GEOTECHNICAL INVESTIGATION
PROPOSED COMMERCIAL/INDUSTRIAL BUILDING

NWC Cedar Avenue and Orange Street
San Bernardino County, California
for
Thrifty Oil Company

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

October 15, 2014

Thrifty Oil Company
13116 Imperial Highway
Santa Fe Springs, California 90670

Attention: Mr. Julien Hoisington

Project No.: **14G190-1**

Subject: **Geotechnical Investigation**
Proposed Commercial/Industrial Building
NWC Cedar Avenue and Orange Street
San Bernardino County, 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.

REGISTERED PROFESSIONAL ENGINEER
DANIEL W. NIELSEN
No. 77915
CIVIL
STATE OF CALIFORNIA

Daniel W. Nielsen, RCE 77915
Project Engineer

John A. Seminara, GE 2294
Principal Engineer

Distribution: (3) 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

- Based on our review of readily available photographs from the internet, the eastern two-thirds of the subject site was formerly developed as a residential subdivision. The only apparent remains of the previous development consist of the remnants two of asphaltic concrete roads which traverse the site from north to south, a concrete slab in the center of the site, and foundations from a former building located along the east property line.
- Initial site preparation should include demolition of the remnants of the two asphaltic concrete roads and any other remnants of the previous development including all foundations, floor slabs, utilities, septic systems, and any other subsurface improvements that will not remain in place for use with the new development. Stripping of the existing vegetation including grasses, weeds, and occasional shrubs and trees will also be necessary and should include all vegetation, organic soils, and root masses. These materials should be disposed of offsite. Concrete and asphalt debris may be crushed to a maximum 2-inch particle size, mixed well with the on-site soils, and incorporated into structural fills if desired. Alternatively, it may be feasible to crush these materials into aggregate base.
- The majority of the borings encountered disturbed native alluvium at the ground surface, extending to depths of 2½ to 3½± feet. Artificial fill soils were encountered at one of the boring locations in the southeast potion of the property, extending to a depth of 4½± feet. Both the disturbed alluvium and the artificial fill soils possess variable strengths and densities, and are not considered suitable, in their present state, for the support of the foundations or floor slabs of the new structure.
- The disturbed alluvium and the artificial fill materials are underlain by undisturbed alluvial soils. The results of laboratory testing indicate that some of these soils within the upper 5 to 6 feet possess a moderate potential for hydrocollapse when inundated with water.
- Remedial grading is recommended to be performed within the new building pad area. The existing soils within the building pad area are recommended to be overexcavated to a depth of 3 feet below existing grade. In addition, all existing artificial fill materials should be removed from the new building pad area. In order to provide a relatively uniform subgrade condition, the excavations are also recommended to extend to a depth of at least 3 feet below proposed pad grade. The soils within the proposed foundation influence zones should be overexcavated to a depth of at least 3 feet below proposed foundation bearing grade.
- 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 subgrade should then be scarified to a depth of 12 inches. The overexcavation subgrade should be thoroughly moisture conditioned to a moisture content of 2 to 4 percent above optimum and compacted to 90 percent relative compaction. The previously excavated soils may then be replaced as compacted structural fill.

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

Building Foundations

- Conventional shallow foundations, supported in newly placed compacted fill.
- 2,500 lbs/ft^2 maximum allowable soil bearing pressure.
- Reinforcement consisting of at least four (4) No. 5 rebars (2 top and 2 bottom) in strip footings. Additional reinforcement may be necessary for structural considerations.

Building Floor Slab

- Conventional Slab-on-Grade, at least 5 inches thick.
- Reinforcement is not required for geotechnical considerations. The actual thickness and reinforcement of the floor slabs should be determined by the structural engineer based on the imposed loading.

Pavements

ASPHALT PAVEMENTS (R = 50)					
Materials	**Thickness (inches)**				
	Auto Parking and Drives (TI = 4.0 & 5.0)	Light Truck Traffic (TI = 6.0)	Moderate Truck Traffic (TI = 7.0)	Heavy Truck Traffic (TI = 8.0)	Heavy Truck Traffic (TI = 9.0)
Asphalt Concrete	3	3½	4	5	6
Aggregate Base	3	4	5	5	6
Compacted Subgrade	12	12	12	12	12

PORTLAND CEMENT CONCRETE PAVEMENTS (R = 50)				
Materials	**Thickness (inches)**			
	Autos & Light Truck Traffic (TI = 5.0 & 6.0)	Moderate Truck Traffic (TI =7.0)	Heavy Truck Traffic (TI =8.0)	Heavy Truck Traffic (TI =9.0)
PCC	5	6½	8	9
Compacted Subgrade (95% Relative 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. 14P297, dated June 26, 2014. 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 slabs, 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 northwest corner of Cedar Avenue and Orange Street in Bloomington, an unincorporated area of San Bernardino County, California. The site is bounded to the north by a railroad easement, to the east by Cedar Avenue, to the south by Orange Street, and to the west by Linden Avenue. The general location of the site is illustrated on the Site Location Map, included as Plate 1 in Appendix A of this report.

The site consists of an irregular shaped parcel, 20± acres in size. The site is currently vacant and undeveloped except for the remnants of two unmaintained asphaltic concrete roads, both of which trend north-south, a concrete slab in the center of the site with dimensions of approximately 20 feet by 20 feet, and foundations of a former building with dimensions of approximately 45 feet by 36 feet. Smaller portions of concrete slabs are also present in the eastern portion of the site. One of these roads appears to be coincident with the alignment of Orchard Street, if the alignment of Orchard Street were projected north of Orange Street. The second abandoned road appears to coincide with the alignment of Magnolia Street, which terminates on the north side of the Interstate 10 Freeway, if this alignment were to be projected south of Interstate 10. Both of these roads extend southward from the northern property line and terminate north of the side walk on the north side of Orange Street. Both roads are in poor condition with numerous large cracks. Ground surface cover throughout the site generally consists of exposed soil with sparse to moderate native grass and weed growth. Occasional trees are scattered throughout the central portion of the site.

As part of our research for this project, we have reviewed several readily available historic aerial photographs from the internet. The earliest available photograph, from the year 1938, indicates that the eastern two-thirds of the site was a citrus orchard. On the next available photograph, from the year 1948, the citrus orchard is no longer present, and the some residences are present in the southern portions of the eastern two-thirds of the site. Several photographs between 1948 and 1980 appear to indicate that the central and eastern portions of the site were developed as a residential subdivision, but all of the residences were demolished prior to 1994, the time of the next available photograph. The two roads presently remaining on the site appear to have been paved sometime between the taking of the photographs from the years 1948 and 1959. The western one-third of the subject site appears to be vacant and undeveloped in all of the aerial photographs we reviewed.

Topographic information for the subject site was obtained from a conceptual grading plan provided by Hall & Foreman, Inc. Based on this plan, the maximum site elevation of 1091.0 feet mean sea level (msl) is located in the northeast corner of the site and the minimum site elevation of 1079.0 feet msl is located in the southwest corner of the site. The site slopes downward to the south at a gradient of less than 2± percent. The overall topographic relief of the site is approximately 12 feet.

3.2 Proposed Development

A conceptual site plan prepared by HPA Architecture, dated July 29, 2014, was provided to our office. Based on this plan, the proposed development will consist of one (1) new commercial/industrial building, 371,308± ft^2 in size. Loading docks are proposed on the south and northeast sides of the building. It is assumed that the building will be surrounded by asphaltic concrete pavements for parking and drive lanes and Portland cement concrete pavements for the loading dock area. It is also assumed that several landscape planters and concrete flatwork will be included throughout the site.

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. The construction may include a second floor mezzanine office. Based on the assumed construction, maximum column and wall loads are expected to be on the order of 80 kips and 3 to 5 kips per linear foot, respectively.

The conceptual grading plan indicates the building will possess a sloping floor, with finished floor grades ranging between 1086.05 to 1088.02 feet msl. No significant amounts of below grade construction, such as basements or crawl spaces, are expected to be included in the proposed development. Based on the conceptual grading plan, cuts of up to 2± feet and fills up to 4± feet are expected to be necessary to achieve the proposed building pad grades.

4.0 SUBSURFACE EXPLORATION

4.1 Scope of Exploration/Sampling Methods

The subsurface exploration conducted for this project consisted of ten (10) borings advanced to depths of 5 to 30± feet below currently existing site grades. All of the borings were logged during drilling by a member of our staff.

The borings were advanced with hollow-stem augers, by a conventional truck-mounted drilling rig. Representative bulk and in-situ soil samples were taken during drilling. Relatively undisturbed in-situ samples were taken with a split barrel "California Sampler" containing a series of one inch long, 2.416± inch diameter brass rings. This sampling method is described in ASTM Test Method D-3550. In-situ samples were also taken using a 1.4± inch inside diameter split spoon sampler, in general accordance with ASTM D-1586. Both of these samplers are driven into the ground with successive blows of a 140-pound weight falling 30 inches. The blow counts obtained during driving are recorded for further analysis. Bulk samples were collected in plastic bags to retain their original moisture content. The relatively undisturbed ring samples were placed in molded plastic sleeves that were then sealed and transported to our laboratory.

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

4.2 Geotechnical Conditions

Artificial Fill

Artificial fill soils were encountered at the ground surface, at Boring No. B-3, extending to a depth of 4½± feet below the existing site grade. The fill soils consist of loose silty fine sands and medium dense gravelly fine to coarse sands. The fill materials possess moderate debris content, including asphaltic concrete fragments, and a disturbed appearance, resulting in their classification as fill.

Alluvium

Native alluvium was encountered beneath the fill materials at Boring Nos. B-1 and at the ground surface at all of the remaining borings. At several of the borings, the near surface alluvium possesses a disturbed appearance and are identified as disturbed alluvium on the boring logs. These disturbed soils generally consist of loose to medium dense silty fine sands with occasional traces of fine gravel. These soils resemble the composition and color of the native alluvium encountered at similar depths, but possess a slightly disturbed appearance. Undisturbed alluvial

soils were encountered at all of the boring locations, beneath the disturbed alluvium, artificial fill, or at the ground surface. Undisturbed alluvium extends to at least the maximum depth explored of 30± feet. The native alluvium generally consists of loose to medium dense silty fine sands, underlain by interbedded strata of medium dense to dense, fine to coarse sands, gravelly fine to coarse sands, fine sands, and silty fine sands. Occasional cobbles were observed in the auger spoils at some of the boring locations.

Groundwater

Free water was not encountered during the drilling of any of the borings. Delayed readings with the open boreholes were not possible due to caving within the borings. Based on the lack of any water within the borings, and the moisture contents of the recovered soil samples, the static groundwater table is considered to have existed at a depth in excess of 30± feet at the time of the subsurface exploration.

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 Boring Logs and are periodically referenced throughout this report.

In-situ Density and Moisture Content

The density has been determined for selected relatively undisturbed ring samples. These densities were determined in general accordance with the method presented in ASTM D-2937. The results are recorded as dry unit weight in pounds per cubic foot. 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 Boring Logs.

Consolidation

Selected soil samples have been tested to determine their consolidation potential, in accordance with ASTM D-2435. The testing apparatus is designed to accept either natural or remolded samples in a one-inch high ring, approximately 2.416 inches in diameter. Each sample is then loaded incrementally in a geometric progression and the resulting deflection is recorded at selected time intervals. Porous stones are in contact with the top and bottom of the sample to permit the addition or release of pore water. The samples are typically inundated with water at an intermediate load to determine their potential for collapse or heave. The results of the consolidation testing are plotted on Plates C-1 through C-8 in Appendix C of this report.

Maximum Dry Density and Optimum Moisture Content

A representative bulk sample has been tested for its maximum dry density and optimum moisture contents. The results have been obtained using the Modified Proctor procedure, per ASTM D-1557 and are presented on Plate C-9 in Appendix C of this report. This test is 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.

Expansion Index

The expansion potential of the on-site soils was determined in general accordance with ASTM D-4829 as required by the California Building Code (CBC). The testing apparatus is designed to accept a 4-inch diameter, 1-in high, remolded sample. The sample is initially remolded to 50± 1

percent saturation and then loaded with a surcharge equivalent to 144 pounds per square foot. The sample is then inundated with water, and allowed to swell against the surcharge. The resultant swell or consolidation is recorded after a 24-hour period. The results of the EI testing are as follows:

Sample Identification	Expansion Index	Expansive Potential
B-4 @ 0 to 5 feet	0	Non-Expansive

Soluble Sulfates

Representative samples of the near-surface soils were submitted to a subcontracted analytical laboratory for determination of soluble sulfate content. Soluble 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 result of the soluble sulfate testing is presented below, and is discussed further in a subsequent section of this report.

Sample Identification	Soluble Sulfates (%)	ACI 318 Classification
B-1 @ 0 to 5 feet	0.009	Negligible
B-7 @ 0 to 5 feet	0.007	Negligible

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 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. Therefore, the possibility of significant fault rupture on the site is considered to be low.

Seismic Design Parameters

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 2013 edition of the California Building Code (CBC). The 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.

The 2013 CBC Seismic Design Parameters have been generated using U.S. Seismic Design Maps, a web-based software application developed by the United States Geological Survey. This software application, available at the USGS web site, calculates seismic design parameters in accordance with the 2013 CBC, utilizing a database of deterministic site accelerations at 0.01 degree intervals. The table below is a compilation of the data provided by the USGS application. A copy of the output generated from this program is included in Appendix E of this report. A

copy of the Design Response Spectrum, as generated by the USGS application is also included in Appendix E. Based on this output, the following parameters may be utilized for the subject site:

2013 CBC SEISMIC DESIGN PARAMETERS

Parameter		Value
Mapped Spectral Acceleration at 0.2 sec Period	S_S	1.550
Mapped Spectral Acceleration at 1.0 sec Period	S_1	0.671
Site Class	---	D
Site Modified Spectral Acceleration at 0.2 sec Period	S_{MS}	1.550
Site Modified Spectral Acceleration at 1.0 sec Period	S_{M1}	1.006
Design Spectral Acceleration at 0.2 sec Period	S_{DS}	1.033
Design Spectral Acceleration at 1.0 sec Period	S_{D1}	0.671

Liquefaction

Liquefaction is the loss of the 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 grain size 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). Clayey (cohesive) soils or soils which possess clay particles (d<0.005mm) in excess of 20 percent (Seed and Idriss, 1982) are generally not considered to be susceptible to liquefaction, nor are those soils which are above the historic static groundwater table.

The California Geological Survey (CGS) has not yet conducted detailed seismic hazards mapping in the area of the subject site. The general liquefaction susceptibility of the site was determined by research of the San Bernardino County Official Land Use Plan, General Plan, Geologic Hazard Overlay. The map for the Fontana Quadrangle, which is identified as Map No. FH29, indicates that the subject site is not located within a liquefaction hazard zone. Based on the mapping performed by the county of San Bernardino and the subsurface conditions encountered at the boring locations, liquefaction is not considered to be a design concern for this project.

6.2 Geotechnical Design Considerations

General

The majority of the borings encountered disturbed native alluvium at the ground surface, extending to depths of 2½ to 3½± feet. Artificial fill soils were encountered at one of the boring

locations in the southeast potion of the property, extending to a depth of 4½± feet. Both the disturbed alluvium and the artificial fill soils possess variable strengths and densities, and are not considered suitable, in their present state, for the support of the foundations or floor slabs of the new structure. Furthermore, the results of laboratory testing indicate that the fill soils and native alluvium within the upper 5 to 6± feet possess a minor potential for consolidation, and a moderate potential for collapse when inundated with water. Therefore, remedial grading is recommended to remove the disturbed alluvium, artificial fill soils, and the upper portion of the undisturbed native alluvium. These soils may be replaced as engineered fill.

Settlement

The recommended remedial grading will remove the potentially compressible, variable strength undocumented fill soils and a portion of the native alluvial soils, and replace them as compacted structural fill. The native 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 structure. Therefore, following completion of the recommended grading, post-construction settlements are expected to be within tolerable limits.

Expansion

The results of expansion index testing indicates that the near-surface soils possess a very low expansion potential (EI = 0). 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 possess concentrations of soluble sulfates which indicate a negligible potential to attack concrete, in accordance with the American Concrete Institute (ACI) Publication 318-05 Building Code Requirements for Structural Concrete and Commentary, Section 4.3. Therefore, no specialized concrete mix designs are considered warranted, with regard to sulfate protection. We do, however, recommend that additional sulfate testing be conducted after the completion of rough grading.

Shrinkage/Subsidence

Removal and recompaction of the near surface fill soils is estimated to result in an average shrinkage of 12 to 17 percent. 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 boring 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 boring 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

Remnants of two asphaltic concrete streets, concrete slabs, and building foundations are present at the ground surface at the site. Additionally, our review of readily available aerial photographs indicates that the eastern two-thirds of the site was previously developed as a residential subdivision. Initial site preparation should include the demolition of the existing roads, slabs, and foundations. Site demolition should also include any utilities, septic systems, and any other subsurface improvements associated with the previous development of the site. Debris resultant from demolition should be disposed of offsite. Alternatively, concrete and asphalt debris may be crushed to a maximum 2-inch particle size, mixed with the on-site soils, and reused as compacted structural fill. It may also be feasible to crush these materials for use as crushed miscellaneous base (CMB).

Initial site preparation should include stripping of any topsoil, vegetation and organic debris on the site. Based on conditions observed at the time of the subsurface exploration, this will include native grass and weed growth and occasional shrubs and trees. These materials should be disposed of off-site. The actual extent of stripping should be determined in the field by a representative of the geotechnical engineer, based on the organic content and the stability of the encountered materials.

Treatment of Existing Soils: Building Pad

Remedial grading should be performed within the new building pad area, in order to remove the potentially disturbed and collapsible native alluvium and the existing undocumented fill soils. The artificial fill materials extend to depths as great as 4½ feet at one of the boring locations in the southeast portion of the site. Overexcavation within the building pad area is recommended to extend to a minimum depth of at least 3 feet below existing grade. In order to provide for a relatively uniform layer of compacted structural fill, the overexcavation is also recommended to extend to a depth of at least 3 feet below the proposed pad grades. The overexcavation should also extend to a sufficient depth to remove any disturbed soils or artificial fill materials.

Where not encompassed within the general building pad overexcavation, additional overexcavation should be performed within the influence zones of the new foundations, extending to a depth of 3 feet below proposed bearing grade.

The overexcavation area 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 incorporates 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 additional fill materials or 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 moisture conditioned to achieve a moisture content of 2 to 4 percent above optimum moisture content, to a depth of at least 24 inches below the overexcavation subgrade. 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.

Treatment of Existing Soils: Retaining Walls and Site Walls

The existing soils within the areas of proposed retaining walls should be overexcavated to a depth of 3 feet below foundation bearing grade and replaced as compacted structural fill, as discussed above for the proposed building pad. Any undocumented fill soils should also be removed from the retaining wall areas. Subgrade soils in areas of non-retaining site walls should be overexcavated to a depth of 2 feet below proposed bearing grade. In both cases, 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. The previously excavated soils may then be replaced as compacted structural fill.

Treatment of Existing Soils: Parking Areas

Based on economic considerations, overexcavation of the existing soils in the new parking 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 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. The subgrade soils should then be scarified to a depth of 12± inches, moisture conditioned to 2 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 alluvial 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 and drive areas 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 mitigate the extent of potentially compressible soils and undocumented 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 graded in a manner similar to that described for the building area.

Fill Placement

- Fill soils should be placed in thin (6± inches), near-horizontal lifts, moisture conditioned to 2 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 CBC and the grading code of the county of San Bernardino.
- 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.

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 soils should be compacted to at least 90 percent of the ASTM D-1557 maximum dry density. As an alternative, a clean sand (minimum Sand Equivalent of 30) may be placed within trenches and compacted in place (jetting or flooding is not recommended). Compacted trench backfill should conform to the requirements of the local grading code, and more restrictive requirements may be indicated by the county of San Bernardino. 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 generally consist of silty fine sands underlain by well graded sands. These materials will likely be subject to caving within shallow excavations. Where caving occurs within shallow excavations, flattened excavation slopes may be sufficient to provide excavation stability. On a preliminary basis, the inclination of temporary slopes should not exceed 2h:1v. 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. All excavation activities on this site should be conducted in accordance with Cal-OSHA regulations.

