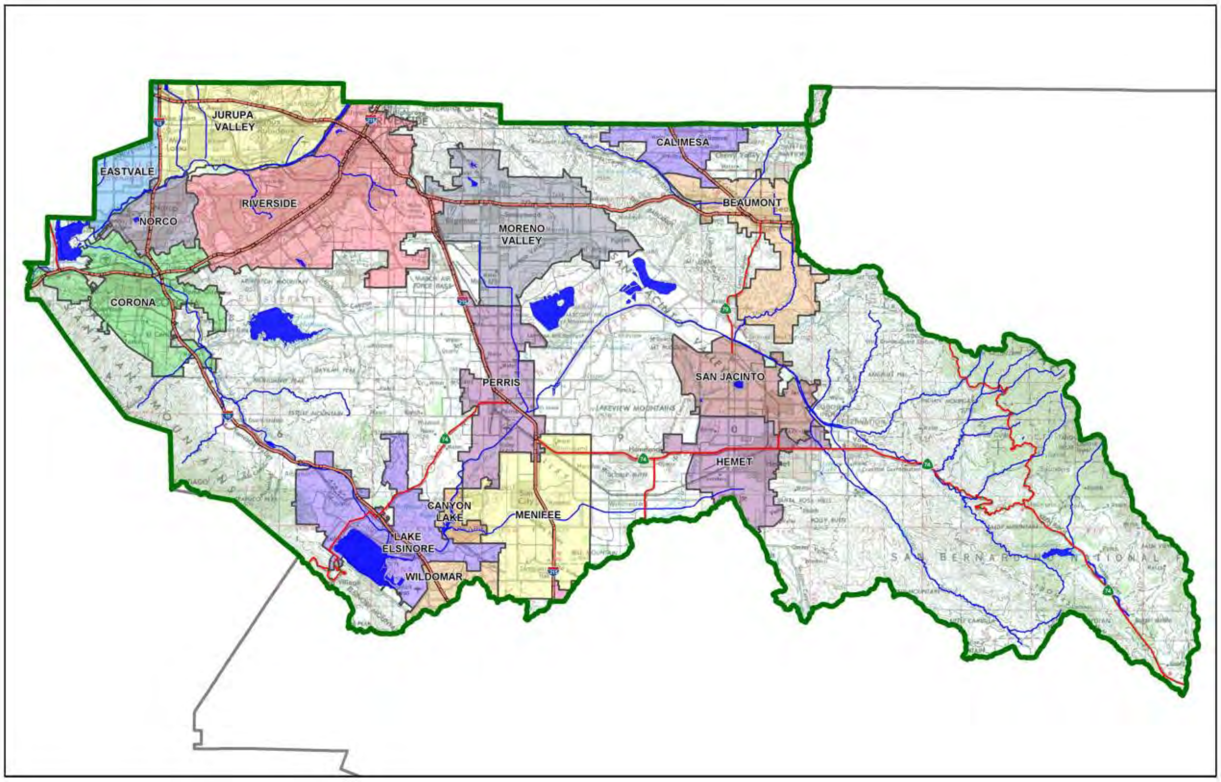
Project Specific Water Quality Management Plan

*A Template for Projects located within the **Santa Ana Watershed** Region of Riverside County*

Project Title: **Rider IV Distribution Center**

Design Review/Case No: **19-00006**



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☒ **Preliminary**
☐ **Final**

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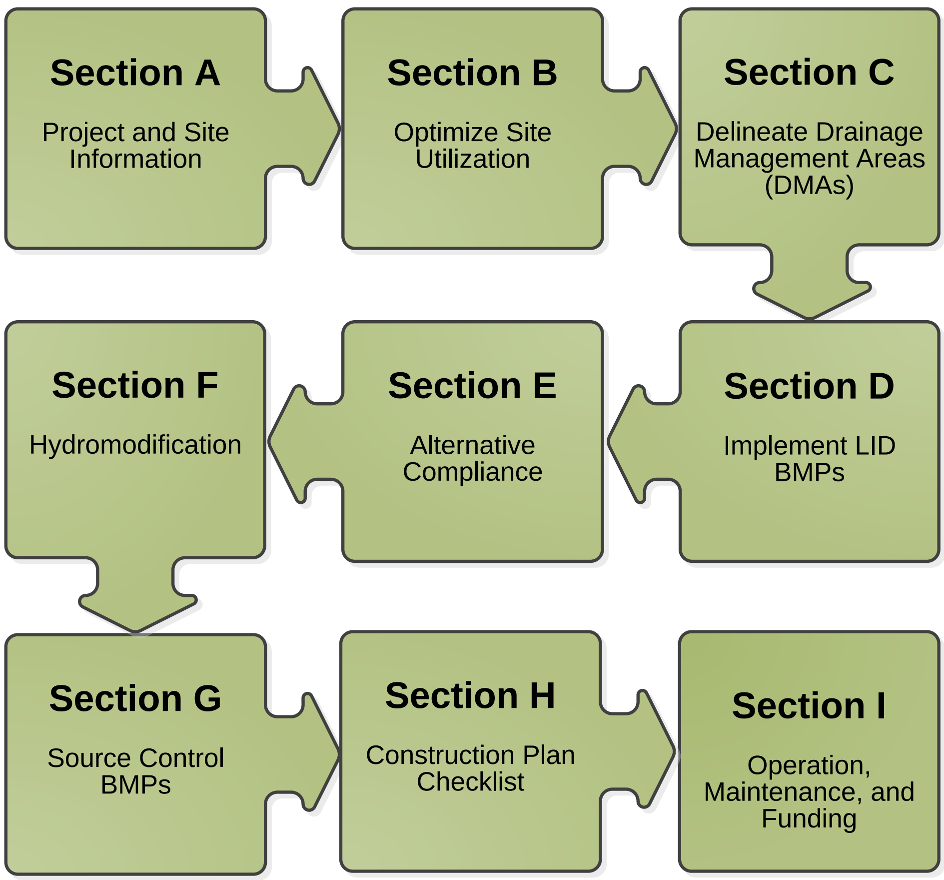
Prepared for Compliance with
Regional Board Order No. **R8-2010-0033**

A Brief Introduction

This Project-Specific WQMP Template for the **Santa Ana Region** has been prepared to help guide you in documenting compliance for your project. Because this document has been designed to specifically document compliance, you will need to utilize the WQMP Guidance Document as your "how-to" manual to help guide you through this process. Both the Template and Guidance Document go hand-in-hand, and will help facilitate a well prepared Project-Specific WQMP. Below is a flowchart for the layout of this Template that will provide the steps required to document compliance.



OWNER'S CERTIFICATION

This Project-Specific Water Quality Management Plan (WQMP) has been prepared for IDI Logistics by Albert A. Webb Associates for the Rider IV Distribution Center project (19-00006).

This WQMP is intended to comply with the requirements of City of Perris for Water Quality Ordinance 1194 which includes the requirement for the preparation and implementation of a Project-Specific WQMP.

The undersigned, while owning the property/project described in the preceding paragraph, shall be responsible for the implementation and funding of this WQMP and will ensure that this WQMP is amended as appropriate to reflect up-to-date conditions on the site. In addition, the property owner accepts responsibility for interim operation and maintenance of Stormwater BMPs until such time as this responsibility is formally transferred to a subsequent owner. This WQMP will be reviewed with the facility operator, facility supervisors, employees, tenants, maintenance and service contractors, or any other party (or parties) having responsibility for implementing portions of this WQMP. At least one copy of this WQMP will be maintained at the project site or project office in perpetuity. The undersigned is authorized to certify and to approve implementation of this WQMP. The undersigned is aware that implementation of this WQMP is enforceable under City of Perris Water Quality Ordinance (Municipal Code Section1194).

"I, the undersigned, certify under penalty of law that the provisions of this WQMP have been reviewed and accepted and that the WQMP will be transferred to future successors in interest."

Owner's Signature	Date
Owner's Printed Name	Owner's Title/Position

PREPARER'S CERTIFICATION

"The selection, sizing and design of stormwater treatment and other stormwater quality and quantity control measures in this plan meet the requirements of Regional Water Quality Control Board Order No. **R8-2010-0033** and any subsequent amendments thereto."

Preparer's Signature	Date
DJ Arellano, P.E.	Senior Engineer
Preparer's Printed Name	Preparer's Title/Position

Preparer's Licensure:

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Section A: Project and Site Information

PROJECT INFORMATION		
Type of Project:	Commercial/Industrial	
Planning Area:	Mead Valley Area Plan (RCIP)	
Community Name:	Mead Vally Area Plan (RCIP)	
Development Name:	Rider Distribution Center IV	
PROJECT LOCATION		
Latitude & Longitude (DMS): 38°49’53.76”, 117°12”51.28”		
Project Watershed and Sub-Watershed: Santa Ana River, San Jacinto River		
APN(s): 303-160-002, 303-160-003, 303-160-007, 303-160-009, 303-160-010		
Map Book and Page No.: Thomas Bros. Map Page 777, Grid H2 & Grid H3 & Grid J2 & Grid J3		
PROJECT CHARACTERISTICS		
Proposed or Potential Land Use(s)		Commercial/Industrial
Proposed or Potential SIC Code(s): **1541** (General Contractors-Industrial Buildings & Warehouses), **4225** (General Warehousing & Storage)		1541, 4225
Area of Impervious Project Footprint (SF)		1,018,217
Total Area of proposed Impervious Surfaces within the Project Limits (SF)/or Replacement		1,018,217
Does the project consist of offsite road improvements?		☒ Y ☐ N
Does the project propose to construct unpaved roads?		☐ Y ☒ N
Is the project part of a larger common plan of development (phased project)?		☐ Y ☒ N
EXISTING SITE CHARACTERISTICS		
Total area of existing Impervious Surfaces within the project limits (SF)		0
Is the project located within any MSHCP Criteria Cell?		☐ Y ☒ N
If so, identify the Cell number:		N/A
Are there any natural hydrologic features on the project site?		☐ Y ☒ N
Is a Geotechnical Report attached?		☒ Y ☐ N
If no Geotech. Report, list the NRCS soils type(s) present on the site (A, B, C and/or D)		N/A
What is the Water Quality Design Storm Depth for the project?		0.63

Project Description

The project is proposing an industrial facility on approximately 26.4 acres of vacant land. The site is bound by Redlands Avenue to the west, the proposed Rider II development to the south, Perris Valley Storm Drain Channel, and Morgan Street to the north. The existing elevations across the site vary from 1443.5 at the southeast corner to 1445.8 at the northwest corner (NAVD88 datum). The site currently slopes at approximately 0.2% from the northwest corner to the southeast corner. The existing drainage pattern for the site and the general area is characterized by sheet flows that follow the slope towards the southeast corner and outfall into Perris Valley Storm Drain.

This project proposes for all on-site runoff to surface flow through the site utilizing ribbon gutters, curb and gutters, grate inlets, and subsurface storm drain systems. The storm drain systems will be used to convey flows into a proposed water quality storage basin before being pumped into a proposed bio-retention basin; both are located in the south-east corner of the project site. Line A and Line B will both drain to storage basin – though they will have different connections to Lateral G-2. For each storm drain, a manhole with an adverse grade pipe downstream will be placed to ensure the tributary water quality volume is retained. Higher flows will overcome the adverse grade and discharge into lateral G-2. (See Preliminary Drainage Report for more information)

After the captured flows have been pumped and treated by the bio-retention basin, they will drain into Lateral G-2. The pump station will discharge at an appropriate flowrate to drain the water quality storage basin in 48 hours. The bio-retention basin was sized given the water quality volume, a bio-retention media filtration/treatment rate of 5.0 in/hr (found from LA County MS4 Attachment H) with a factor of safety of 2, and a mandatory drawdown time of 48 hours. We calculated the required bio-retention area to be 4,800 square feet. The required pump will need to be at least 100 gpm.

Maps and Site Plans

When completing your Project-Specific WQMP, include a map of the local vicinity and existing site. In addition, include all grading, drainage, landscape/plant palette and other pertinent construction plans in Appendix 2. At a **minimum**, your WQMP Site Plan should include the following:

- Drainage Management Areas
- Proposed Structural BMPs
- Drainage Path
- Drainage Infrastructure, Inlets, Overflows
- Source Control BMPs
- Buildings, Roof Lines, Downspouts
- Impervious Surfaces
- Standard Labeling

Use your discretion on whether or not you may need to create multiple sheets or can appropriately accommodate these features on one or two sheets. Keep in mind that the Co-Permittee plan reviewer must be able to easily analyze your project utilizing this template and its associated site plans and maps.

A.1 Identify Receiving Waters

Using Table A.1 below, list in order of upstream to downstream, the receiving waters that the project site is tributary to. Continue to fill each row with the Receiving Water's 303(d) listed impairments (if any), designated beneficial uses, and proximity, if any, to a RARE beneficial use. Include a map of the receiving waters in Appendix 1.

Table A.1 Identification of Receiving Waters

Receiving Waters	EPA Approved 303(d) List Impairments	Designated Beneficial Uses	Proximity to RARE Beneficial Use
Perris Valley Storm Drain	None	None	Not a water body classified as RARE
San Jacinto River (Reach 3)(HU#802.11)	None	MUN,AGR, GWR, REC1, REC2, WARM, WILD	Not a water body classified as RARE
San Jacinto River (Reach 2)(HU#802.11)	None	MUN,AGR, GWR, REC1, REC2, WARM, WILD	Not a water body classified as RARE
Canyon Lake (HU#802.11, 802.12)	Nutrients, Pathogens	MUN,AGR, GWR, REC1, REC2, WARM, WILD	Not a water body classified as RARE
San Jacinto River (Reach 1)(HU#802.32)	None	MUN,AGR, GWR, REC1, REC2, WARM, WILD	Not a water body classified as RARE
Lake Elsinore (HU#802.31)	PCBs, (Organic Compounds), Nutrients, Organic Enrichment (Low DO) , Sediment Toxicity, Unknown Toxicity	REC1, REC2, WARM, WILD	Not a water body classified as RARE

A.2 Additional Permits/Approvals required for the Project:

Table A.2 Other Applicable Permits

Agency	Permit Required	
State Department of Fish and Game, 1602 Streambed Alteration Agreement	☐ Y	☒ N
State Water Resources Control Board, Clean Water Act (CWA) Section 401 Water Quality Cert.	☐ Y	☒ N
US Army Corps of Engineers, CWA Section 404 Permit	☐ Y	☒ N
US Fish and Wildlife, Endangered Species Act Section 7 Biological Opinion	☐ Y	☒ N
Statewide Construction General Permit Coverage	☒ Y	☐ N
Statewide Industrial General Permit Coverage	☒ Y	☐ N
Western Riverside MSHCP Consistency Approval (e.g., JPR, DBESP)	☐ Y	☒ N
Other *(please list in the space below as required)* Grading Permit	☒ Y	☐ N

If yes is answered to any of the questions above, the Co-Permittee may require proof of approval/coverage from those agencies as applicable including documentation of any associated requirements that may affect this Project-Specific WQMP.

Section B: Optimize Site Utilization (LID Principles)

Review of the information collected in Section 'A' will aid in identifying the principal constraints on site design and selection of LID BMPs as well as opportunities to reduce imperviousness and incorporate LID Principles into the site and landscape design. For example, constraints might include impermeable soils, high groundwater, groundwater pollution or contaminated soils, steep slopes, geotechnical instability, high-intensity land use, heavy pedestrian or vehicular traffic, utility locations or safety concerns. Opportunities might include existing natural areas, low areas, oddly configured or otherwise unbuildable parcels, easements and landscape amenities including open space and buffers (which can double as locations for bioretention BMPs), and differences in elevation (which can provide hydraulic head). Prepare a brief narrative for each of the site optimization strategies described below. This narrative will help you as you proceed with your LID design and explain your design decisions to others.

The 2010 Santa Ana MS4 Permit further requires that LID Retention BMPs (Infiltration Only or Harvest and Use) be used unless it can be shown that those BMPs are infeasible. Therefore, it is important that your narrative identify and justify if there are any constraints that would prevent the use of those categories of LID BMPs. Similarly, you should also note opportunities that exist which will be utilized during project design. Upon completion of identifying Constraints and Opportunities, include these on your WQMP Site plan in Appendix 1.

Site Optimization

The following questions are based upon Section 3.2 of the WQMP Guidance Document. Review of the WQMP Guidance Document will help you determine how best to optimize your site and subsequently identify opportunities and/or constraints, and document compliance.

Did you identify and preserve existing drainage patterns? If so, how? If not, why?

The project site naturally drains towards the south. The project proposes to preserve the existing drainage pattern by placing the water quality storage and treatment basins on the southeast side of the project.

Did you identify and protect existing vegetation? If so, how? If not, why?

The project site will not protect the existing vegetation as it is currently vacant with very little vegetation.

Did you identify and preserve natural infiltration capacity? If so, how? If not, why?

There are unfavorable soil conditions for infiltration per the attached soils and infiltration report. They recommend using an infiltration rate of 1.0 in/hr which is less than the required 1.6 in/hr for infiltration BMP's.

Did you identify and minimize impervious area? If so, how? If not, why?

Yes, impervious area was minimized given the proposed site usage and required materials. The project intends to provide the standard amount of cover.

Did you identify and disperse runoff to adjacent pervious areas? If so, how? If not, why?

Runoff will be dispersed to the water quality storage and treatment basins. There are also areas that will utilize self-retention.

Section C: Delineate Drainage Management Areas (DMAs)

Utilizing the procedure in Section 3.3 of the WQMP Guidance Document which discusses the methods of delineating and mapping your project site into individual DMAs, complete Table C.1 below to appropriately categorize the types of classification (e.g., Type A, Type B, etc.) per DMA for your project site. Upon completion of this table, this information will then be used to populate and tabulate the corresponding tables for their respective DMA classifications.

Table C.1 DMA Classifications

DMA Name or ID	Surface Type(s)[1]	Area (Sq. Ft.)	DMA Type
L-A	LANDSCAPE	20,506	D
R-A	ROOF	567,098	D
H-A	HARDSCAPE	451,119	D
BMP-A	LANDSCAPE	4,800	D
SR-A	LANDSCAPE	105,229	B

[1]Reference Table 2-1 in the WQMP Guidance Document to populate this column

Table C.2 Type 'A', Self-Treating Areas

DMA Name or ID	Area (Sq. Ft.)	Stabilization Type	Irrigation Type (if any)

Table C.3 Type 'B', Self-Retaining Areas

Self-Retaining Area				Type 'C' DMAs that are draining to the Self-Retaining Area		
DMA Name/ ID	Post-project surface type	Area (square feet) [A]	Storm Depth (inches) [B]	DMA Name/ID	[C] from Table C.4 = [C]	Required Retention Depth (inches) [D]
SR-A	LANDSCAPE	105,229	0.63	N/A	N/A	N/A

$$[D] = [B] + \frac{[B] \cdot [C]}{[A]}$$

Table C.4 Type 'C', Areas that Drain to Self-Retaining Areas*

DMA					Receiving Self-Retaining DMA		
DMA Name/ ID	Area (square feet) [A]	Post-project surface type	Runoff factor [B]	Product [C] = [A] x [B]	DMA name /ID	Area (square feet) [D]	Ratio [C]/[D]

*Drainage to self-retaining areas might be possible. The landscaped areas are subject to minor revisions prior to final engineering; table will be completed in full during final engineering if design warrants.

Table C.5 Type 'D', Areas Draining to BMPs

DMA Name or ID	BMP Name or ID
L-A	BMP-A
R-A	BMP-A
H-A	BMP-A
BMP-A	BMP-A

<u>Note</u>: More than one drainage management area can drain to a single LID BMP, however, one drainage management area may not drain to more than one BMP.

Section D: Implement LID BMPs

D.1 Infiltration Applicability

Is there an approved downstream 'Highest and Best Use' for stormwater runoff (see discussion in Chapter 2.4.4 of the WQMP Guidance Document for further details)? ☐ Y ☒ N

If yes has been checked, Infiltration BMPs shall not be used for the site. If no, continue working through this section to implement your LID BMPs. It is recommended that you contact your Co-Permittee to verify whether or not your project discharges to an approved downstream 'Highest and Best Use' feature.

Geotechnical Report

A Geotechnical Report or Phase I Environmental Site Assessment may be required by the Copermittee to confirm present and past site characteristics that may affect the use of Infiltration BMPs. In addition, the Co-Permittee, at their discretion, may not require a geotechnical report for small projects as described in Chapter 2 of the WQMP Guidance Document. If a geotechnical report has been prepared, include it in Appendix 3. In addition, if a Phase I Environmental Site Assessment has been prepared, include it in Appendix 4.

Is this project classified as a small project consistent with the requirements of Chapter 2 of the WQMP Guidance Document? ☐ Y ☒ N

Infiltration Feasibility

Table D.1 below is meant to provide a simple means of assessing which DMAs on your site support Infiltration BMPs and is discussed in the WQMP Guidance Document in Chapter 2.4.5. Check the appropriate box for each question and then list affected DMAs as applicable. If additional space is needed, add a row below the corresponding answer.

Table D.1 Infiltration Feasibility

Does the project site…	YES	NO
…have any DMAs with a seasonal high groundwater mark shallower than 10 feet?		X
If Yes, list affected DMAs:		
…have any DMAs located within 100 feet of a water supply well?		X
If Yes, list affected DMAs:		
…have any areas identified by the geotechnical report as posing a public safety risk where infiltration of stormwater could have a negative impact?		X
If Yes, list affected DMAs:		
…have measured in-situ infiltration rates of less than 1.6 inches / hour?	X	
If Yes, list affected DMAs: DMA-A		
…have significant cut and/or fill conditions that would preclude in-situ testing of infiltration rates at the final infiltration surface?		X
If Yes, list affected DMAs:		
…geotechnical report identify other site-specific factors that would preclude effective and safe infiltration?		X
Describe here:		

If you answered "Yes" to any of the questions above for any DMA, Infiltration BMPs should not be used for those DMAs and you should proceed to the assessment for Harvest and Use below.

D.2 Harvest and Use Assessment

Please check what applies*:*

☒ Reclaimed water will be used for the non-potable water demands for the project.

☐Downstream water rights may be impacted by Harvest and Use as approved by the Regional Board (verify with the Copermittee).

☐The Design Capture Volume will be addressed using Infiltration Only BMPs. In such a case, Harvest and Use BMPs are still encouraged, but it would not be required if the Design Capture Volume will be infiltrated or evapotranspired.

If any of the above boxes have been checked, Harvest and Use BMPs need not be assessed for the site. If neither of the above criteria applies, follow the steps below to assess the feasibility of irrigation use, toilet use and other non-potable uses (e.g., industrial use).

Irrigation Use Feasibility

Complete the following steps to determine the feasibility of harvesting stormwater runoff for Irrigation Use BMPs on your site:

Step 1: Identify the total area of irrigated landscape on the site, and the type of landscaping used.

Total Area of Irrigated Landscape: Insert Area (Acres)

Type of Landscaping (Conservation Design or Active Turf): List Landscaping Type

Step 2: Identify the planned total of all impervious areas on the proposed project from which runoff might be feasibly captured and stored for irrigation use. Depending on the configuration of buildings and other impervious areas on the site, you may consider the site as a whole, or parts of the site, to evaluate reasonable scenarios for capturing and storing runoff and directing the stored runoff to the potential use(s) identified in Step 1 above.

Total Area of Impervious Surfaces: Insert Area (Acres)

Step 3: Cross reference the Design Storm depth for the project site (see Exhibit A of the WQMP Guidance Document) with the left column of Table 2-3 in Chapter 2 to determine the minimum area of Effective Irrigated Area per Tributary Impervious Area (EIATIA).

Enter your EIATIA factor: EIATIA Factor

Step 4: Multiply the unit value obtained from Step 3 by the total of impervious areas from Step 2 to develop the minimum irrigated area that would be required.

Minimum required irrigated area: Insert Area (Acres)

Step 5: Determine if harvesting stormwater runoff for irrigation use is feasible for the project by comparing the total area of irrigated landscape (Step 1) to the minimum required irrigated area (Step 4).

Minimum required irrigated area (Step 4)	Available Irrigated Landscape (Step 1)
Insert Area (Acres)	Insert Area (Acres)

Toilet Use Feasibility

Complete the following steps to determine the feasibility of harvesting stormwater runoff for toilet flushing uses on your site:

Step 1: Identify the projected total number of daily toilet users during the wet season, and account for any periodic shut downs or other lapses in occupancy:

Projected Number of Daily Toilet Users: Number of daily Toilet Users

Project Type: Enter 'Residential', 'Commercial', 'Industrial' or 'Schools'

Step 2: Identify the planned total of all impervious areas on the proposed project from which runoff might be feasibly captured and stored for toilet use. Depending on the configuration of buildings and other impervious areas on the site, you may consider the site as a whole, or parts of the site, to evaluate reasonable scenarios for capturing and storing runoff and directing the stored runoff to the potential use(s) identified in Step 1 above.

Total Area of Impervious Surfaces: Insert Area (Acres)

Step 3: Enter the Design Storm depth for the project site (see Exhibit A) into the left column of Table 2-1 in Chapter 2 to determine the minimum number or toilet users per tributary impervious acre (TUTIA).

Enter your TUTIA factor: TUTIA Factor

Step 4: Multiply the unit value obtained from Step 3 by the total of impervious areas from Step 2 to develop the minimum number of toilet users that would be required.

Minimum number of toilet users: Required number of toilet users

Step 5: Determine if harvesting stormwater runoff for toilet flushing use is feasible for the project by comparing the Number of Daily Toilet Users (Step 1) to the minimum required number of toilet users (Step 4).

Minimum required Toilet Users (Step 4)	**Projected number of toilet users (Step 1)**
Insert Area (Acres)	Insert Area (Acres)

Other Non-Potable Use Feasibility

Are there other non-potable uses for stormwater runoff on the site (e.g. industrial use)? See Chapter 2 of the Guidance for further information. If yes, describe below. If no, write N/A.

Insert narrative description here.

Step 1: Identify the projected average daily non-potable demand, in gallons per day, during the wet season and accounting for any periodic shut downs or other lapses in occupancy or operation.

Average Daily Demand: Projected Average Daily Use (gpd)

Step 2: Identify the planned total of all impervious areas on the proposed project from which runoff might be feasibly captured and stored for the identified non-potable use. Depending on the configuration of buildings and other impervious areas on the site, you may consider the site as a whole, or parts of the site, to evaluate reasonable scenarios for capturing and storing runoff and directing the stored runoff to the potential use(s) identified in Step 1 above.

Total Area of Impervious Surfaces: Insert Area (Acres)

Step 3: Enter the Design Storm depth for the project site (see Exhibit A) into the left column of Table 2-3 in Chapter 2 to determine the minimum demand for non-potable uses per tributary impervious acre.

Enter the factor from Table 2-3: Enter Value

Step 4: Multiply the unit value obtained from Step 4 by the total of impervious areas from Step 3 to develop the minimum number of gallons per day of non-potable use that would be required.

Minimum required use: Minimum use required (gpd)

Step 5: Determine if harvesting stormwater runoff for other non-potable use is feasible for the project by comparing the Number of Daily Toilet Users (Step 1) to the minimum required number of toilet users (Step 4).

Minimum required non-potable use (Step 4)	**Projected average daily use (Step 1)**
Minimum use required (gpd)	Projected Average Daily Use (gpd)

If Irrigation, Toilet and Other Use feasibility anticipated demands are less than the applicable minimum values, Harvest and Use BMPs are not required and you should proceed to utilize LID Bioretention and Biotreatment, unless a site-specific analysis has been completed that demonstrates technical infeasibility as noted in D.3 below.

D.3 Bioretention and Biotreatment Assessment

Other LID Bioretention and Biotreatment BMPs as described in Chapter 2.4.7 of the WQMP Guidance Document are feasible on nearly all development sites with sufficient advance planning.

Select one of the following:

☒ LID Bioretention/Biotreatment BMPs will be used for some or all DMAs of the project as noted below in Section D.4 (note the requirements of Section 3.4.2 in the WQMP Guidance Document).

☐ A site-specific analysis demonstrating the technical infeasibility of all LID BMPs has been performed and is included in Appendix 5. If you plan to submit an analysis demonstrating the technical infeasibility of LID BMPs, request a pre-submittal meeting with the Copermittee to discuss this option. Proceed to Section E to document your alternative compliance measures.

D.4 Feasibility Assessment Summaries

From the Infiltration, Harvest and Use, Bioretention and Biotreatment Sections above, complete Table D.2 below to summarize which LID BMPs are technically feasible, and which are not, based upon the established hierarchy.

Table D.2 LID Prioritization Summary Matrix

DMA Name/ID	LID BMP Hierarchy				No LID (Alternative Compliance)
	1. Infiltration	2. Harvest and use	3. Bioretention	4. Biotreatment	
DMA-A	☐	☐	☒	☐	☐
	☐	☐	☐	☐	☐
	☐	☐	☐	☐	☐
	☐	☐	☐	☐	☐
	☐	☐	☐	☐	☐
	☐	☐	☐	☐	☐

For those DMAs where LID BMPs are not feasible, provide a brief narrative below summarizing why they are not feasible, include your technical infeasibility criteria in Appendix 5, and proceed to Section E below to document Alternative Compliance measures for those DMAs. Recall that each proposed DMA must pass through the LID BMP hierarchy before alternative compliance measures may be considered.

