench excavations to maximum depths of approximately 9 feet and 10 feet, respectively. It is reasonable to assume the influence of building foundations (e.g., loading dock wall footings, column footings, etc.) will influence/load native soils deeper than 10 feet below existing grade and that the infiltration of stormwater will percolate into soils deeper than 10 feet. The lack of subsurface data to depths beyond the proposed bottom of the infiltration system should be justified. Additional field work (i.e., deeper borings) and laboratory testing are suggested to confirm the preliminary subsurface assumptions and geotechnical recommendations.

Third Party Review of Southern California Geotechnical, Geotechnical Investigation, Proposed Warehouse, 4416 Azusa Canyon Road, Irwindale, California, Project No. 20G105-1, dated February 14, 2020.

Comment No. 1 – Subsurface explorations for the geotechnical evaluation and infiltration evaluation were performed via open trench excavations to maximum depths of approximately 9 feet and 10 feet, respectively. It is reasonable to assume the influence of building foundations (e.g., loading dock wall footings, column footings, etc.) will influence/load native soils deeper than 10 feet below existing grade and that the infiltration of stormwater will percolate into soils deeper than 10 feet. The lack of subsurface data to depths to/beyond the anticipated influence of the proposed building foundations should be justified. Additional field work (i.e., deeper borings) and laboratory testing are suggested to confirm the preliminary subsurface assumptions and geotechnical recommendations.

Comment No. 2 – The potential geotechnical restraints or hazards, if any, associated with proximity of the subject site to the deep excavation known as the adjacent Olive Pit mine should be addressed.

Comment No. 3 – The potential impacts to adjacent (offsite) properties, structures and improvements as a result of site grading and construction and additional recommendations to protect these offsite improvements should be addressed, as necessary.

Comment No. 4 – The potential for hydro collapse of the dry to damp granular alluvial soils should be addressed.

Comment No. 5 – Expansion potential laboratory testing should be performed at the completion of grading to verify the preliminary assumptions.

<u>Closure</u>

Please note, this letter is based on our review of the referenced report and limited site visit only.

Our proposed services were performed to the general standard of care of geotechnical consulting in Southern California; no other warranty is expressed or implied. This geotechnical third-party review report has been prepared for the sole use of Placeworks, its subsidiaries and affiliates may be relied upon by any of same.

The opportunity of submitting this third-party review is sincerely appreciated. Should you have any questions, please do not hesitate to contact this office.

Sincerely,

LGC Geotechnical, Inc.

REGISTERED PROFESSIONAL ENGINEER
RYAN L. DOUGLAS
No. 3147
GEOTECHNICAL
★ STATE OF CALIFORNIA ★

Ryan Douglas, PE, GE 3147
Project Engineer

RLD/BPP/amm

Attachment: References

Distribution: (1) Addressee (electronic copy)

References

California Division of Mines and Geology (CDMG) 1997, Geologic Map of the Baldwin Park 7.5-Minute Quadrangle, Los Angeles County, California, Open File Report 98-30, dated 1997.

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________, 1999, State of California Seismic Hazard Zones, Baldwin Park Quadrangle, Official Map, scale: 1:24,000, dated March 25, 1999.

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Historic Aerials, 2021, HistoricAerials.com: Aerial Photographs and Topographic Maps by Netronline, retrieved April 28, 2021.

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________, 2020b, Geotechnical Investigation, Proposed Warehouse, 4416 Azusa Canyon Road, Irwindale, California, Project No. 20G105-1, dated February 14, 2020.