Groundwater

The static groundwater table at this site is considered to exist at a depth in excess of 30± feet. Therefore, groundwater is not expected to impact the 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 structural fill soils used to replace existing unsuitable near surface soils. These new structural fill soils are expected to extend to depths of at least 3 feet below proposed foundation bearing grade. These soils will be underlain by 1± foot of additional soil that has been densified and moisture conditioned in place. Based on this subsurface profile, the proposed building may be supported on conventional shallow foundations.

Building Foundation Design Parameters

New square and rectangular footings may be designed as follows:

- Maximum, net allowable soil bearing pressure: 2,500 lbs/ft^2.
- Minimum wall/column footing width: 14 inches/24 inches.
- Minimum longitudinal steel reinforcement within strip footings: Four (4) No. 5 rebars (2 top and 2 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 pressures 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 2 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: 0.30

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 2,500 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 structure may be constructed as a conventional slab-on-grade supported on newly placed structural fill, extending to a depth of at least 3 feet below finished pad grade. Based on geotechnical considerations, the floor slab may be designed as follows:

- Minimum slab thickness: 5 inches.

- Minimum slab reinforcement: Not required for geotechnical considerations. 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 then minimum slab underlayment should consist of a moisture vapor barrier constructed below the entire area of the proposed slab where such floor coverings will be used. 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. 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. Where moisture sensitive floor coverings are not anticipated, the vapor barrier may be omitted.

- Moisture condition the floor slab subgrade soils to 2 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 small (less than 3 to 5± feet in height) retaining walls may be required to facilitate the new site grades. It is also expected that some retaining walls will be required in the new loading dock areas. The parameters recommended for use in the design of these walls are presented below.

Retaining Wall Design Parameters

Based on the soil conditions encountered at the boring locations, the following parameters may be used in the design of new retaining walls for this site. The following parameters assume that only the on-site soils will be utilized for retaining wall backfill. The near surface soils generally consist of silty fine sands underlain by well graded sands. Based on their composition, the on-site soils have been assigned a friction angle of 30 degrees.

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 Silty Sands and Sands
Internal Friction Angle (ϕ)		30°
Unit Weight		125 lbs/ft^3
Equivalent Fluid Pressure:	Active Condition (level backfill)	42 lbs/ft^3
	Active Condition (2h:1v backfill)	67 lbs/ft^3
	At-Rest Condition (level backfill)	63 lbs/ft^3

Regardless of the backfill type, the walls should be designed using a soil-footing coefficient of friction of 0.30 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.

Retaining Wall Foundation Design

The retaining wall foundations should be supported within newly placed compacted structural fill, extending to a depth of at least 2 feet below the proposed bearing grade. 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

It is recommended that a prefabricated drainage composite such as the MiraDRAIN 6000XL (or approved equivalent), which is specifically designed for use behind retaining walls, be placed against the face of the retaining walls. The drainage composite should be installed in accordance with the manufacturer's specifications and extend from the top of the retaining wall footing to within 1 foot of the ground surface on the back side of the retaining wall. If the backfill soils are 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.

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.

Seismic Lateral Earth Pressures

In accordance with the 2013 CBC, any walls retaining 6 or more feet (in height) of soil must be designed for seismic lateral earth pressures. If walls retaining 6 feet or more are required for this site, the geotechnical engineer should be contacted for supplementary seismic lateral earth pressure recommendations.

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 near surface soils generally consist of silty fine sands underlain by well graded sands. Based on their classification, these materials are expected to possess good pavement support characteristics, with R-values in the range of 50 to 60. Since R-value testing was not included in the scope of services for this project, the subsequent pavement design is based upon a conservatively assumed R-value of 50. 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 = 50)					
Materials	**Thickness (inches)**				
	Auto Parking and Drives (TI = 4.0 & 5.0)	Light Truck Traffic (TI = 6.0)	Moderate Truck Traffic (TI = 7.0)	Heavy Truck Traffic (TI = 8.0)	Heavy Truck Traffic (TI = 9.0)
Asphalt Concrete	3	3½	4	5	6
Aggregate Base	3	4	5	5	6
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 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 & Light Truck Traffic (TI = 5.0 & 6.0)	Moderate Truck Traffic (TI =7.0)	Heavy Truck Traffic (TI =8.0)	Heavy Truck Traffic (TI =9.0)
PCC	5	6½	8	9
Compacted Subgrade	12	12	12	12

The concrete should have a 28-day compressive strength of at least 3,000 psi. The maximum joint spacing within all of the PCC pavements is recommended to be equal to or less than 30 times the pavement thickness.

7.0 GENERAL COMMENTS

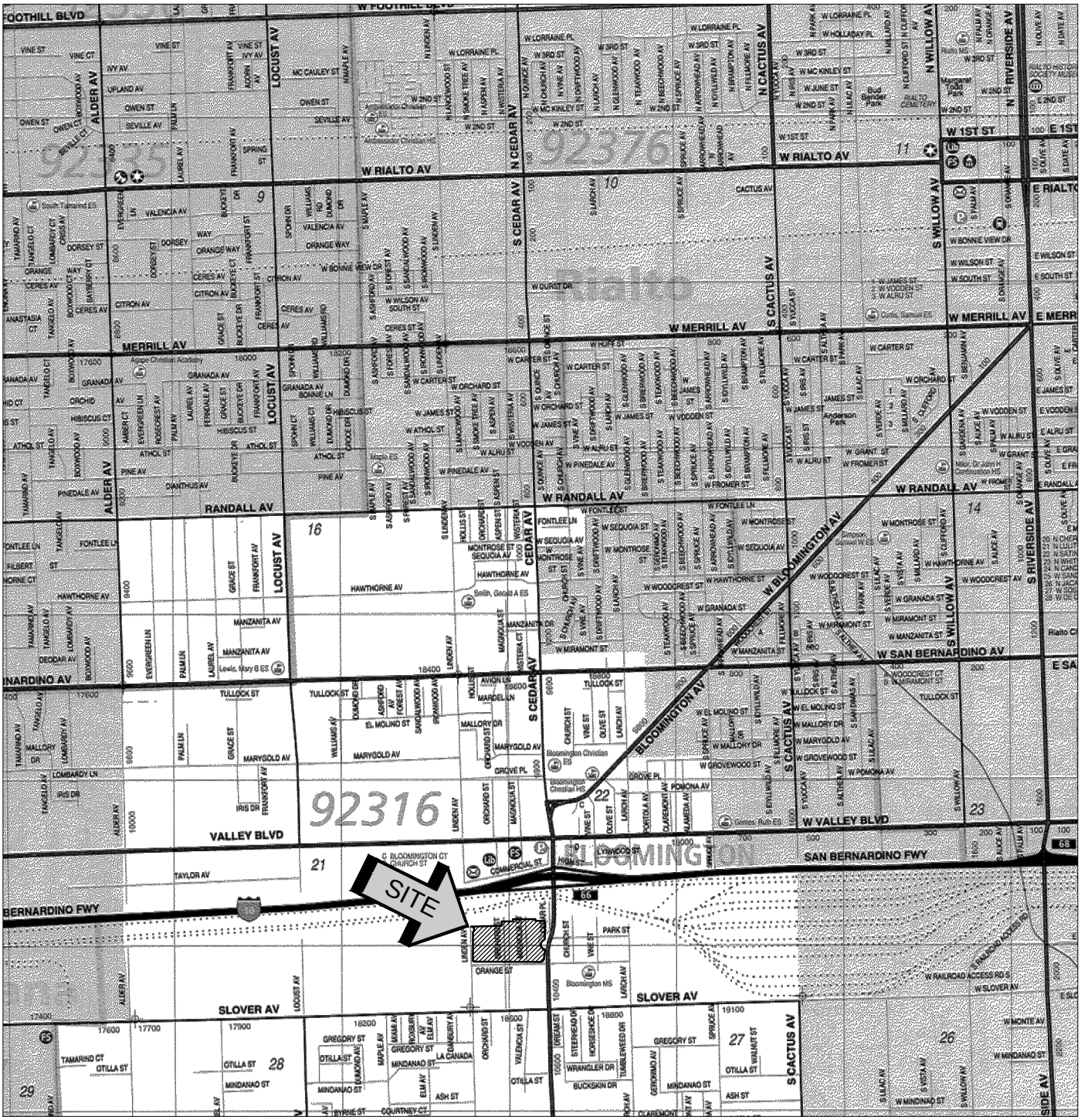
This report has been prepared as an instrument of service for use by the client, in order to aid in the evaluation of this property and to assist the architects and engineers in the design and preparation of the project plans and specifications. This report may be provided to the contractor(s) and other 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 boring 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.

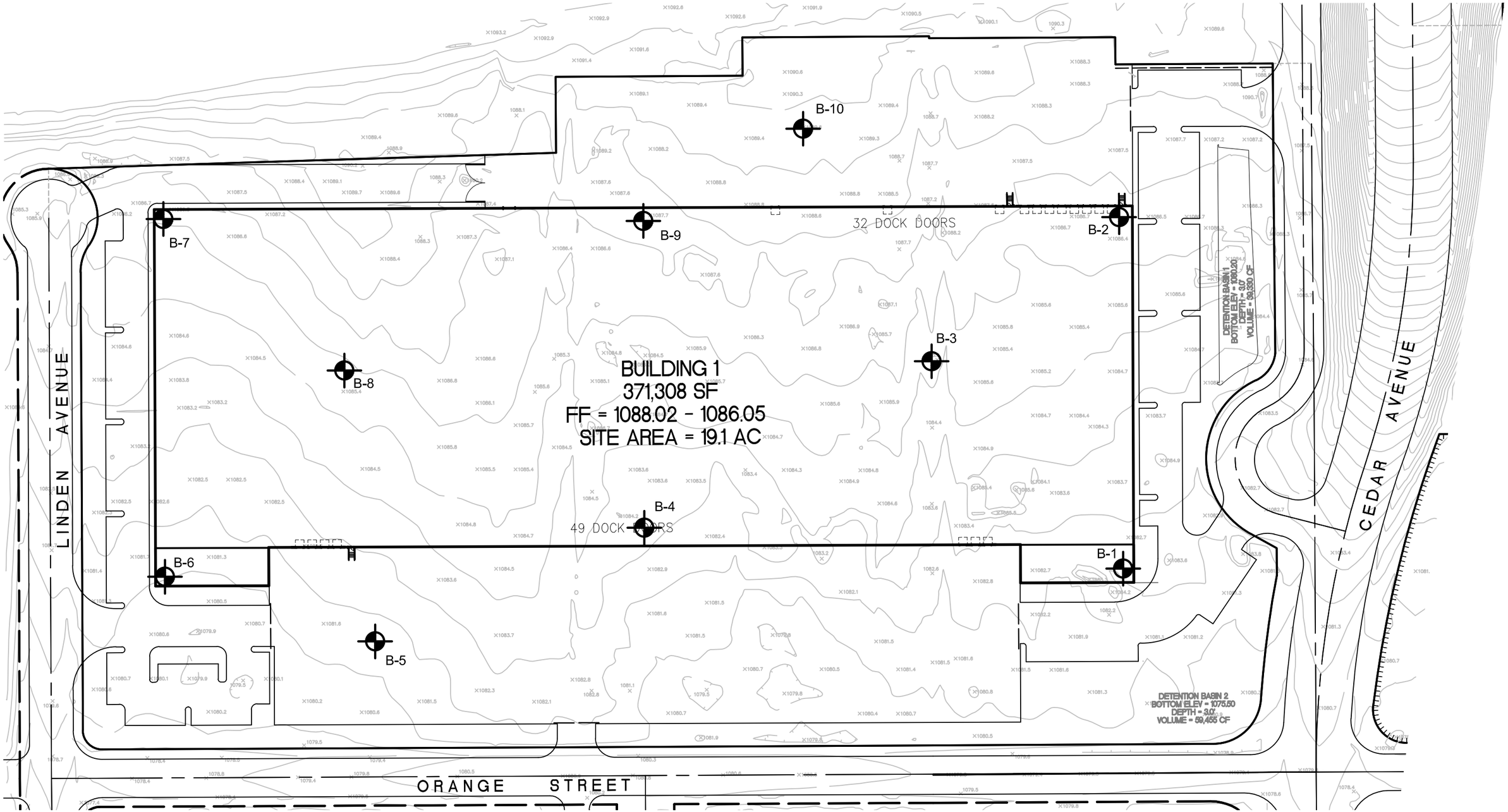
APPENDIX A



SOURCE: SAN BERNARDINO COUNTY
THOMAS GUIDE, 2013

ITE LOCATION MAP
PROPOSED COMMERCIAL/INDUSTRIAL BUILDING
(SAN BERNARDINO COUNTY), CALIFORNIA
SCALE: 1" = 2400'
DRAWN: KVC CHKD: JAS
SCG PROJECT 14G190-1
PLATE

SOUTHERN CALIFORNIA GEOTECHNICAL



GEOTECHNICAL LEGEND

APPROXIMATE BORING LOCATION

NORTH

NOTE: BASE MAP PREPARED BY HALL & FOREMAN, INC.

BORING LOCATION PLAN

PROPOSED COMMERCIAL/INDUSTRIAL BUILDING

SAN BERNARDINO COUNTY, CALIFORNIA

SCALE: 1" = 100'

DRAWN: RF
CHKD: JAS

SCG PROJECT
14G190-1

PLATE 2

SOUTHERN CALIFORNIA GEOTECHNICAL

APPENDIX B

BORING LOG LEGEND

SAMPLE TYPE	GRAPHICAL SYMBOL	SAMPLE DESCRIPTION
AUGER		SAMPLE COLLECTED FROM AUGER CUTTINGS, NO FIELD MEASUREMENT OF SOIL STRENGTH. (DISTURBED)
CORE		ROCK CORE SAMPLE: TYPICALLY TAKEN WITH A DIAMOND-TIPPED CORE BARREL. TYPICALLY USED ONLY IN HIGHLY CONSOLIDATED BEDROCK.
GRAB		SOIL SAMPLE TAKEN WITH NO SPECIALIZED EQUIPMENT, SUCH AS FROM A STOCKPILE OR THE GROUND SURFACE. (DISTURBED)
CS		CALIFORNIA SAMPLER: 2-1/2 INCH I.D. SPLIT BARREL SAMPLER, LINED WITH 1-INCH HIGH BRASS RINGS. DRIVEN WITH SPT HAMMER. (RELATIVELY UNDISTURBED)
NSR		NO RECOVERY: THE SAMPLING ATTEMPT DID NOT RESULT IN RECOVERY OF ANY SIGNIFICANT SOIL OR ROCK MATERIAL.
SPT		STANDARD PENETRATION TEST: SAMPLER IS A 1.4 INCH INSIDE DIAMETER SPLIT BARREL, DRIVEN 18 INCHES WITH THE SPT HAMMER. (DISTURBED)
SH		SHELBY TUBE: TAKEN WITH A THIN WALL SAMPLE TUBE, PUSHED INTO THE SOIL AND THEN EXTRACTED. (UNDISTURBED)
VANE		VANE SHEAR TEST: SOIL STRENGTH OBTAINED USING A 4 BLADED SHEAR DEVICE. TYPICALLY USED IN SOFT CLAYS-NO SAMPLE RECOVERED.

COLUMN DESCRIPTIONS

DEPTH: Distance in feet below the ground surface.

SAMPLE: Sample Type as depicted above.

BLOW COUNT: Number of blows required to advance the sampler 12 inches using a 140 lb hammer with a 30-inch drop. 50/3" indicates penetration refusal (>50 blows) at 3 inches. WH indicates that the weight of the hammer was sufficient to push the sampler 6 inches or more.

POCKET PEN.: Approximate shear strength of a cohesive soil sample as measured by pocket penetrometer.

GRAPHIC LOG: Graphic Soil Symbol as depicted on the following page.

DRY DENSITY: Dry density of an undisturbed or relatively undisturbed sample in lbs/ft^3.

MOISTURE CONTENT: Moisture content of a soil sample, expressed as a percentage of the dry weight.

LIQUID LIMIT: The moisture content above which a soil behaves as a liquid.

PLASTIC LIMIT: The moisture content above which a soil behaves as a plastic.

PASSING #200 SIEVE: The percentage of the sample finer than the #200 standard sieve.

UNCONFINED SHEAR: The shear strength of a cohesive soil sample, as measured in the unconfined state.

SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVEL AND GRAVELLY SOILS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
				GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
		GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	CLEAN SANDS (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
				SP	POORLY-GRADED SANDS, GRAVELLY SAND, LITTLE OR NO FINES
		SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	SILTY SANDS, SAND - SILT MIXTURES
				SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY
				OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-1

JOB NO.: 14G190
PROJECT: Proposed C/I Bldg
LOCATION: San Bernardino County, California

DRILLING DATE: 9/8/14
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Eric Torres

WATER DEPTH: n/a
CAVE DEPTH: 20 feet
READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
	X	7			DISTURBED ALLUVIUM: Brown Silty fine Sand, trace coarse Sand, trace fine Gravel, loose-dry to damp		2					
	X	20			ALLUVIUM: Brown Silty fine Sand, trace medium to coarse Sand, trace fine Gravel, medium dense-damp		4					
5												
	X	17			Gray Brown to Brown fine to coarse Sand, trace to little fine to coarse Gravel, trace Silt, occasional Cobbles, medium dense-dry to damp		2					
	X	26					2					
10												
	X	45			@ 13½ to 15 feet, dense		2					
15												
					Brown Silty fine Sand, medium dense-moist							
	X	16					10					
20												
					Gray Brown fine to medium Sand, trace fine to coarse Gravel, trace coarse Sand, medium dense-damp							
	X	23					3					
25												
					Boring Terminated at 25'							

TBL 14G190.GPJ SOCALGEO.GDT 10/17/14

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-2

JOB NO.: 14G190
PROJECT: Proposed C/I Bldg
LOCATION: San Bernardino County, California

DRILLING DATE: 9/8/14
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Eric Torres

WATER DEPTH: n/a
CAVE DEPTH: 20 feet
READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
		19			ALLUVIUM: Brown Silty fine Sand, trace fine Gravel, medium dense-dry to damp @ 1 to 2 feet, slightly porous	108	2					
		36				108	3					
5		36			Brown fine to coarse Sand, little to some fine to coarse Gravel, trace Silt, medium dense-dry	124	1					
		35			Gray Brown fine to coarse Sand, little fine to coarse Gravel, trace Silt, occasional Cobbles, medium dense to dense-dry to damp	115	3					
10		42				120	4					
15		31					2					
20		26					3					
					Gray Brown fine to medium Sand, trace coarse Sand, trace fine Gravel, medium dense to dense-damp							
25		28					3					
30		31					3					
					Boring Terminated at 30'							

TBL 14G190.GPJ SOCALGEO.GDT 10/17/14

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-3

JOB NO.: 14G190
PROJECT: Proposed C/I Bldg
LOCATION: San Bernardino County, California

DRILLING DATE: 9/8/14
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Eric Torres

WATER DEPTH: n/a
CAVE DEPTH: 10 feet
READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					3± inches Asphaltic concrete, no discernible Aggregate base							
	X	9			FILL: Dark Red Brown to Brown Silty fine Sand, trace fine Gravel, trace Asphalt fragments, loose-damp to moist	109	6					
	X	23			FILL: Brown Gravelly fine to coarse Sand, little Silt, medium dense-damp	116	4					
5	X	14			ALLUVIUM: Gray Brown Gravelly fine to coarse Sand, trace Silt, loose-dry to damp	113	3					
	X	38			@ 7 to 15 feet, medium dense to dense	118	3					
10	X	46				120	2					
15	X	29					3					
					Boring Terminated at 15'							

TBL 14G190.GPJ SOCALGEO.GDT 10/17/14

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-4

JOB NO.: 14G190
PROJECT: Proposed C/I Bldg
LOCATION: San Bernardino County, California

DRILLING DATE: 9/8/14
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Eric Torres