N/A

D.5 LID BMP Sizing

Each LID BMP must be designed to ensure that the Design Capture Volume will be addressed by the selected BMPs. First, calculate the Design Capture Volume for each LID BMP using the V_{BMP} worksheet in Appendix F of the LID BMP Design Handbook. Second, design the LID BMP to meet the required V_{BMP} using a method approved by the Copermittee. Utilize the worksheets found in the LID BMP Design Handbook or consult with your Copermittee to assist you in correctly sizing your LID BMPs. Complete Table D.3 below to document the Design Capture Volume and the Proposed Volume for each LID BMP. Provide the completed design procedure sheets for each LID BMP in Appendix 6. You may add additional rows to the table below as needed.

Table D.3 DCV Calculations for LID BMPs

DMA Type/ID	DMA Area (square feet)	Post-Project Surface Type	Effective Impervious Fraction, I_f	DMA Runoff Factor	DMA Areas x Runoff Factor	*Design Storm Depth (in)*	*Design Capture Volume, V_{BMP} (cubic feet)*	*Proposed Volume on Plans (cubic feet)*
	[A]		[B]	[C]	[A] x [C]			
L-A	*20,506*	*LANDSCAPE*	*0.1*	*0.11*	*2,265.1*			
R-A	*567,098*	*ROOF*	*1.0*	*0.89*	*505,851.4*			
H-A	*451,119*	*HARDSCAPE*	*1.0*	*0.89*	*402,398.1*			
BMP-A	*4,800*	*LANDSCAPE*	*0.1*	*0.11*	*530.2*			
SR-A	*105,229*	*LANDSCAPE*						
	1,148,752				911,044.8	0.63	47,829.9	47,850

[B], [C] is obtained as described in Section 2.3.1 of the WQMP Guidance Document
[E] is obtained from Exhibit A in the WQMP Guidance Document
[G] is obtained from a design procedure sheet, such as in LID BMP Design Handbook and placed in Appendix 6

Section E: Alternative Compliance (LID Waiver Program)

LID BMPs are expected to be feasible on virtually all projects. Where LID BMPs have been demonstrated to be infeasible as documented in Section D, other Treatment Control BMPs must be used (subject to LID waiver approval by the Copermittee). Check one of the following Boxes:

☒ LID Principles and LID BMPs have been incorporated into the site design to fully address all Drainage Management Areas. No alternative compliance measures are required for this project and thus this Section is not required to be completed.

- *Or* -

☐ The following Drainage Management Areas are unable to be addressed using LID BMPs. A site-specific analysis demonstrating technical infeasibility of LID BMPs has been approved by the Co-Permittee and included in Appendix 5. Additionally, no downstream regional and/or sub-regional LID BMPs exist or are available for use by the project. The following alternative compliance measures on the following pages are being implemented to ensure that any pollutant loads expected to be discharged by not incorporating LID BMPs, are fully mitigated.

All DMAs will be treated using a bio-retention basin.

E.1 Identify Pollutants of Concern

Utilizing Table A.1 from Section A above which noted your project's receiving waters and their associated EPA approved 303(d) listed impairments, cross reference this information with that of your selected Priority Development Project Category in Table E.1 below. If the identified General Pollutant Categories are the same as those listed for your receiving waters, then these will be your Pollutants of Concern and the appropriate box or boxes will be checked on the last row. The purpose of this is to document compliance and to help you appropriately plan for mitigating your Pollutants of Concern in lieu of implementing LID BMPs.

Table E.1 Potential Pollutants by Land Use Type

Priority Development Project Categories and/or Project Features (check those that apply)	General Pollutant Categories							
	Bacterial Indicators	Metals	Nutrients	Pesticides	Toxic Organic Compounds	Sediments	Trash & Debris	Oil & Grease
☐ Detached Residential Development	P	N	P	P	N	P	P	P
☐ Attached Residential Development	P	N	P	P	N	P	P	P(2)
☒ Commercial/Industrial Development	P(3)	P	P(1)	P(1)	P(5)	P(1)	P	P
☐ Automotive Repair Shops	N	P	N	N	P(4, 5)	N	P	P
☐ Restaurants (>5,000 ft^2)	P	N	N	N	N	N	P	P
☐ Hillside Development (>5,000 ft^2)	P	N	P	P	N	P	P	P
☒ Parking Lots (>5,000 ft^2)	P(6)	P	P(1)	P(1)	P(4)	P(1)	P	P
☐ Retail Gasoline Outlets	N	P	N	N	P	N	P	P
Project Priority Pollutant(s) of Concern	☒	☐	☒	☐	☒	☐	☐	☐

P = Potential

N = Not Potential

(1) *A potential Pollutant if non-native landscaping exists or is proposed onsite; otherwise not expected*

(2) *A potential Pollutant if the project includes uncovered parking areas; otherwise not expected*

(3) *A potential Pollutant is land use involving animal waste*

(4) *Specifically petroleum hydrocarbons*

(5) *Specifically solvents*

(6) *Bacterial indicators are routinely detected in pavement runoff*

E.2 Stormwater Credits

Projects that cannot implement LID BMPs but nevertheless implement smart growth principles are potentially eligible for Stormwater Credits. Utilize Table 3-8 within the WQMP Guidance Document to identify your Project Category and its associated Water Quality Credit. If not applicable, write N/A.

Table E.2 Water Quality Credits

Qualifying Project Categories	Credit Percentage[2]
N/A	
Total Credit Percentage[1]	

[1]Cannot Exceed 50%
[2]Obtain corresponding data from Table 3-8 in the WQMP Guidance Document

E.3 Sizing Criteria

After you appropriately considered Stormwater Credits for your project, utilize Table E.3 below to appropriately size them to the DCV, or Design Flow Rate, as applicable. Please reference Chapter 3.5.2 of the WQMP Guidance Document for further information.

Table E.3 Treatment Control BMP Sizing

DMA Type/ID	DMA Area (square feet)	Post-Project Surface Type	Effective Impervious Fraction, I_f	DMA Runoff Factor	DMA Area x Runoff Factor		***Enter BMP Name / Identifier Here***		
	[A]		[B]	[C]	[A] x [C]				
						Design Storm Depth (in)	*Minimum Design Capture Volume or Design Flow Rate (cubic feet or cfs)*	*Total Storm Water Credit % Reduction*	*Proposed Volume or Flow on Plans (cubic feet or cfs)*
N/A									
	A_T = Σ[A]				Σ= [D]	[E]	$[F] = \frac{[D]x[E]}{[G]}$	[F] X (1-[H])	[I]

[B], [C] is obtained as described in Section 2.3.1 from the WQMP Guidance Document
[E] is obtained from Exhibit A in the WQMP Guidance Document
[G] is for Flow-Based Treatment Control BMPs [G] = 43,560, for Volume-Based Control Treatment BMPs, [G] = 12
[H] is from the Total Credit Percentage as Calculated from Table E.2 above
[I] as obtained from a design procedure sheet from the BMP manufacturer and should be included in Appendix 6

E.4 Treatment Control BMP Selection

Treatment Control BMPs typically provide proprietary treatment mechanisms to treat potential pollutants in runoff, but do not sustain significant biological processes. Treatment Control BMPs must have a removal efficiency of a medium or high effectiveness as quantified below:

- **High**: equal to or greater than 80% removal efficiency
- **Medium**: between 40% and 80% removal efficiency

Such removal efficiency documentation (e.g., studies, reports, etc.) as further discussed in Chapter 3.5.2 of the WQMP Guidance Document, must be included in Appendix 6. In addition, ensure that proposed Treatment Control BMPs are properly identified on the WQMP Site Plan in Appendix 1.

Table E.4 Treatment Control BMP Selection

Selected Treatment Control BMP Name or ID[1]	Priority Pollutant(s) of Concern to Mitigate[2]	Removal Efficiency Percentage[3]
N/A		

[1] Treatment Control BMPs must not be constructed within Receiving Waters. In addition, a proposed Treatment Control BMP may be listed more than once if they possess more than one qualifying pollutant removal efficiency.
[2] Cross Reference Table E.1 above to populate this column.
[3] As documented in a Co-Permittee Approved Study and provided in Appendix 6.

Section F: Hydromodification

F.1 Hydrologic Conditions of Concern (HCOC) Analysis

Once you have determined that the LID design is adequate to address water quality requirements, you will need to assess if the proposed LID Design may still create a HCOC. Review Chapters 2 and 3 (including Figure 3-7) of the WQMP Guidance Document to determine if your project must mitigate for Hydromodification impacts. If your project meets one of the following criteria which will be indicated by the check boxes below, you do not need to address Hydromodification at this time. However, if the project does not qualify for Exemptions 1, 2 or 3, then additional measures must be added to the design to comply with HCOC criteria. This is discussed in further detail below in Section F.2.

HCOC EXEMPTION 1: The Priority Development Project disturbs less than one acre. The Copermittee has the discretion to require a Project-Specific WQMP to address HCOCs on projects less than one acre on a case by case basis. The disturbed area calculation should include all disturbances associated with larger common plans of development.

Does the project qualify for this HCOC Exemption? ☐ Y ☒ N

If Yes, HCOC criteria do not apply.

HCOC EXEMPTION 2: The volume and time of concentration[1] of storm water runoff for the post-development condition is not significantly different from the pre-development condition for a 2-year return frequency storm (a difference of 5% or less is considered insignificant) using one of the following methods to calculate:

- Riverside County Hydrology Manual
- Technical Release 55 (TR-55): Urban Hydrology for Small Watersheds (NRCS 1986), or derivatives thereof, such as the Santa Barbara Urban Hydrograph Method
- Other methods acceptable to the Co-Permittee

Does the project qualify for this HCOC Exemption? ☐ Y ☒ N

If Yes, report results in Table F.1 below and provide your substantiated hydrologic analysis in Appendix 7.

Table F.1 Hydrologic Conditions of Concern Summary

	2 year – 24 hour		
	Pre-condition	**Post-condition**	**% Difference**
Time of Concentration	INSERT VALUE	INSERT VALUE	INSERT VALUE
Volume (Cubic Feet)	INSERT VALUE	INSERT VALUE	INSERT VALUE

[1] Time of concentration is defined as the time after the beginning of the rainfall when all portions of the drainage basin are contributing to flow at the outlet.

HCOC EXEMPTION 3: All downstream conveyance channels to an adequate sump (for example, Prado Dam, Lake Elsinore, Canyon Lake, Santa Ana River, or other lake, reservoir or naturally erosion resistant feature) that will receive runoff from the project are engineered and regularly maintained to ensure design flow capacity; no sensitive stream habitat areas will be adversely affected; or are not identified on the Co-Permittees Hydromodification Sensitivity Maps.

Does the project qualify for this HCOC Exemption? ☐ Y ☒ N

If Yes, HCOC criteria do not apply and note below which adequate sump applies to this HCOC qualifier:

F.2 HCOC Mitigation

If none of the above HCOC Exemption Criteria are applicable, HCOC criteria is considered mitigated if they meet one of the following conditions:

a. Additional LID BMPS are implemented onsite or offsite to mitigate potential erosion or habitat impacts as a result of HCOCs. This can be conducted by an evaluation of site-specific conditions utilizing accepted professional methodologies published by entities such as the California Stormwater Quality Association (CASQA), the Southern California Coastal Water Research Project (SCCRWP), or other Co-Permittee approved methodologies for site-specific HCOC analysis.

b. The project is developed consistent with an approved Watershed Action Plan that addresses HCOC in Receiving Waters.

c. Mimicking the pre-development hydrograph with the post-development hydrograph, for a 2-year return frequency storm. Generally, the hydrologic conditions of concern are not significant, if the post-development hydrograph is no more than 10% greater than pre-development hydrograph. In cases where excess volume cannot be infiltrated or captured and reused, discharge from the site must be limited to a flow rate no greater than 110% of the pre-development 2-year peak flow.

Be sure to include all pertinent documentation used in your analysis of the items a, b or c in Appendix 7.

The project is located within the approved Hydromodification exempt area based on the approved HCOC Applicability Map (approved April 20, 2017) Furnished by the Santa Ana Region Co-Permittees.

Section G: Source Control BMPs

Source control BMPs include permanent, structural features that may be required in your project plans — such as roofs over and berms around trash and recycling areas — and Operational BMPs, such as regular sweeping and "housekeeping", that must be implemented by the site's occupant or user. The MEP standard typically requires both types of BMPs. In general, Operational BMPs cannot be substituted for a feasible and effective permanent BMP. Using the Pollutant Sources/Source Control Checklist in Appendix 8, review the following procedure to specify Source Control BMPs for your site:

1. ***Identify Pollutant Sources*****:** Review Column 1 in the Pollutant Sources/Source Control Checklist. Check off the potential sources of Pollutants that apply to your site.

2. ***Note Locations on Project-Specific WQMP Exhibit*****:** Note the corresponding requirements listed in Column 2 of the Pollutant Sources/Source Control Checklist. Show the location of each Pollutant source and each permanent Source Control BMP in your Project-Specific WQMP Exhibit located in Appendix 1.

3. ***Prepare a Table and Narrative*****:** Check off the corresponding requirements listed in Column 3 in the Pollutant Sources/Source Control Checklist. In the left column of Table G.1 below, list each potential source of runoff Pollutants on your site (from those that you checked in the Pollutant Sources/Source Control Checklist). In the middle column, list the corresponding permanent, Structural Source Control BMPs (from Columns 2 and 3 of the Pollutant Sources/Source Control Checklist) used to prevent Pollutants from entering runoff. **Add additional narrative** in this column that explains any special features, materials or methods of construction that will be used to implement these permanent, Structural Source Control BMPs.

4. ***Identify Operational Source Control BMPs:*** To complete your table, refer once again to the Pollutant Sources/Source Control Checklist. List in the right column of your table the Operational BMPs that should be implemented as long as the anticipated activities continue at the site. Copermittee stormwater ordinances require that applicable Source Control BMPs be implemented; the same BMPs may also be required as a condition of a use permit or other revocable Discretionary Approval for use of the site.

Table G.1 Permanent and Operational Source Control Measures

Potential Sources of Runoff pollutants	Permanent Structural Source Control BMPs	Operational Source Control BMPs
A. On-site storm drain catch basins and grated inlets. Locations are shown on the PWQMP Exhibit in Appendix 1.	*On-site storm drain signage will utilize language, "No Dumping Drains to River", or equally approved text that is consistent with the City of Perris' requirements. Landscape area drains surrounded by vegetation will not be signed. Catch Basin Markers may be available from the Riverside County Flood Control and Water District Conservation District, call 951-955-1200 to verify.*	*Maintain and periodically repaint or replace inlet markings.* *Provide stormwater pollution prevention information to new site owners, lessees, or operators.* *See applicable operational BMPs in Fact Sheet SC-44, "Drainage System Maintenance," in Appendix 10 (CASQA Stormwater Quality Handbook at* *www.cabmphandbooks.com*

	On-site drainage structures, including all storm drain clean outs, area drains, inlets, catch basins, inlet & outlet structures, forebays, & water treatment control basins shall be inspected and maintained on a regular basis to insure their operational adequacy.	*Include the following in lessee agreements: "Tenants shall not allow anyone to discharge anything to storm drains or to store or deposit materials so as to create a potential discharge to storm drains"* *Maintenance should include removal of trash, debris, & sediment and the repair of any deficiencies or damage that may impact water quality.*
B. Interior floor drains and elevator shaft sump	*The interior floor drains and elevator shaft sump pumps will be plumbed to sanitary sewer*	*Inspect and maintain drains to prevent blockages and overflow.*
C. Landscape/Outdoor Pesticide Use	*The final landscape shall be designed to accomplish all of the following:* *Preserve existing native trees, shrubs and ground cover to the maximum extent possible.* *Design landscape to minimize irrigation and runoff, to promote surface infiltration where appropriate and to minimize the use of fertilizers and pesticides that can contribute to stormwater pollution.* *Where landscaped areas are used to retain or detain stormwater, specify plants that are tolerant of saturated soil conditions.* *Consider using pest-resistant plants, especially adjacent to hardscape.* *To insure successful establishments, select plants appropriate to site, soils, slopes, climate, sun, wind, rain, land use, air movement, ecological consistency and plant interactions.* *Pesticide usage should be at a necessary minimum and be consistent with the instructions contained on product labels and with the regulations administered by the State Department of Pesticide Regulation.* *Pesticides should be used at an absolute minimum or not at all in the retention/infiltration basin. If used, it*	*Maintain landscaping using minimum or no pesticides* *See applicable operational BMPs in "What you should know for…. Landscape and Gardening" at http://rcflood.org/stormwater and Appendix 10.* *Provide IPM information to new owners, lessees and operators.* *Landscape maintenance should include mowing, weeding, trimming, removal of trash & debris, repair of erosion, re-vegetation, and removal of cut & dead vegetation.* *Irrigation maintenance should include the repair of leaky or broken sprinkler heads, the maintaining of timing apparatus accuracy, and the maintaining of shut off valves in good working order.*

	should not be applied in close proximity to the rainy season.	
D. Refuse Trash Storage areas	*Trash container storage areas shall be paved with an impervious surface, designed not to allow run-on from adjoining areas, designed to divert drainage from adjoining roofs and pavements from the surrounding area, and screened or walled to prevent off-site transport of trash.* *Trash dumpsters (containers) shall be leak proof and have attached covers or lids.* *Trash enclosures shall be roofed per City standards and the details on the PWQMP Exhibit in Appendix 1.* *Trash compactors shall be roofed and set on a concrete pad per City standards. The pad shall be a minimum of one foot larger all around than the trash compactor and sloped to drain to a sanitary sewer line. Connection of trash area drains to the MS4 is prohibited.* *See CASQA SD-32 BMP Fact Sheets in Appendix 10 for additional information.* *Signs shall be posted on or near dumpsters with the words "Do not dump hazardous materials here" or similar.*	*Adequate number of receptacles shall be provided. Inspect receptacles regularly; repair or replace leaky receptacles. Keep receptacles covered. Prohibit/prevent dumping of liquid or hazardous wastes. Post "no hazardous materials" signs. Inspect and pick up litter daily and clean up spills immediately. Keep spill control materials available on-site. See Fact Sheet SC-34, in Appendix 10, "Waste Handling and Disposal" in the CASQA Stormwater Quality Handbook at www.cabmphandbooks.com*
E. Loading Docks	*Loading docks will not be covered and are 4 feet above finished pavement surface.* *Spill kits are to be kept on-site at all times per SC-11.*	*Move loaded and unloaded items indoors as soon as possible.* *Inspect for accumulated trash and debris. Implement good housekeeping procedures on a regular basis. Sweep areas clean instead of using wash water. Loading docks will be kept in a clean and orderly condition, through a regular program of sweeping and litter control, and immediate clean up of any spills or broken containers. Property owner will ensure that loading docks will be swept as needed. Cleanup procedures will not include the use of wash-down water. Property owner will be responsible for implementation of loading dock housekeeping procedures* *See the Fact Sheet SC-30, in Appendix 10, "Outdoor Loading*

		and Unloading" in the CASQA Stormwater Quality Handbooks at www.cabmphandbooks.com
F. Fire Sprinkler Test Water	*Provide a means to drain fire sprinkler test water to the sanitary sewer.*	*See the note in the Fact Sheet SC-41, in Appendix 10, "Building and Grounds Maintenance", in the CASQA Stormwater Quality Handbooks at www.cabmphandbooks.com*
G. Miscellaneous Drain or Wash Water or Other Sources *Boiler drain lines* *Condensate drain lines* *Rooftop equipment* *Drainage sumps* *Roofing, gutters and trim* *Other sources*	*Boiler drain lines shall be directly or indirectly connected to the sanitary sewer system and may not discharge to the storm drain system* *Condensate drain lines may discharge to landscaped areas if the flow is small enough that runoff will not occur.* *Condensate drain lines may not discharge to the storm drain system.* *Rooftop equipment with potential to produce pollutants shall be roofed and/or have secondary containment.* *Any drainage sumps on-site shall feature a sediment sump to reduce the quantity of sediment in pumped water.* *Avoid roofing, gutters and trim made of copper of other unprotected metals that may leach into runoff.* *Include controls for other sources as specified by local reviewer.*	
H. Plazas, sidewalks, and parking lots	*Spill kits are to be kept on-site at all times per SC-11.*	*Sweep plazas, sidewalks, and parking lots regularly to prevent accumulation of litter and debris. Collect debris from pressure washing to prevent entry into the storm drain system. Collect washwater containing any cleaning agent or degreaser and discharge to the sanitary sewer not to a storm drain.*

Section H: Construction Plan Checklist

Populate Table H.1 below to assist the plan checker in an expeditious review of your project. The first two columns will contain information that was prepared in previous steps, while the last column will be populated with the corresponding plan sheets. This table is to be completed with the submittal of your final Project-Specific WQMP.

Table H.1 Construction Plan Cross-reference

BMP No. or ID	BMP Identifier and Description	Corresponding Plan Sheet(s)
*	*	*

Note that the updated table — or Construction Plan WQMP Checklist — is **only a reference tool** to facilitate an easy comparison of the construction plans to your Project-Specific WQMP. Co-Permittee staff can advise you regarding the process required to propose changes to the approved Project-Specific WQMP.

***To be completed during final engineering.**

Section I: Operation, Maintenance and Funding

The Copermittee will periodically verify that Stormwater BMPs on your site are maintained and continue to operate as designed. To make this possible, your Copermittee will require that you include in Appendix 9 of this Project-Specific WQMP:

1. A means to finance and implement facility maintenance in perpetuity, including replacement cost.
2. Acceptance of responsibility for maintenance from the time the BMPs are constructed until responsibility for operation and maintenance is legally transferred. A warranty covering a period following construction may also be required.
3. An outline of general maintenance requirements for the Stormwater BMPs you have selected.
4. Figures delineating and designating pervious and impervious areas, location, and type of Stormwater BMP, and tables of pervious and impervious areas served by each facility. Geo-locating the BMPs using a coordinate system of latitude and longitude is recommended to help facilitate a future statewide database system.
5. A separate list and location of self-retaining areas or areas addressed by LID Principles that do not require specialized O&M or inspections but will require typical landscape maintenance as noted in Chapter 5, pages 85-86, in the WQMP Guidance. Include a brief description of typical landscape maintenance for these areas.

Your local Co-Permittee will also require that you prepare and submit a detailed Stormwater BMP Operation and Maintenance Plan that sets forth a maintenance schedule for each of the Stormwater BMPs built on your site. An agreement assigning responsibility for maintenance and providing for inspections and certification may also be required.

Details of these requirements and instructions for preparing a Stormwater BMP Operation and Maintenance Plan are in Chapter 5 of the WQMP Guidance Document.

Maintenance Mechanism: WQMP Covenant and Agreement

Will the proposed BMPs be maintained by a Home Owners' Association (HOA) or Property Owners Association (POA)?

☐ Y ☒ N

Include your Operation and Maintenance Plan and Maintenance Mechanism in Appendix 9. Additionally, include all pertinent forms of educational materials for those personnel that will be maintaining the proposed BMPs within this Project-Specific WQMP in Appendix 10.

***More information to be provided during final engineering.**

Appendix 1: Maps and Site Plans

Location Map, WQMP Site Plan and Receiving Waters Map

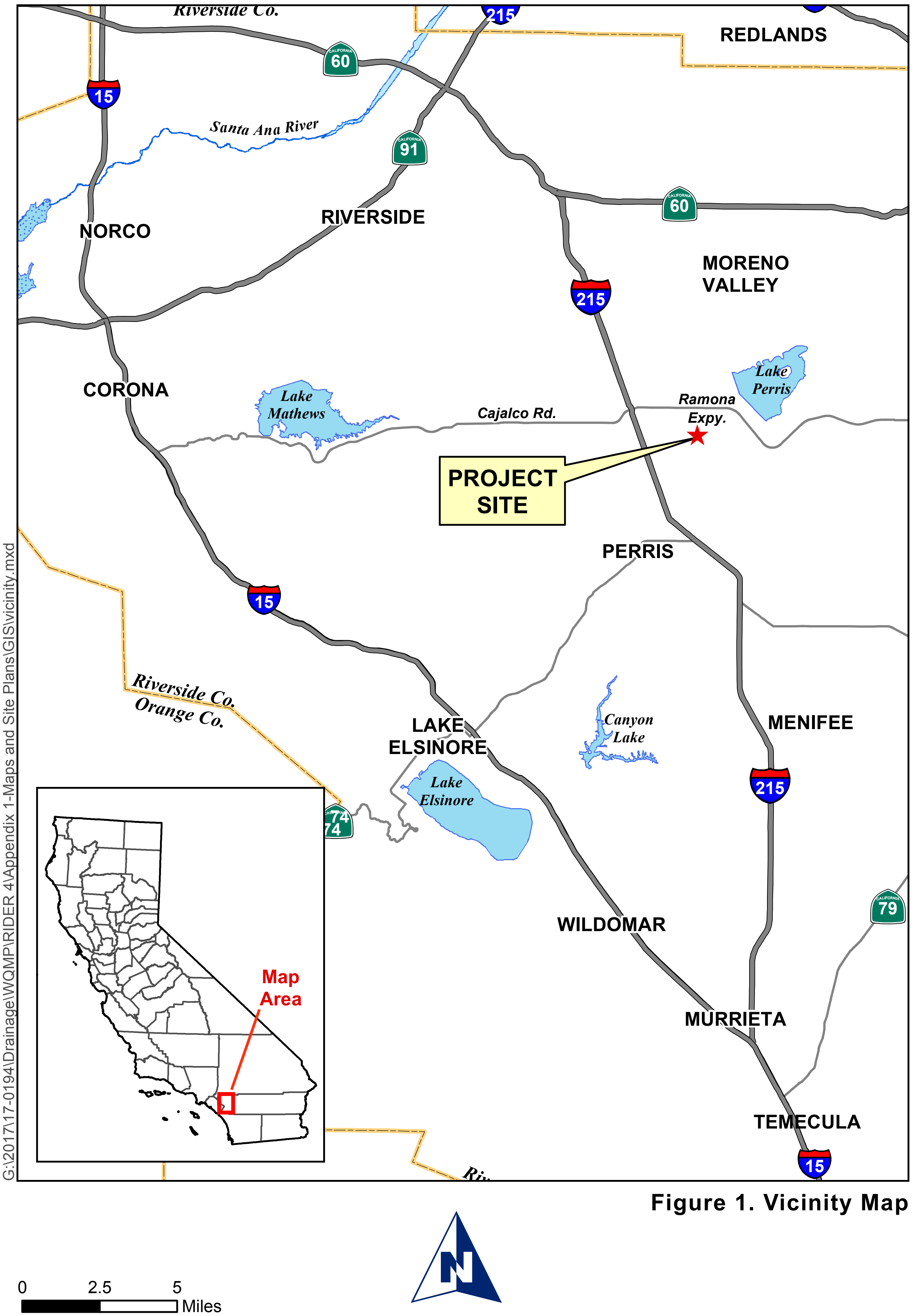

Figure 1. Vicinity Map

Albert A. WEBB Associates

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Sources: ESRI / USGS 7.5min Quad
DRGs: PERRIS

Figure 2. USGS Topography Map

0 1,000 2,000
Feet

Albert A. WEBB Associates

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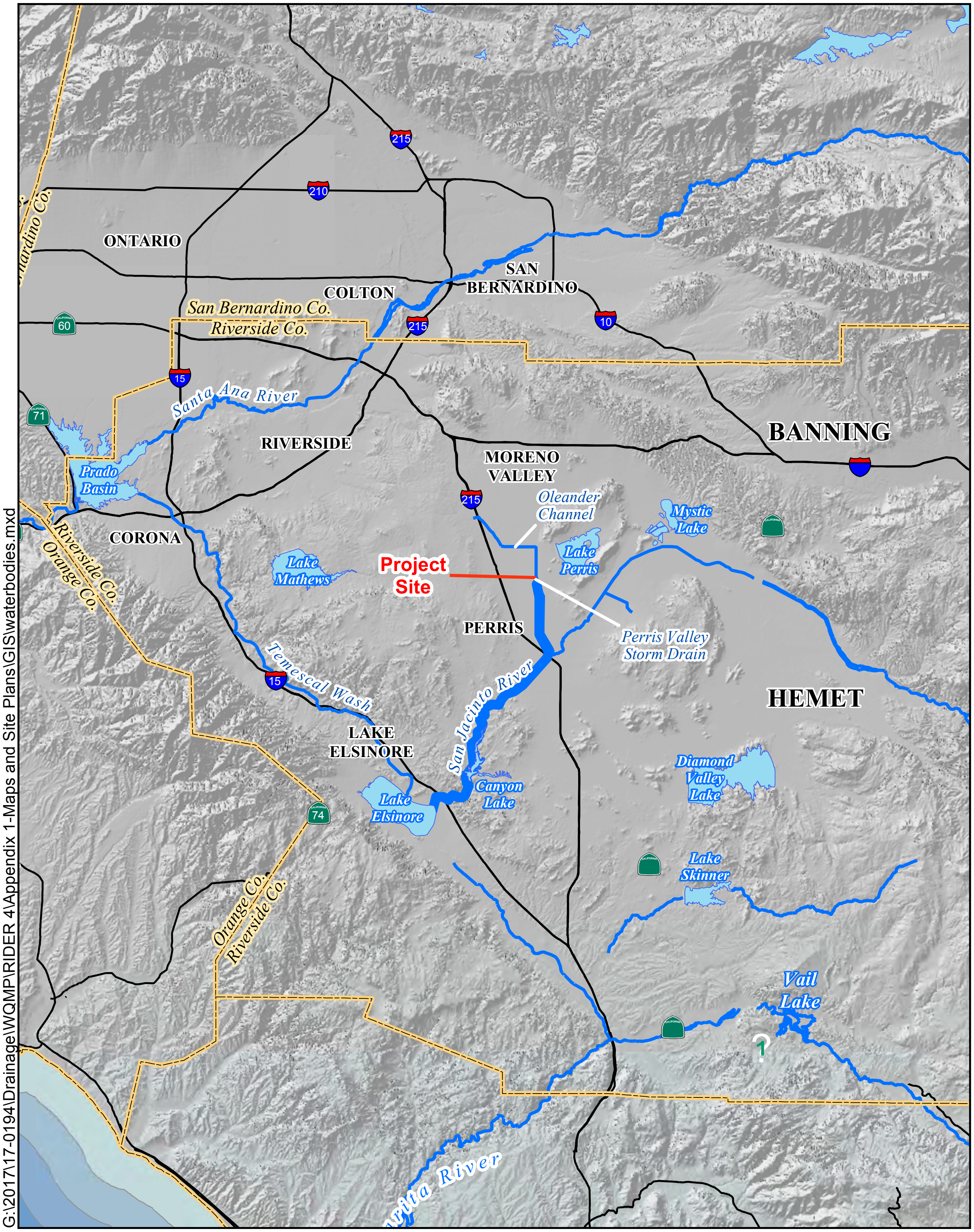
Sources: County of Riverside GIS, 2013;
Eagle Aerial, April 2012.