WATER DEPTH: n/a
CAVE DEPTH: 6 feet
READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
		8			DISTURBED ALLUVIUM: Brown Silty fine Sand, trace fine Gravel, loose-dry to damp		2					
		16			ALLUVIUM: Brown Silty fine to coarse Sand, trace to little fine Gravel, medium dense-dry to damp		2					
5		11					2					
		9			Gray Brown Silty fine Sand, loose-moist		12					
10		36			Gray Brown Silty fine Sand, trace medium to coarse Sand, trace fine Gravel, medium dense-moist	117	8					
		27			Gray Brown fine to coarse Sand, little to some fine to coarse Gravel, medium dense to dense-dry to damp		3					
15												
		39					2					
20					Boring Terminated at 20'							

TBL 14G190.GPJ SOCALGEO.GDT 10/17/14

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-5

JOB NO.: 14G190
PROJECT: Proposed C/I Bldg
LOCATION: San Bernardino County, California

DRILLING DATE: 9/8/14
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Eric Torres

WATER DEPTH: n/a
CAVE DEPTH: n/a
READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
	X	10			DISTURBED ALLUVIUM: Brown Silty fine Sand, loose to medium dense-dry		1					
	X	17			ALLUVIUM: Brown Silty fine Sand, trace medium to coarse Sand, trace fine to coarse Gravel, medium dense-dry		1					
5					Boring Terminated at 5'							

TBL 14G190.GPJ SOCALGEO.GDT 10/17/14

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-6

JOB NO.: 14G190
PROJECT: Proposed C/I Bldg
LOCATION: San Bernardino County, California

DRILLING DATE: 9/8/14
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Eric Torres

WATER DEPTH: n/a
CAVE DEPTH: 20 feet
READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
		17			DISTURBED ALLUVIUM: Brown Silty fine Sand, trace medium to coarse Sand, trace fine Gravel, medium dense-dry	114	1					
		34			ALLUVIUM: Gray Brown to Brown Silty fine Sand, trace medium to coarse Sand, trace fine to coarse Gravel, occasional Cobbles, medium dense-dry to damp	119	2					
5		45			Gray Brown fine to coarse Sand, trace fine to coarse Gravel, trace Silt, occasional Cobbles, medium dense to dense-damp to moist	117	2					
		40					7					
10		57			Gray Brown fine to medium Sand, trace fine to corase Gravel, dense-dry to damp	127	2					
15		17			Gray Brown fine Sand, trace medium Sand, medium dense-damp		5					
20		16					6					
25		32			Gray Brown fine to medium Sand, trace coarse Sand, trace fine to coarse Gravel, dense-damp		3					
30		18			Gray fine Sand, trace Silt, medium dense-moist		8					
					Boring Terminated at 30'							

TBL 14G190.GPJ SOCALGEO.GDT 10/17/14

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-7

JOB NO.: 14G190
PROJECT: Proposed C/I Bldg
LOCATION: San Bernardino County, California

DRILLING DATE: 9/8/14
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Eric Torres

WATER DEPTH: n/a
CAVE DEPTH: 17 feet
READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
		12			DISTURBED ALLUVIUM: Brown Silty fine Sand, trace fine Gravel, medium dense-dry to damp		2					
		19			ALLUVIUM: Gray Brown fine to coarse Sand, trace fine to coarse Gravel, trace Silt, medium dense-dry to damp		2					
5												
		24			Gray Brown Gravelly fine to coarse Sand, medium dense-dry to damp		2					
		24					2					
10												
		19					2					
15					Gray Brown fine to medium Sand, trace coarse Sand, medium dense-dry to damp		2					
					Gray Brown fine Sandy Silt, trace Iron oxide staining, medium dense-very moist							
		18					41					
20												
					Gray Brown to Brown Gravelly fine to coarse Sand, trace Silt, medium dense-damp							
		19					3					
					Brown Silty fine Sand, medium dense-moist		9					
25												
					Boring Terminated at 25'							

TBL 14G190.GPJ SOCALGEO.GDT 10/17/14

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-8

JOB NO.: 14G190
PROJECT: Proposed C/I Bldg
LOCATION: San Bernardino County, California

DRILLING DATE: 9/8/14
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Eric Torres

WATER DEPTH: n/a
CAVE DEPTH: 6 feet
READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
		29			DISTURBED ALLUVIUM: Dark Brown Silty fine Sand, trace fine Gravel, medium dense-dry	112	1					
		32			ALLUVIUM: Brown Silty fine Sand, trace fine Gravel, slightly porous, medium dense-damp	108	4					
5		31				106	3					
		35				117	3					
		30			Gray Brown Gravelly fine to coarse Sand, medium dense-dry to damp @ 9 to 10 feet, damp to moist	113	2					
10												
		56			Gray Brown fine to coarse Sand, litlte to some fine to coarse Gravel, very dense-dry to damp		2					
15												
					Boring Terminated at 15'							

TBL 14G190.GPJ SOCALGEO.GDT 10/17/14

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-9

JOB NO.: 14G190
PROJECT: Proposed C/I Bldg
LOCATION: San Bernardino County, California

DRILLING DATE: 9/8/14
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Eric Torres

WATER DEPTH: n/a
CAVE DEPTH: 13 feet
READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
	X	17	1.0		ALLUVIUM: Light Brown Silty fine Sand, trace coarse Sand, slightly porous, medium dense-damp		4					
	X	22			Gray Brown Gravelly fine to coarse Sand, medium dense to dense-dry to damp		2					
5												
	X	38					2					
	X	21					2					
10												
					Gray Brown fine to coarse Sand, trace fine Gravel, medium dense-dry to damp							
	X	25					2					
15												
					Gray Brown Gravelly fine to coarse Sand, medium dense-dry to damp							
	X	20					2					
20												
					Boring Terminated at 20'							

TBL 14G190.GPJ SOCALGEO.GDT 10/17/14

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-10

JOB NO.: 14G190
PROJECT: Proposed C/I Bldg
LOCATION: San Bernardino County, California

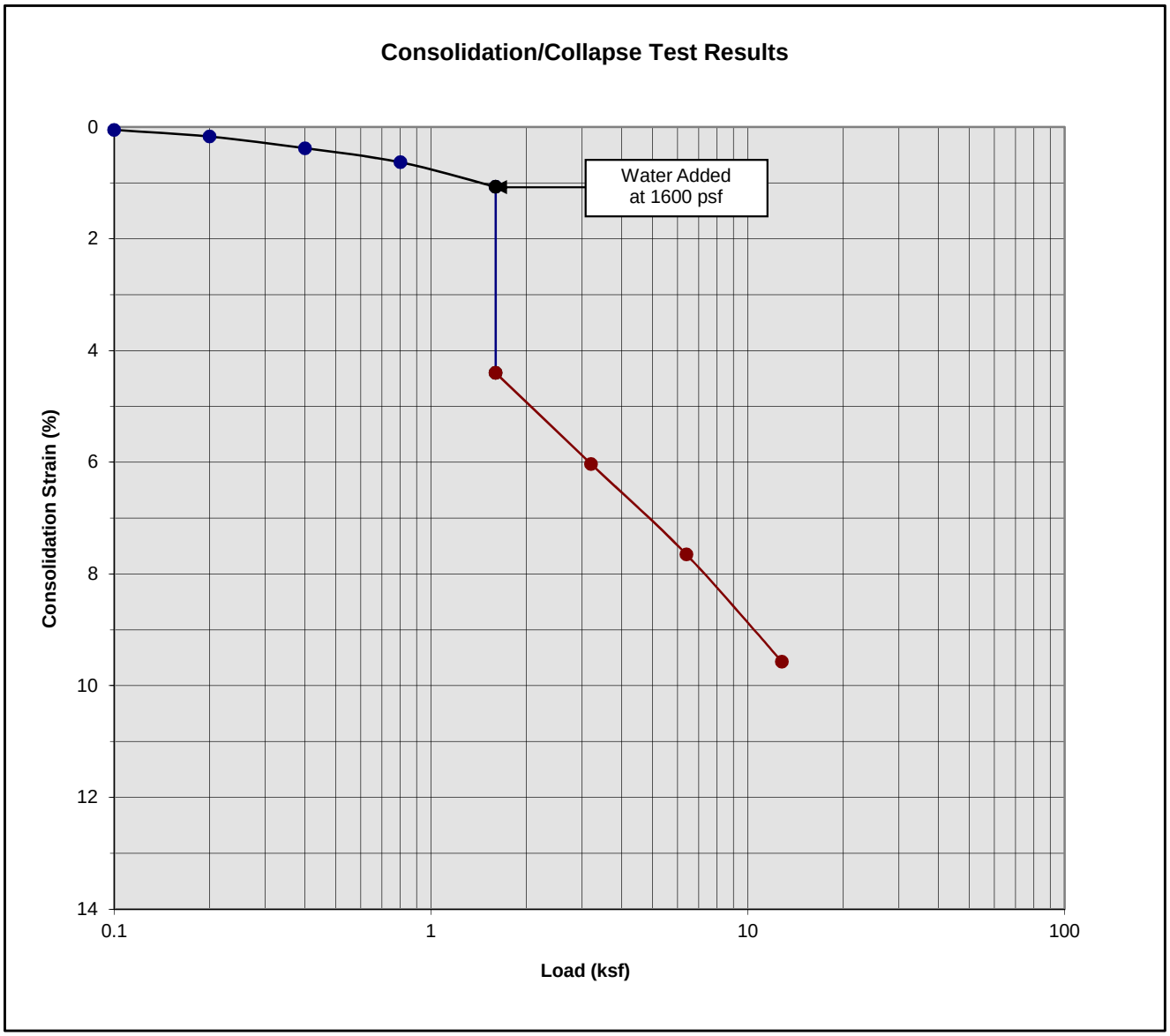
DRILLING DATE: 9/8/14
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Eric Torres

WATER DEPTH: n/a
CAVE DEPTH: n/a
READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
	X	18			ALLUVIUM: Brown Silty fine Sand, medium dense-damp		3					
	X	14			@ 3½ to 5 feet, trace fine Gravel		3					
5					Boring Terminated at 5'							

TBL 14G190.GPJ SOCALGEO.GDT 10/17/14

APPENDIX C

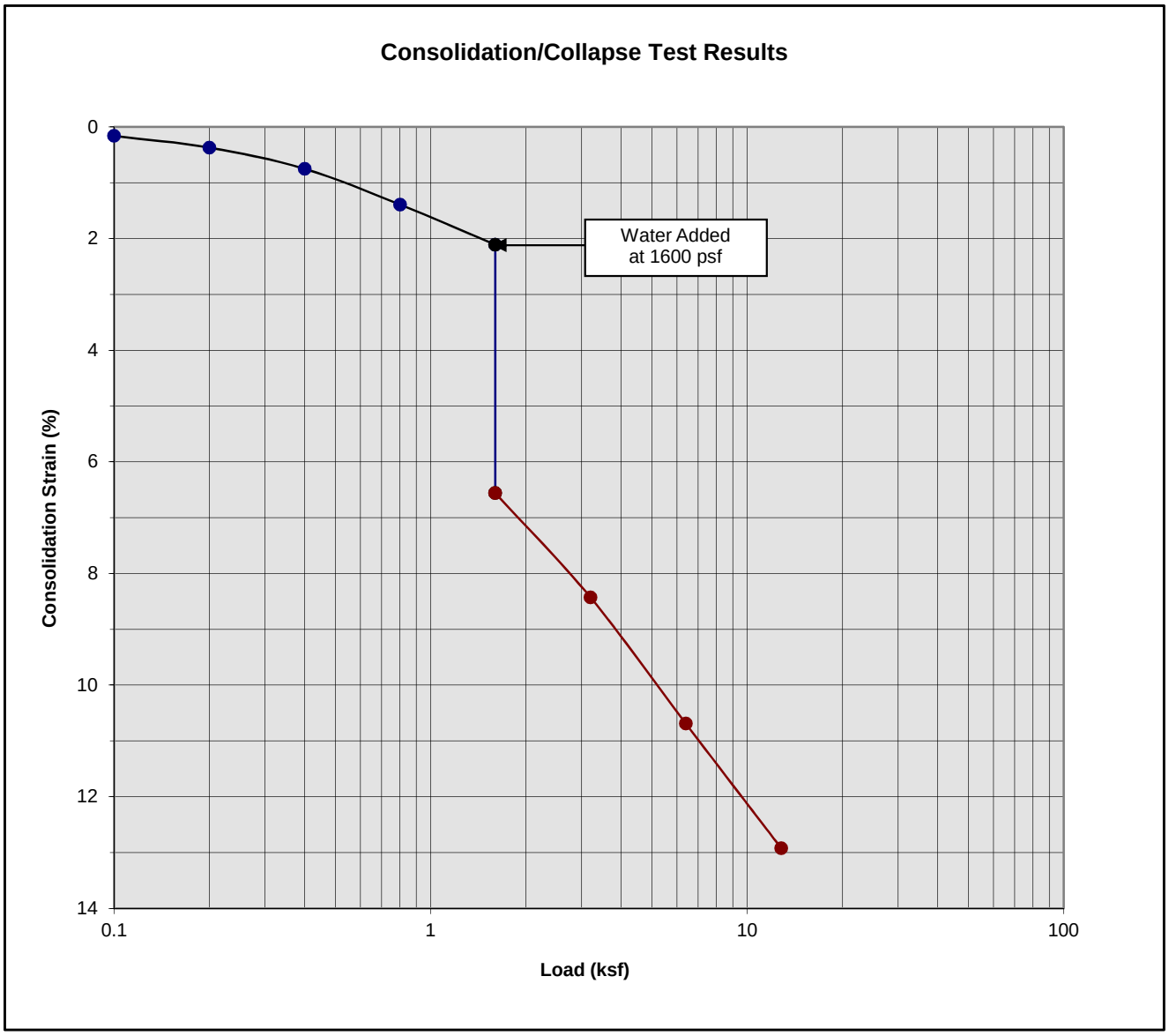


Classification: FILL: Dark Red Brown to Brown Silty fine Sand

Boring Number:	B-3	Initial Moisture Content (%)	6
Sample Number:	---	Final Moisture Content (%)	16
Depth (ft)	1 to 2	Initial Dry Density (pcf)	108.9
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	119.7
Specimen Thickness (in)	1.0	Percent Collapse (%)	3.33

Proposed C/I Bldg
San Bernardino County, California
Project No. 14G190
PLATE C- 1

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

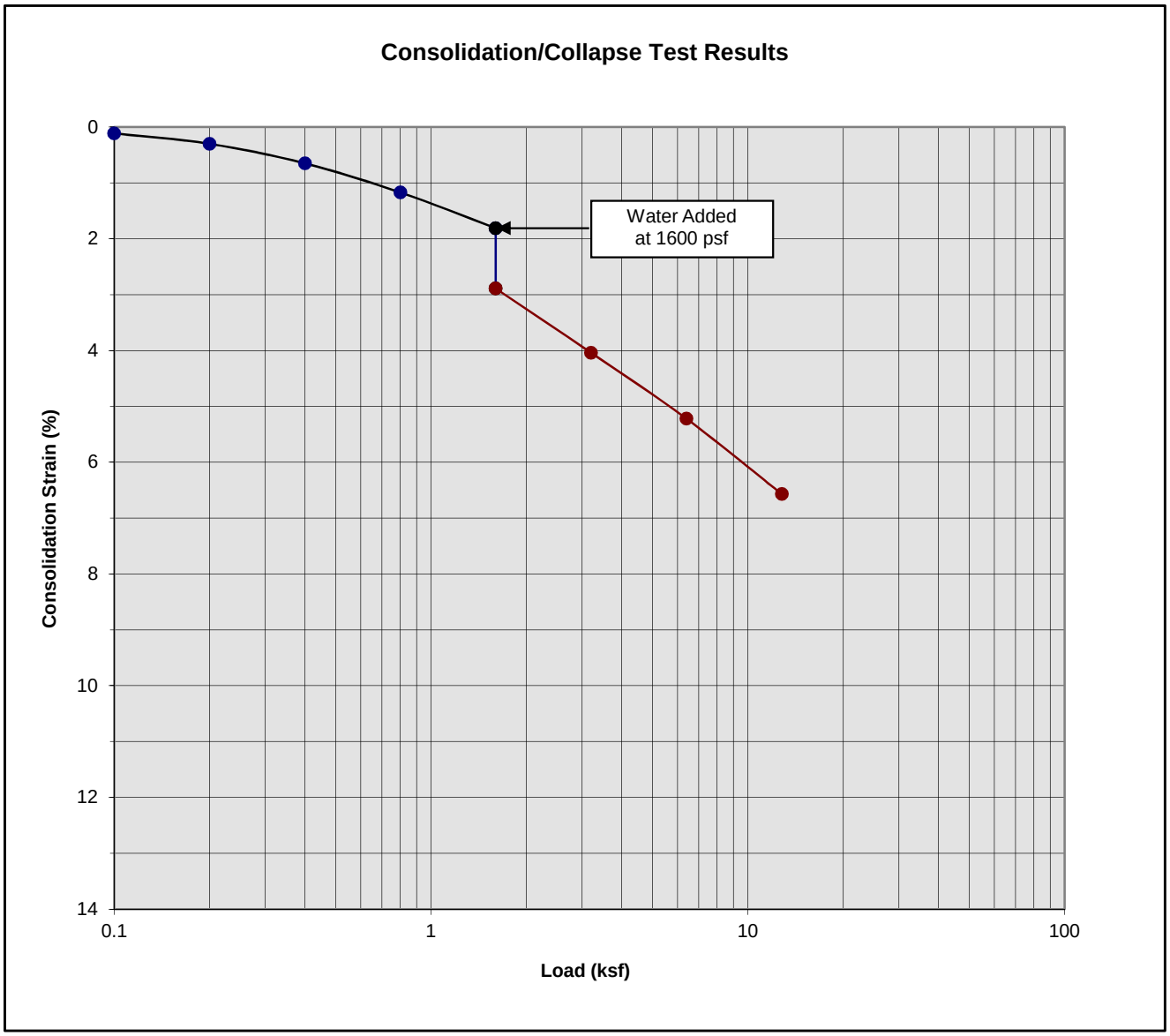


Classification: FILL: Brown Gravelly fine to coarse Sand, little to some Silt

Boring Number:	B-3	Initial Moisture Content (%)	4
Sample Number:	---	Final Moisture Content (%)	11
Depth (ft)	3 to 4	Initial Dry Density (pcf)	115.8
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	132.8
Specimen Thickness (in)	1.0	Percent Collapse (%)	4.45

Proposed C/I Bldg
San Bernardino County, California
Project No. 14G190
PLATE C- 2

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

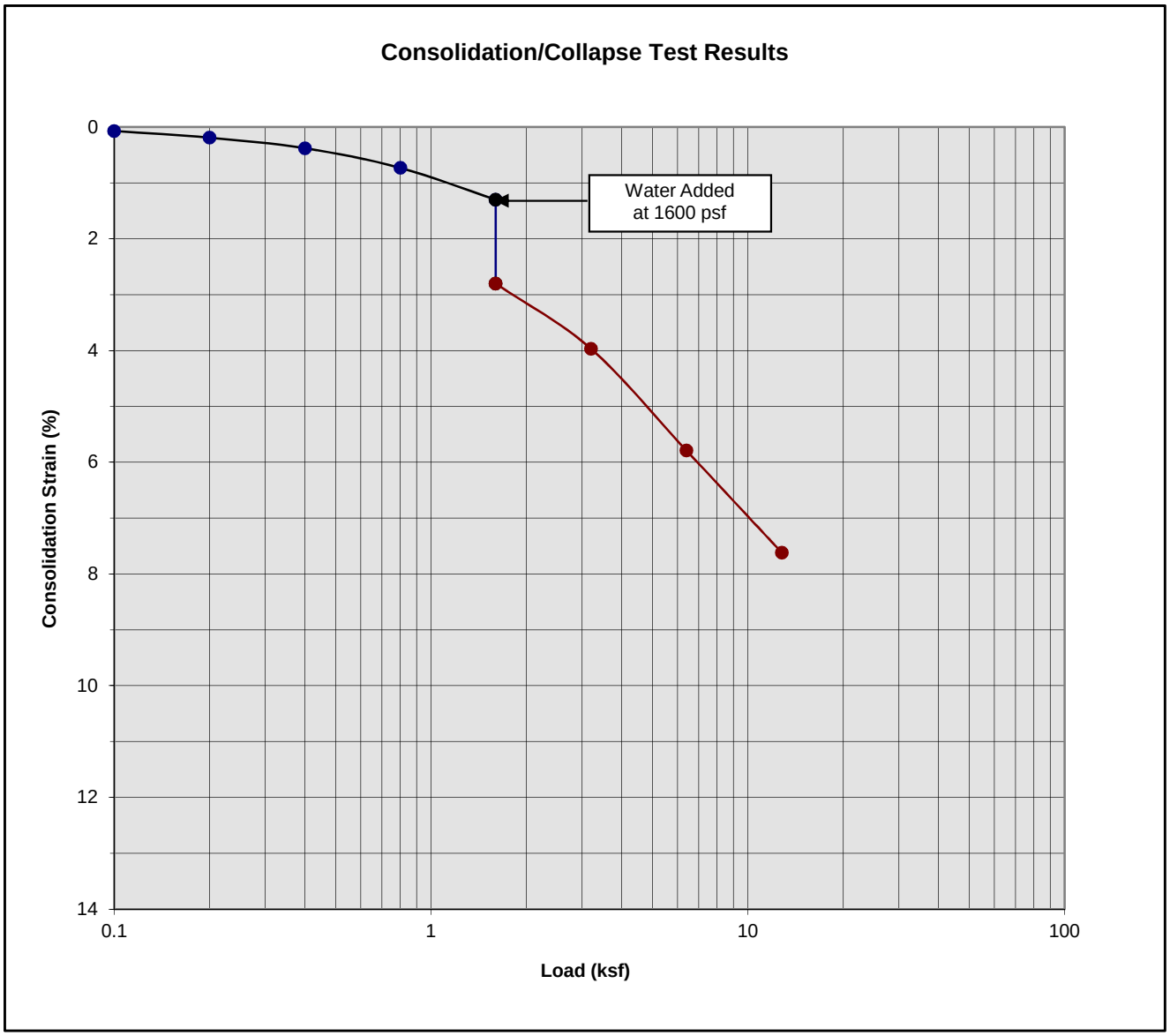


Classification: Gray Brown Gravelly fine to coarse Sand, trace Silt

Boring Number:	B-3	Initial Moisture Content (%)	3
Sample Number:	---	Final Moisture Content (%)	12
Depth (ft)	5 to 6	Initial Dry Density (pcf)	112.6
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	120.6
Specimen Thickness (in)	1.0	Percent Collapse (%)	1.08

Proposed C/I Bldg
San Bernardino County, California
Project No. 14G190
PLATE C- 3

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

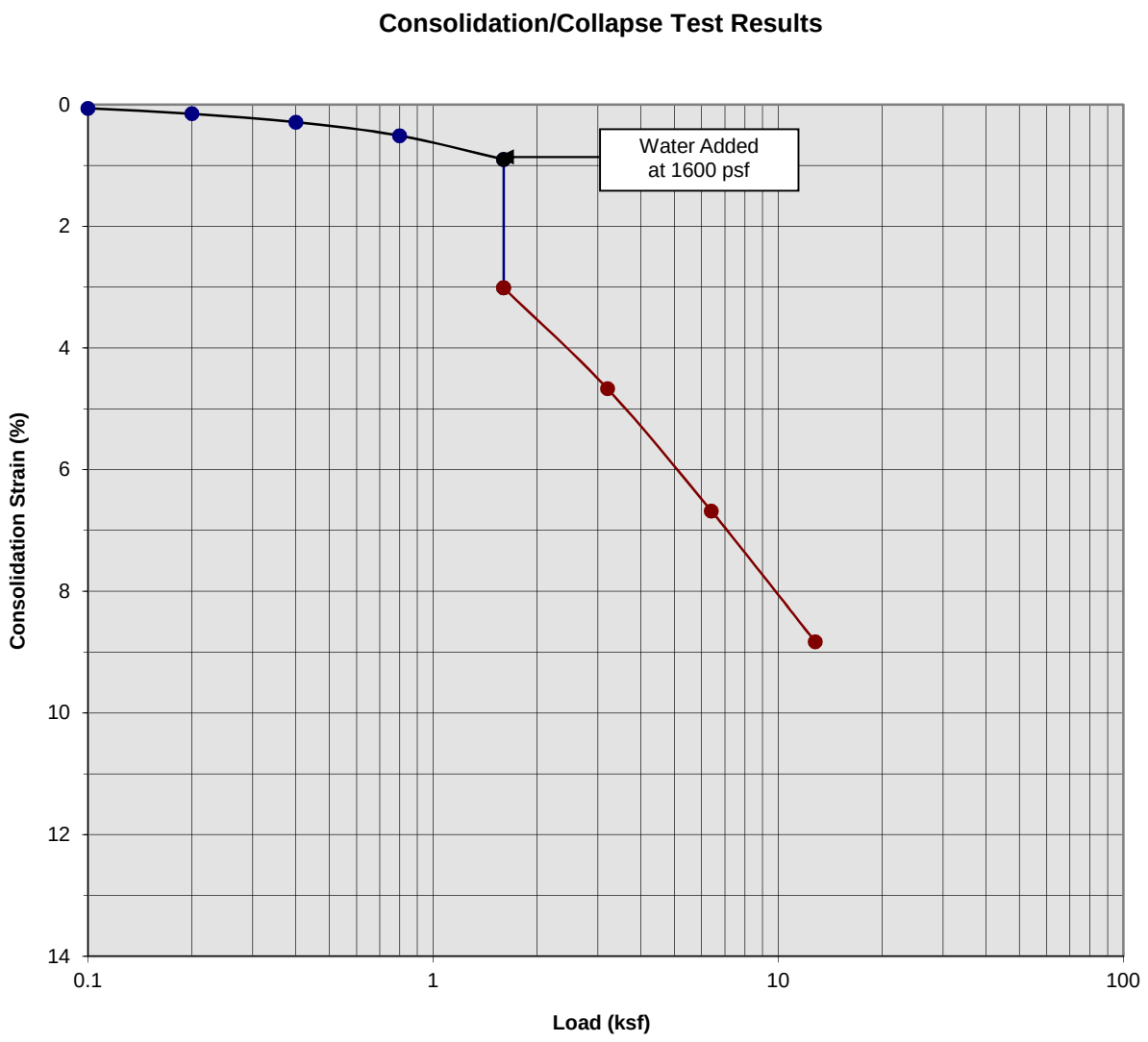


Classification: Gray Brown Gravelly fine to coarse Sand, trace Silt

Boring Number:	B-3	Initial Moisture Content (%)	3
Sample Number:	---	Final Moisture Content (%)	11
Depth (ft)	7 to 8	Initial Dry Density (pcf)	117.5
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	127.2
Specimen Thickness (in)	1.0	Percent Collapse (%)	1.50

Proposed C/I Bldg
San Bernardino County, California
Project No. 14G190
PLATE C- 4

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

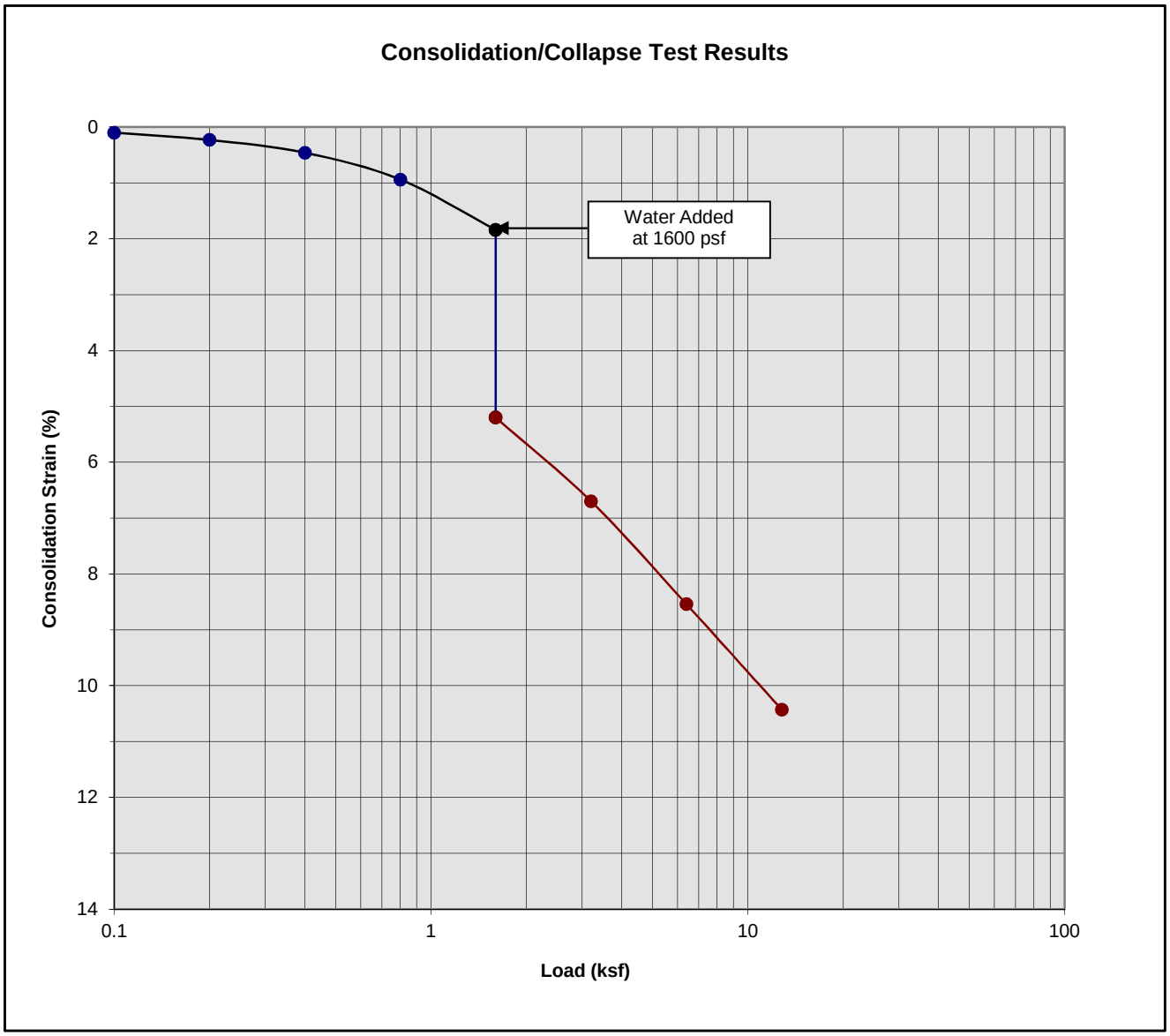


Classification: Dark Brown Silty fine Sand, trace fine Gravel

Boring Number:	B-8	Initial Moisture Content (%)	1
Sample Number:	---	Final Moisture Content (%)	16
Depth (ft)	1 to 2	Initial Dry Density (pcf)	112.7
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	123.6
Specimen Thickness (in)	1.0	Percent Collapse (%)	2.11

Proposed C/I Bldg
San Bernardino County, California
Project No. 14G190
PLATE C- 5

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

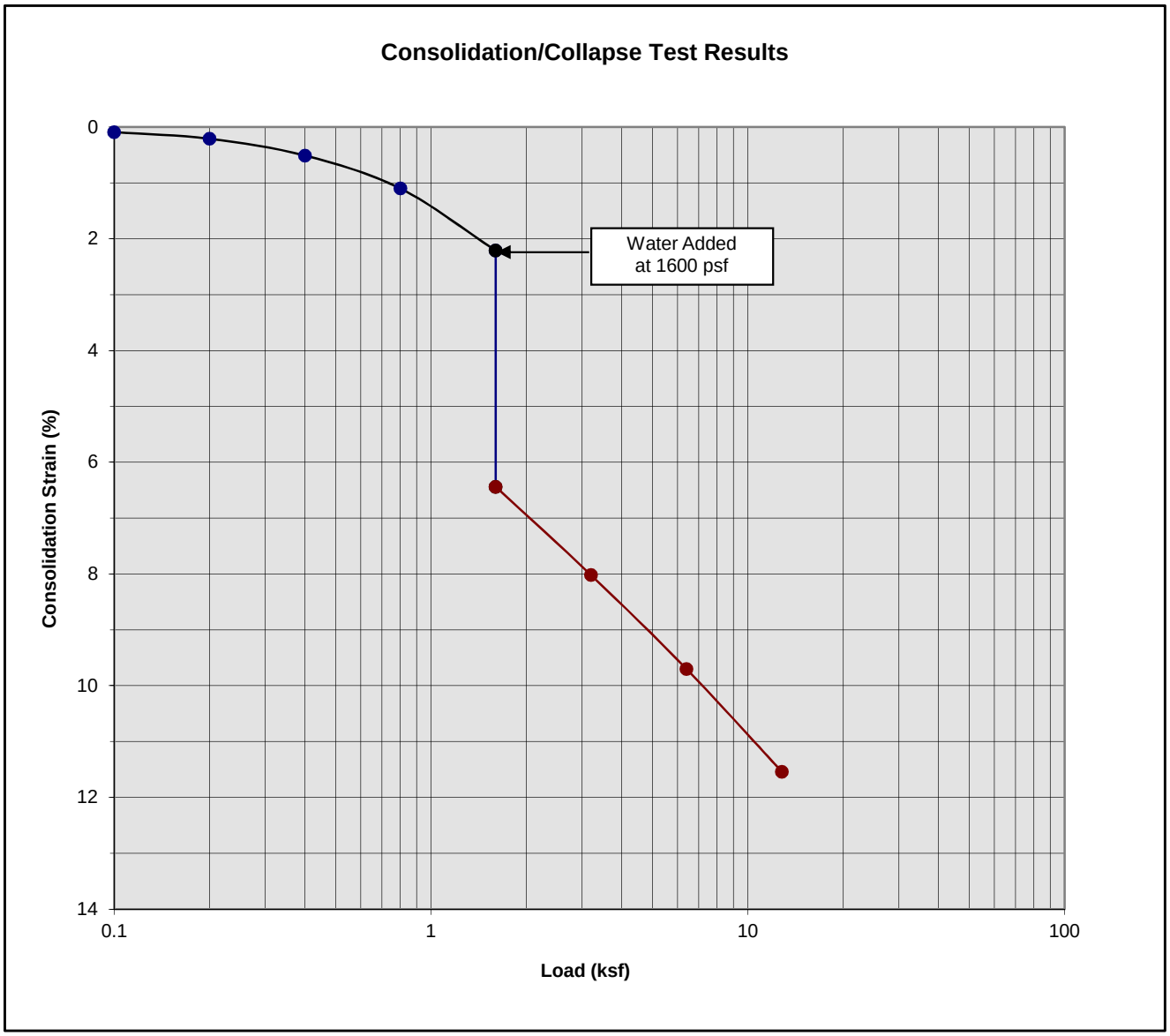


Classification: Brown Silty fine Sand, trace fine Gravel

Boring Number:	B-8	Initial Moisture Content (%)	4
Sample Number:	---	Final Moisture Content (%)	17
Depth (ft)	3 to 4	Initial Dry Density (pcf)	108.2
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	119.5
Specimen Thickness (in)	1.0	Percent Collapse (%)	3.36

Proposed C/I Bldg
San Bernardino County, California
Project No. 14G190
PLATE C- 6

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

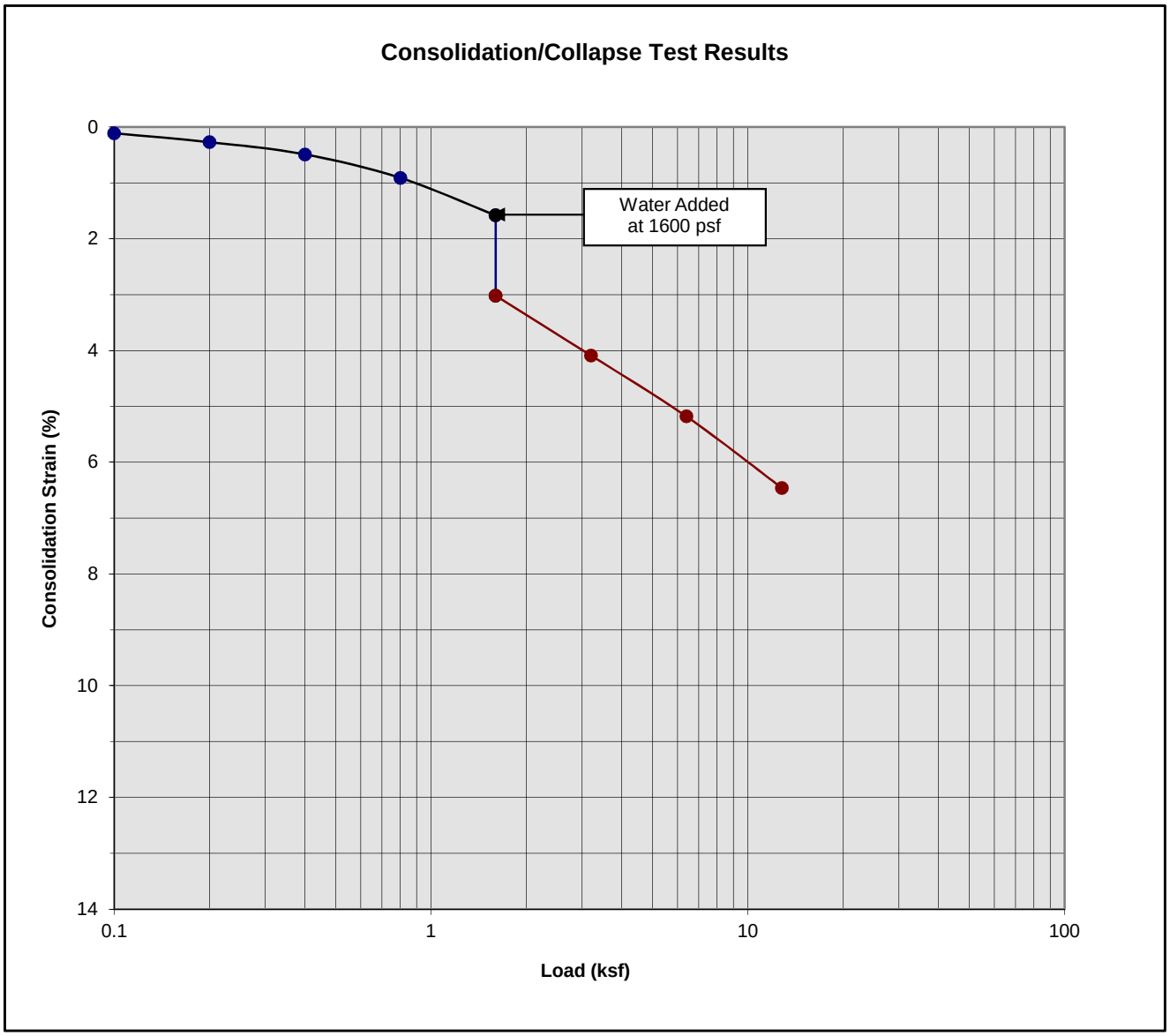


Classification: Brown Silty fine Sand, trace fine Gravel

Boring Number:	B-8	Initial Moisture Content (%)	3
Sample Number:	---	Final Moisture Content (%)	16
Depth (ft)	5 to 6	Initial Dry Density (pcf)	105.5
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	119.2
Specimen Thickness (in)	1.0	Percent Collapse (%)	4.23

Proposed C/I Bldg
San Bernardino County, California
Project No. 14G190
PLATE C- 7