Figure 3. Aerial Photograph

0 400 800
Feet

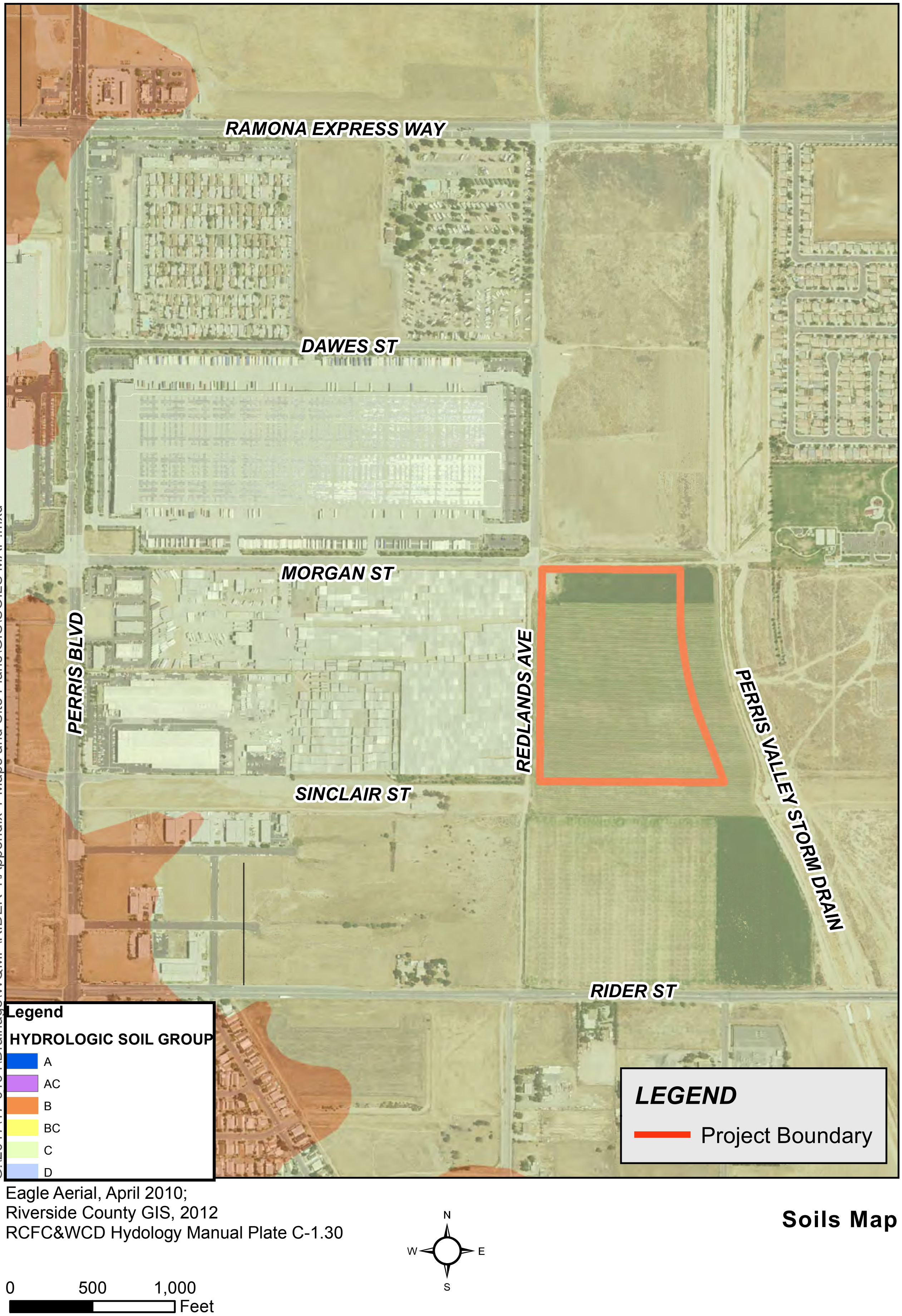


Sources: USGS 30 Meter DEM;
USGS Digital Line Graph

Figure 4. Receiving Waterbodies

0 2 4 6
Miles

Flowpath



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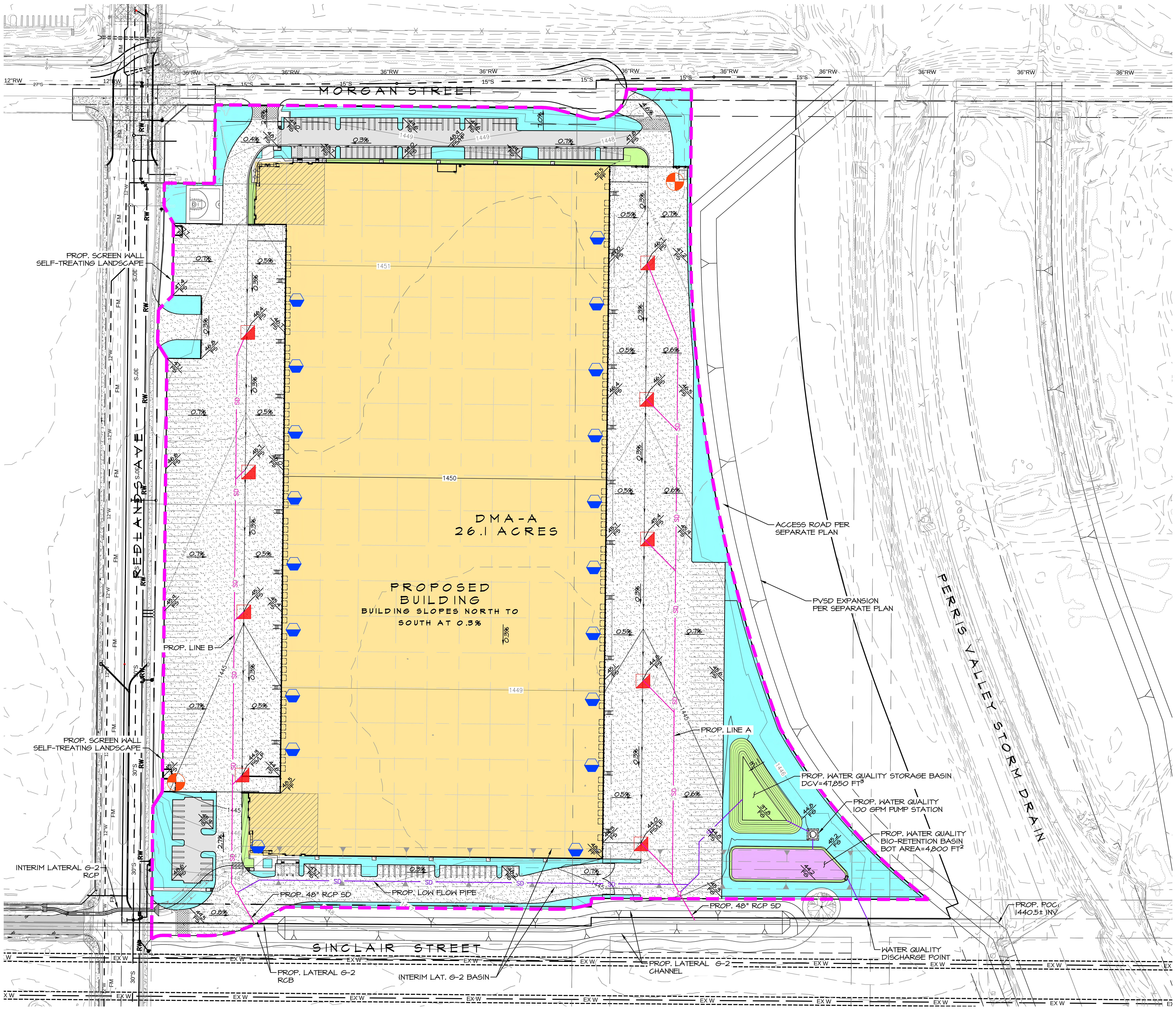
Eagle Aerial, April 2010;
Riverside County GIS, 2012
RCFC&WCD Hydology Manual Plate C-1.30

Soils Map

Albert A. WEBB Associates



LEGEND

- DRAINAGE MANAGEMENT BOUNDARY
- LANDSCAPING
- ROOF
- CONCRETE OR ASPHALT
- SELF RETAINING
- PROPOSED MODULAR WETLAND SYSTEM
- FLOW DIRECTION
- SD — STORM DRAIN PIPE
- SD — LOW FLOW STORM DRAIN PIPE
- ROOF DRAIN DOWNSPOUT
- CURB OPENING/CUT
- STORM INLET
- TRASH ENCLOSURE

DRAINAGE MANAGEMENT AREAS

LEGEND	DMA-ID	TYPE	AREA (SF)
	L-A	LANDSCAPE	20,506
	R-A	ROOF	567,098
	H-A	HARDSCAPE	451,119
	BMP-A	LANDSCAPE	4,800
	SR-A	SELF-RETAINING LANDSCAPE	105,229

GENERAL NOTES

1. THIS PRELIMINARY WATER QUALITY REPORT IS BASED ON THE CURRENT AVAILABLE INFORMATION AND IS SUBJECT TO MINOR MODIFICATIONS.
2. ALL SELF-RETAINING AREAS ARE DEPRESSED A MINIMUM OF 2-INCHES. SEE SHEET 2 FOR FURTHER SELF-RETAINING DETAILS.
3. THE PROJECT SITE PROVIDES 12.2% OF LANDSCAPE COVER AND APPROXIMATELY 80% OF THE LANDSCAPE AREAS WILL BE USED AS SELF-RETAINING.

N
W E
S

1" = 80'
40 0 80 160 240

CITY OF PERRIS

POST CONSTRUCTION BMP SITE MAP
RIDER DISTRIBUTION CENTER IV
19-00006

SCALE: 1"=80'	ALBERT A. WEBB ASSOCIATES	ENGINEERING CONSULTANTS 3788 McCRAY STREET RIVERSIDE CA. 92506 PH. (951) 686-1070 FAX (951) 788-1256	W.O. 17-0358
DATE: 9/19/19			SHEET 1 OF 2 SHEETS
DESIGNED: TSW			
CHECKED: DJA			
PLN CK REF:			DWG. NO.
F.B.			

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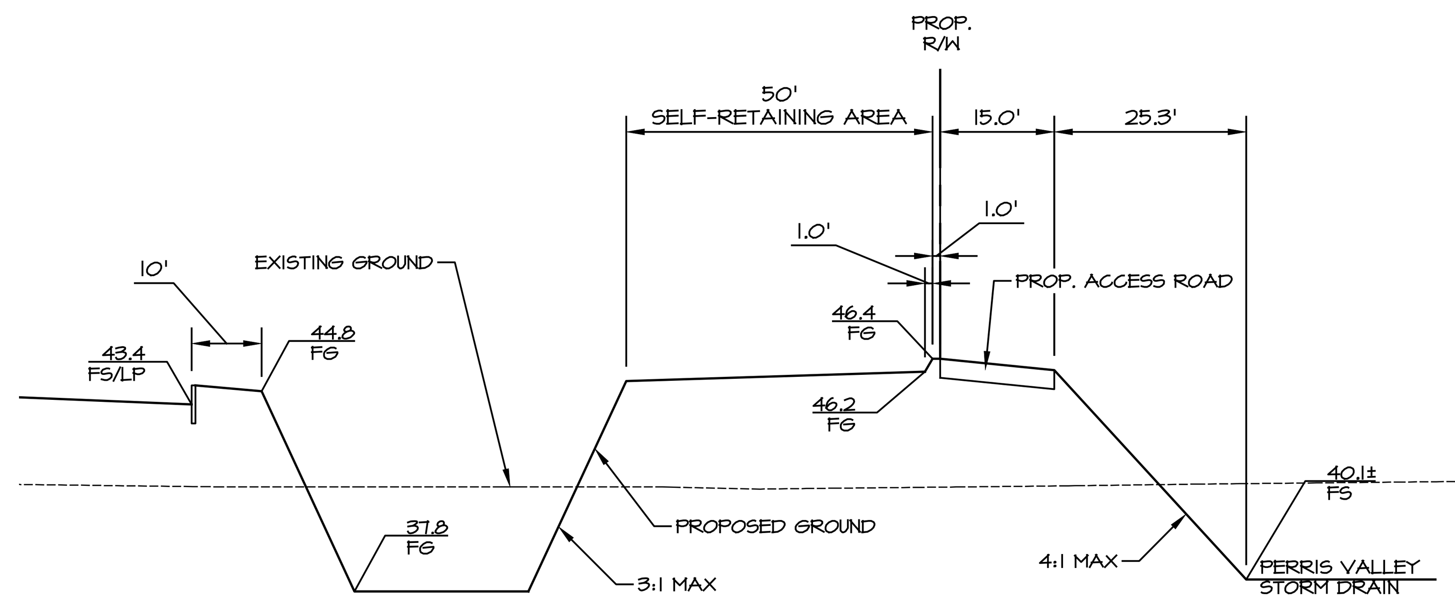
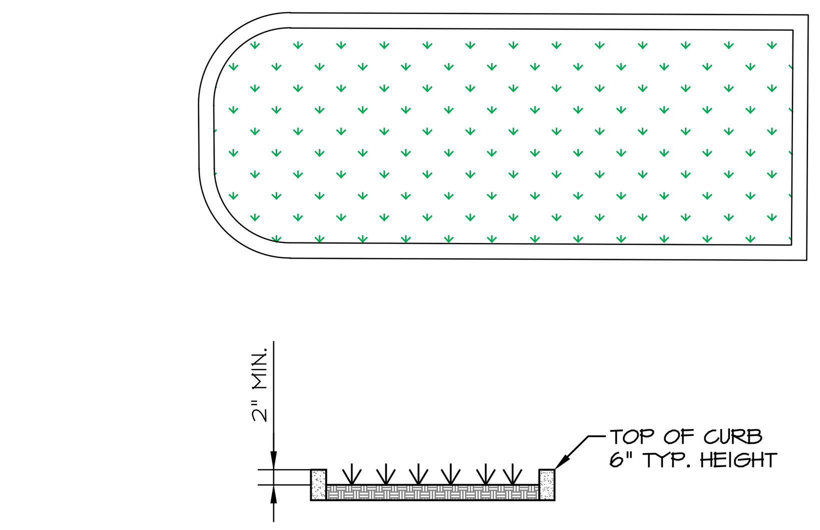


WATER QUALITY STORAGE BASIN ALONG PVSD FRONTAGE TYPICAL SECTION

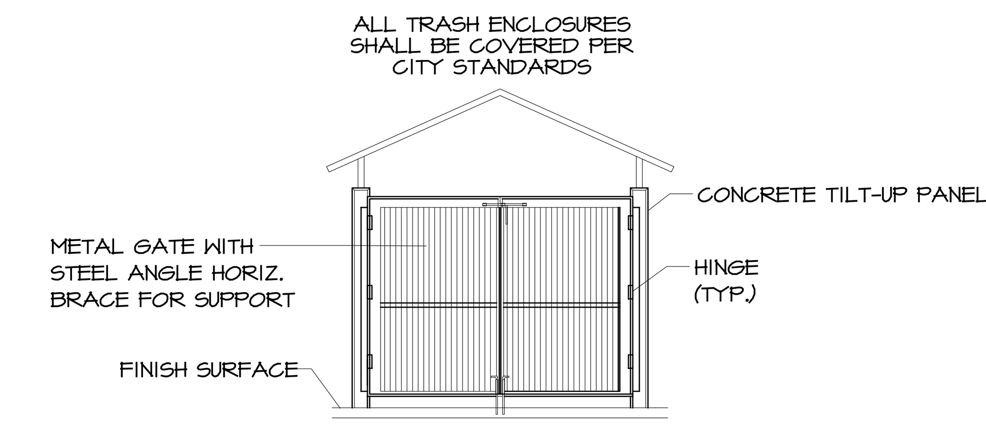
NTS



SELF-RETAINING FINGER ISLAND TYPICAL SECTION

NTS

ALL SELF-RETAINING AREAS WILL BE DEPRESSED A MINIMUM OF 2-INCHES



TRASH ENCLOSURE GATE ELEVATION

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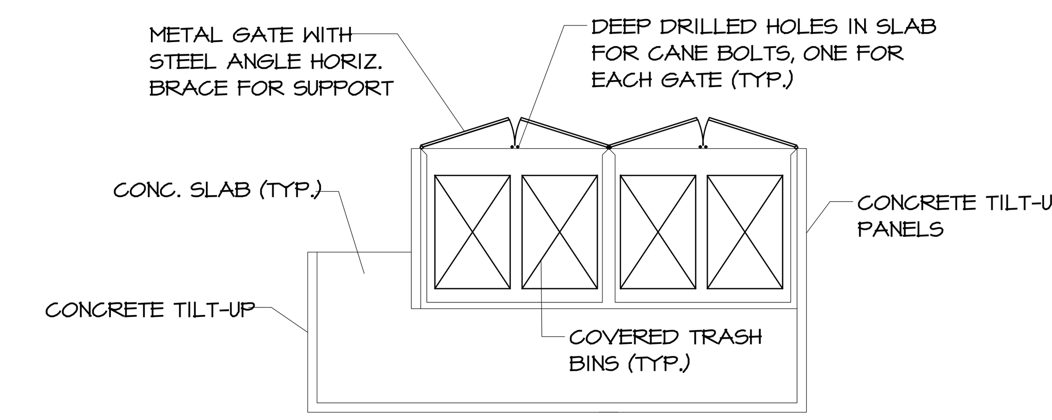
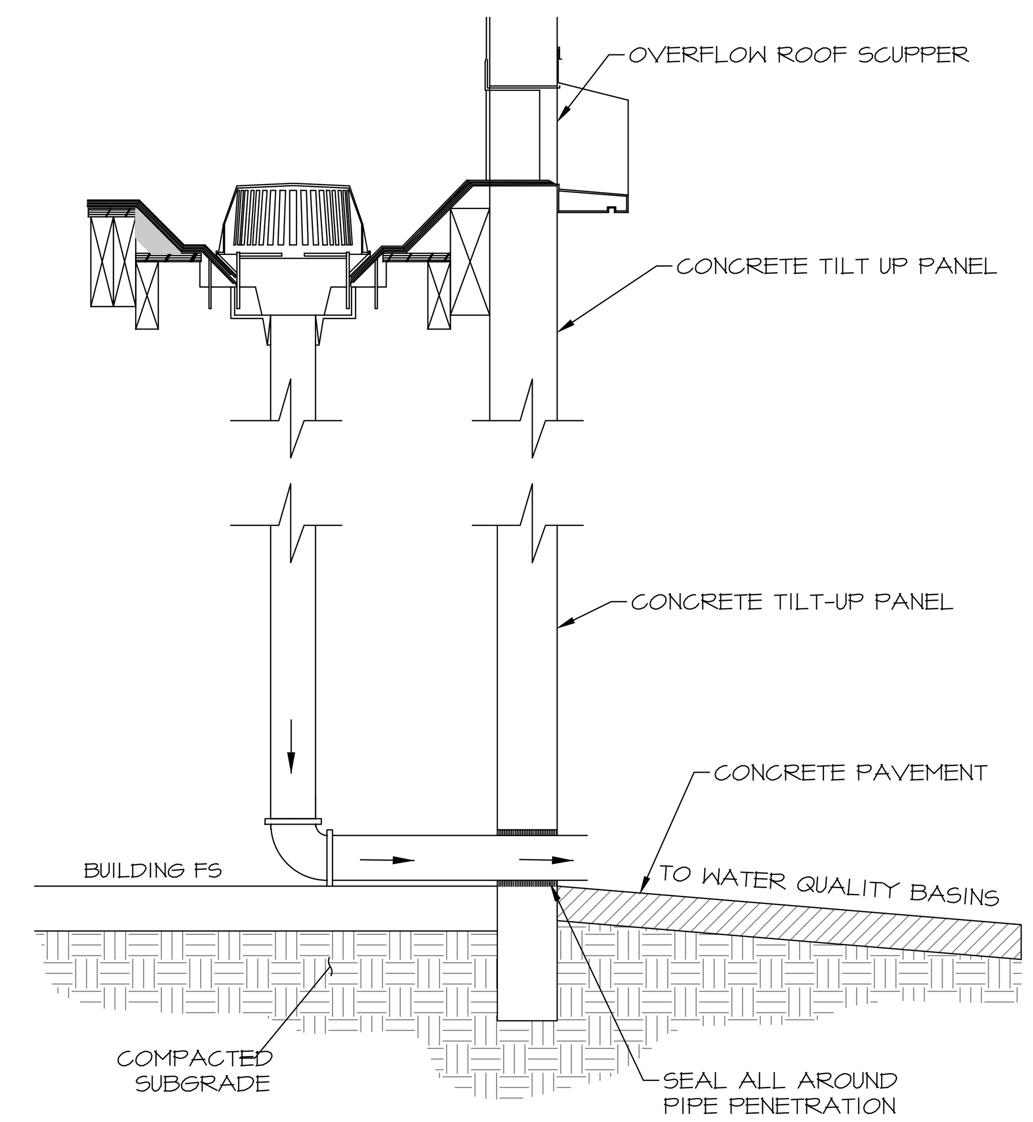


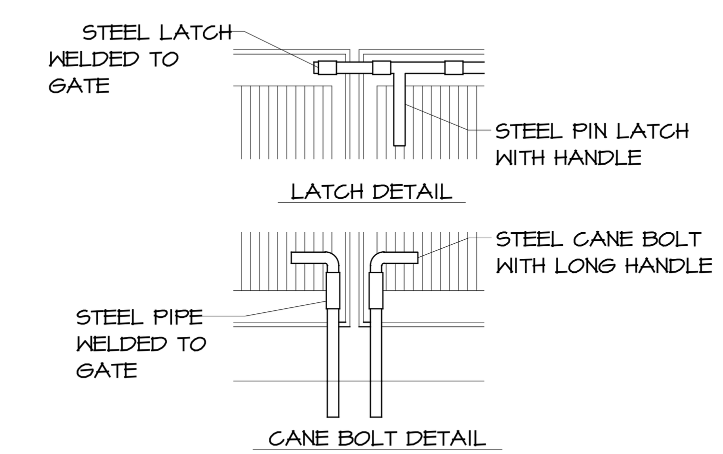
TRASH ENCLOSURE PLAN DETAIL

N.T.S.



ROOF DRAIN DETAIL

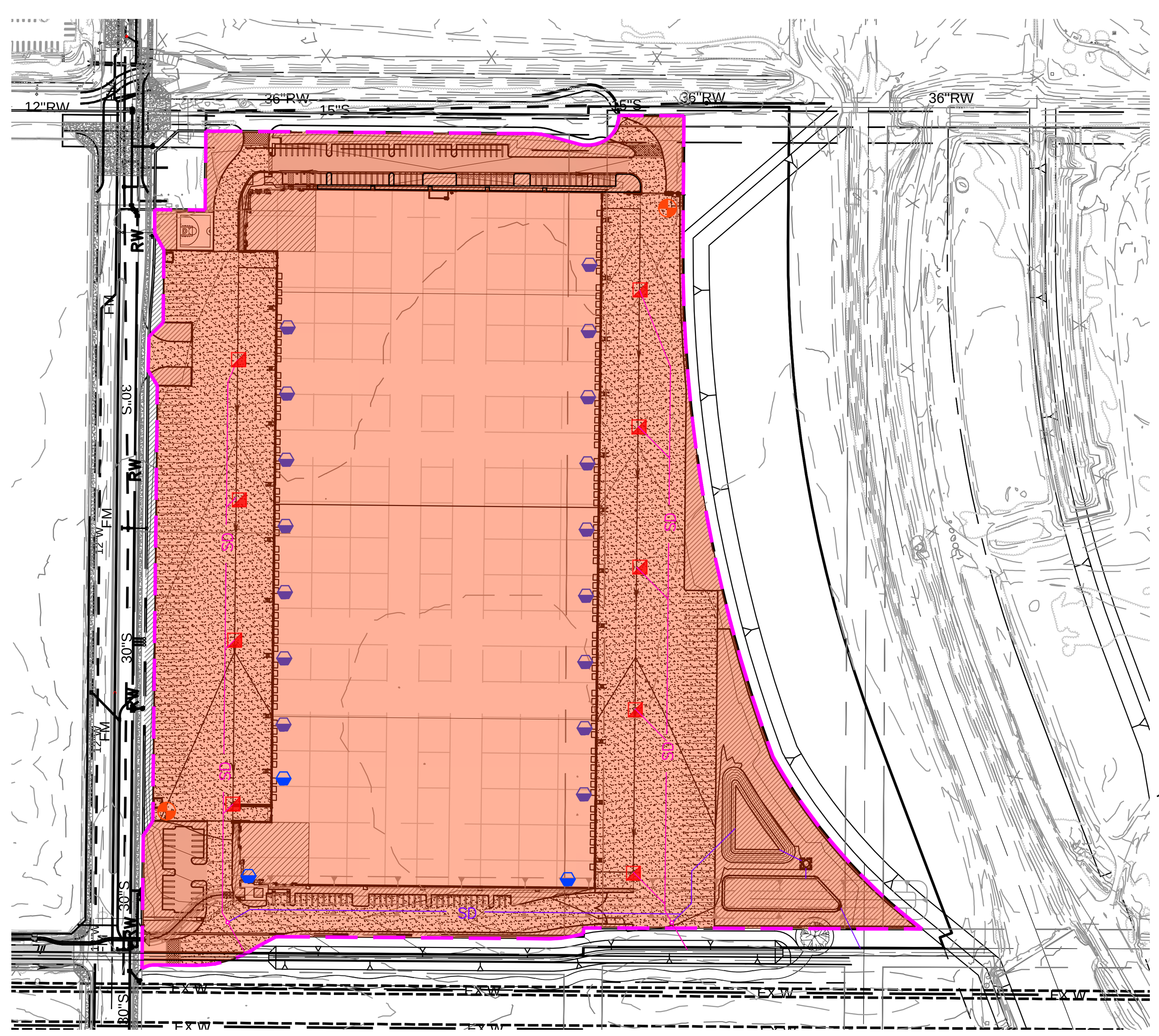
N.T.S.



NOTE:
LATCH AND CANE TO BE AT EXTERIOR SIDE OF GATES

TRASH ENCLOSURE GATE LATCHES DETAIL

N.T.S.



DMA SITE MAP

DMA-A

N
W WEBB E
S

1" = 200'

100 0 200 400 600

CITY OF PERRIS

POST CONSTRUCTION DMA MAP & DETAILS
RIDER DISTRIBUTION CENTER IV
19-00006

SCALE: AS SHOWN	ALBERT A. WEBB ASSOCIATES	ENGINEERING CONSULTANTS 3788 McCRAY STREET RIVERSIDE CA. 92506 PH. (951) 686-1070 FAX (951) 788-1256	W.O. 17-0358
DATE: 9/19/19			SHEET 2
DESIGNED: TSW			OF 2 SHEETS
CHECKED: DJA			DWG. NO.
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Appendix 2: Construction Plans

Grading and Drainage Plans

Appendix 3: Soils Information

Geotechnical Study and Other Infiltration Testing Data

GEOTECHNICAL INVESTIGATION
RIDER 4 – PROPOSED
COMMERCIAL/INDUSTRIAL BUILDING

SEC Redlands Avenue at Morgan Street
Perris, California
for
IDI Gazeley

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

November 30, 2017

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

IDI Gazeley
8 Corporate Park, Suite 300-34
Irvine, California 92606

Attention: Mr. Stephen Hollis

Project No.: **17G206-1**

Subject: **Geotechnical Investigation**
Rider 4 – Proposed Commercial/Industrial Building
SEC Redlands Avenue at Morgan Street
Perris, California

Dear Mr. Hollis:

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.