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation



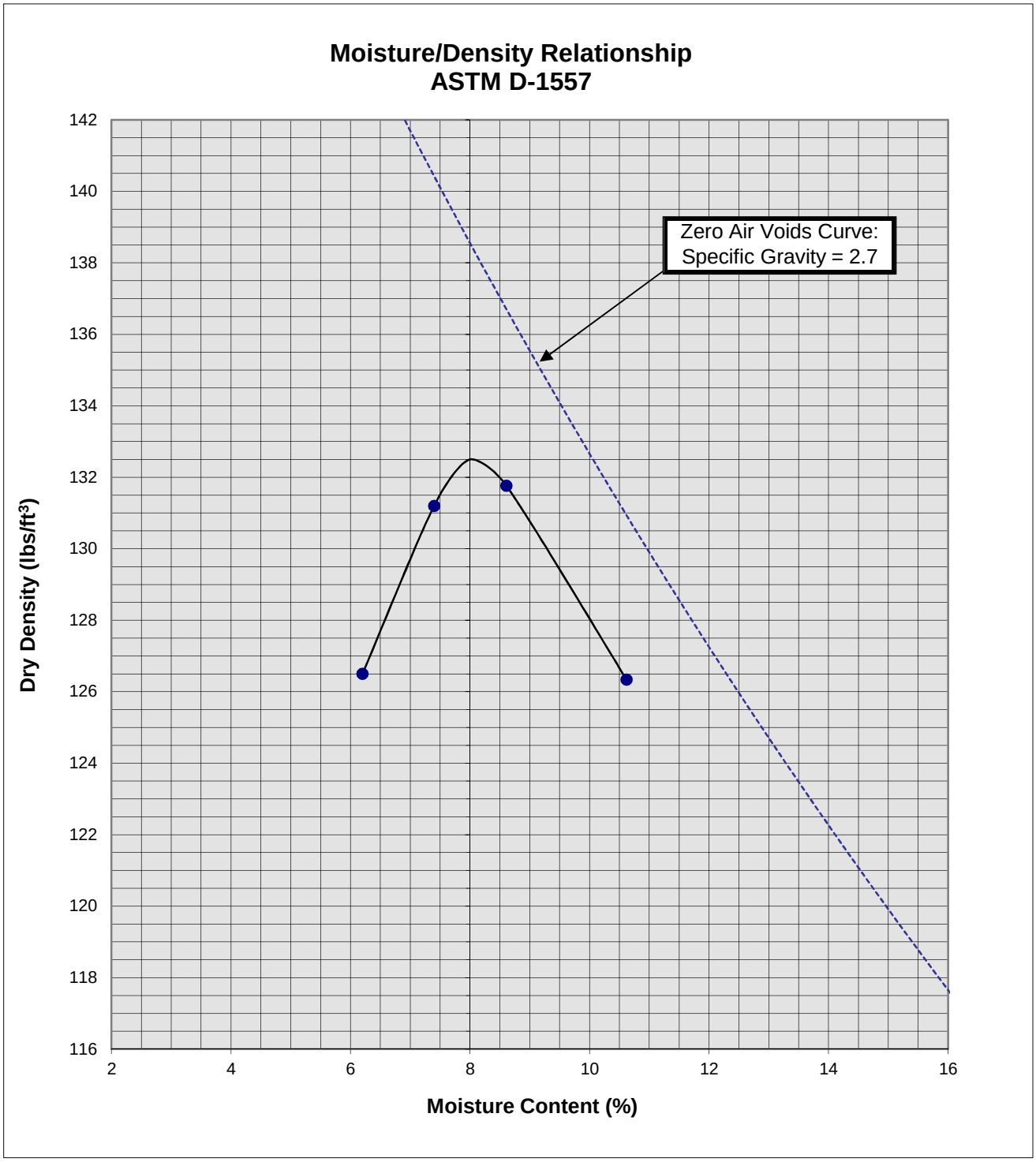
Classification: Gray Brown Gravelly fine to coarse Sand

Boring Number:	B-8	Initial Moisture Content (%)	3
Sample Number:	---	Final Moisture Content (%)	11
Depth (ft)	7 to 8	Initial Dry Density (pcf)	117.8
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	125.8
Specimen Thickness (in)	1.0	Percent Collapse (%)	1.44

Proposed C/I Bldg
San Bernardino County, California
Project No. 14G190

PLATE C- 8

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation



Soil ID Number	B-1 @ 0 tp 5
Optimum Moisture (%)	8
Maximum Dry Density (pcf)	132.5
Soil Classification	Brown Silty fine Sand, trace fine Gravel

Proposed C/I Bldg
San Bernardino County, California
Project No. 14G190
PLATE C-9

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.

Compacted Fills

- 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.

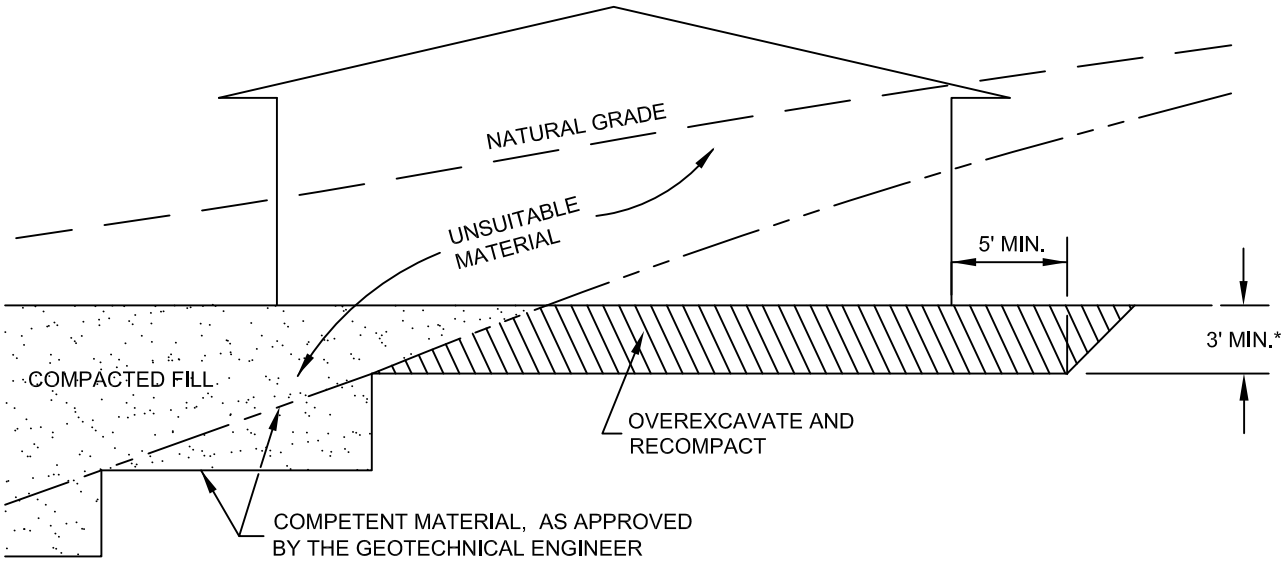
Subdrains

- 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.

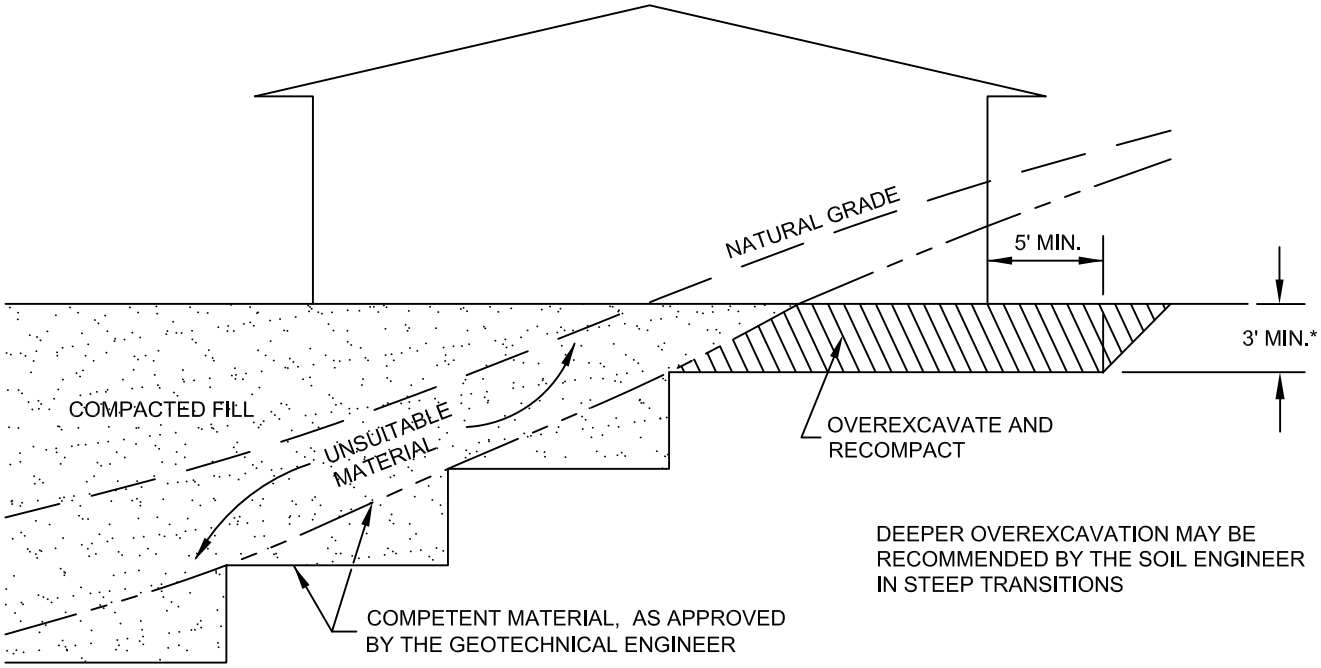
- Plastic pipe may be used for subdrains provided it is Schedule 40 or SDR 35 or equivalent. Pipe should be protected against breakage, typically by placement in a square-cut (backhoe) trench or as recommended by the manufacturer.

- 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



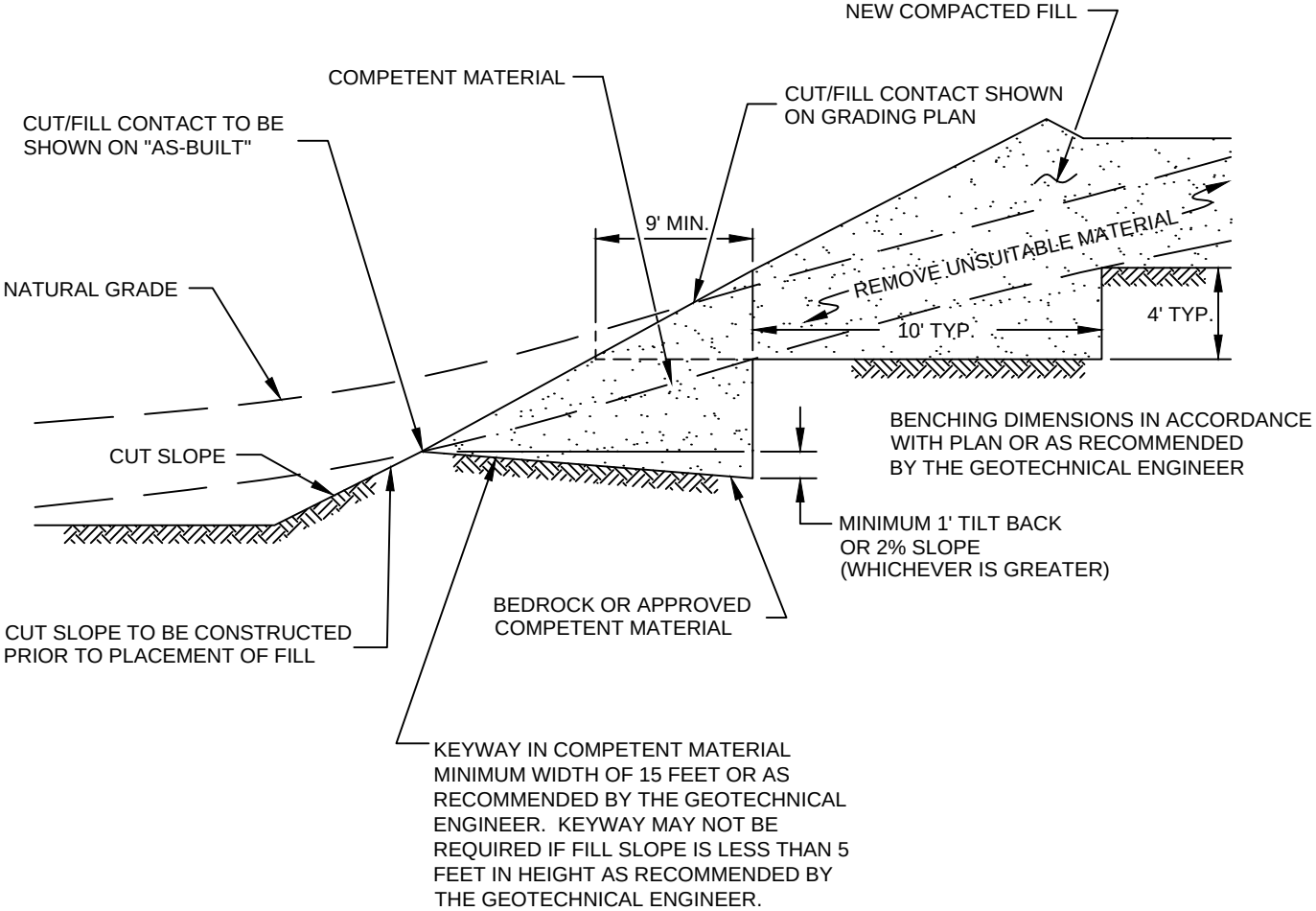
CUT/FILL LOT (TRANSITION)



*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

SOUTHERN CALIFORNIA GEOTECHNICAL



□ILL ABO□E C□T □LOPE □ETAIL

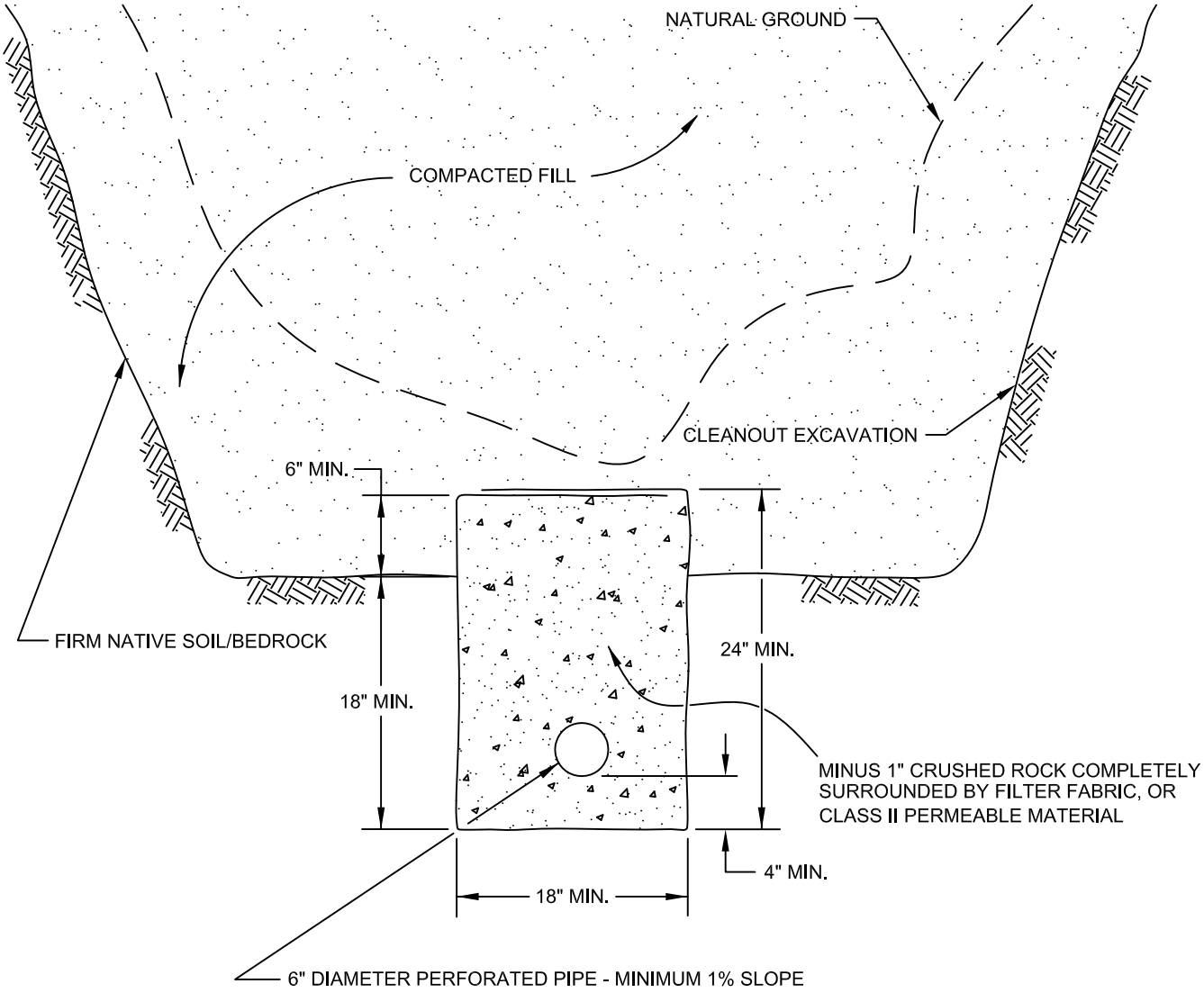
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE □-2

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

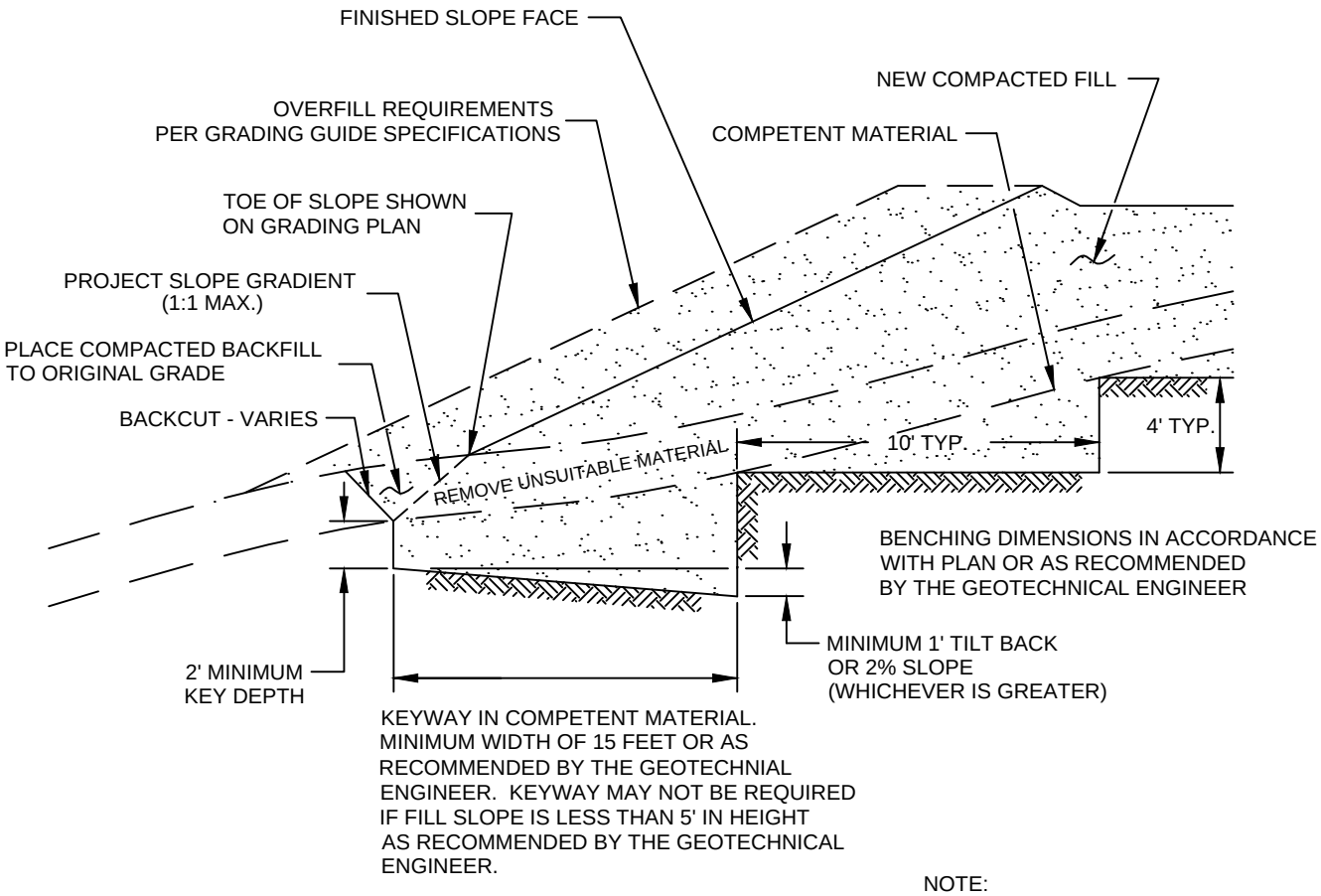
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-3

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.

□ILL ABO□E NAT□RAL □LOPE □ETAIL

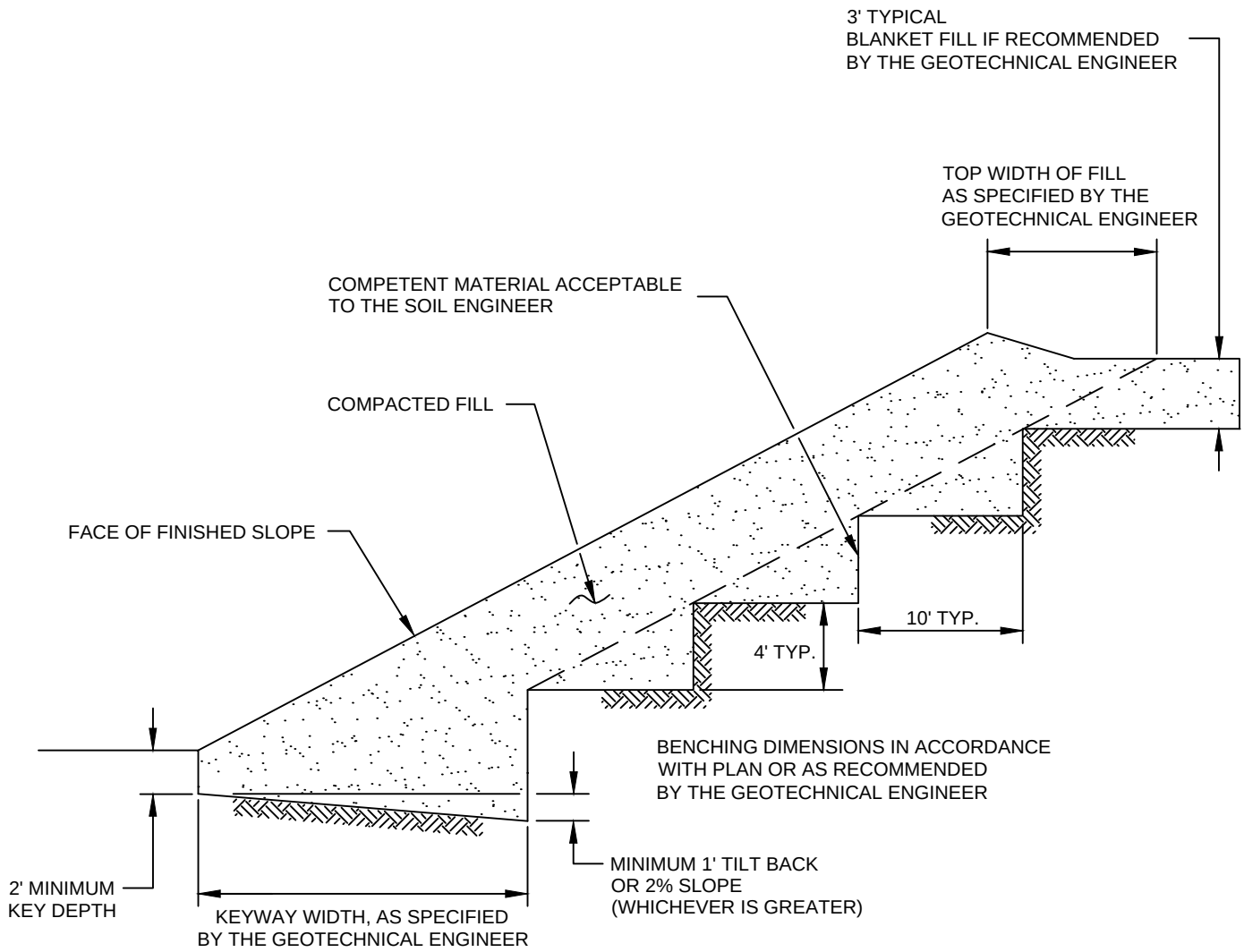
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE □-□

SOUTHERN CALIFORNIA GEOTECHNICAL



□TABILI□ATION □ILL □ETAIL

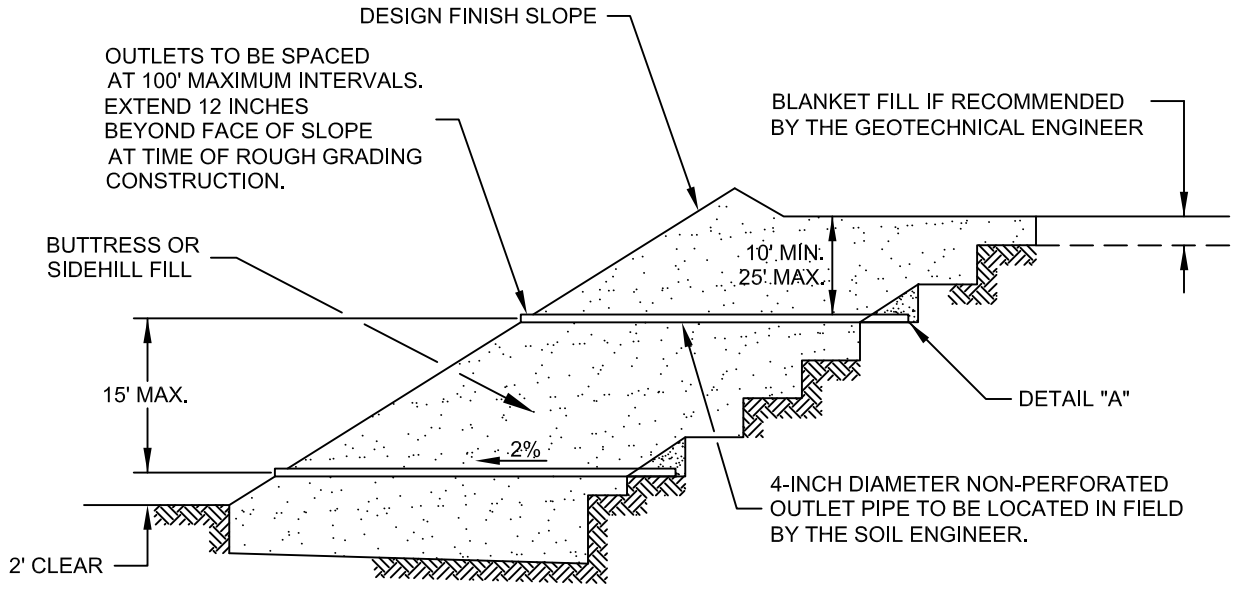
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE □-□

SOUTHERN CALIFORNIA GEOTECHNICAL



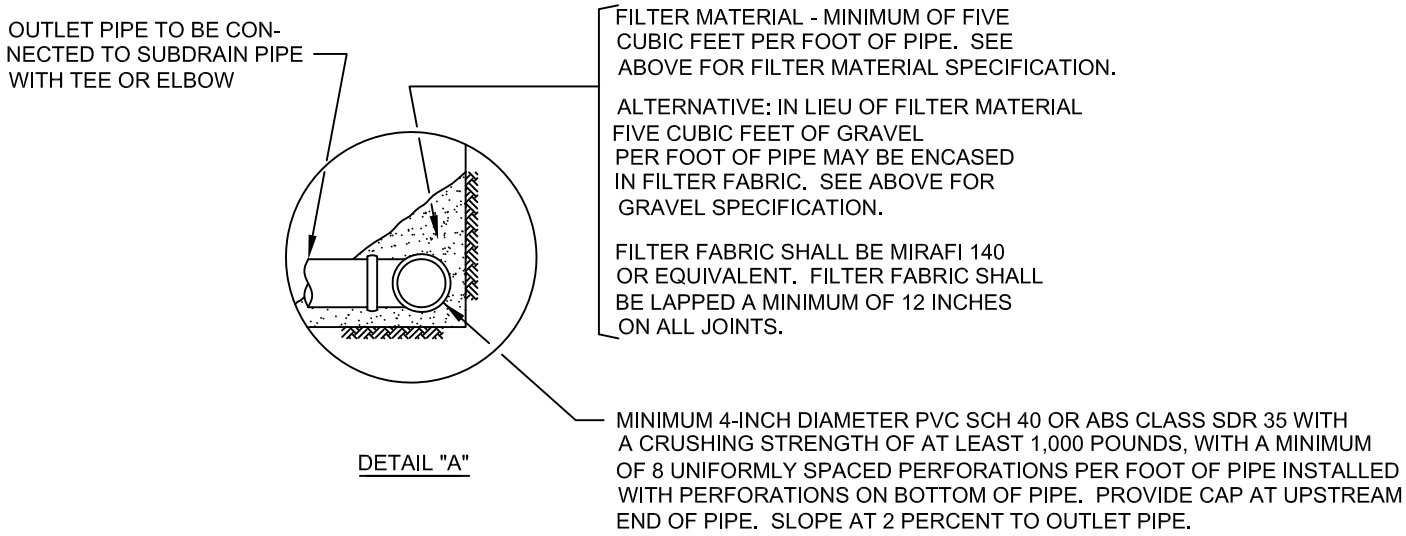
"FILTER MATERIAL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT: (CONFORMS TO EMA STD. PLAN 323)

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



DETAIL "A"

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

SLOPE FILL SUBDRAINS

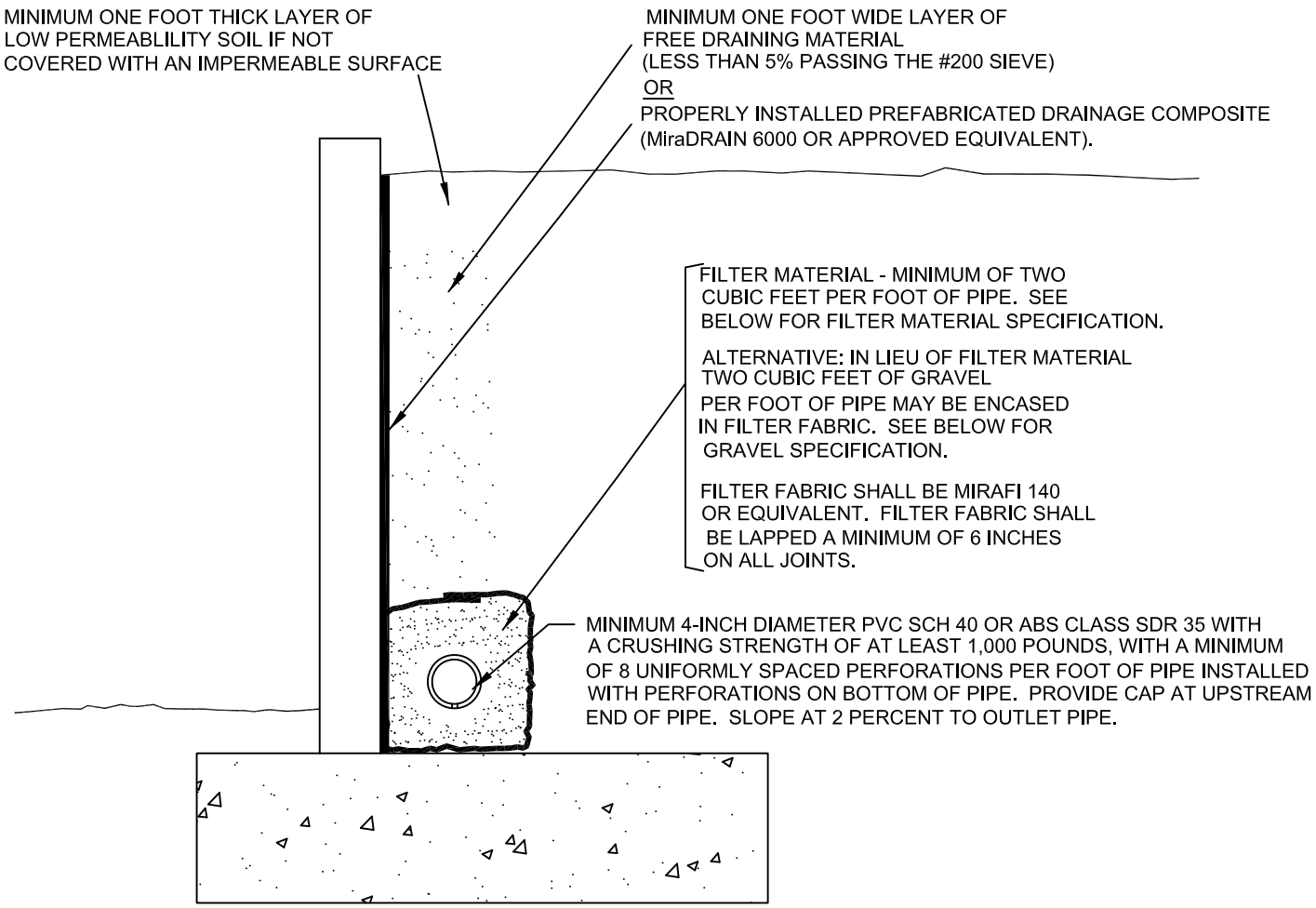
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-6

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

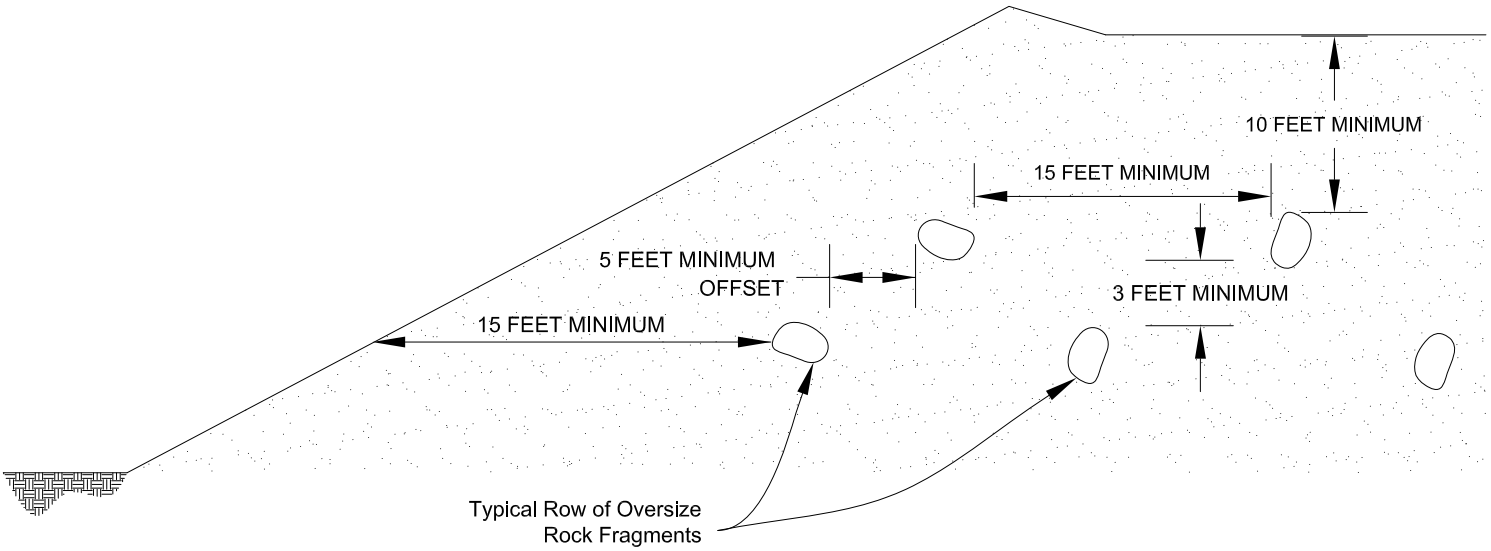
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

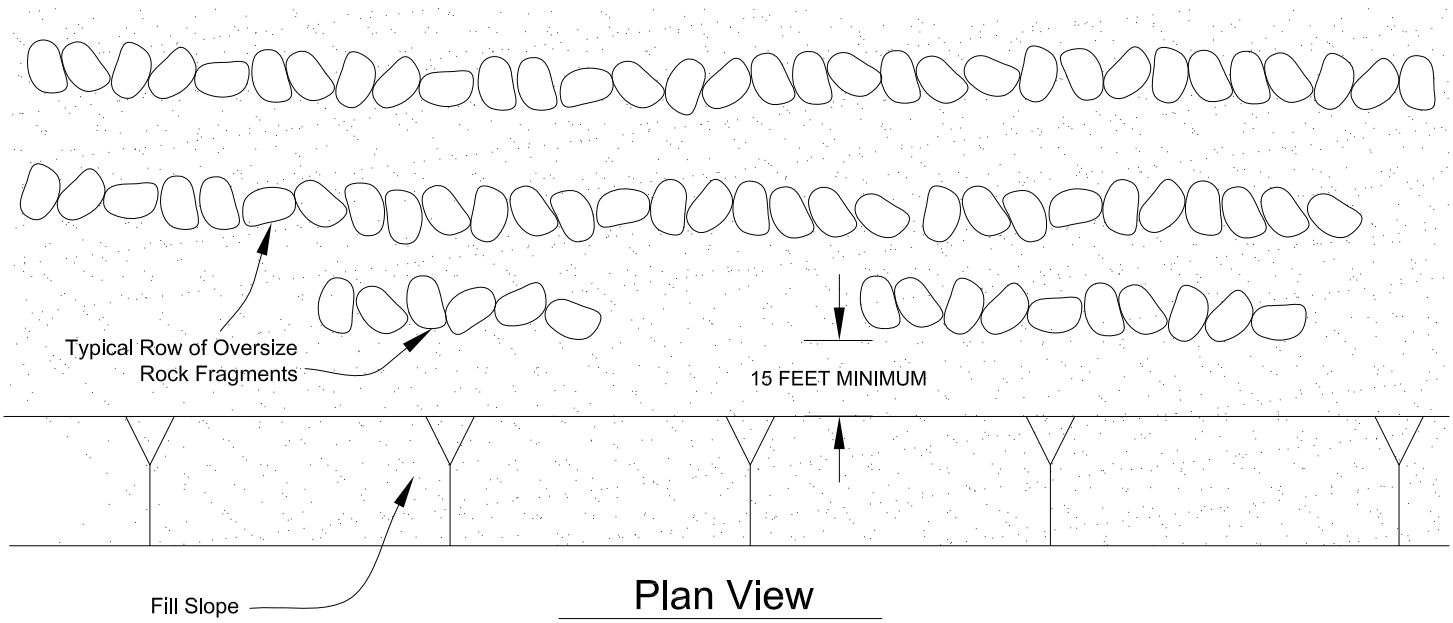
DRAWN: JAS
CHKD: GKM

PLATE D-7

SOUTHERN CALIFORNIA GEOTECHNICAL



Section View



Plan View

PLACEMENT OF OVERSIZED MATERIAL

GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: PM
CHKD: GKM

PLATE D-8

SOUTHERN CALIFORNIA GEOTECHNICAL

APPENDIX E

USGS Design Maps Summary Report

User–Specified Input

Report Title Proposed C/I Building
Fri October 17, 2014 17:14:27 UTC

Building Code Reference Document ASCE 7-10 Standard
(which utilizes USGS hazard data available in 2008)

Site Coordinates 34.06623°N, 117.39852°W

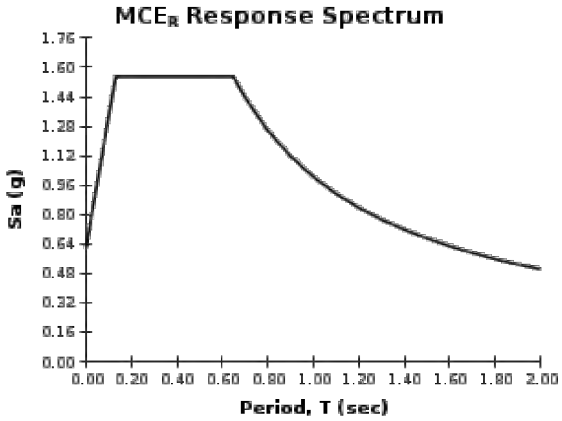
Site Soil Classification Site Class D – "Stiff Soil"

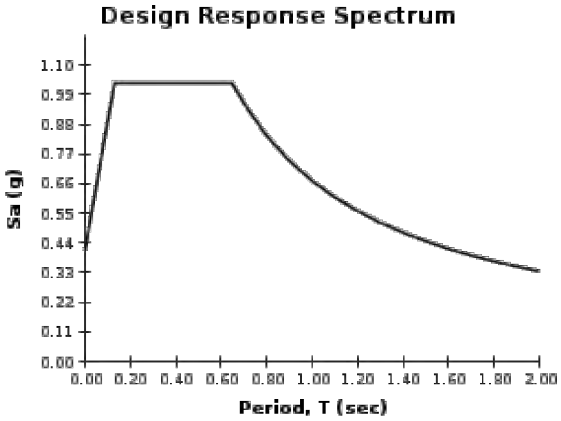
Risk Category I/II/III

USGS–Provided Output

S_S =	1.550 g	S_{MS} =	1.550 g	S_{DS} =	1.033 g
S_1 =	0.671 g	S_{M1} =	1.006 g	S_{D1} =	0.671 g

For information on how the SS and S1 values above have been calculated from probabilistic (risk-targeted) and deterministic ground motions in the direction of maximum horizontal response, please return to the application and select the "2009 NEHRP" building code reference document.