Gregory K. Mitchell, GE 2364
Principal Engineer

REGISTERED PROFESSIONAL ENGINEER
GREGORY K. MITCHELL
No. 2364
GEOTECHNICAL
STATE OF CALIFORNIA

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Principal Engineer

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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.

Geotechnical Design Considerations

- The subject site is located within an area of moderate liquefaction susceptibility.
- Our site-specific liquefaction evaluation included two borings extended to depths of 50± feet. Liquefiable soils were encountered at both of these boring locations.
- The potential liquefaction-induced settlements range from 0.8± inches to 2.6± inches.
- Based on the estimated magnitude of the differential settlements, the proposed structure may be supported on shallow foundations. Additional design considerations related to the potentially liquefiable soils are presented within the text of this report.

Site Preparation

- Initial site preparation should include stripping of the existing crop stubble as well as any existing native grass and weed growth.
- The near-surface soils generally consist of low expansive native alluvium which possesses a moderate potential for consolidation/collapse. Therefore, remedial grading is recommended to remove the upper portion of the near-surface native alluvium and replace these soils as compacted structural fill. The recommended remedial grading will reduce potential differential settlements by replacing collapsible/compressible soils as compacted structural fill.
- The proposed building area should be overexcavated to a depth of at least 4 feet below existing grade and to a depth of 4 feet below proposed building pad subgrade elevation. Within the foundation influence zones, the overexcavation should extend to a depth of at least 3 feet below proposed foundation bearing grade. The overexcavation should extend horizontally at least 5 feet beyond the building and foundation perimeters.
- After the overexcavation has been completed, the resulting subgrade soils should be evaluated by the geotechnical engineer to identify any additional soils that should be removed. Resulting subgrade should then be scarified to a depth of 12 inches and moisture conditioned to 2 to 4 percent above optimum. The previously excavated soils may then be replaced as compacted structural fill. All structural fill soils should be compacted to at least 90 percent of the ASTM D-1557 maximum dry density.
- The new pavement and flatwork 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 six (6) No. 5 rebars (3 top and 3 bottom) in strip footings, due to the presence of medium expansive soils and minor amounts of liquefaction-induced settlement. Additional reinforcement may be necessary for structural considerations.

Building Floor Slab

- Conventional Slab-on-Grade, 6 inches thick.
- Modulus of Subgrade Reaction: k = 100 psi/in.
- Minimum slab reinforcement: Reinforcement of the floor slab should consist of No. 3 bars at 18-inches on center in both directions due to the presence of medium expansive soils and minor amounts of liquefaction-induced settlement. The actual floor slab reinforcement should be determined by the structural engineer, based upon the imposed loading.

Pavements

ASPHALT PAVEMENTS (R = 30)					
Materials	**Thickness (inches)**				
	Auto Parking and Auto Drive Lanes (TI = 4.0 to 5.0)	Truck Traffic			
		TI = 6.0	TI = 7.0	TI = 8.0	TI = 9.0
Asphalt Concrete	3	4	4	5	6
Aggregate Base	6	7	10	11	12
Compacted Subgrade	12	12	12	12	12

PORTLAND CEMENT CONCRETE PAVEMENTS (R = 30)				
Materials	**Thickness (inches)**			
	Autos and Light Truck Traffic (TI = 6.0)	Truck Traffic		
		TI = 7.0	TI = 8.0	TI = 9.0
PCC	5	5½	6½	8
Compacted Subgrade (95% minimum compaction)	12	12	12	12

2.0 SCOPE OF SERVICES

The scope of services performed for this project was in accordance with our Proposal No. 17P383, dated October 10, 2017. The scope of services included a visual site reconnaissance, subsurface exploration, field and laboratory testing, and geotechnical engineering analysis to provide criteria for preparing the design of the building foundations, building floor slab, and parking lot pavements along with site preparation recommendations and construction considerations for the proposed development. Based on the location of this site, this investigation also included a site-specific liquefaction evaluation. 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 site is located at the southeast corner of Redlands Avenue and Morgan Street in Perris, California. The site is bounded to the north by vacant lot, to the west by Redlands Avenue, to the south by an agricultural field, and to the east by a flood channel. The general location of the site is illustrated on the Site Location Map, enclosed as Plate 1 in Appendix A of this report.

The subject site consists of an irregular-shaped parcel, approximately 37.93± acres in size. The site is currently being utilized or was recently utilized as an agricultural field. The current ground surface cover consists of exposed soil and extensive crop stubble. There is an existing water pump station located at the northwest corner of the site.

Detailed topographic information was obtained from a conceptual site plan prepared by Albert A. Webb Associates, the project civil engineer. This plan indicates that the overall site topography generally slopes downward to the southeast at an estimated gradient of less than 1 percent. The maximum site elevation is 1448± feet mean sea level (msl) located in the northwestern corner of the subject site, and the minimum site elevation is 1443± feet msl in the southeastern corner of the subject site.

3.2 Proposed Development

A site plan for the proposed development was provided to our office by the client. The plan indicates that the site will be developed with one (1) new warehouse building. The building will be located in the center of the site and will be 540,913± ft^2 in size. The building will be constructed in a cross-dock configuration with loading docks along both the east and west sides of the building. It is expected that the building will be surrounded by asphaltic concrete pavements for parking and drive lanes and Portland cement concrete pavements in the loading dock areas. Several landscape planters and concrete flatwork are expected to be included throughout the site.

Detailed structural information has not been provided. It is assumed that the building will be a single-story structure of tilt-up concrete construction, typically supported on conventional shallow foundations with a concrete slab-on-grade floor. Based on the assumed construction, maximum column and wall loads are expected to be on the order of 100 kips and 4 to 7 kips per linear foot, respectively.

Based on the existing topography, it is expected that cuts and fills of less than 5± feet will be necessary to achieve the new site grades. No significant amounts of below grade construction, such as basements or crawl spaces, are expected to be included in the proposed development.

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 50± feet below currently existing site grades. All of the borings were logged during drilling by a member of our staff.

All 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

Alluvium

Native alluvial soils were encountered at the ground surface at all ten (10) of the boring locations. The near-surface alluvium generally consists of loose to medium dense silty fine sands and fine sandy silts, extending to depths of 3 to 12± feet. At greater depths, the alluvium generally consists of stiff to very stiff silty clays and clayey silts. These materials generally possess elevated moisture contents. Interbedded layers of medium dense to dense sandy silts and silty sands as well as stiff to very stiff silty clays and clayey silts extend to at least the maximum depth explored of 50± feet.

Groundwater

Free water was encountered during drilling at a depth of 34± feet. Based on the water level measurements and the moisture contents of the recovered soil samples, the static groundwater table is considered to have existed at a depth of 34± feet below existing site grades at the time of the subsurface investigation.

As part of our research, we reviewed available groundwater data in order to determine the historic high groundwater level for the site. The primary reference used to determine the groundwater depths in this area is the California Department of Water Resources website, http://www.water.ca.gov/waterdatalibrary/. Several monitoring wells are located within a mile radius of the subject site with high groundwater level readings ranging from 26 to 108± feet from the ground surface. Therefore, the high groundwater depth of 26± feet (February 2012) reported in a monitoring well located 0.9± miles southeast of the subject site is considered to be conservative with respect to the recent site conditions.

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 content. 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. These tests are generally used to compare the in-situ densities of undisturbed field samples, and for later compaction testing. Additional testing of other soil types or soil mixes may be necessary at a later date.

Expansion Index

The expansion potential of the on-site soils was determined in general accordance with ASTM D-4829. 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-3 @ 0 to 5 feet	28	Low
B-6 @ 0 to 5 feet	39	Low

Soluble Sulfates

Representative samples of the near-surface soils have been 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 results of the soluble sulfate testing are presented below, and are discussed further in a subsequent section of this report.

Sample Identification	Soluble Sulfates (%)	Sulfate Classification
B-2 @ 0 to 5 feet	0.006	Negligible
B-3 @ 0 to 5 feet	0.012	Negligible
B-6 @ 0 to 5 feet	0.001	Negligible
B-7 @ 0 to 5 feet	0.001	Negligible

Grain Size Analysis

Limited grain size analyses have been performed on several selected samples, in accordance with ASTM D-1140. These samples were washed over a #200 sieve to determine the percentage of fine-grained material in each sample, which is defined as the material which passes the #200 sieve. 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 laboratory tests are shown on the attached boring logs.

Organic Content Testing

Selected soil samples have been tested to determine their organic content, in accordance with ASTM Test Method 2974. The results of the testing are as follows:

Sample Identification	Organic Content
B-2 @ 0 to 5 feet	1.27%
B-7 @ 0 to 5 feet	0.77%

6.0 CONCLUSIONS AND RECOMMENDATIONS

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

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

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

6.1 Seismic Design Considerations

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

Faulting and Seismicity

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

Seismic Design Parameters

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 2016 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 2016 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 2016 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:

2016 CBC SEISMIC DESIGN PARAMETERS

Parameter		Value
Mapped Spectral Acceleration at 0.2 sec Period	S_S	1.500
Mapped Spectral Acceleration at 1.0 sec Period	S_1	0.600
Site Class	---	D
Site Modified Spectral Acceleration at 0.2 sec Period	S_{MS}	1.500
Site Modified Spectral Acceleration at 1.0 sec Period	S_{M1}	0.900
Design Spectral Acceleration at 0.2 sec Period	S_{DS}	1.000
Design Spectral Acceleration at 1.0 sec Period	S_{D1}	0.600

*The 2016 CBC requires that Site Class F be assigned to any profile containing soils vulnerable to potential failure or collapse under seismic loading, such as liquefiable soils. For Site Class F, the site coefficients are to be determined in accordance with Section 11.4.7 of ASCE 7-10. However, Section 20.3.1 of ASCE 7-10 indicates that for sites with structures having a fundamental period of vibration equal to or less than 0.5 seconds, the site coefficient factors (F_a and F_v) may be determined using the standard procedures. The seismic design parameters tabulated above were calculated using the site coefficient factors for Site Class D, assuming that the fundamental period of the structure is less than 0.5 seconds. However, the results of the liquefaction evaluation indicate that the subject site is underlain by potentially liquefiable soils. Therefore, if the proposed structure has a fundamental period greater than 0.5 seconds, a site specific seismic hazards analysis would be required and additional subsurface exploration would be necessary.

Ground Motion Parameters

For the liquefaction evaluation, we utilized a site acceleration consistent with maximum considered earthquake ground motions, as required by the 2016 CBC. The peak ground acceleration (PGA) was determined in accordance with Section 11.8.3 of ASCE 7-10. The parameter PGA_M is the maximum considered earthquake geometric mean (MCE_G) PGA, multiplied by the appropriate site coefficient from Table 11.8-1 of ASCE 7-10. The web-based software application U.S. Seismic Design Maps (described in the previous section) was used to determine PGA_M, which is 0.5g. A portion of the program output is included as Plate E-2 of this report. An associated earthquake magnitude was obtained from the USGS Unified Hazard Tool, Interactive Deaggregation application available on the USGS website. The deaggregated modal magnitude is 7.13, based on the peak ground acceleration and soil classification D.

Liquefaction

The Riverside County GIS website indicates that the subject site is located within a zone of moderate liquefaction susceptibility. Based on this mapping, the scope of this investigation included additional subsurface exploration, laboratory testing, and engineering analysis in order to determine the site-specific liquefaction potential.

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

The liquefaction analysis was conducted in accordance with the requirements of Special Publication 117A (CDMG, 2008), and currently accepted practice (SCEC, 1997). The liquefaction potential of the subject site was evaluated using the empirical method developed by Boulanger and Idriss (Boulanger and Idriss, 2008). This method predicts the earthquake-induced liquefaction potential of the site based on a given design earthquake magnitude and peak ground acceleration at the subject site. This procedure essentially compares the cyclic resistance ratio (CRR) [the cyclic stress ratio required to induce liquefaction for a cohesionless soil stratum at a given depth] with the earthquake-induced cyclic stress ratio (CSR) at that depth from a specified design earthquake (defined by a peak ground surface acceleration and an associated earthquake moment magnitude). CRR is determined as a function of the corrected SPT N-value $(N_1)_{60\text{-}cs}$, adjusted for fines content. The factor of safety against liquefaction is defined as CRR/CSR. Based on Special Publication 117A, a factor of safety of at least 1.3 is required in order to demonstrate that a given soil stratum is non-liquefiable. Additionally, in accordance with Special Publication 117A, clayey soils which do not meet the criteria for liquefiable soils defined by Bray and Sancio (2006), loose soils with a plasticity index (PI) less than 12 and moisture content greater than 85% of the liquid limit, are considered to be insusceptible to liquefaction. Non-sensitive soils with a PI greater than 18 are also considered non-liquefiable.

As part of the liquefaction evaluation, Boring Nos. B-1 and B-8 were extended to depths of 50± feet. The liquefaction analysis procedure is tabulated on the spreadsheet forms included in Appendix F of this report, using the data obtained from these borings. The liquefaction potential of the site was analyzed utilizing a PGA_M of 0.50g for a magnitude 7.13 seismic event.

The historic high groundwater depth was obtained from the California Department of Water Resources website, http://www.water.ca.gov/waterdatalibrary/, which indicates a historic high groundwater depth in the vicinity of the subject site of approximately 26 feet. Free water was encountered during the drilling of Boring No. B-1 and B-8 at a depth of 34 feet. Therefore, the historic high groundwater table was conservatively assumed to exist at a depth of 26± feet.

If liquefiable soils are identified, the potential settlements that could occur as a result of liquefaction are determined using the equation for volumetric strain due to post-cyclic reconsolidation (Yoshimine et. al, 2006). This procedure uses an empirical relationship between the induced cyclic shear strain and the corrected N-value to determine the expected volumetric strain of saturated sands subjected to earthquake shaking. This analysis is also documented on the spreadsheets included in Appendix F.

Conclusions and Recommendations

Potentially liquefiable soils were encountered at both of the 50-foot deep boring locations. The liquefiable strata at Boring No. B-1 are present between depths of 28 and 37± feet. At Boring No. B-8, the liquefiable soils exist between depths of 37 and 47± feet. The remaining soil strata encountered below the historic high groundwater table either possess adequate factors of safety, or are considered non-liquefiable due to their cohesive characteristics and the results of the Atterberg limits testing with respect to the requirements of Special Publication 117A. Settlement analyses were conducted for the potentially liquefiable strata. The results of the settlement analyses indicate the following total deformations:

- Boring No. B-1: 0.81 inches
- Boring No. B-8: 2.62 inches

Based on the results of the settlement analyses, differential settlements are expected to be on the order of 1.5± inches or less. The estimated differential settlement can be assumed to occur across a distance of 100 feet, indicating a maximum angular distortion of approximately 0.001 inches per inch.

Based on our understanding of the proposed development, it is considered feasible to support the proposed structure on shallow foundations. Such a foundation system can be designed to resist the effects of the anticipated differential settlements, to the extent that the structures would not catastrophically fail. Designing the proposed structure to remain completely undamaged during a seismic event that could occur once every 2475 years (the code-specified return period used in the liquefaction analysis) is not considered to be economically feasible. Based on this understanding, the use of shallow foundation systems is considered to be the most economical means of supporting the proposed structure.

In order to support the proposed structure on shallow foundations (such as spread footings) the structural engineer should verify that the structure would not catastrophically fail due to the predicted dynamic differential settlements. Any utility connections to the structure should be designed to withstand the estimated differential settlements. It should also be noted that minor to moderate repairs, including re-leveling, restoration of utility connections, repair of damaged drywall and stucco, etc., would likely be required after occurrence of the liquefaction-induced settlements.

The use of a shallow foundation system, as described in this report, is typical for buildings of this type, where they are underlain the extent of liquefiable soils encountered at this site. The post-liquefaction damage that could occur within the building proposed for this site will also be typical of similar buildings in the vicinity of this project. However, if the owner determines that this level of potential damage is not acceptable, other geotechnical and structural options are available, including the use of ground improvement or mat foundations.

6.2 Geotechnical Design Considerations

General

The subsurface conditions encountered at the boring locations generally consist of variable strength native alluvium. The results of laboratory testing indicate that the near surface alluvium (within the upper 3 to 5± feet) possesses a potential for moderate collapse when exposed to moisture infiltration as well as excessive consolidation when exposed to load increases in the range of those that will be exerted by the new foundations. By visual examination, the majority of the near surface samples also possess calcareous nodules and veining throughout, and appear to be weakly cemented. Cemented soils with low relative densities are generally prone to settlement due to collapse when inundated with water. Based on these conditions, remedial grading will be necessary within the proposed building area to provide a subgrade suitable for support of the new foundations and floor slab. The remedial grading will also serve to create more uniform support characteristics across any cut/fill transitions.

Settlement

The recommended remedial grading will remove the compressible/collapsible near-surface native alluvium, and replace these materials as compacted structural fill. The native soils that will remain in place below the recommended depth of overexcavation will not be subject to significant load increases from the foundations of the new structure. Provided that the recommended remedial grading is completed, the post-construction static settlements of the proposed structure are expected to be within tolerable limits.

Expansion

Laboratory testing performed on representative samples of the near surface soils indicates that these materials possess a low expansion potential (EI = 28 and 39). Based on the presence of expansive soils at this site, care should be given to proper moisture conditioning of all building pad subgrade soils to a moisture content of 2 to 4 percent above the ASTM D-1557 optimum during site grading. In addition to adequately moisture conditioning the subgrade soils and fill soils during grading, special care must be taken to maintaining moisture content of these soils at 2 to 4 percent above the optimum moisture content. This will require the contractor to frequently moisture condition these soils throughout the grading process, unless grading occurs during a period of relatively wet weather.

Shrinkage/Subsidence

Removal and recompaction of the near-surface native fill soils is estimated to result in an average shrinkage of 8 to 13 percent. It should be noted that the potential shrinkage estimate is based on dry density testing performed on small-diameter samples taken at the boring locations. If a more accurate and precise shrinkage estimate is desired, SCG can perform a shrinkage study involving several excavated test-pits where in-place densities are determined using in-situ testing methods instead of laboratory density testing on small-diameter samples. Please contact SCG for details and a cost estimate regarding a shrinkage study, if desired.

Minor ground subsidence is expected to occur in the soils below the zone of removal, due to settlement and machinery working. The subsidence is estimated to be 0.10 feet.

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

It is recommended that we be provided with copies of the grading and foundation 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

Initial site preparation should include stripping of any surficial vegetation and organic soils. Based on conditions encountered at the time of the subsurface exploration, minor stripping of the crop stubble and native grass and weed growth is expected to be necessary. These materials should be disposed of offsite. The actual extent of site stripping should be determined in the field by the geotechnical engineer, based on the organic content and stability of the encountered materials.

Treatment of Existing Soils: Building Pad

Remedial grading should be performed within the proposed building pad area in order to remove the existing potentially compressible/collapsible native alluvium. It is recommended that the overexcavation extend to a depth of at least 4 feet below existing grade and to a depth of at least 4 feet below proposed grade, whichever is greater. Within the influence zones of the new foundations, the overexcavation should extend to a depth of at least 3 feet below proposed foundation bearing grade.

The overexcavation areas should extend at least 5 feet beyond the building perimeter, and to an extent equal to the depth of fill below the new foundations. If the proposed structure 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 overexcavation areas 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, overly moist, or low density native soils are encountered at the base of the overexcavation.

Based on conditions encountered at the exploratory boring locations, moist to very moist soils may be encountered at or near the base of the recommended overexcavation. Scarification and air drying of these materials may be sufficient to obtain a stable subgrade. However, if highly unstable soils are identified, and if the construction schedule does not allow for delays associated with drying, mechanical stabilization, usually consisting of coarse crushed stone and/or geotextile, may be necessary. Concrete and asphalt debris that is crushed to a 3 to 6-inch particle size may also be feasible to use as a subgrade stabilization material. If unstable subgrade conditions are encountered, the geotechnical engineer should be contacted for supplementary recommendations.

After a suitable overexcavation subgrade has been achieved, the exposed soils should be scarified to a depth of at least 12 inches and moisture conditioned or air dried to achieve a moisture content of 2 to 4 percent above optimum moisture content. The subgrade soils should then be recompacted to at least 90 percent of the ASTM D-1557 maximum dry density. The building pad areas may then be raised to grade with previously excavated soils or imported, structural fill. All structural fill soils present within the proposed building area should be compacted to at least 90 percent of the ASTM D-1557 maximum dry density.

Treatment of Existing Soils: Retaining Walls and Site Walls

The existing soils within the areas of proposed retaining and non-retaining site walls should be overexcavated to a depth of at least 3 feet below foundation bearing grade and replaced as compacted structural fill. Any undocumented fill soils should also be removed from the retaining wall areas. 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 surficial alluvial 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 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 completely mitigate the extent of existing collapsible and compressible alluvium in the parking areas. As such, settlement and associated pavement distress could occur. Typically, repair of such distressed areas involves significantly lower costs than completely mitigating these soils at the time of construction. If the owner cannot tolerate the risk of such settlements, the parking and drive areas should be overexcavated to a depth of 2 feet below proposed pavement subgrade elevation, with the resulting soils replaced as compacted structural fill.

Treatment of Existing Soils: Flatwork Areas

Subgrade preparation in the new flatwork 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.

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 city of Perris.
- 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 low expansive (EI < 50), 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 city of Perris. 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 low to moderate strength silty sands, sandy silts, clayey silts and sandy clays. These materials may be subject to minor 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.

Expansive Soils

Based on the results of laboratory testing, the near surface soils have been determined to be low expansive. Based on the presence of expansive soils at this site, care should be given to proper moisture conditioning of all building pad subgrade soils to a moisture content of 2 to 4 percent above the Modified Proctor optimum during site grading. All imported fill soils should have low expansive characteristics. In addition to adequately moisture conditioning the subgrade soils and fill soils during grading, special care must be taken to maintain moisture content of these soils at 2 to 4 percent above the Modified Proctor optimum. This will require the contractor to frequently moisture condition these soils throughout the grading process, unless grading occurs during a period of relatively wet weather.

Due to the presence of expansive soils at this site, provisions should be made to limit the potential for surface water to penetrate the soils immediately adjacent to the structure. These provisions should include directing surface runoff into rain gutters and area drains, reducing the extent of landscaped areas around the structure, and sloping the ground surface away from the building. Where possible, it is recommended that landscaped planters not be located immediately adjacent to the building. If landscaped planters around the buildings are necessary, it is recommended that drought tolerant plants or a drip irrigation system be utilized, to minimize the potential for deep moisture penetration around the structures. Presented below is a list of additional soil moisture control recommendations that should be considered by the owner, developer, and civil engineer:

- Ponding and areas of low flow gradients in unpaved walkways, grass and planter areas should be avoided. In general, minimum drainage gradients of 2 percent should be maintained in unpaved areas.
- Bare soil within five feet of proposed structures should be sloped at a minimum five percent gradient away from the structure (about three inches of fall in five feet), or the same area could be paved with a minimum surface gradient of one percent. Pavement is preferable.
- Decorative gravel ground cover tends to provide a reservoir for surface water and may hide areas of ponding or poor drainage. Decorative gravel is, therefore, not recommended and should not be utilized for landscaping unless equipped with a subsurface drainage system designed by a licensed landscape architect.
- Positive drainage devices, such as graded swales, paved ditches, and catch basins should be installed at appropriate locations within the area of proposed development.
- Concrete walks and flatwork should not obstruct the free flow of surface water to the appropriate drainage devices.
- Area drains should be recessed below grade to allow free flow of water into the drain. Concrete or brick flatwork joints should be sealed with mortar or flexible mastic.
- Gutter and downspout systems should be installed to capture all discharge from roof areas. Downspouts should discharge directly into a pipe or paved surface system to be conveyed offsite.
- Enclosed planters adjoining, or in close proximity to proposed structures, should be sealed at the bottom and provided with subsurface collection systems and outlet pipes.
- Depressed planters should be raised with soil to promote runoff (minimum drainage gradient two percent or five percent, see above), and/or equipped with area drains to eliminate ponding.
- Drainage outfall locations should be selected to avoid erosion of slopes and/or properly armored to prevent erosion of graded surfaces. No drainage should be directed over or towards adjoining slopes.
- All drainage devices should be maintained on a regular basis, including frequent observations during the rainy season to keep the drains free of leaves, soil and other debris.
- Landscape irrigation should conform to the recommendations of the landscape architect and should be performed judiciously to preclude either soaking or excessive drying of the foundation soils. This should entail regular watering during the drier portions of the year and little or no irrigation during the rainy season. Automatic sprinkler systems should, therefore, be switched to manual operation during the rainy season. Good irrigation practice typically requires frequent application of limited quantities of water that are sufficient to sustain plant growth, but do not excessively wet the soils. Ponding and/or run-off of irrigation water are indications of excessive watering.

Other provisions, as determined by the landscape architect or civil engineer, may also be appropriate.

<u>Groundwater</u>

The static groundwater table is considered to exist at a depth of 34± feet below existing grade. 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 extending to depths of at least 3 feet below foundation bearing grade. Based on this subsurface profile, and based on the design considerations presented in Section 6.1 of this report, the proposed structure may be supported on conventional shallow foundations.

Foundation Design Parameters

New square and rectangular footings may be designed as follows:

- Maximum, net allowable soil bearing pressure: 2,500 lbs/ft^2.

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

- Minimum longitudinal steel reinforcement within strip footings: Six (6) No. 5 rebars (3 top and 3 bottom), due to the presence of expansive soils and moderate amounts of liquefaction-induced settlement.

- 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 1/3 when considering short duration wind or seismic loads. The minimum steel reinforcement recommended above is based on standard geotechnical practice. Additional rigidity may be necessary for structural considerations, or to resist the effects of the liquefaction-induced differential settlements, as discussed in Section 6.1. 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 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 static 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, under static conditions. 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. These settlements are in addition to the liquefaction-induced settlements previously discussed in Section 6.1 of this report.

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: 275 lbs/ft^3
- Friction Coefficient: 0.28

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 soils. 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, and based on the design considerations presented in Section 6.1 of this report, the floor of the proposed structure may be constructed as a conventional slab-on-grade supported on newly placed structural fill, extending to a depth of at least 4 feet below finished pad grade. Based on geotechnical considerations, the floor slab may be designed as follows:

- Minimum slab thickness: 6 inches.

- Modulus of Subgrade Reaction: 100 lbs/in^3.

- Minimum slab reinforcement: No. 3 bars at 18-inches on-center, in both directions, due to presence of expansive soils and potentially liquefiable soils, at this site. The actual floor slab reinforcement should be determined by the structural engineer, based upon the imposed loading, and the potential liquefaction-induced settlements.

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

- 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 Exterior Flatwork Design and Construction

Subgrades which will support new exterior slabs-on-grade for sidewalks, patios, and other concrete flatwork, should be prepared in accordance with the recommendations contained in the ***Grading Recommendations*** section of this report. Based on geotechnical considerations, exterior slabs on grade may be designed as follows:

- Minimum slab thickness: 4½ inches.

- Minimum slab reinforcement: No. 3 bars at 18 inches on center, in both directions.

- The flatwork at building entry areas should be structurally connected to the perimeter foundation that is recommended to span across the door opening. This recommendation is designed to reduce the potential for differential movement at this joint.

- Moisture condition the slab subgrade soils to at least 2 to 4 percent of optimum moisture content, to a depth of at least 12 inches. Adequate moisture conditioning should be verified by the geotechnical engineer 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.

- Control joints should be provided at a maximum spacing of 8 feet on center in two directions for slabs and at 6 feet on center for sidewalks. Control joints are intended to direct cracking. Minor cracking of exterior concrete slabs on grade should be expected.

Expansion or felt joints should be used at the interface of exterior slabs on grade and any fixed structures to permit relative movement.

6.8 Retaining Wall Design and Construction

Although not indicated on the site plan, some small (less than 6 feet in height) retaining walls may be required to facilitate the new site grades and in the 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 clayey silts and sandy clays. Based on their composition, the on-site soils have been assigned a friction angle of 28 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 Clayey Sands, Silty Sands and Sandy Silts
Internal Friction Angle (ϕ)		28°
Unit Weight		121 lbs/ft^3
Equivalent Fluid Pressure:	Active Condition (level backfill)	44 lbs/ft^3
	Active Condition (2h:1v backfill)	77 lbs/ft^3
	At-Rest Condition (level backfill)	65 lbs/ft^3

The walls should be designed using a soil-footing coefficient of friction of 0.28 and an equivalent passive pressure of 275 lbs/ft^3. The structural engineer should incorporate appropriate factors of safety in the design of the retaining walls.
The active earth pressure may be used for the design of retaining walls that do not directly support structures or support soils that in turn support structures and which will be allowed to deflect. The at-rest earth pressure should be used for walls that will not be allowed to deflect such as those which will support foundation bearing soils, or which will support foundation loads directly.