SOURCE: U.S. GEOLOGICAL SURVEY (USGS)
<http://geohazards.usgs.gov/designmaps/us/application.php>

EI MIC E IGN PARAMETER

PROPOSED COMMERCIAL/INDUSTRIAL BUILDING

SAN BERNARDINO COUNTY, CALIFORNIA

DRAWN: KVC
CHKD: JAS

SCG PROJECT
14G190-1

PLATE E-

SOUTHERN CALIFORNIA GEOTECHNICAL

October 30, 2014

Thrifty Oil Company
13116 Imperial Highway
Santa Fe Springs, California 90670

Attention: Mr. Julien Hoisington

Project No.: **14G190-2**

Subject: **Results of Infiltration Testing**
Proposed Commercial/Industrial Building
NWC of Orange Street and Cedar Avenue,
San Bernardino County, California

Gentlemen:

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. 14P297 dated June 26, 2014. The scope of services included surface reconnaissance, subsurface exploration, field testing, and geotechnical engineering analysis to determine the infiltration rate of the on-site soils. 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 northwest corner of Orange Street and Cedar Avenue in Bloomington, an unincorporated area of San Bernardino County, California. The site is bounded to west by Linden Avenue, to the south by Orange Street, to the east by Cedar Avenue, and to the north by a 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, 20± acres in size. The site is currently vacant and undeveloped except for the remnants of two unmaintained asphaltic concrete roads, both of which trend north-south, a concrete slab in the center of the site with dimensions of approximately 20 feet by 20 feet, and foundations of a former building with dimensions of approximately 45 feet by 36 feet. Smaller portions of concrete slabs are also present in the eastern portion of the site. One of these roads appears to be coincident with the alignment of Orchard Street, if the alignment of Orchard Street were projected north of Orange Street. The second abandoned road appears to coincide with the alignment of Magnolia Street, which terminates on the north side of the Interstate 10 Freeway, if this alignment were to be projected south of Interstate 10. Both of these roads extend southward from the northern property line and terminate north of the sidewalk on the north side of Orange Street. Both roads are in poor

condition with numerous large cracks. Ground surface cover throughout the site generally consists of exposed soil with sparse to moderate native grass and weed growth. Occasional trees are scattered throughout the central portion of the site.

Topographic information for the subject site was obtained from Hall & Foreman, Inc. Based on this plan, the maximum site elevation of 1091.0 feet mean sea level (msl) is located in the northeast corner of the site and the minimum site elevation of 1079.0 feet msl is located in the southwest corner of the site. The site slopes downward to the south at a gradient of less than 2± percent. The overall topographic relief of the site is approximately 12 feet.

Proposed Development

Based on a site plan prepared by Hall & Foreman, Inc., the site will be developed with one (1) new commercial/industrial building, approximately 371,308 ft^2 in size. Truck loading docks will be constructed along the south side and eastern part of the north side of the building. The building will be surrounded by Portland cement concrete pavements in the truck loading dock areas and asphaltic concrete pavements in the automobile parking and drive lanes. Landscape planter areas and concrete flatwork may be included throughout the site.

Based on the conversations with the project civil engineer, the site will utilize on-site storm water infiltration systems to dispose of storm water at the subject site. The storm water infiltration systems will consist of two (2) detention basins located in the southeast and northeast corners of the site. Based on the conversation with the project civil engineer, the bottom of the basins will be constructed at 8 to 10± feet below existing site grades (at an elevation of 1075.0± feet msl and 1070.5± feet msl, respectfully). We were requested to perform infiltration testing within the areas of the detention basin.

Concurrent Study

Southern California Geotechnical, Inc. (SCG) is currently conducting a geotechnical investigation of the subject site. As a part of this study, a total of ten (10) borings were advanced to depths of 5 to 30± feet below existing site grades. All of the borings were advanced with hollow-stem augers, by a conventional truck-mounted drilling rig.

Artificial fill soils were encountered at the ground surface, at Boring No. B-3, extending to a depth of 4± feet below the existing site grade. The fill soils consist of loose silty fine sands and medium dense gravely fine to coarse sands. Native alluvium was encountered beneath the fill materials at Boring Nos. B-1 and at the ground surface at all of the remaining borings. The near surface alluvium, within the upper 2 to 3 feet of the ground surface, possess a disturbed appearance and are identified as disturbed alluvium on the boring logs. These disturbed soils generally consist of loose to medium dense silty fine sands with occasional traces of fine gravel. These soils resemble the composition and color of the native alluvium encountered at similar depths, but possess a slightly disturbed appearance. Undisturbed alluvial soils were encountered at all of the boring locations, beneath the disturbed alluvium, artificial fill, or at the ground surface. Undisturbed alluvium extends to at least the maximum depth explored of 30± feet. The native alluvium generally consists of loose to medium dense silty fine sands, underlain by interbedded strata of medium dense to dense, fine to coarse sands, gravelly fine to coarse sands, fine sands, and silty fine sands.

Free water was not encountered during drilling of any of the ten (10) borings. Based on the lack of any water within the borings, and the moisture contents of the recovered soil samples, the static groundwater table is considered to have existed at a depth in excess of 30± feet at the time of the subsurface exploration.

Subsurface Exploration

Scope of Exploration

The subsurface exploration consisted of two (2) trenches excavated with a rubber tire backhoe, extending to depths of 8 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 tests (identified as I-1 and I-2) are indicated on the Infiltration Test Location Plan, enclosed as Plate 2 of this report.

Geotechnical Conditions

Native alluvial soils were encountered at the ground surface at Infiltration Test Nos. I-1 and I-2 extending to the maximum depth explored of 10± feet below existing site grades. The near-surface alluvial soils at the test locations generally consist of loose silty fine sands and silty fine to coarse sands with trace fine gravel extending to a depth of 5± feet below existing site grades. At greater depths the alluvial soils generally consist of medium dense gravelly fine to coarse sand and fine to coarse sandy gravel with abundant cobbles and trace silt. Groundwater was not encountered at any of the infiltration test trench locations. The Trench Logs, which illustrate the conditions encountered at the trench locations, are included with this report.

Infiltration Testing

We understand that the results of the infiltration testing will be used to prepare a preliminary design for the proposed detention/infiltration basin that will be used to store and/or dispose of storm water at the subject site. As previously stated, 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, the outer ring was driven 3± inches into the soil at the base of the 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

The infiltration testing was performed at Infiltration Trench Nos. I-1 and I-2. 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 were maintained using 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 test.

The schedule for readings was determined based on the observed soil type at the base of each trench. Due to the gravel content within the infiltration test locations, the readings for the infiltration tests were taken at intervals of 2 to 4 minutes. The water volume readings are presented on the spreadsheets enclosed with this report. The infiltration rates for each of the timed intervals are also tabulated on the spreadsheets.

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

Infiltration Test No.	**Elevation of Test (ft msl)**	**Soil Description**	**Infiltration Rate (inches/hr)**
I-1	1071.0	Gravelly fine to coarse Sand, trace Silt	7.9
I-2	1078.0	Fine to coarse Sandy Gravel, trace Silt	12.1

Laboratory Testing

Grain Size Analysis

The grain size distribution of selected soils 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 these tests are presented at the end of this report.

Design Recommendations

A total of two (2) infiltration tests were performed at the subject site. As noted above, the infiltration rates at the two locations are 7.9 to 12.1 inches per hour. These rates are typical for the encountered soil types. Variations between the two rates may be attributed to the varying gravel content and relative densities of the soils in the two locations.

Based on the relative densities and varying gravel content encountered at various locations and depths throughout the site, **we recommend a design infiltration rate of 8.0 inches per hour be used in the design of both detention basins** located in the northeast and southeast regions of the site if the bottom of the basins are constructed at elevations of 1078.0 feet msl and 1071.0 feet msl, respectfully, or deeper. It may be prudent for the designer to add a factor of safety, at his discretion.

The design of the infiltration systems should be performed by the project civil engineer, in accordance with the San Bernardino County guidelines. However, it is recommended that the system be constructed so as to facilitate removal of silt and clay, or other deleterious materials from any water that may enter the storm water infiltration 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 rate.** It should be noted that the recommended infiltration rates are based on infiltration testing at two discrete locations and the overall infiltration rate of the storm water infiltration system could vary considerably.

The near-surface soils consist of silty sands. It is possible, and likely, for the silty sands encountered above the native alluvial sandy soils to migrate down the slopes of the detention/infiltration basin to the bottom of the detention/infiltration basin which would decrease the infiltration rate of the system. **Therefore, the recommended infiltration rates are contingent upon the basin being designed as to prevent silt or clay from migrating down the slopes to the base of the detention/infiltration basin.** We recommend that the slopes of the basin be protected with vegetation or fabric overlaid with rock to help prevent the bottom of the detention/infiltration basin from collecting excess silt and/or clay content.

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

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. If possible, all of 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 systems.

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,

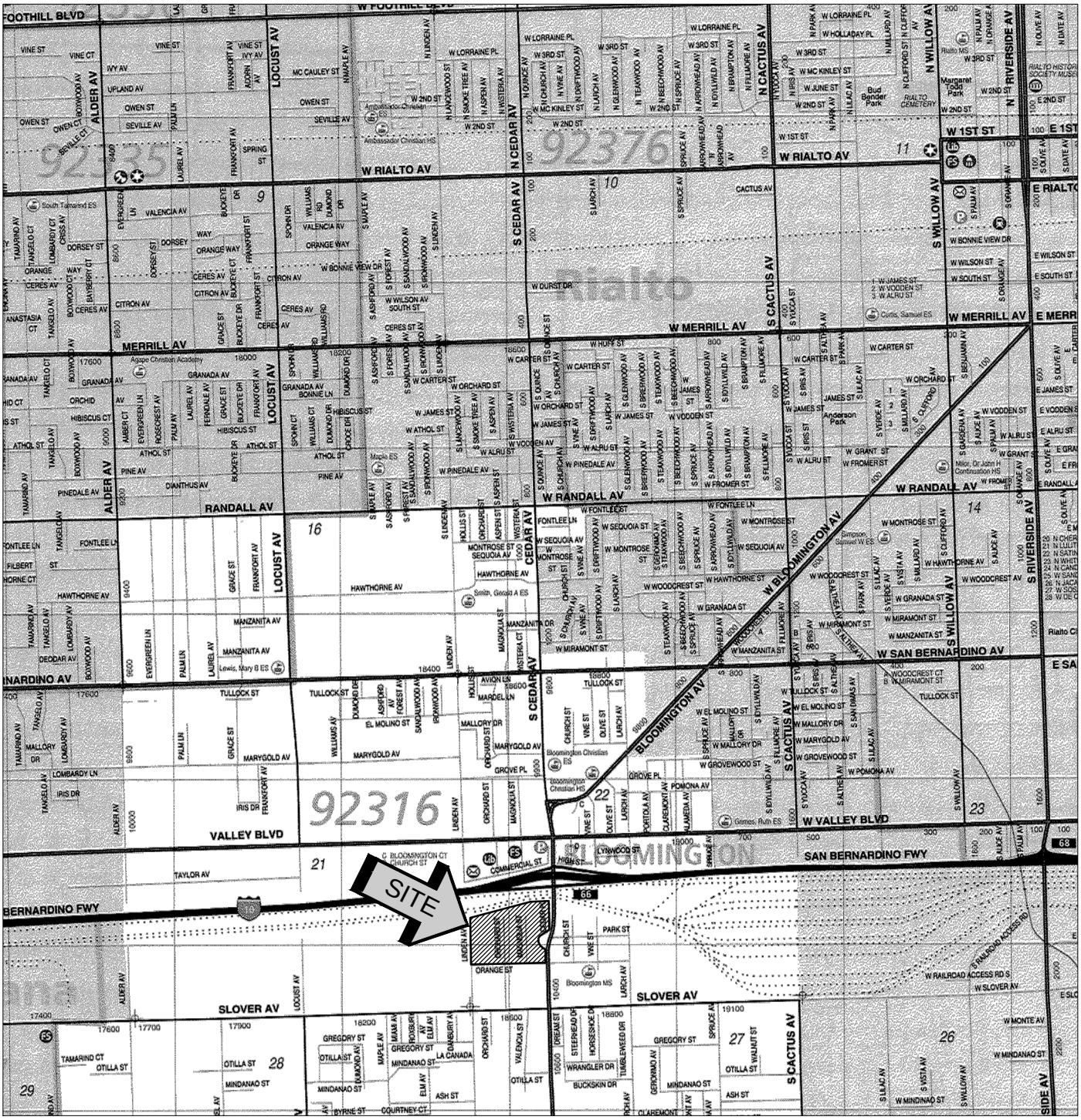
SOUTHERN CALIFORNIA GEOTECHNICAL, INC.

Ricardo Frias
Staff Engineer

John A. Seminara, GE 2294
Principal Engineer

Distribution: (1) Addressee

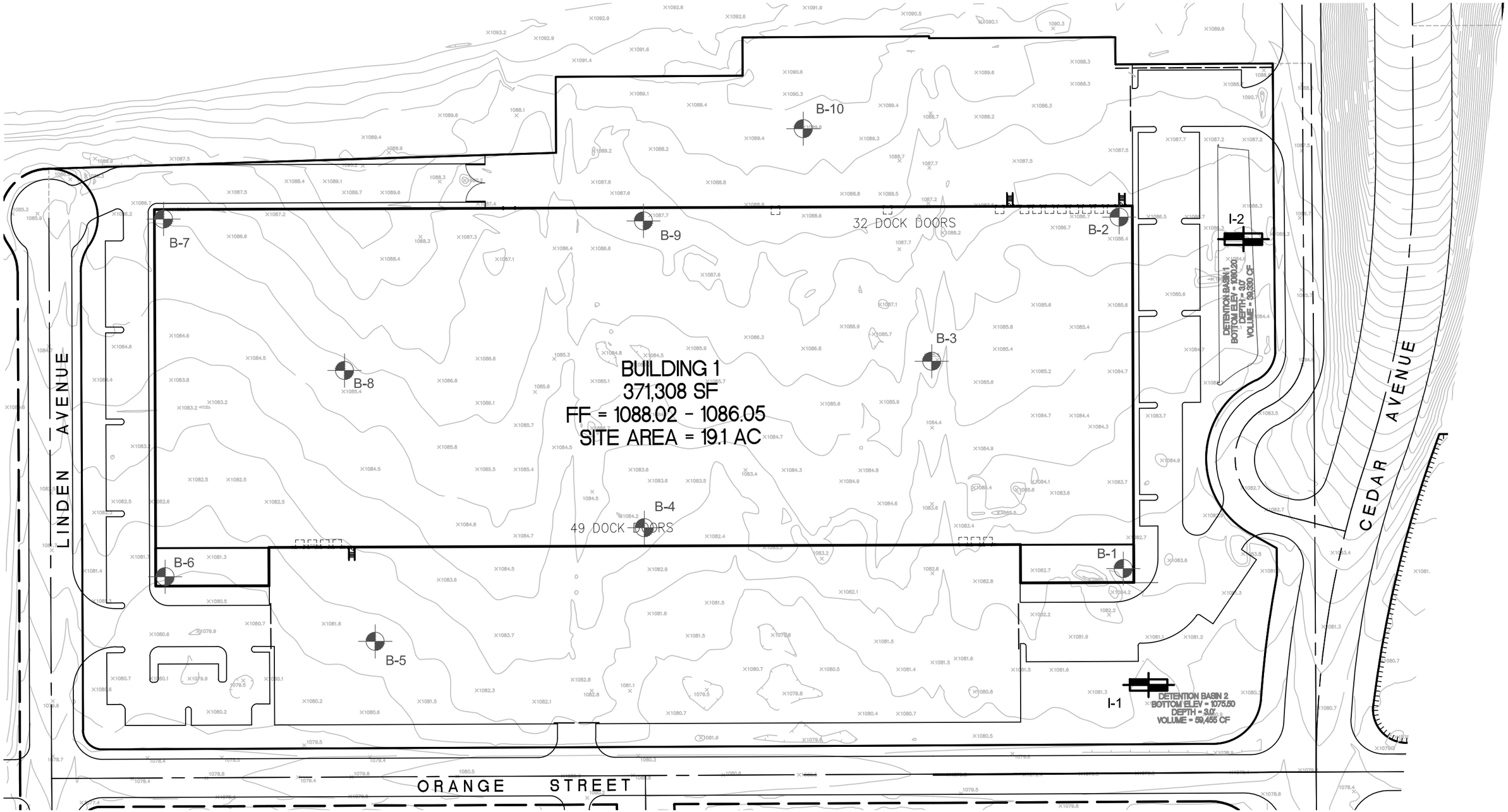
Enclosures: Plate 1 Site Location Map
Plate 2 Infiltration Test Location Plan
Trench Logs (2 pages)
Infiltration Test Results Spreadsheets (2 pages)
Grain Size Analysis Graphs (2 pages)



SOURCE: SAN BERNARDINO COUNTY
THOMAS GUIDE, 2013

SITE LOCATION MAP
PROPOSED COMMERCIAL/INDUSTRIAL BUILDING
SAN BERNARDINO COUNTY, CALIFORNIA
SCALE: 1" = 2400'
DRAWN: RF CHKD: JAS
SCG PROJECT 14G190-2
PLATE

SOUTHERN CALIFORNIA GEOTECHNICAL



GEOTECHNICAL LEGEND

APPROXIMATE INFILTRATION TEST LOCATION

APPROXIMATE BORING LOCATION FROM CONCURRENT STUDY (SCG PROJECT NO. 14G190-1)

NORTH
SoCalGeo

NOTE: BASE MAP PREPARED BY HALL & FOREMAN, INC.