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

Seismic Lateral Earth Pressures

In accordance with the 2016 CBC, any retaining walls more than 6 feet in height must be designed for seismic lateral earth pressures. If walls 6 feet or more are required for this site, the geotechnical engineer should be contacted for supplementary seismic lateral earth pressure recommendations.

Retaining Wall Foundation Design

The retaining wall foundations should be supported within newly placed compacted structural fill, extending to a depth of at least 3 feet below proposed foundation 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

On-site soils may be used to backfill the retaining walls, provided that they are low expansive (EI < 50). All backfill material placed within 3 feet of the back wall face should have a particle size no greater than 3 inches. The retaining wall backfill materials should be well graded.

It is recommended that a minimum 1 foot thick layer of free-draining granular material (less than 5 percent passing the No. 200 sieve) be placed against the face of the retaining walls. This material should 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. This material should be approved by the geotechnical engineer. In lieu of the 1 foot thick layer of free-draining material, a properly installed prefabricated drainage composite such as the MiraDRAIN 6000XL (or approved equivalent), which is specifically designed for use behind retaining walls, may be used. If the layer of free-draining material is not covered by an impermeable surface, such as a structure or pavement, a 12-inch thick layer of a low permeability soil should be placed over the backfill to reduce surface water migration to the underlying soils. The layer of free draining granular material should be separated from the backfill soils by a suitable geotextile, approved by the geotechnical engineer.

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

Subsurface Drainage

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

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

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

6.9 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 sandy silts and silty sands. These soils are generally considered to possess fair pavement support characteristics with an estimated R-values of 30 to 40. R-value testing was outside the scope of services. The subsequent pavement design is therefore based upon an assumed R-value of 30. 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 = 30)					
Materials	**Thickness (inches)**				
	Auto Parking and Auto Drive Lanes (TI = 4.0 to 5.0)	Truck Traffic			
		TI = 6.0	TI = 7.0	TI = 8.0	TI = 9.0
Asphalt Concrete	3	4	4	5	6
Aggregate Base	6	7	10	11	12
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 (R = 30)				
Materials	**Thickness (inches)**			
	Autos and Light Truck Traffic (TI = 6.0)	Truck Traffic		
		TI = 7.0	TI = 8.0	TI = 9.0
PCC	5	5½	6½	8
Compacted Subgrade (95% minimum compaction)	12	12	12	12

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

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.

8.0 REFERENCES

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Seed, H. B., and Idriss, I. M., "Simplified Procedure for Evaluating Soil Liquefaction Potential using field Performance Data," Journal of the Soil Mechanics and Foundations Division, American Society of Civil Engineers, September 1971, pp. 1249-1273.

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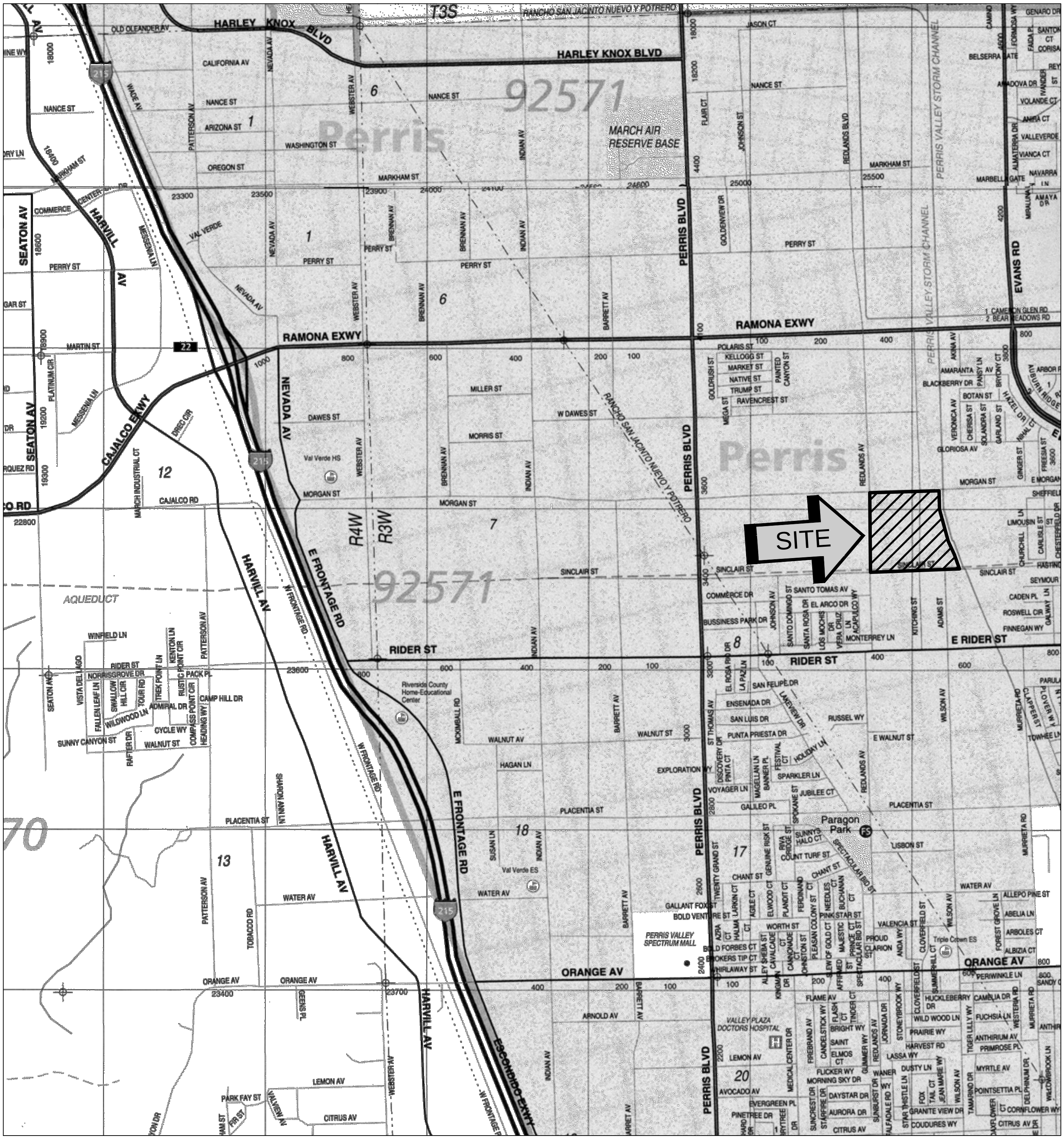
Tokimatsu K., and Seed, H. B., "Evaluation of Settlements in Sands Due to Earthquake Shaking," Journal of the Geotechnical Engineering Division, American society of Civil Engineers, Volume 113, No. 8, August 1987, pp. 861-878.

Tokimatsu, K. and Yoshimi, Y., "*Empirical Correlations of Soil Liquefaction Based on SPT N-value and Fines Content,"* Seismological Research Letters, Eastern Section Seismological Society Of America, Volume 63, Number 1, p. 73.

Youd, T. L. and Idriss, I. M. (Editors), "Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils," Salt Lake City, UT, January 5-6 1996, NCEER Technical Report NCEER-97-0022, Buffalo, NY.

APPENDIX A



SOURCE: RIVERSIDE COUNTY
THOMAS GUIDE, 2013

SITE LOCATION MAP

RIDER 4 - PROPOSED COMMERCIAL/INDUSTRIAL BUILDING

PERRIS, CALIFORNIA

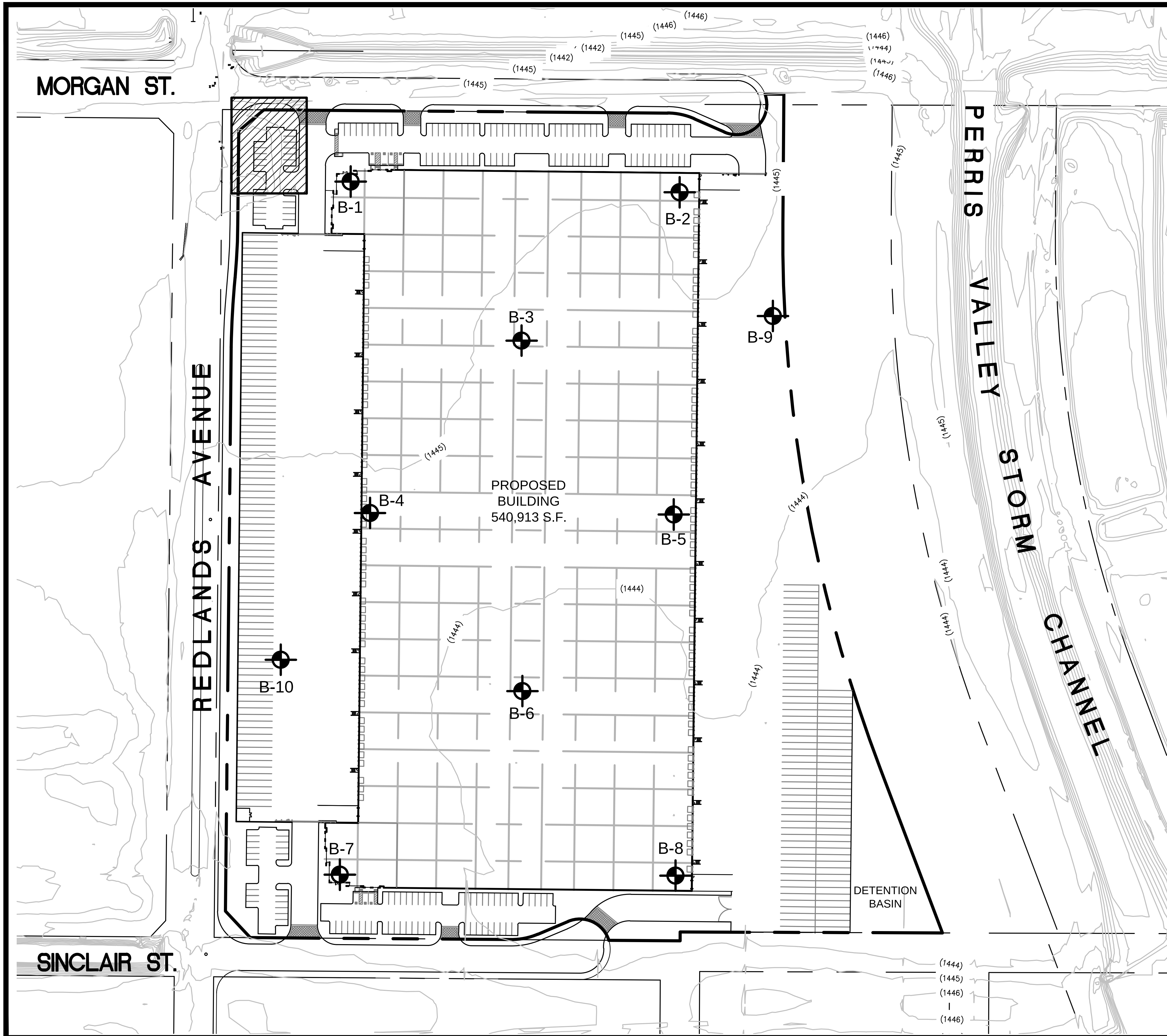
SCALE: 1" = 2400'

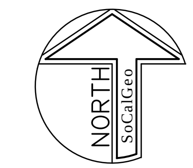
DRAWN: SAM
CHKD: GKM

SCG PROJECT
17G206-1

PLATE 1

SOUTHERN
CALIFORNIA
GEOTECHNICAL





GEOTECHNICAL LEGEND

APPROXIMATE BORING LOCATION

EXISTING PUMP STATION

NOTE: SITE PLAN PREPARED BY ALBERT A. WEBB ASSOCIATES.

BORING LOCATION PLAN

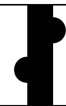
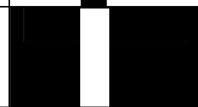
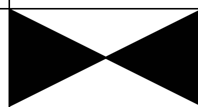
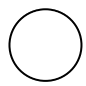
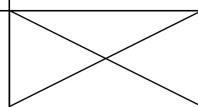
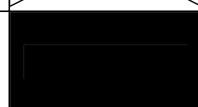
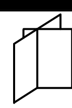
RIDER 4 - PROPOSED COMMERCIAL/INDUSTRIAL BUILDING

PERRIS, CALIFORNIA

SCALE: 1" = 150'	SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
DRAWN: JLL CHKD: GKM	
SCG PROJECT 17G206-1	
PLATE 2	

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		TYPICAL DESCRIPTIONS
			GRAPH	LETTER	
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

BORING NO. B-1

JOB NO.: 17G206	DRILLING DATE: 11/9/17	WATER DEPTH: 34 feet
PROJECT: Rider 4	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: ---
LOCATION: Perris, California	LOGGED BY: Jason Hiskey	READING TAKEN: 30 mins

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: 1445 feet MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
	X	9			ALLUVIUM: Gray Brown fine Sandy Silt, trace Clay, fine root fibers, slightly porous, loose-damp		8					
	X	18			Light Gray Brown Silty fine Sand to fine Sandy Silt, trace Clay, abundant calcareous nodules/veining, medium dense-moist to very moist		21					
5	X	13					17					
	X	21					22					
10												
	X	16	3.5		Brown Silty Clay, abundant calcareous nodules/veining, very stiff-moist		16					
15												
	X	19			Brown Silty fine Sand, trace Clay, medium dense-moist		13					
20												
	X	53	2.0		Brown Clayey fine Sand to fine Sandy Clay, very dense to hard-damp to moist		12					
25												
	X	17			Brown Silty fine to medium Sand, medium dense-moist		11			24		
30												
	X	16			Brown fine Sandy Silt, some Clay, medium dense-moist to wet		17			59		

TBL 17G206.GPJ SOCALGEO.GDT 11/30/17

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-1

JOB NO.: 17G206
PROJECT: Rider 4
LOCATION: Perris, California

DRILLING DATE: 11/9/17
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jason Hiskey

WATER DEPTH: 34 feet
CAVE DEPTH: ---
READING TAKEN: 30 mins

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION (Continued)	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					Brown fine Sandy Silt, some Clay, medium dense-moist to wet							
					Brown Silty fine to medium Sand, medium dense-wet							
	X	29					16					
40												
					Brown Silty fine Sand, trace Clay, trace medium Sand, medium dense-very moist							
	X	25					17			32		
45												
	X	20					15			32		
50												
					Boring Terminated at 50'							

TBL 17G206.GPJ SOCALGEO.GDT 11/30/17

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-10

JOB NO.: 17G206	DRILLING DATE: 11/9/17	WATER DEPTH: Dry
PROJECT: Rider 4	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: ---
LOCATION: Perris, California	LOGGED BY: Jason Hiskey	READING TAKEN: At completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: 1444 feet MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					ALLUVIUM: Brown fine Sandy Silt, trace Clay, fine root fibers, trace to little calcareous veining/nodules, dense-damp to moist							
	X	31					12					
	X	34					13					
5												
					Boring Terminated at 5'							

TBL 17G206.GPJ SOCALGEO.GDT 11/30/17

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-2

JOB NO.: 17G206
PROJECT: Rider 4
LOCATION: Perris, California

DRILLING DATE: 11/9/17
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jason Hiskey

WATER DEPTH: Dry
CAVE DEPTH: 14 feet
READING TAKEN: At completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: 1445 feet MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					ALLUVIUM: Light Gray Brown fine Sandy Silt, trace fine root fibers, slightly porous, loose-damp							
		13				92	9					
					Light Gray Brown Silty fine Sand, abundant calcareous nodules, medium dense-damp							
		29				94	10					
					Gray Brown Silty Clay to Clayey Silt, abundant calcareous nodules/veining, very stiff-very moist							
5		16	3.0			78	23					
		41	4.0			66	44					
					Brown Silty Clay, abundant calcareous nodules/veining, hard-damp to moist							
		35	4.5+			109	15					
10												
					Brown fine Sandy Silt, some calcareous nodules, very stiff-damp to moist							
		22					12					
15												
					Brown Silty fine to medium Sand, trace calcareous nodules, medium dense-damp							
		25					8					
20												
					Brown Silty fine Sand, trace Clay, dense-damp to moist							
		38					12					
25												
					Boring Terminated at 25'							

TBL 17G206.GPJ SOCALGEO.GDT 11/30/17

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-3

JOB NO.: 17G206
PROJECT: Rider 4
LOCATION: Perris, California

DRILLING DATE: 11/9/17
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jason Hiskey

WATER DEPTH: Dry
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: 1444 feet MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					ALLUVIUM: Gray Brown fine Sandy Silt, little Clay, trace calcareous veining, slightly porous, loose-moist							
	X	14				89	13					EI = 28 @ 0 to 5 feet
	X	25	4.0			88	17					
					Light Gray Brown Clayey Silt, trace fine Sand, abundant calcareous nodules/veining, porous, very stiff to hard-moist to very moist							
5	X	15	3.5			82	23					
					Light Brown Silty fine Sand, trace medium Sand, trace calcareous nodules/veining, medium dense-damp							
	X	25				108	7					
					Brown Clayey Silt, trace calcareous nodules/veining, very stiff-moist to very moist							
	X	24	3.0			107	17					
10												
					Brown fine Sandy Clay, trace calcareous nodules/veining, very stiff-damp to moist							
	X	20	4.5+				16					
15					Boring Terminated at 15'							

TBL 17G206.GPJ SOCALGEO.GDT 11/30/17

SOUTHERN CALIFORNIA GEOTECHNICAL

A California Corporation

BORING NO. B-4

JOB NO.: 17G206
PROJECT: Rider 4
LOCATION: Perris, California

DRILLING DATE: 11/9/17
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jason Hiskey

WATER DEPTH: Dry
CAVE DEPTH: 9 feet
READING TAKEN: At completion

FIELD RESULTS / LABORATORY RESULTS

SURFACE ELEVATION: 1444 feet MSL

DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
	X	19			ALLUVIUM: Gray Brown fine Sandy Silt, trace Clay, fine root fibers, medium dense-moist		12					
5	X	27			Brown Silty fine Sand, trace calcareous nodules/veining, medium dense-damp		8					
	X	11			Brown fine Sandy Silt, little Clay, some calcareous nodules/veining, medium dense-moist to very moist		15					
10	X	20	3.5		Brown Clayey Silt, trace fine Sand, stiff to very stiff-moist to very moist		19					
15	X	14	1.5				19					
20	X	35			Brown Clayey fine Sand, trace Silt, trace calcareous nodules, dense-damp to moist		12					

Boring Terminated at 20'

TBL 17G206.GPJ SOCALGEO.GDT 11/30/17

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A California Corporation

JOB NO.: 17G206
PROJECT: Rider 4
LOCATION: Perris, California

DRILLING DATE: 11/9/17
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jason Hiskey

WATER DEPTH: Dry
CAVE DEPTH: 11 feet
READING TAKEN: At completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: 1444 feet MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
		12			ALLUVIUM: Light Brown fine Sandy Silt, trace Clay, trace calcareous nodules/veining, loose-damp	95	7					
		12	3.0		Gray Brown Clayey Silt, trace to little fine Sand, abundant calcareous nodules/veining, stiff to very stiff-very moist	80	19					
5		23	2.5			79	20					
		17	3.5			90	23					
10		21	4.5+		Brown Silty Clay, trace fine Sand, abundant calcareous nodules/veining, hard-very moist	93	25					
15		17	2.5		Brown fine Sandy Clay, trace to little Silt, very stiff-moist to very moist		21					
20		30			Brown Clayey fine Sand, trace Silt, medium dense to dense-moist		11					

Boring Terminated at 20'

TBL 17G206.GPJ SOCALGEO.GDT 11/30/17

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-6

JOB NO.: 17G206
PROJECT: Rider 4
LOCATION: Perris, California

DRILLING DATE: 11/9/17
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jason Hiskey

WATER DEPTH: Dry
CAVE DEPTH: 11 feet
READING TAKEN: At completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: 1443 feet MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					ALLUVIUM: Gray fine Sandy Silt, trace to little Clay, abundant calcareous nodules/veining, medium dense-damp							
	X	22				96	8					EI = 39 @ 0 to 5'
					Gray Brown Clayey Silt, trace fine Sand, some calcareous nodules/veining, stiff to very stiff-moist to very moist							
	X	20	4.0			86	27					
5	X	22	1.5			119	13					
	X	14	4.5+			105	16					
					Brown fine Sandy Clay, stiff to very stiff-moist to very moist							
	X	15	4.5+			115	15					
10												
	X	19	2.0				19					
15					Boring Terminated at 15'							

TBL 17G206.GPJ SOCALGEO.GDT 11/30/17

BORING NO. B-7

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

JOB NO.: 17G206
PROJECT: Rider 4
LOCATION: Perris, California

DRILLING DATE: 11/9/17
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jason Hiskey

WATER DEPTH: Dry
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: 1444 feet MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					ALLUVIUM: Brown fine Sandy Silt, trace Clay, fine root fibers, slighlty porous, loose to medium dense-damp to moist							
		13				101	10					
		37				109	12					
5		23				111	12					
		36			Brown Silty fine Sand, trace medium Sand, trace calcareous nodules/veining, medium dense-damp	111	8					
10		23			Gray Brown fine Sandy Silt, trace Clay, medium dense-damp to moist	116	12					
					Brown Silty Clay, some calcareous nodules, stiff-very moist							
15		13	3.0				21					
					Brown Clayey fine Sand, trace Silt, calcareous nodules, medium dense-moist							
20		20					14					
25		34					13					
					Boring Terminated at 25'							

TBL 17G206.GPJ SOCALGEO.GDT 11/30/17

TEST BORING LOG

PLATE B-7

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-8

JOB NO.: 17G206
PROJECT: Rider 4
LOCATION: Perris, California

DRILLING DATE: 11/9/17
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jason Hiskey

WATER DEPTH: 34 feet
CAVE DEPTH: ---
READING TAKEN: 30 mins

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: 1443 feet MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
	X	12			ALLUVIUM: Light Gray fine Sandy Silt, slightly porous, trace fine root fibers, medium dense-damp		8					
	X	10			Light Gray Brown Clayey Silt, abundant calcareous nodules/veining, stiff-very moist		28					
5	X	10	1.0				32					
	X	12			Brown Silty fine Sand, trace calcareous nodules, medium dense-damp		10					
10												
	X	11					10					
15												
					Brown fine Sandy Silt, trace Clay, medium dense to dense-moist							
	X	32					13					
20												
	X	27					13					
25												
	X	28					15					
30												
					Brown Clayey fine Sand, medium dense-damp							
	X	21			@ 34 feet, wet		14			44		

TEST BORING LOG

BORING NO. B-8

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

JOB NO.: 17G206	DRILLING DATE: 11/9/17	WATER DEPTH: 34 feet
PROJECT: Rider 4	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: ---
LOCATION: Perris, California	LOGGED BY: Jason Hiskey	READING TAKEN: 30 mins

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION (Continued)	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					Brown Clayey fine Sand, medium dense-damp							
					Brown Silty fine to medium Sand, medum dense-wet							
	X	14					13			21		
40			3.0		Light Brown fine Sandy Clay, some Silt, stiff-very moist		18			55		
					Brown Clayey fine sand, trace medium Sand, medium dense-very moist							
	X	14					16			42		
45												
					Brown fine to medium Sandy Clay, hard-very moist							
	X	31	3.5				15					
50												
					Boring Terminated at 50'							

TBL 17G206.GPJ SOCALGEO.GDT 11/30/17

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-9

JOB NO.: 17G206
PROJECT: Rider 4
LOCATION: Perris, California

DRILLING DATE: 11/9/17
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jason Hiskey

WATER DEPTH: Dry
CAVE DEPTH: ---
READING TAKEN: At completion

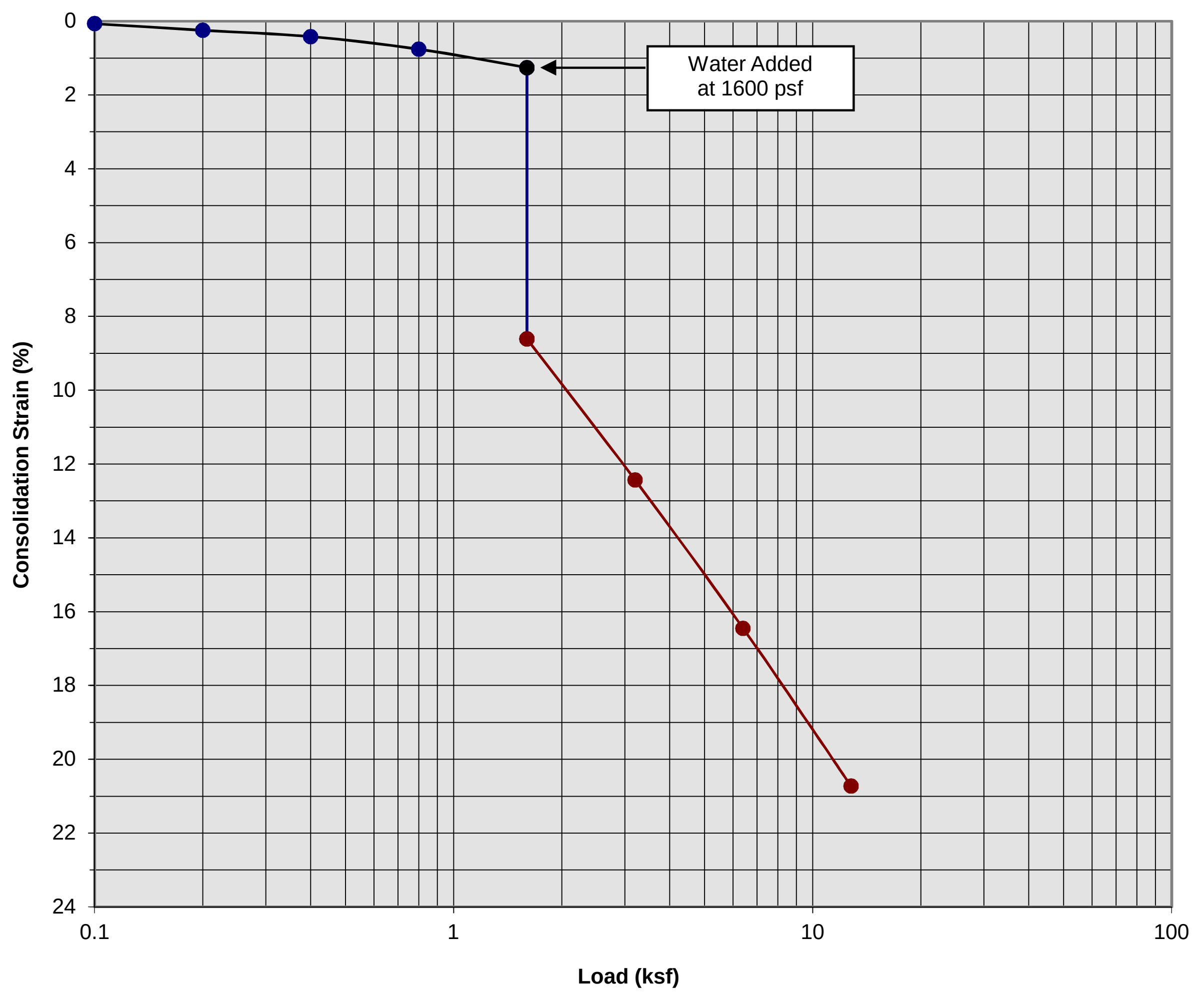
FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: 1444 feet MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
	X	6			ALLUVIUM: Light Gray Brown Silty fine Sand, trace Clay, slightly porous, trace fine root fibers, loose-damp		7					
5	X	6			Gray Brown fine Sandy Silt, loose-moist		14					
					Boring Terminated at 5'							

TBL 17G206.GPJ SOCALGEO.GDT 11/30/17

TEST BORING LOG **PLATE B-9**

APPENDIX C

Consolidation/Collapse Test Results



Classification: Light Gray Brown fine Sandy Silt,

Boring Number:	B-2	Initial Moisture Content (%)	9
Sample Number:	---	Final Moisture Content (%)	19
Depth (ft)	1 to 2	Initial Dry Density (pcf)	92.0
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	115.3
Specimen Thickness (in)	1.0	Percent Collapse (%)	7.35

Rider 4
Perris, California
Project No. 17G206
PLATE C- 1

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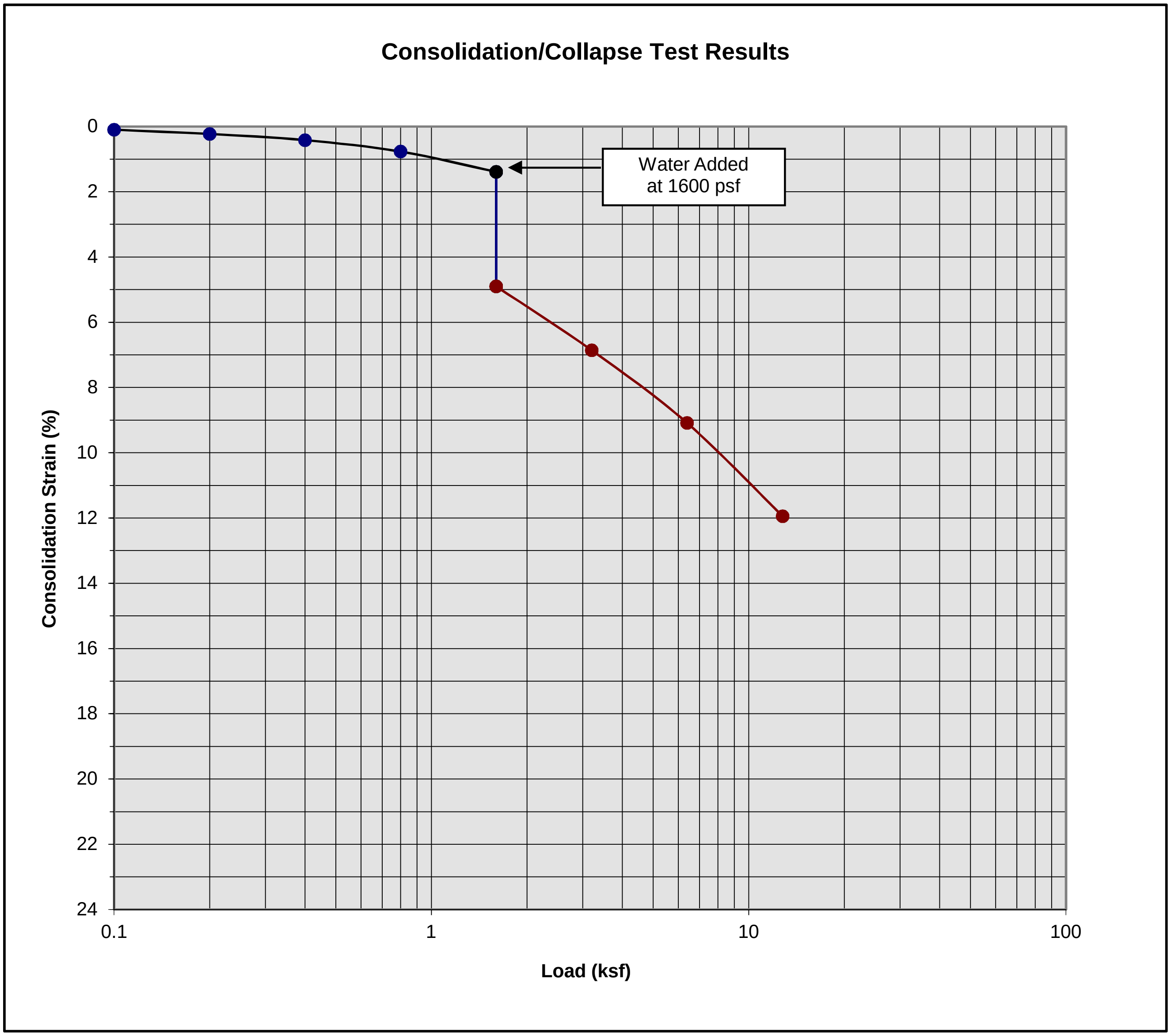

Classification: Light Gray Brown Silty fine Sand

Boring Number:	B-2	Initial Moisture Content (%)	10
Sample Number:	---	Final Moisture Content (%)	22
Depth (ft)	3 to 4	Initial Dry Density (pcf)	93.4
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	105.8
Specimen Thickness (in)	1.0	Percent Collapse (%)	3.51

Rider 4
Perris, California
Project No. 17G206
PLATE C- 2

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL
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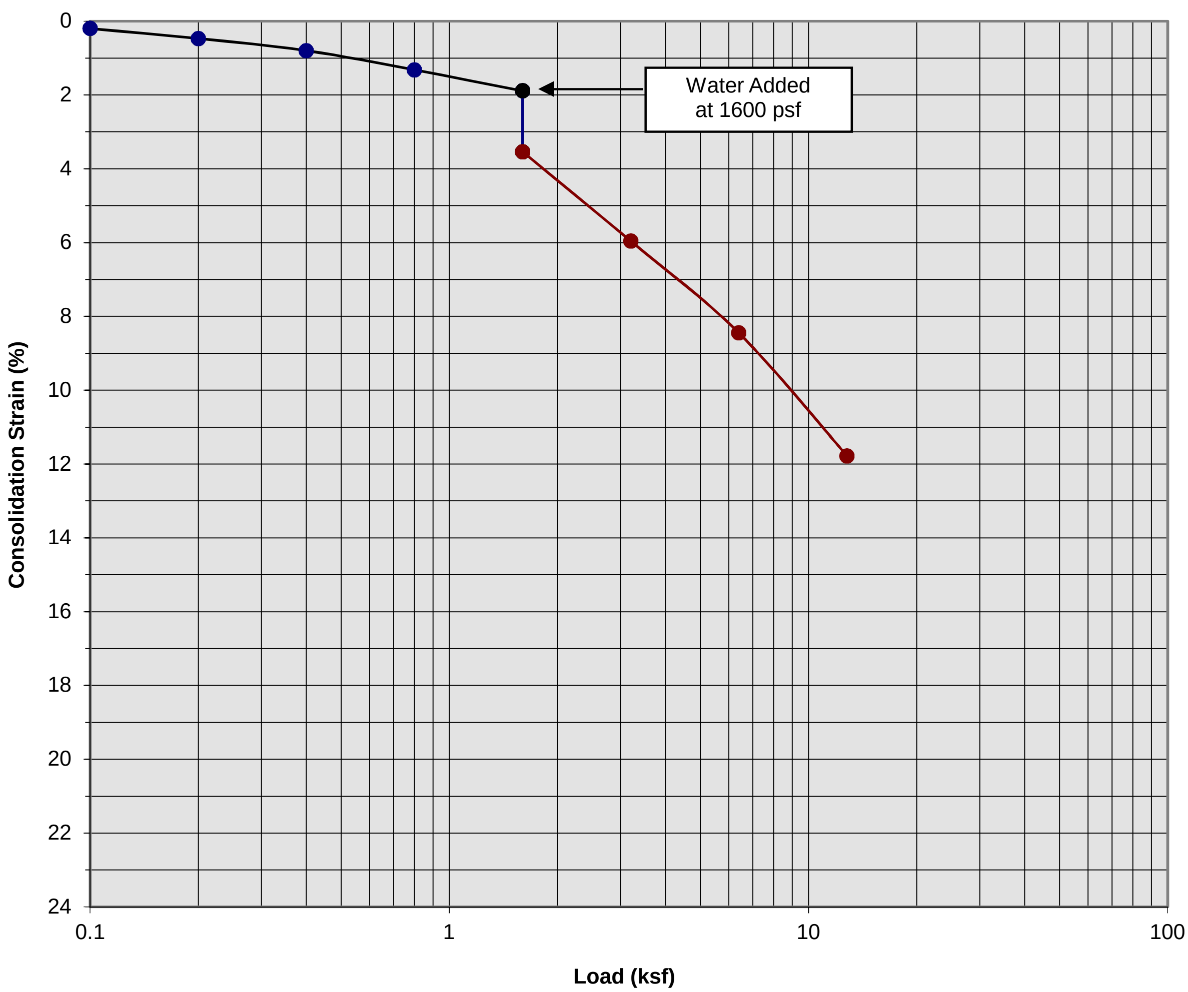
Consolidation/Collapse Test Results



Classification: Gray Brown Silty Clay to Clayey Silt

Boring Number:	B-2	Initial Moisture Content (%)	23
Sample Number:	---	Final Moisture Content (%)	35
Depth (ft)	5 to 6	Initial Dry Density (pcf)	77.1
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	87.5
Specimen Thickness (in)	1.0	Percent Collapse (%)	1.65

Rider 4
Perris, California
Project No. 17G206
PLATE C- 3

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL
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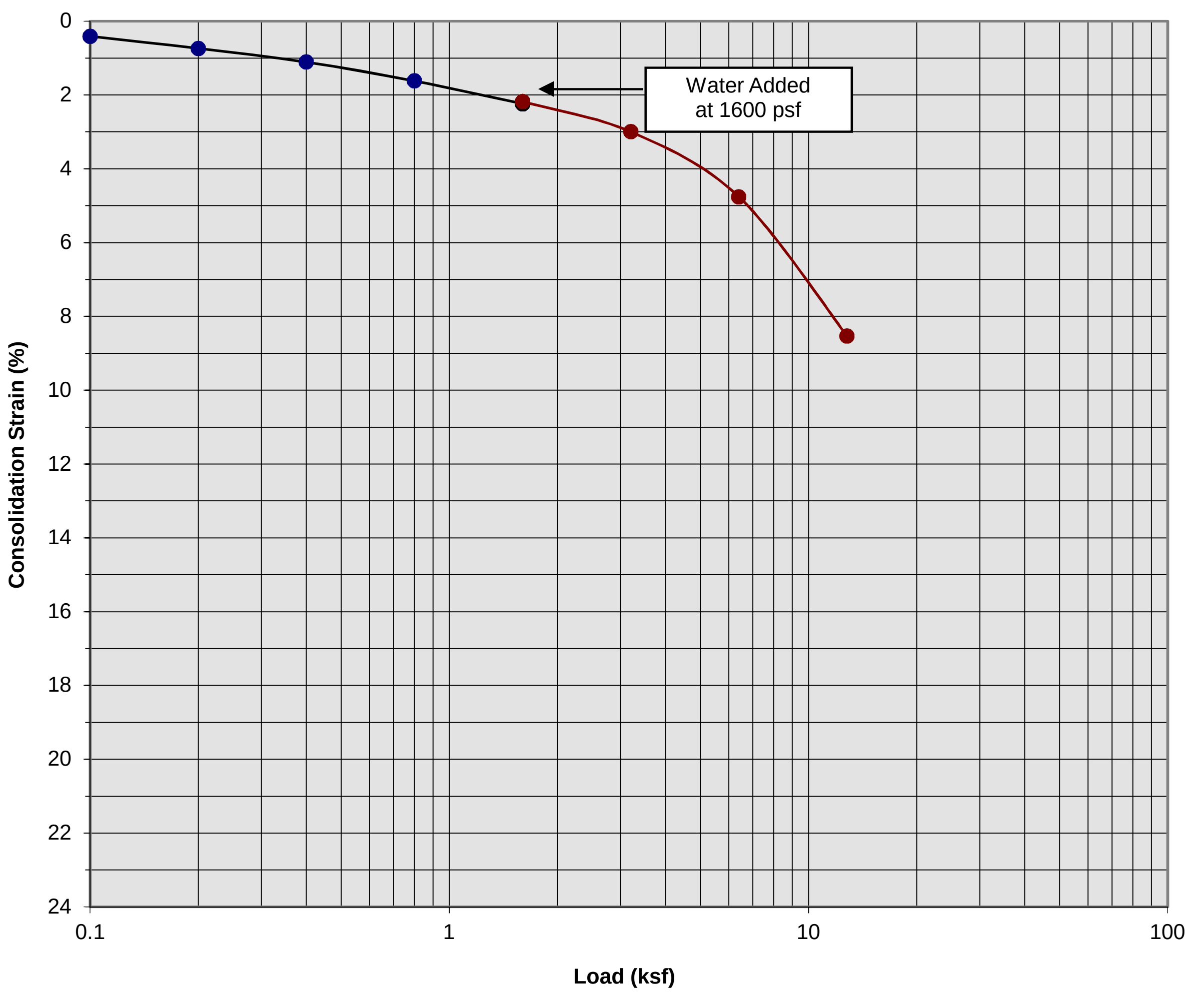
Consolidation/Collapse Test Results



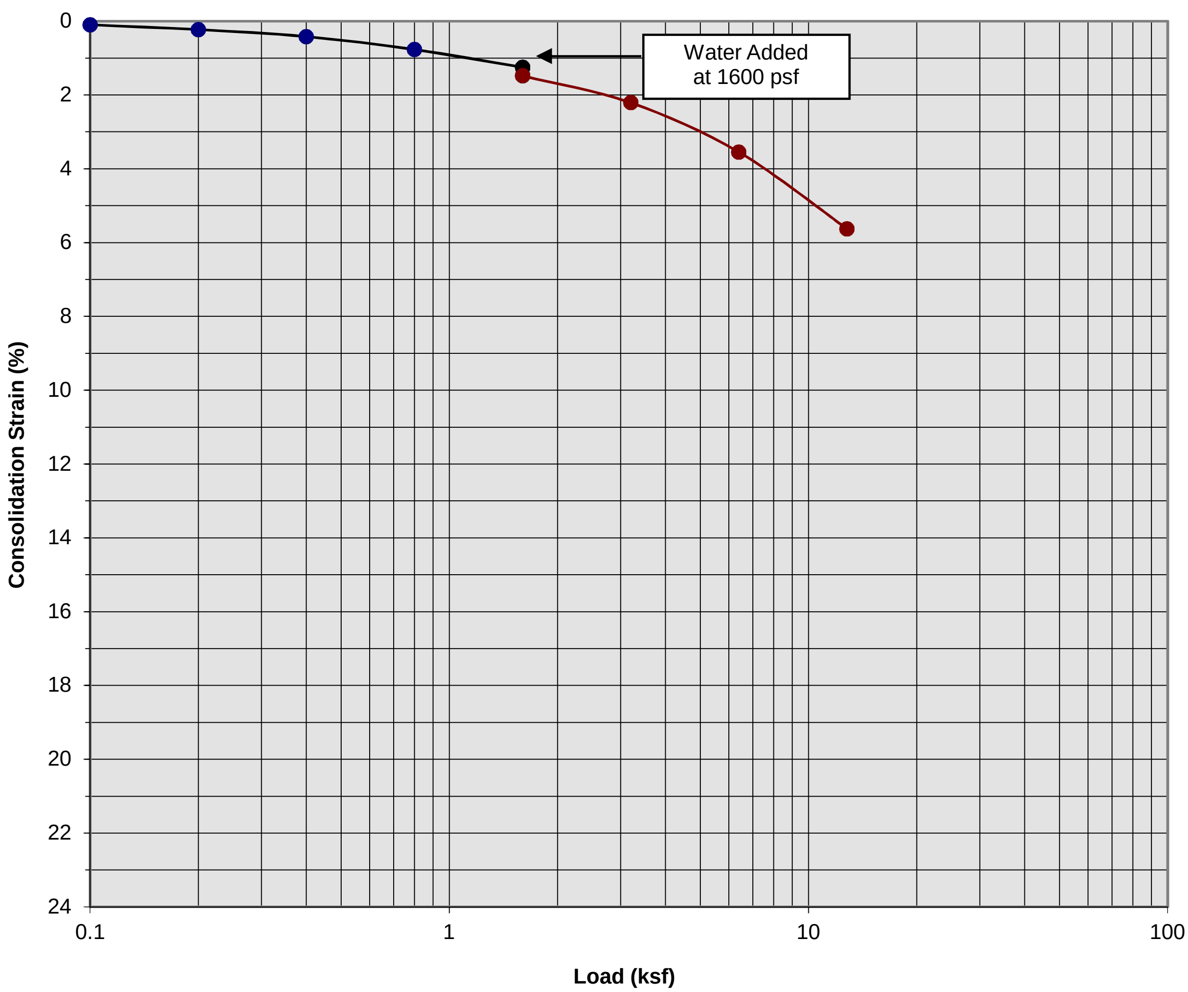
Classification: Gray Brown Silty Clay to Clayey Silt

Boring Number:	B-2	Initial Moisture Content (%)	45
Sample Number:	---	Final Moisture Content (%)	54
Depth (ft)	7 to 8	Initial Dry Density (pcf)	67.4
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	73.3
Specimen Thickness (in)	1.0	Percent Collapse (%)	-0.06

Rider 4
Perris, California
Project No. 17G206
PLATE C- 4

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

Consolidation/Collapse Test Results

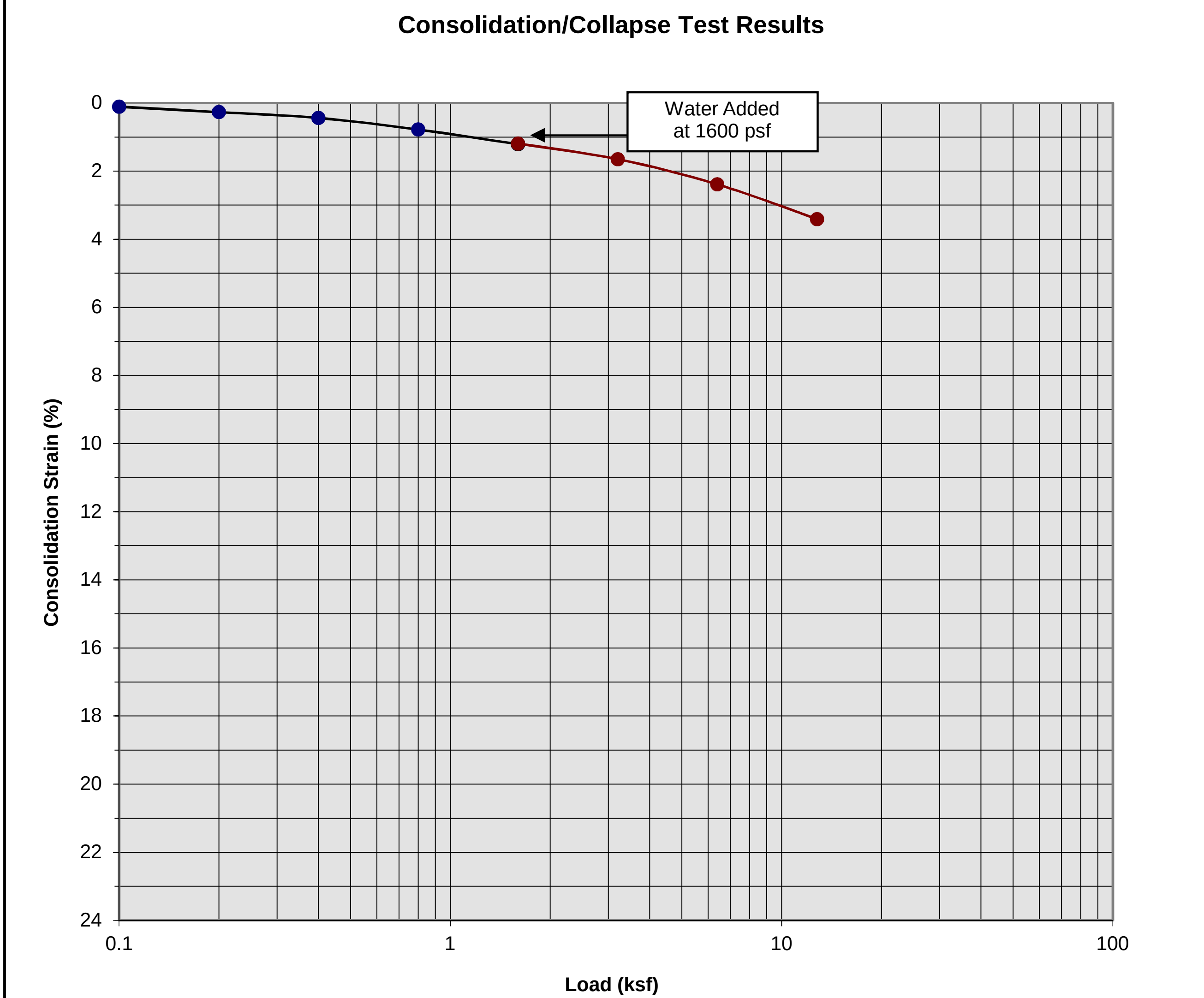


Classification: Gray Brown Clayey Silt, trace fine Sand

Boring Number:	B-6	Initial Moisture Content (%)	27
Sample Number:	---	Final Moisture Content (%)	32
Depth (ft)	3 to 4	Initial Dry Density (pcf)	86.4
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	91.6
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.23

Rider 4
Perris, California
Project No. 17G206
PLATE C- 5

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL A California Corporation



Classification: Gray Brown Clayey Silt, trace fine Sand

Boring Number:	B-6	Initial Moisture Content (%)	13
Sample Number:	---	Final Moisture Content (%)	16
Depth (ft)	5 to 6	Initial Dry Density (pcf)	118.7
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	122.6
Specimen Thickness (in)	1.0	Percent Collapse (%)	-0.02

Rider 4
Perris, California
Project No. 17G206
PLATE C- 6

SOUTHERN CALIFORNIA GEOTECHNICAL
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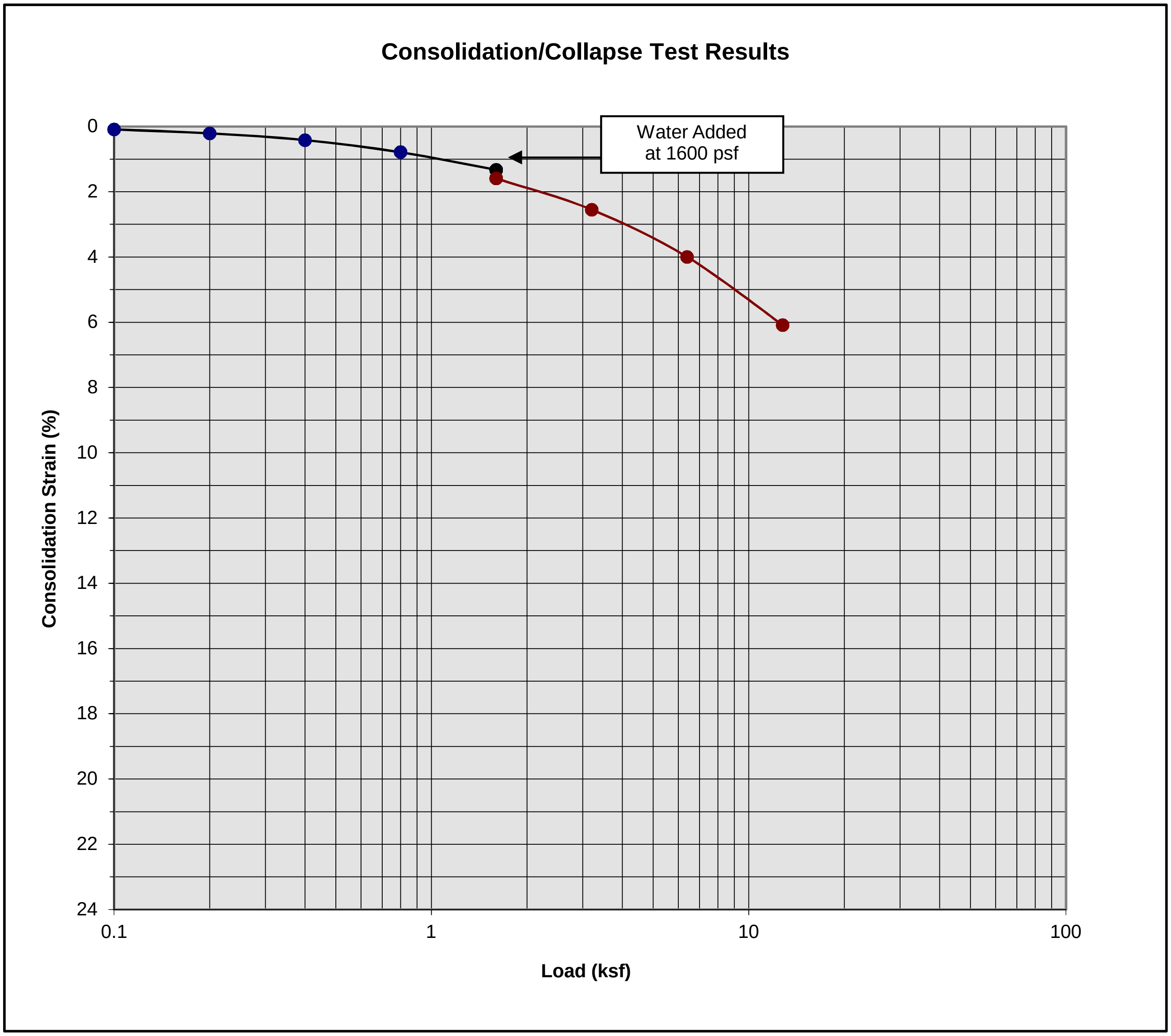
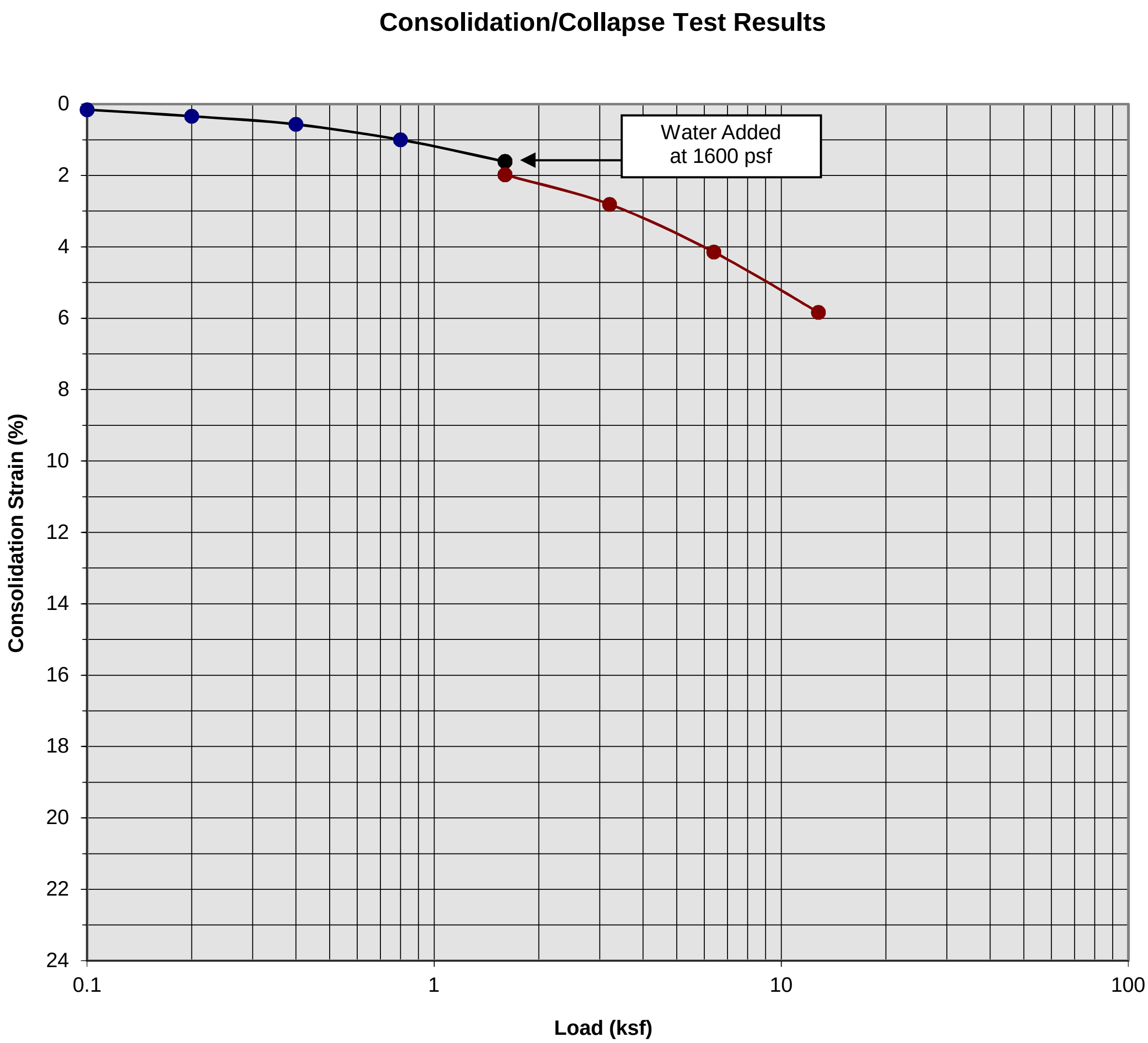


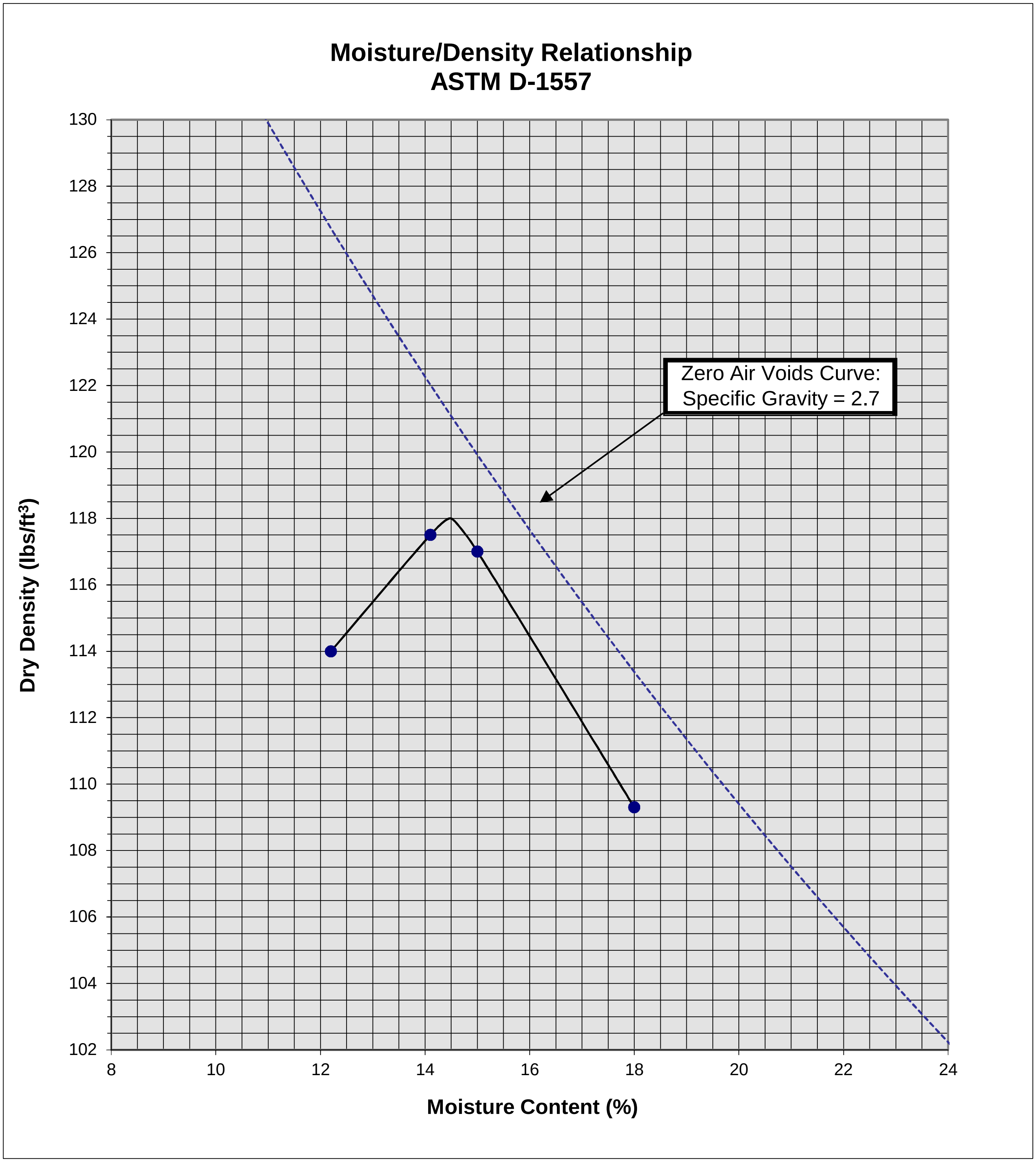
Classification: Gray Brown Clayey Silt, trace fine Sand

Boring Number:	B-6	Initial Moisture Content (%)	16
Sample Number:	---	Final Moisture Content (%)	22
Depth (ft)	7 to 8	Initial Dry Density (pcf)	106.0
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	113.0
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.26



Classification: Brown fine Sandy Clay

Boring Number:	B-6	Initial Moisture Content (%)	15
Sample Number:	---	Final Moisture Content (%)	16
Depth (ft)	9 to 10	Initial Dry Density (pcf)	114.8
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	122.8
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.37



Soil ID Number		B-3 @ 0 to 5'
Optimum Moisture (%)		14.5
Maximum Dry Density (pcf)		118
Soil Classification	Gray Brown fine Sandy Silt, little Clay, occ calcareous nodules	

Rider 4
Perris, California
Project No. 17G206
PLATE C-9

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

APPENDIX D

GRADING GUIDE SPECIFICATIONS

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

General

- The Earthwork Contractor is responsible for the satisfactory completion of all earthwork in accordance with the plans and geotechnical reports, and in accordance with city, county, and applicable building codes.