INFILTRATION TRENCH LOCATION PLAN
PROPOSED COMMERCIAL/INDUSTRIAL BUILDING
SAN BERNARDINO COUNTY, CALIFORNIA
SCALE: 1" = 100'
DRAWN: RF CHKD: JAS
SCG PROJECT 14G190-2
PLATE 2

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL

SOUTHERN CALIFORNIA GEOTECHNICAL

TRENCH NO.
I-□

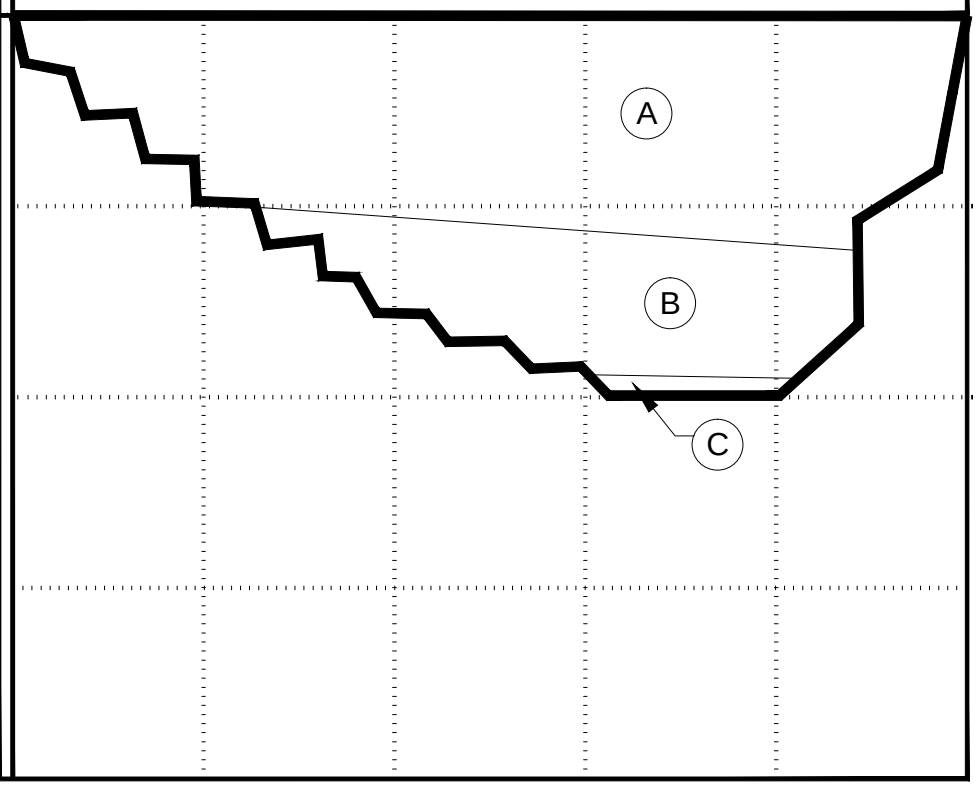
JOB NO.: 14G190-2
PROJECT: Proposed Commercial/Industrial Building
LOCATION: San Bernardino County, CA
DATE: 09-10-2014

EQUIPMENT USED: Backhoe
LOGGED BY: Ricardo Frias
ORIENTATION: N 80 E
TOP OF TRENCH ELEVATION: 1081.0 ft

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

DEPTH	SAMPLE	DRY DENSITY (PCF)	MOISTURE (%)	EARTH MATERIALS DESCRIPTION
				A: ALLUVIUM: Brown Silty fine Sand to fine Sandy Silt, trace coarse Sand, trace Mica, loose to medium dense - dry
				B: ALLUVIUM: Light Brown to Brown fine to coarse Sand, trace Silt, loose to medium dense - damp
				C:ALLUVIUM: Light Brown Gravelly fine to coarse Sand, trace Silt, occasional Cobbles, medium dense - dry to damp
5				
10				Trench Terminated @ 10 feet
15				

GRAPHIC REPRESENTATION



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

TRENCH LOG

PLATE B-□

SOUTHERN CALIFORNIA GEOTECHNICAL

TRENCH NO. I-2

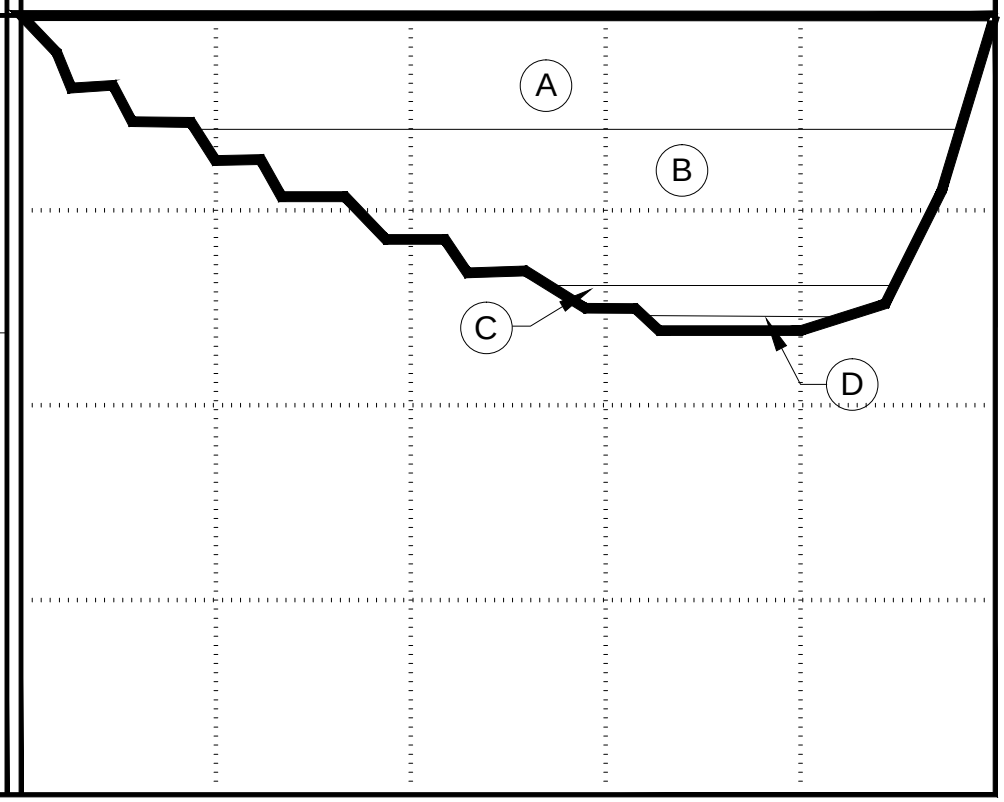
JOB NO.: 14G190-2
PROJECT: Proposed Commercial/Industrial Building
LOCATION: San Bernardino County, CA
DATE: 09-10-2014

EQUIPMENT USED: Backhoe
LOGGED BY: Ricardo Frias
ORIENTATION: N 75 E
TOP OF TRENCH ELEVATION: 1086.0 ft

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

DEPTH	SAMPLE	DRY DENSITY (PCF)	MOISTURE (%)	EARTH MATERIALS DESCRIPTION
				A: ALLUVIUM: Gray Brown fine to medium Sandy Silt, trace fine Gravel, loose to medium dense - dry
				B: ALLUVIUM: Brown Silty fine to coarse Sand, some fine Gravel, some Cobbles, loose to medium dense - dry
5				C: ALLUVIUM: Brown fine to medium Gravelly fine to coarse Sand, trace Silt, occasional Cobbles, medium dense - damp
				D: ALLUVIUM: Gray Brown fine to coarse Sandy Gravel, trace Silt, occasional Cobbles, medium dense to dense - dry to damp
10				Trench Terminated @ 8 feet
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

INFILTRATION CALCULATIONS

Project Name	Proposed Commercial/Industrial Building
Project Location	San Bernardino County, CA
Project Number	14G190-2
Engineer	Ricardo Frias

Infiltration Test No: I-1

Constants	Diameter (ft)	Area (ft^2)	Area (cm^2)
Inner	1	0.79	730
Annular	2	2.36	2189

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

Test Interval		Time (hr)	Time 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	9:00 AM	1	1550	450	5200	1050	37.00	28.78	14.57	11.33
	Final	9:01 AM	**1**	2000		6250					
2	Initial	9:01 AM	1	2000	400	6250	950	32.89	26.04	12.95	10.25
	Final	9:02 AM	**2**	2400		7200					
3	Initial	9:02 AM	1	2400	300	7200	950	24.67	26.04	9.71	10.25
	Final	9:03 AM	**3**	2700		8150					
4	Initial	9:03 AM	5	0	1275	0	4500	20.97	24.67	8.26	9.71
	Final	9:08 AM	**8**	1275		4500					
5	Initial	9:09 AM	5	0	1300	1800	4400	21.38	24.12	8.42	9.50
	Final	9:14 AM	**14**	1300		6200					
6	Initial	9:15 AM	5	0	1250	1500	4150	20.56	22.75	8.09	8.96
	Final	9:20 AM	**20**	1250		5650					
7	Initial	9:20 AM	5	1200	1250	2000	4200	20.56	23.03	8.09	9.07
	Final	9:25 AM	**25**	2450		6200					
8	Initial	9:26 AM	5	0	1250	1700	4150	20.56	22.75	8.09	8.96
	Final	9:31 AM	**31**	1250		5850					
9	Initial	9:32 AM	5	0	1225	1600	4200	20.15	23.03	7.93	9.07
	Final	9:37 AM	**37**	1225		5800					
10	Initial	9:37 AM	5	1550	1225	2000	4150	20.15	22.75	7.93	8.96
	Final	9:42 AM	**42**	2775		6150					
11	Initial	9:43 AM	5	300	1225	1700	4150	20.15	22.75	7.93	8.96
	Final	9:48 AM	**48**	1525		5850					
12	Initial	9:49 AM	5	0	1225	1500	4050	20.15	22.20	7.93	8.74
	Final	9:54 AM	**54**	1225		5550					
13	Initial	9:55 AM	5	0	1225	1500	4100	20.15	22.48	7.93	8.85
	Final	10:00 AM	**60**	1225		5600					

INFILTRATION CALCULATIONS

Project Name	Proposed Commercial/Industrial Building
Project Location	San Bernardino County, CA
Project Number	14G190-2
Engineer	Ricardo Frias

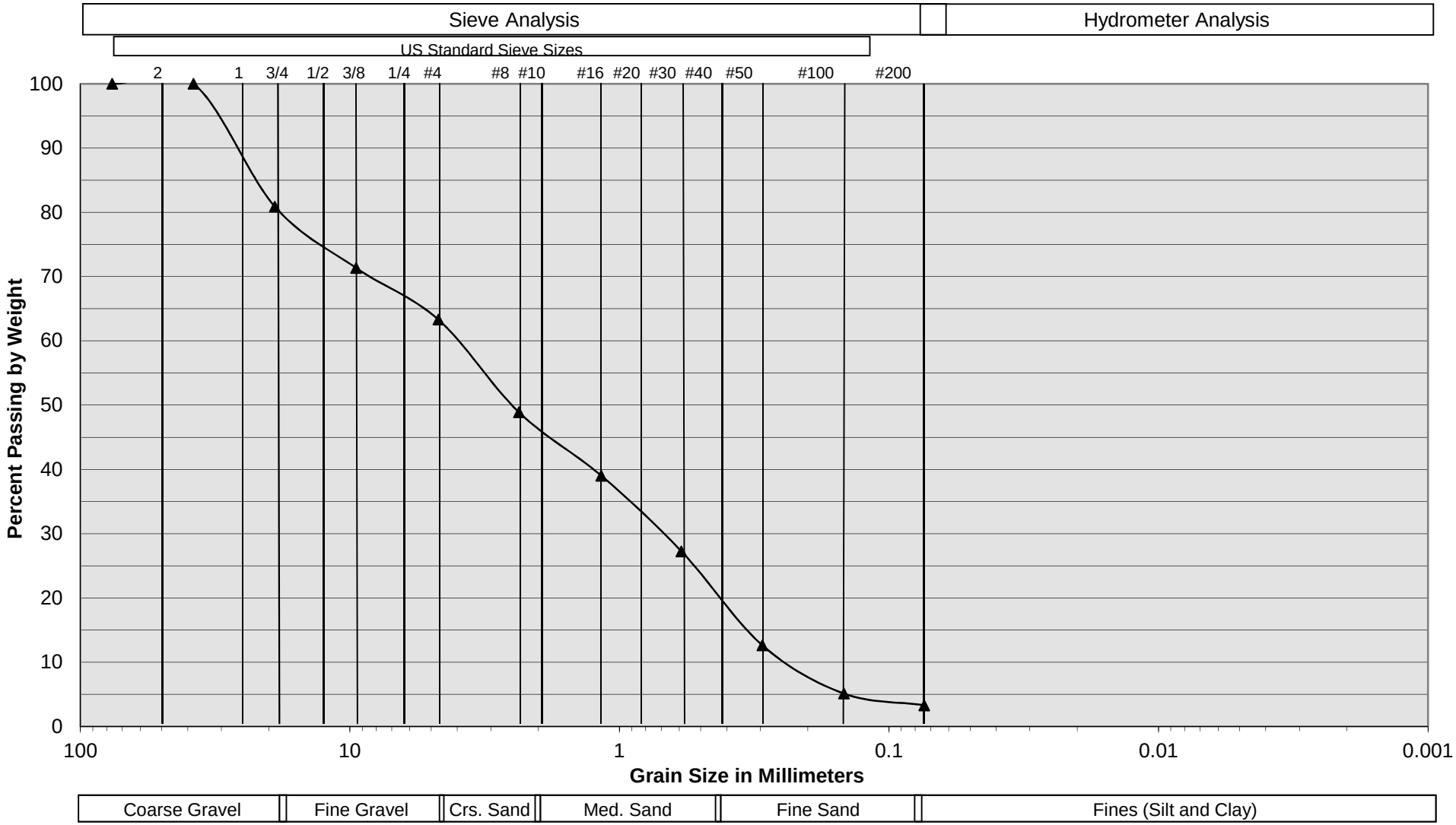
Infiltration Test No: I-2

Constants	Diameter (ft)	Area (ft^2)	Area (cm^2)
Inner	1	0.79	730
Annular	2	2.36	2189

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

Test Interval		Time (hr)	Time 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	11:41 AM	5	0	2250	0	8000	37.00	43.86	14.57	17.27
	Final	11:46 AM	**5**	2250		8000					
2	Initial	11:46 AM	5	0	2150	0	7850	35.36	43.04	13.92	16.94
	Final	11:51 AM	**10**	2150		7850					
3	Initial	11:51 AM	5	0	2100	0	7700	34.54	42.21	13.60	16.62
	Final	11:56 AM	**15**	2100		7700					
4	Initial	11:56 AM	5	0	2100	0	7600	34.54	41.66	13.60	16.40
	Final	12:01 PM	**20**	2100		7600					
5	Initial	12:01 PM	5	0	2050	0	7350	33.72	40.29	13.27	15.86
	Final	12:06 PM	**25**	2050		7350					
6	Initial	12:06 PM	5	0	2050	0	7200	33.72	39.47	13.27	15.54
	Final	12:11 PM	**30**	2050		7200					
7	Initial	12:11 PM	5	0	2000	0	7050	32.89	38.65	12.95	15.22
	Final	12:16 PM	**35**	2000		7050					
8	Initial	12:16 PM	5	0	2000	0	6950	32.89	38.10	12.95	15.00
	Final	12:21 PM	**40**	2000		6950					
9	Initial	12:21 PM	10	0	3950	0	13000	32.48	35.63	12.79	14.03
	Final	12:31 PM	**50**	3950		13000					
10	Initial	12:31 PM	10	0	3800	0	12500	31.25	34.26	12.30	13.49
	Final	12:41 PM	**60**	3800		12500					
11	Initial	12:41 PM	10	0	3850	0	12450	31.66	34.13	12.46	13.44
	Final	12:51 PM	**70**	3850		12450					
12	Initial	12:51 PM	10	0	3850	0	12200	31.66	33.44	12.46	13.17
	Final	1:01 PM	**80**	3850		12200					
13	Initial	1:01 PM	10	0	3800	0	12150	31.25	33.30	12.30	13.11
	Final	1:11 PM	**90**	3800		12150					
14	Initial	1:11 PM	10	0	3750	0	12000	30.84	32.89	12.14	12.95
	Final	1:21 PM	**100**	3750		12000					

Grain Size Distribution



Sample Description	I-1 @ 10'
Soil Classification	Light Brown to Brown Gravelly fine to coarse Sand, trace Silt

Proposed Commercial/Industrial Building
San Bernardino County, California
Project No. 14G190-2
PLATE C- 1

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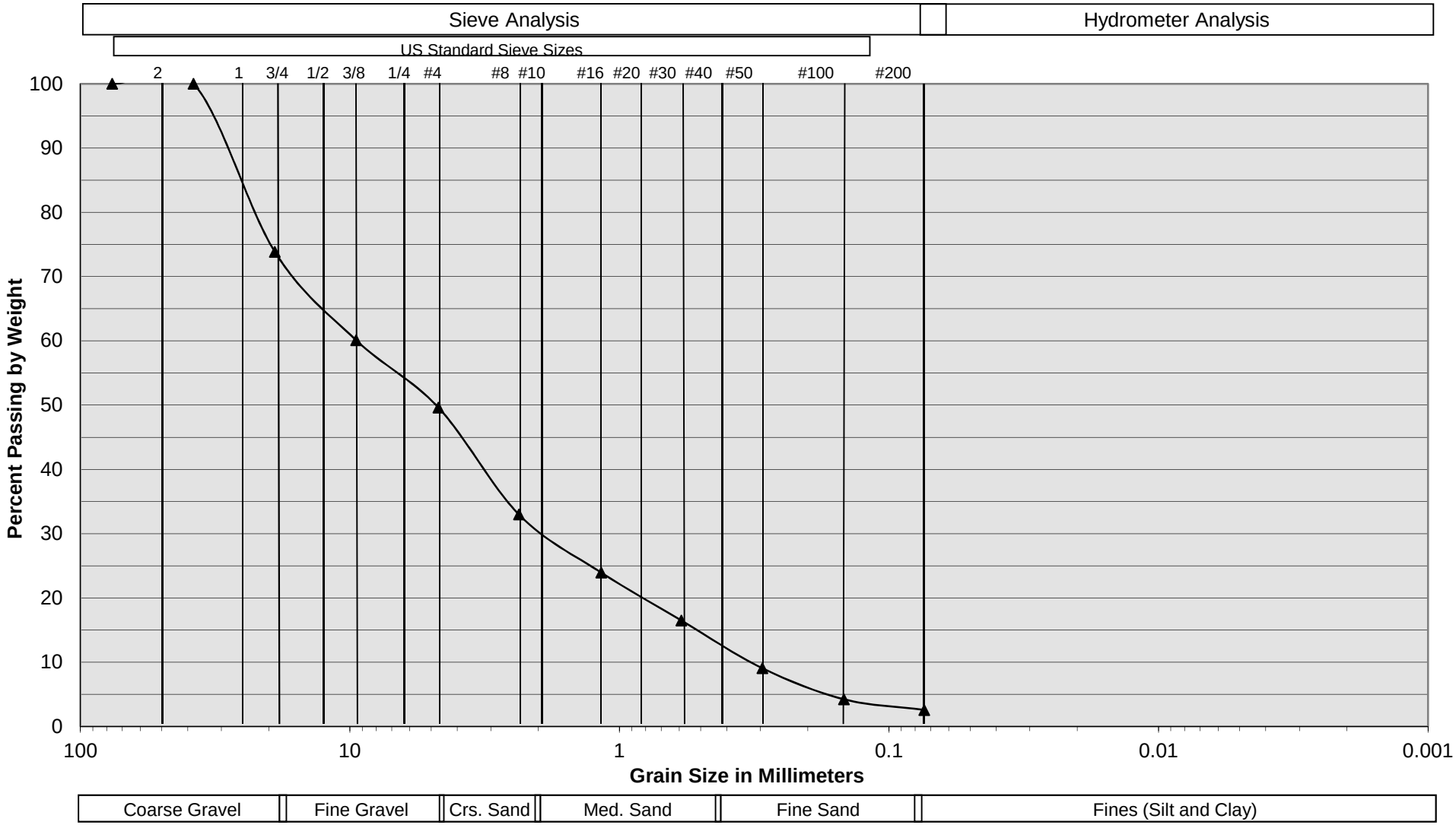
Grain Size Distribution



Sample Description	I-2 @ 8'
Soil Classification	Brown fine to coarse Sandy Gravel, trace Silt

Proposed Commercial/Industrial Building
San Bernardino County, California
Project No. 14G190-2
PLATE C- 2

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