- The Geotechnical Engineer is the representative of the Owner/Builder for the purpose of implementing the report recommendations and guidelines. These duties are not intended to relieve the Earthwork Contractor of any responsibility to perform in a workman-like manner, nor is the Geotechnical Engineer to direct the grading equipment or personnel employed by the Contractor.

- The Earthwork Contractor is required to notify the Geotechnical Engineer of the anticipated work and schedule so that testing and inspections can be provided. If necessary, work may be stopped and redone if personnel have not been scheduled in advance.

- The Earthwork Contractor is required to have suitable and sufficient equipment on the job-site to process, moisture condition, mix and compact the amount of fill being placed to the approved compaction. In addition, suitable support equipment should be available to conform with recommendations and guidelines in this report.

- Canyon cleanouts, overexcavation areas, processed ground to receive fill, key excavations, subdrains and benches should be observed by the Geotechnical Engineer prior to placement of any fill. It is the Earthwork Contractor's responsibility to notify the Geotechnical Engineer of areas that are ready for inspection.

- Excavation, filling, and subgrade preparation should be performed in a manner and sequence that will provide drainage at all times and proper control of erosion. Precipitation, springs, and seepage water encountered shall be pumped or drained to provide a suitable working surface. The Geotechnical Engineer must be informed of springs or water seepage encountered during grading or foundation construction for possible revision to the recommended construction procedures and/or installation of subdrains.

Site Preparation

- The Earthwork Contractor is responsible for all clearing, grubbing, stripping and site preparation for the project in accordance with the recommendations of the Geotechnical Engineer.

- If any materials or areas are encountered by the Earthwork Contractor which are suspected of having toxic or environmentally sensitive contamination, the Geotechnical Engineer and Owner/Builder should be notified immediately.

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

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

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

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

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

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

<u>Compacted Fills</u>

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

Foundations

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

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

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

Fill Slopes

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

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

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

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

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

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

Cut Slopes

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

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

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

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

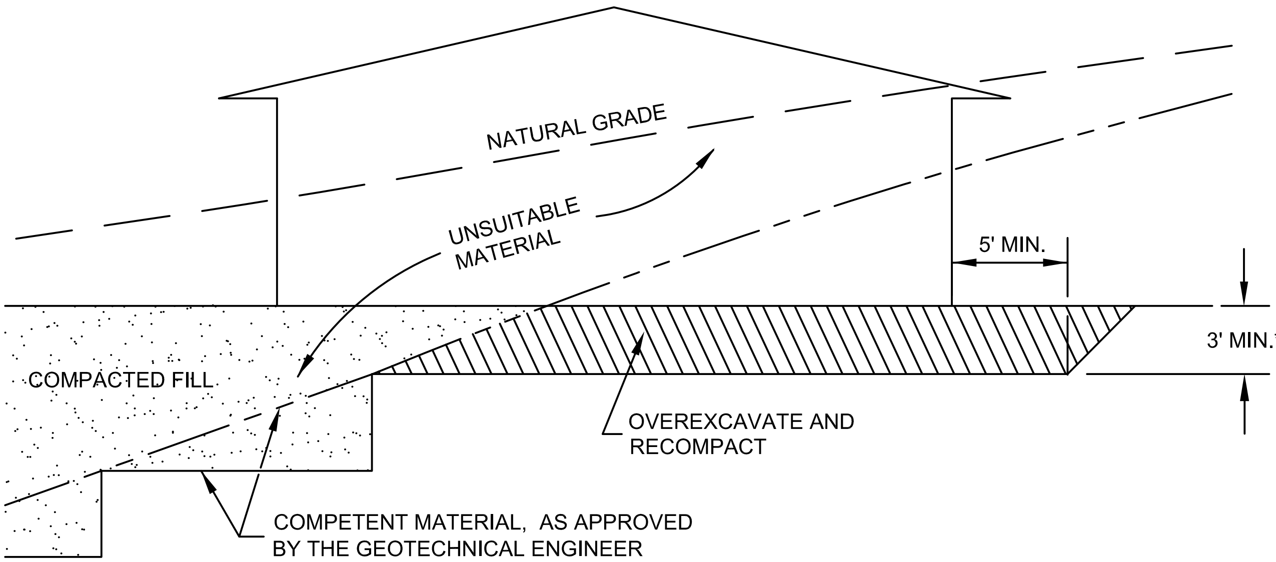
Subdrains

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

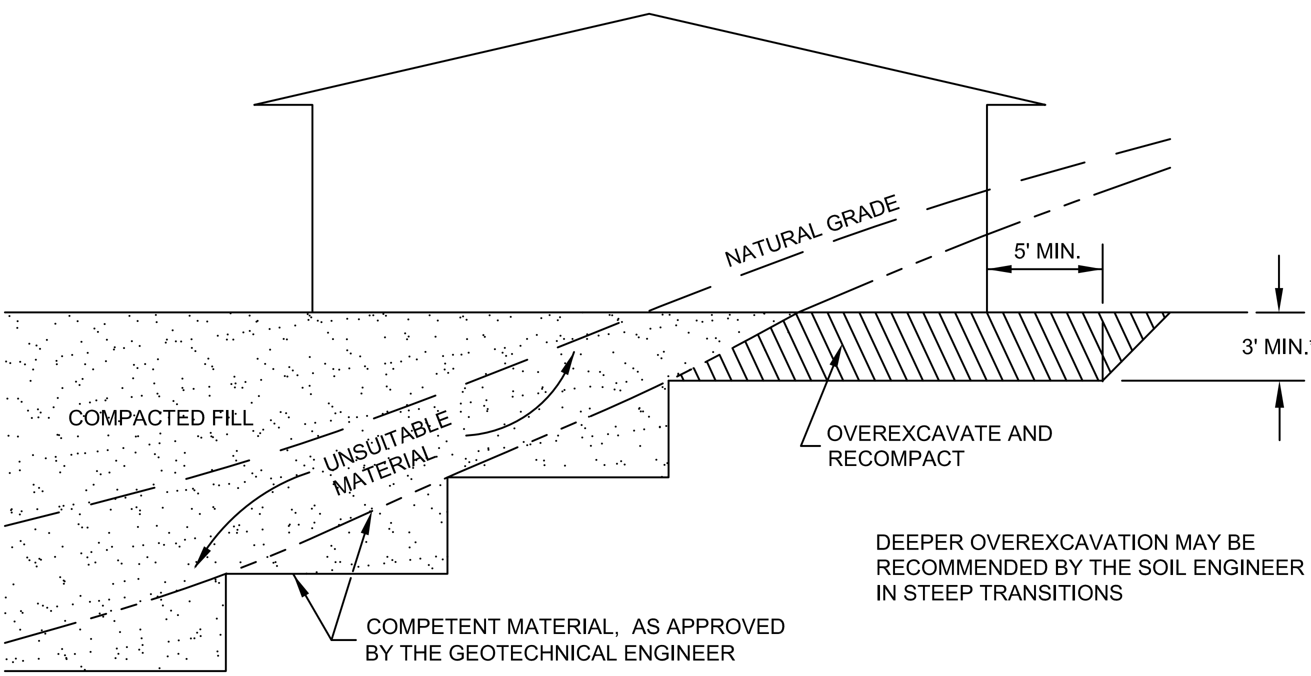
- Plastic pipe may be used for subdrains provided it is Schedule 40 or SDR 35 or equivalent. Pipe should be protected against breakage, typically by placement in a square-cut (backhoe) trench or as recommended by the manufacturer.

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

CUT LOT

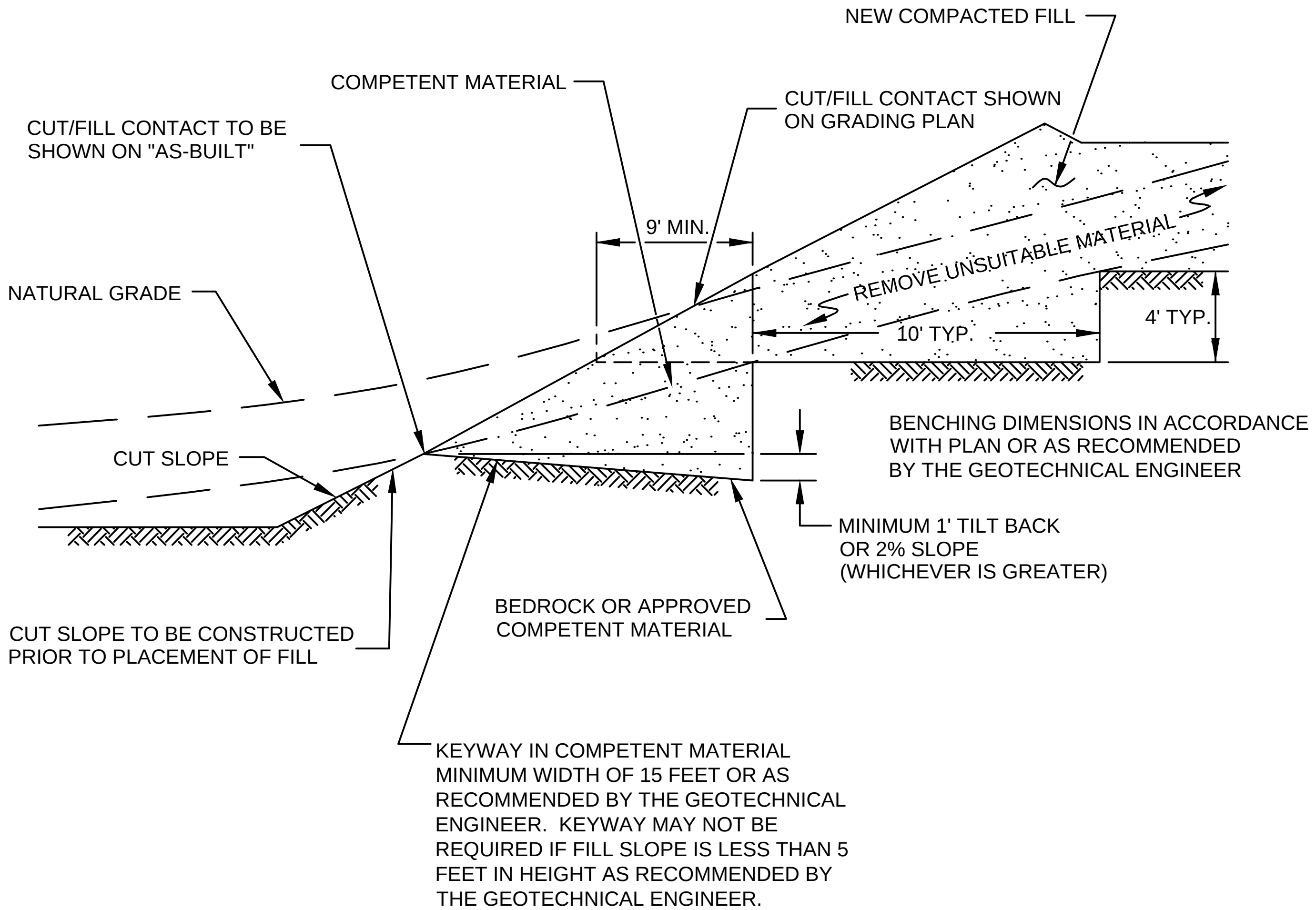


CUT/FILL LOT (TRANSITION)



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

TRANSITION LOT DETAIL	
GRADING GUIDE SPECIFICATIONS	
NOT TO SCALE	SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
DRAWN: JAS CHKD: GKM	
PLATE D-1	



FILL ABOVE CUT SLOPE DETAIL

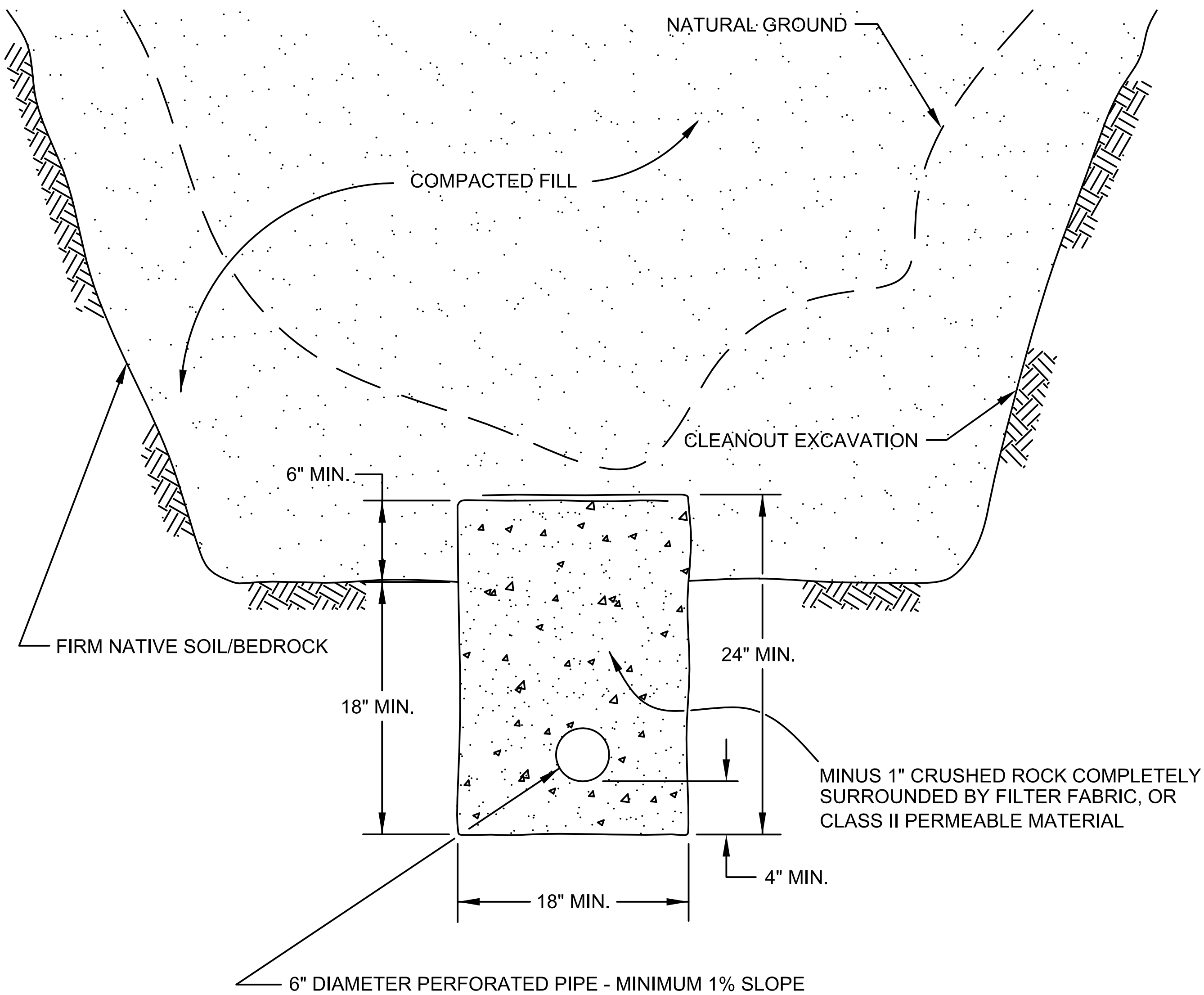
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-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

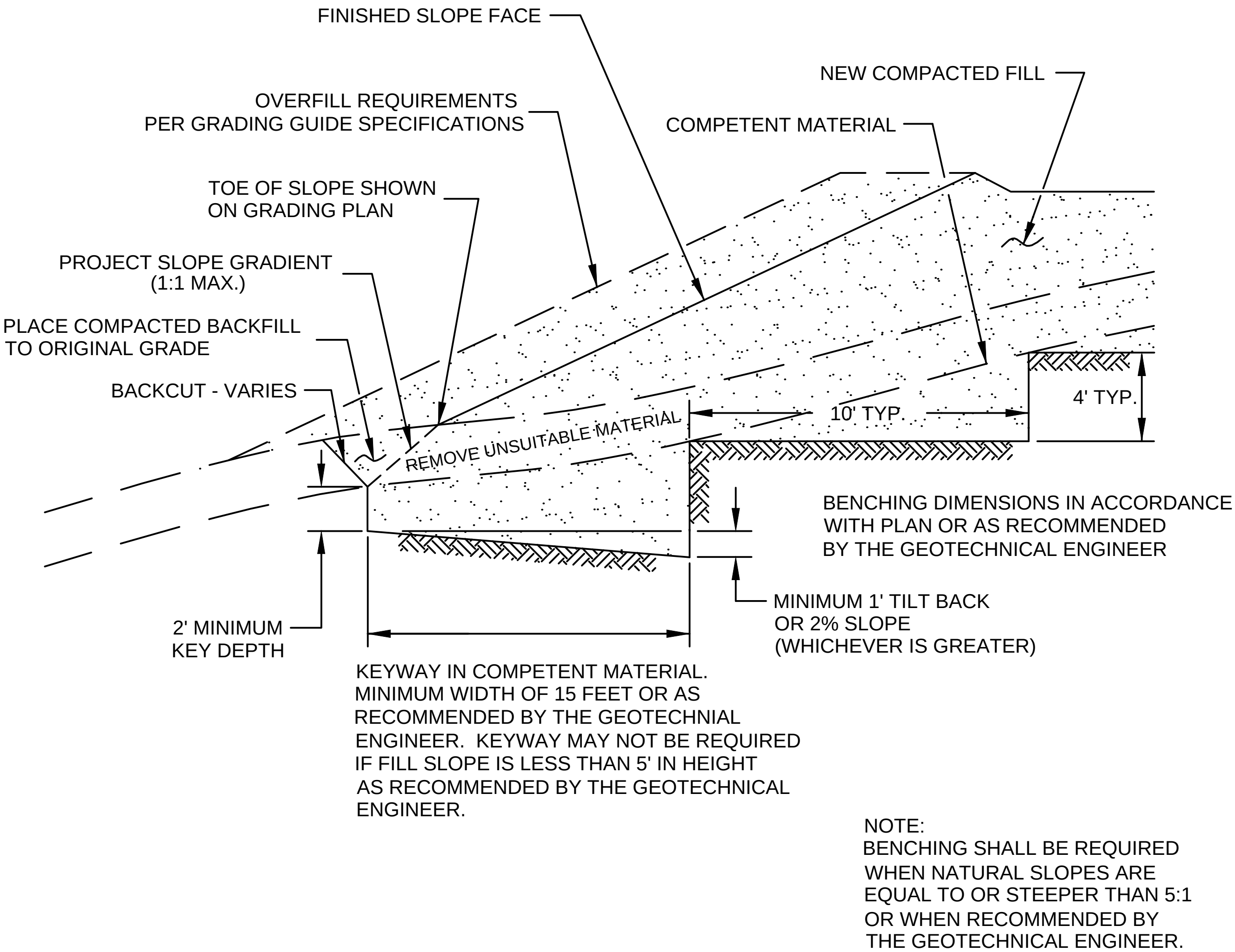
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-3

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL



FILL ABOVE NATURAL SLOPE DETAIL

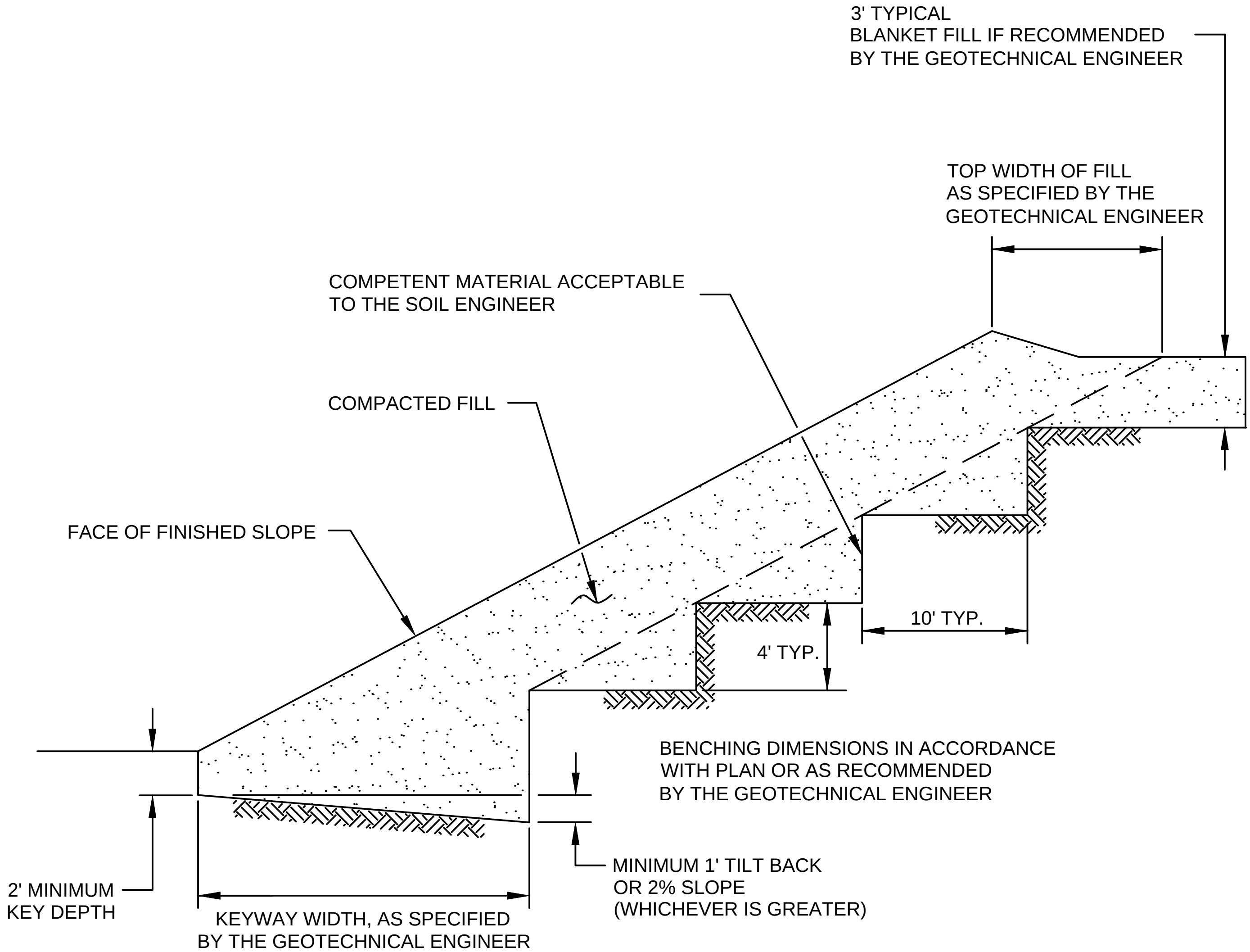
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-4

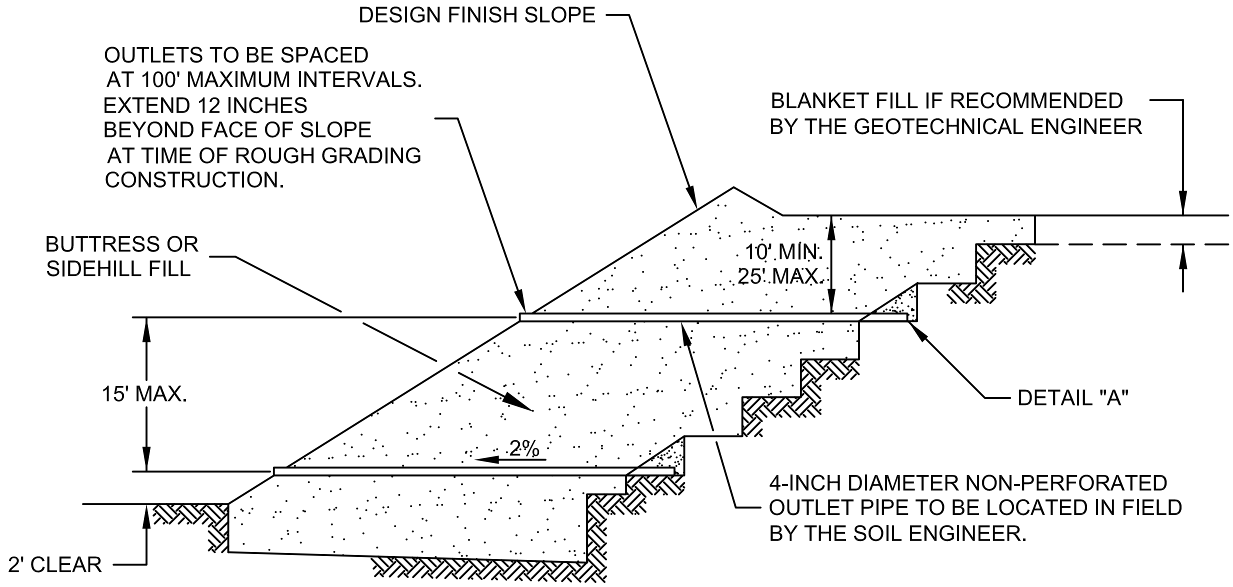
SOUTHERN CALIFORNIA GEOTECHNICAL



STABILIZATION FILL DETAIL

GRADING GUIDE SPECIFICATIONS

NOT TO SCALE	SOUTHERN CALIFORNIA GEOTECHNICAL
DRAWN: JAS CHKD: GKM	
PLATE D-5	



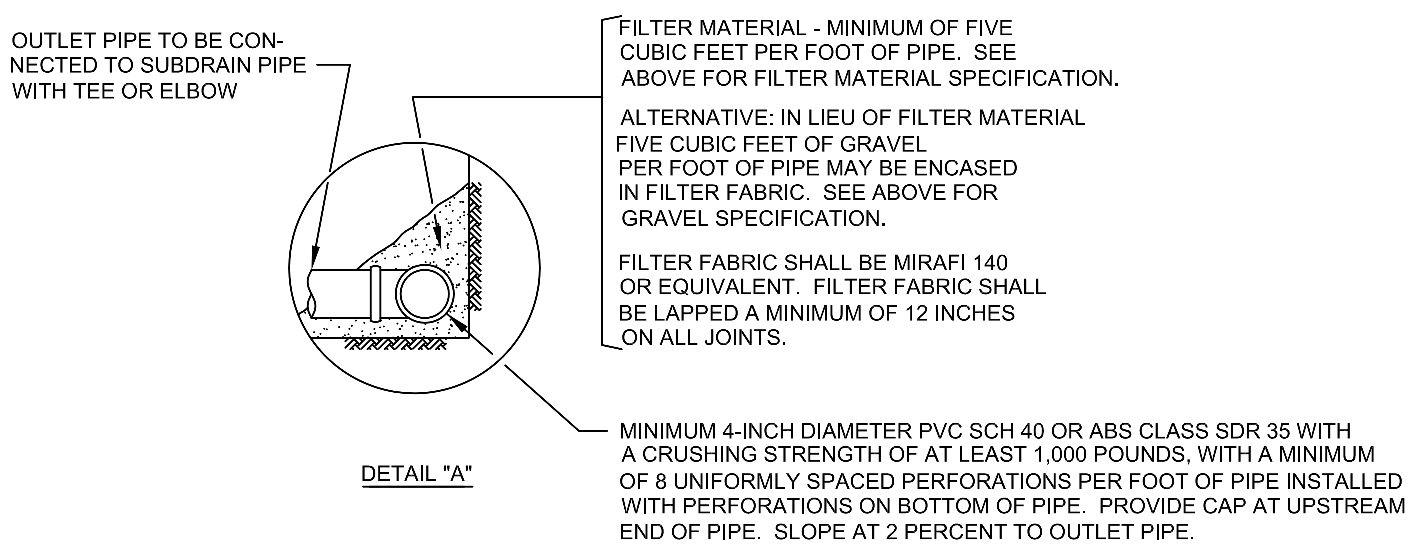
"FILTER MATERIAL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT: (CONFORMS TO EMA STD. PLAN 323)

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

"GRAVEL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT:

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

SAND EQUIVALENT = MINIMUM OF 50



DETAIL "A"

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

SLOPE FILL SUBDRAINS

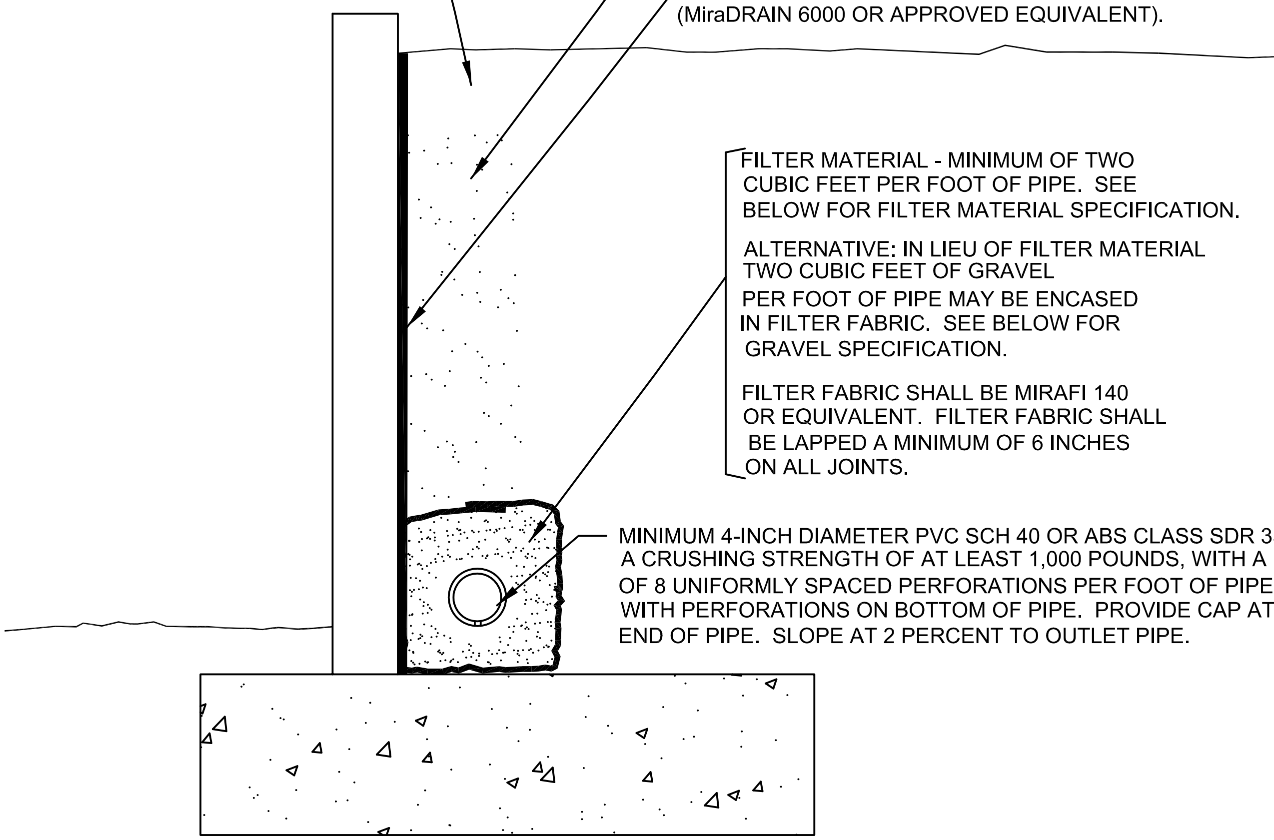
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-6

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL



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

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

"GRAVEL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT:

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

SAND EQUIVALENT = MINIMUM OF 50

RETAINING WALL BACKDRAINS

GRADING GUIDE SPECIFICATIONS

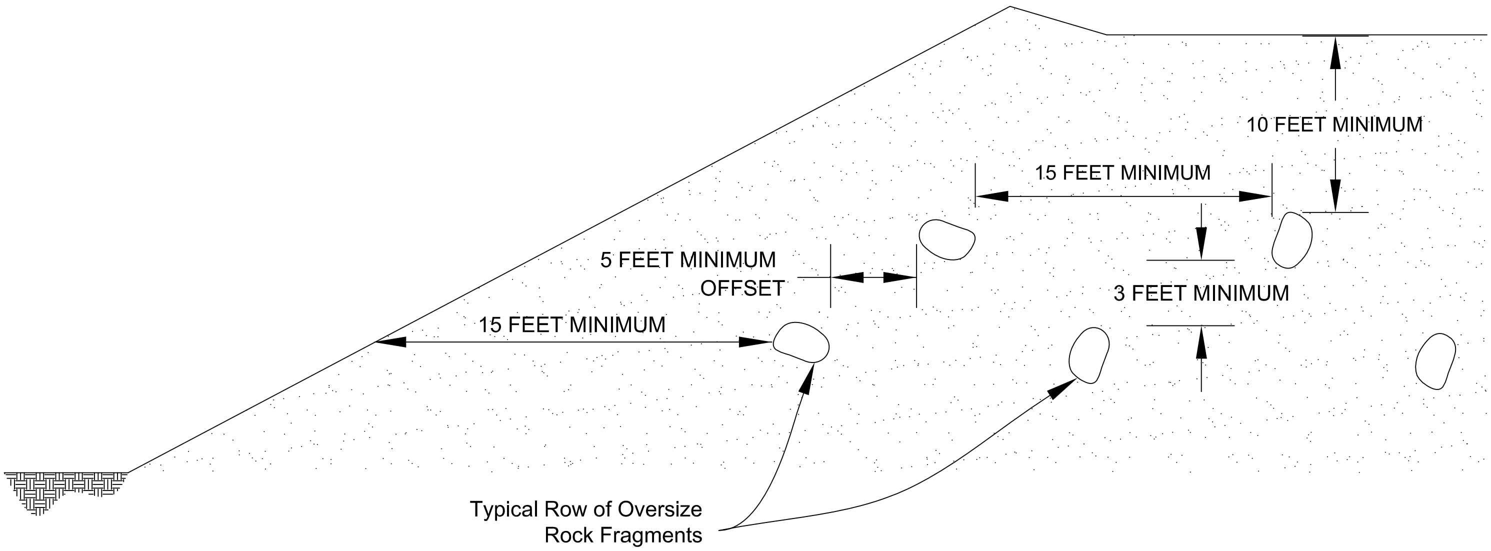
NOT TO SCALE

DRAWN: JAS
CHKD: GKM

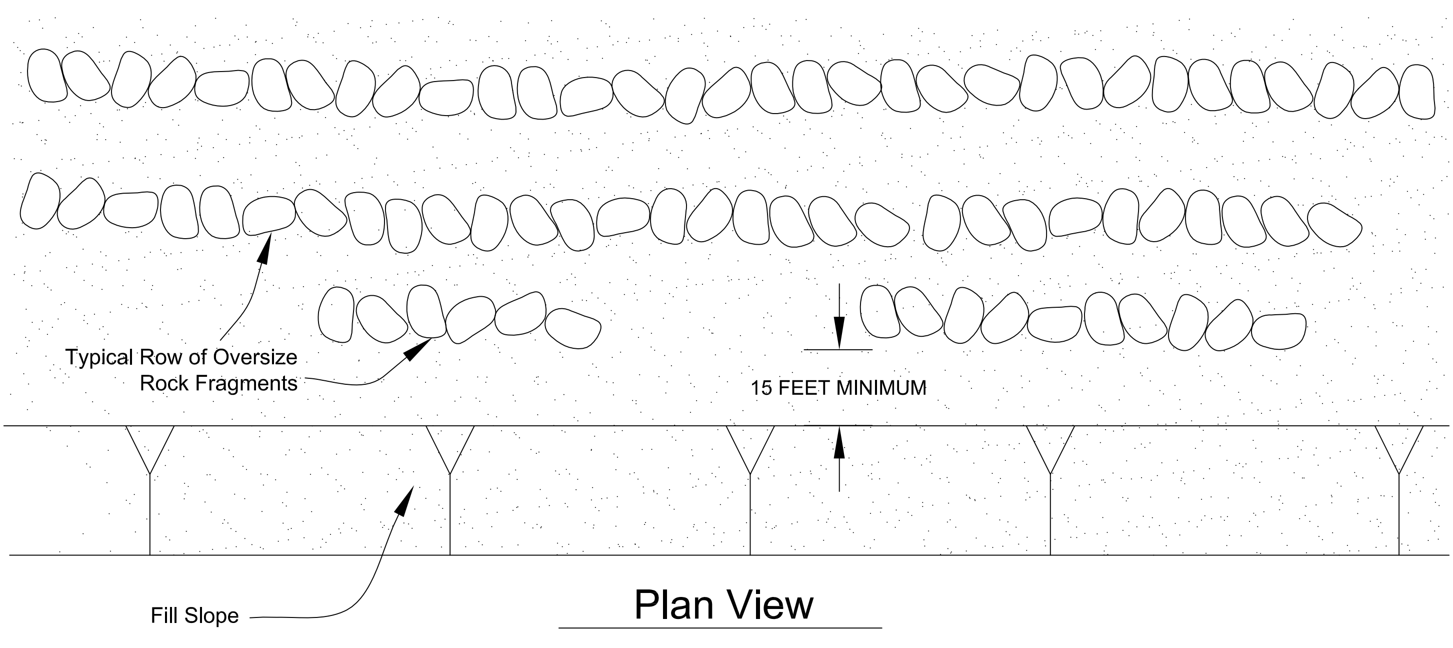
PLATE D-7

SoCalGeo

SOUTHERN CALIFORNIA GEOTECHNICAL



Section View



Plan View

PLACEMENT OF OVERSIZED MATERIAL
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE	SOUTHERN CALIFORNIA GEOTECHNICAL
DRAWN: PM CHKD: GKM	
PLATE D-8	

APPENDIX E

USGS Design Maps Summary Report

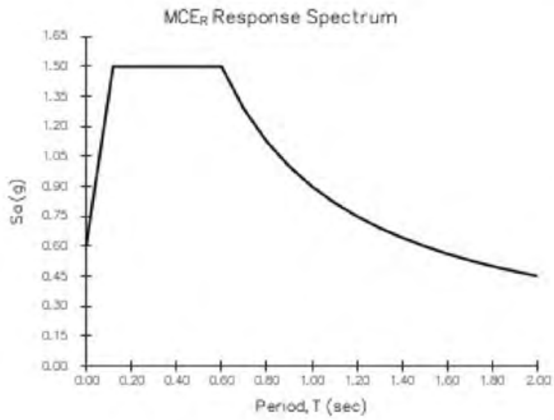
User–Specified Input

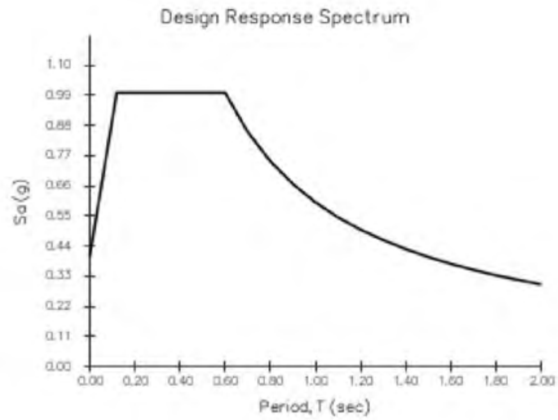
Building Code Reference Document	ASCE 7-10 Standard (which utilizes USGS hazard data available in 2008)
Site Coordinates	33.83516°N, 117.21503°W
Site Soil Classification	Site Class D – "Stiff Soil"
Risk Category	I/II/III

USGS–Provided Output

S_S = 1.500 g	S_{MS} = 1.500 g	S_{DS} = 1.000 g
S_1 = 0.600 g	S_{M1} = 0.900 g	S_{D1} = 0.600 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>

SEISMIC DESIGN PARAMETERS

RIDER 4 - PROPOSED COMMERCIAL/INDUSTRIAL BUILDING

PERRIS, CALIFORNIA

DRAWN: JLH
CHKD: GKM

SCG PROJECT
17G206-1

PLATE E-1

SOUTHERN CALIFORNIA GEOTECHNICAL

Section 11.8.3 — Additional Geotechnical Investigation Report Requirements for Seismic Design Categories D through F

From Figure 22-7 [4] PGA = 0.500

Equation (11.8–1): $PGA_M = F_{PGA}PGA = 1.000 \times 0.500 = 0.5$ g

Table 11.8–1: Site Coefficient F_{PGA}

Site Class	Mapped MCE Geometric Mean Peak Ground Acceleration, PGA				
	PGA ≤ 0.10	PGA = 0.20	PGA = 0.30	PGA = 0.40	PGA ≥ 0.50
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of PGA

For Site Class = D and PGA = 0.500 g, F_{PGA} = 1.000

Section 21.2.1.1 — Method 1 (from Chapter 21 – Site-Specific Ground Motion Procedures for Seismic Design)

From Figure 22-17 [5] C_{RS} = 1.057

From Figure 22-18 [6] C_{R1} = 1.025

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

MCE PEAK GROUND ACCELERATION

RIDER 4 - PROPOSED COMMERCIAL/INDUSTRIAL BUILDING

PERRIS, CALIFORNIA

DRAWN: JLL
CHKD: GKM

SCG PROJECT
17G206-1

PLATE E-2

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL

APPENDIX F

LIQUEFACTION EVALUATION

Project Name	Rider 4	MCE_G Design Acceleration	0.500 (g)
Project Location	Perris, CA	Design Magnitude	7.13
Project Number	17G206	Historic High Depth to Groundwater	26 (ft)
Engineer	DWN	Depth to Groundwater at Time of Drilling	34 (ft)
		Borehole Diameter	6 (in)

Boring No. B-1

Sample Depth (ft)	Depth to Top of Layer (ft)	Depth to Bottom of Layer (ft)	Depth to Midpoint (ft)	Uncorrected SPT N-Value	Unit Weight of Soil (pcf)	Fines Content (%)	Energy Correction	C_B	C_S	C_N	Rod Length Correction	$(N_1)_{60}$	$(N_1)_{60CS}$	Overburden Stress (σ_o) (psf)	Eff. Overburden Stress (Hist. Water) (σ_o') (psf)	Eff. Overburden Stress (Curr. Water) (σ_o') (psf)	Stress Reduction Coefficient (r_d)	MSF	KS	Cyclic Resistance Ratio (M=7.5)	Cyclic Resistance Ratio (M=7.13)	Cyclic Stress Ratio Induced by Design Earthquake	Factor of Safety	Comments
							(1)	(2)	(3)	(4)	(5)	(6)	(7)				(8)	(9)	(10)	(11)	(12)	(13)		
7	0	26	13		120		1.3	1.05	1.1	1.27	0.75	0.0	0.0	1560	1560	1560	0.96	1.01	1.02	0.06	0.06	N/A	N/A	Above Water Table
24.5	26	28	27	53	120		1.3	1.05	1.3	0.97	0.95	86.8	86.8	3240	3178	3240	0.90	1.15	0.88	2.00	2.00	0.30	6.70	Nonliquefiable
29.5	28	32	30	17	120	24	1.3	1.05	1.22	0.81	0.95	21.9	26.9	3600	3350	3600	0.89	1.11	0.92	0.34	0.35	0.31	1.12	**Liquefiable**
34.5	32	37	34.5	16	120	59	1.3	1.05	1.2	0.77	1	20.2	25.8	4140	3610	4109	0.86	1.10	0.91	0.31	0.31	0.32	0.96	**Liquefiable**
39.5	37	42	39.5	29	120		1.3	1.05	1.3	0.81	1	41.7	41.7	4740	3898	4397	0.84	1.15	0.82	2.00	1.89	0.33	5.71	Nonliquefiable
44.5	42	47	44.5	25	120	32	1.3	1.05	1.3	0.79	1	35.1	40.5	5340	4186	4685	0.81	1.15	0.8	2.00	1.84	0.34	5.47	Nonliquefiable
49.5	47	50	48.5	20	120	32	1.3	1.05	1.25	0.74	1	25.3	30.7	5820	4416	4915	0.79	1.13	0.84	0.53	0.51	0.34	1.51	Nonliquefiable

Notes:

(1) Energy Correction for N_{90} of automatic hammer to standard N_{60}

(2) Borehole Diameter Correction (Skempton, 1986)

(3) Correction for split-spoon sampler with room for liners, but liners are absent, (Seed et al., 1984, 2001)

(4) Overburden Correction, Caluclated by Eq. 39 (Boulanger and Idriss, 2008)

(5) Rod Length Correction for Samples <10 m in depth

(6) N-value corrected for energy, borehole diameter, sampler with absent liners, rod length, and overburden

(7) N-value corrected for fines content per Eqs. 75 and 76 (Boulanger and Idriss, 2008)

(8) Stress Reduction Coefficient calculated by Eq. 22 (Boulanger and Idriss, 2008)

(9) Magnitude Scaling Factor calculated by Eqns. A.8 & A.10 (Boulanger and Idriss, 2014)

(10) Overburden Correction Factor calcuated by Eq. 54 (Boulanger and Idriss, 2008)

(11) Calcuated by Eq. 70 (Boulanger and Idriss, 2008)

(12) Calcuated by Eq. 72 (Boulanger and Idriss, 2008)

(13) Calcuated by Eq. 25 (Boulanger and Idriss, 2008)

LIQUEFACTION INDUCED SETTLEMENTS

Project Name	Rider 4
Project Location	Perris, CA
Project Number	17G206
Engineer	DWN

Boring No. B-1

Sample Depth (ft)	Depth to Top of Layer (ft)	Depth to Bottom of Layer (ft)	Depth to Midpoint (ft)	$(N_1)_{60}$	DN for fines cont	$(N_1)_{60\text{-}CS}$	Liquefaction Factor of Safety	Limiting Shear Strain γ_{min}	Parameter $F\alpha$	Maximum Shear Strain γ_{max}	Height of Layer		Vertical Reconsolidation Strain ε_v		Total Deformation of Layer (in)	Comments
				(1)	(2)	(3)	(4)	(5)	(6)	(7)			(8)			
7	0	26	13	0.0	0.0	0.0	N/A	0.50	0.95	0.00	26.00		0.000		0.00	Above Water Table
24.5	26	28	27	86.8	0.0	86.8	6.70	0.00	-4.82	0.00	2.00		0.000		0.00	Nonliquefiable
29.5	28	32	30	21.9	5.0	26.9	1.12	0.07	0.12	0.03	4.00		0.006		0.29	**Liquefiable**
34.5	32	37	34.5	20.2	5.6	25.8	0.96	0.08	0.18	0.04	5.00		0.009		0.52	**Liquefiable**
39.5	37	42	39.5	41.7	0.0	41.7	5.71	0.01	-0.93	0.00	5.00		0.000		0.00	Nonliquefiable
44.5	42	47	44.5	35.1	5.4	40.5	5.47	0.01	-0.84	0.00	5.00		0.000		0.00	Nonliquefiable
49.5	47	50	48.5	25.3	5.4	30.7	1.51	0.04	-0.14	0.01	3.00		0.000		0.00	Nonliquefiable
											Total Deformation (in)				0.81	

Notes:

(1) $(N_1)_{60}$ calculated previously for the individual layer

(2) Correction for fines content per Equation 76 (Boulanger and Idriss, 2008)

(3) Corrected $(N_1)_{60}$ for fines content

(4) Factor of Safety against Liquefaction, calculated previously for the individual layer

(5) Calcuated by Eq. 86 (Boulanger and Idriss, 2008)

(6) Calcuated by Eq. 89 (Boulanger and Idriss, 2008)

(7) Calcuated by Eqs. 90, 91, and 92 (Boulanger and Idriss, 2008)

(8) Volumetric Strain Induced in a Liquefiable Layer, Calcuated by Eq. 96 (Boulanger and Idriss, 2008)
(Strain N/A if Factor of Safety against Liquefaction > 1.3)

LIQUEFACTION EVALUATION

Project Name	Rider 4	MCE_G Design Acceleration	0.500 (g)
Project Location	Perris, CA	Design Magnitude	7.13
Project Number	17G206	Historic High Depth to Groundwater	26 (ft)
Engineer	DWN	Depth to Groundwater at Time of Drilling	34 (ft)
		Borehole Diameter	6 (in)
Boring No.	B-8		

Sample Depth (ft)	Depth to Top of Layer (ft)	Depth to Bottom of Layer (ft)	Depth to Midpoint (ft)	Uncorrected SPT N-Value	Unit Weight of Soil (pcf)	Fines Content (%)	Energy Correction	C_B	C_S	C_N	Rod Length Correction	$(N_1)_{60}$	$(N_1)_{60CS}$	Overburden Stress (σ_o) (psf)	Eff. Overburden Stress (Hist. Water) (σ_o') (psf)	Eff. Overburden Stress (Curr. Water) (σ_o') (psf)	Stress Reduction Coefficient (r_d)	MSF	KS	Cyclic Resistance Ratio (M=7.5)	Cyclic Resistance Ratio (M=7.13)	Cyclic Stress Ratio Induced by Design Earthquake	Factor of Safety	Comments
							(1)	(2)	(3)	(4)	(5)	(6)	(7)				(8)	(9)	(10)	(11)	(12)	(13)		
7	0	26	13		120		1.3	1.05	1.1	1.27	0.75	0.0	0.0	1560	1560	1560	0.96	1.01	1.02	0.06	0.06	N/A	N/A	Above Water Table
24.5	26	28	27	27	120		1.3	1.05	1.3	0.88	0.95	40.1	40.1	3240	3178	3240	0.90	1.15	0.88	2.00	2.00	0.30	6.70	Nonliquefiable
29.5	28	32	30	28	120		1.3	1.05	1.3	0.85	0.95	40.3	40.3	3600	3350	3600	0.89	1.15	0.86	2.00	1.99	0.31	6.43	Nonliquefiable
34.5	32	37	34.5	21	120	44	1.3	1.05	1.3	0.81	1	30.0	35.6	4140	3610	4109	0.86	1.15	0.85	1.27	1.25	0.32	3.88	Nonliquefiable
39.5	37	39.5	38.3	14	120	21	1.3	1.05	1.16	0.73	1	16.3	21.0	4590	3826	4325	0.84	1.07	0.92	0.22	0.21	0.33	0.65	**Liquefiable**
39.5	39.5	42	40.8	14	120	55	1.3	1.05	1.16	0.73	1	16.1	21.8	4890	3970	4469	0.83	1.07	0.91	0.23	0.22	0.33	0.67	**Liquefiable**
44.5	42	47	44.5	14	120	42	1.3	1.05	1.16	0.71	1	15.7	21.3	5340	4186	4685	0.81	1.07	0.9	0.22	0.22	0.34	0.64	**Liquefiable**
49.5	47	50	48.5	31	120		1.3	1.05	1.3	0.79	1	43.5	43.5	5820	4416	4915	0.79	1.15	0.78	2.00	1.80	0.34	5.33	Nonliquefiable

Notes:

(1) Energy Correction for N_{90} of automatic hammer to standard N_{60}

(2) Borehole Diameter Correction (Skempton, 1986)

(3) Correction for split-spoon sampler with room for liners, but liners are absent, (Seed et al., 1984, 2001)

(4) Overburden Correction, Caluclated by Eq. 39 (Boulanger and Idriss, 2008)

(5) Rod Length Correction for Samples <10 m in depth

(6) N-value corrected for energy, borehole diameter, sampler with absent liners, rod length, and overburden

(7) N-value corrected for fines content per Eqs. 75 and 76 (Boulanger and Idriss, 2008)

(8) Stress Reduction Coefficient calculated by Eq. 22 (Boulanger and Idriss, 2008)

(9) Magnitude Scaling Factor calculated by Eqns. A.8 & A.10 (Boulanger and Idriss, 2014)

(10) Overburden Correction Factor calcuated by Eq. 54 (Boulanger and Idriss, 2008)

(11) Calcuated by Eq. 70 (Boulanger and Idriss, 2008)

(12) Calcuated by Eq. 72 (Boulanger and Idriss, 2008)

(13) Calcuated by Eq. 25 (Boulanger and Idriss, 2008)

LIQUEFACTION INDUCED SETTLEMENTS

Project Name	Rider 4
Project Location	Perris, CA
Project Number	17G206
Engineer	DWN

Boring No. B-8

Sample Depth (ft)	Depth to Top of Layer (ft)	Depth to Bottom of Layer (ft)	Depth to Midpoint (ft)	$(N_1)_{60}$	DN for fines cont	$(N_1)_{60\text{-CS}}$	Liquefaction Factor of Safety	Limiting Shear Strain γ_{min}	Parameter $F\alpha$	Maximum Shear Strain γ_{max}	Height of Layer		Vertical Reconsolidation Strain ε_v		Total Deformation of Layer (in)	Comments
				(1)	(2)	(3)	(4)	(5)	(6)	(7)			(8)			
7	0	26	13	0.0	0.0	0.0	N/A	0.50	0.95	0.00	26.00		0.000		0.00	Above Water Table
24.5	26	28	27	40.1	0.0	40.1	6.70	0.01	-0.81	0.00	2.00		0.000		0.00	Nonliquefiable
29.5	28	32	30	40.3	0.0	40.3	6.43	0.01	-0.83	0.00	4.00		0.000		0.00	Nonliquefiable
34.5	32	37	34.5	30.0	5.6	35.6	3.88	0.02	-0.48	0.00	5.00		0.000		0.00	Nonliquefiable
39.5	37	39.5	38.3	16.3	4.6	21.0	0.65	0.14	0.47	0.14	2.50		0.022		0.66	**Liquefiable**
39.5	39.5	42	40.8	16.1	5.6	21.8	0.67	0.13	0.42	0.11	2.50		0.021		0.64	**Liquefiable**
44.5	42	47	44.5	15.7	5.6	21.3	0.64	0.14	0.45	0.13	5.00		0.022		1.31	**Liquefiable**
49.5	47	50	48.5	43.5	0.0	43.5	5.33	0.00	-1.08	0.00	3.00		0.000		0.00	Nonliquefiable
											Total Deformation (in)				2.62	

Notes:

(1) $(N_1)_{60}$ calculated previously for the individual layer

(2) Correction for fines content per Equation 76 (Boulanger and Idriss, 2008)

(3) Corrected $(N_1)_{60}$ for fines content

(4) Factor of Safety against Liquefaction, calculated previously for the individual layer

(5) Calcuated by Eq. 86 (Boulanger and Idriss, 2008)

(6) Calcuated by Eq. 89 (Boulanger and Idriss, 2008)

(7) Calcuated by Eqs. 90, 91, and 92 (Boulanger and Idriss, 2008)

(8) Volumetric Strain Induced in a Liquefiable Layer, Calcuated by Eq. 96 (Boulanger and Idriss, 2008)
(Strain N/A if Factor of Safety against Liquefaction > 1.3)

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

November 30, 2017

IDI Gazeley
8 Corporate Park, Suite 300-34
Irvine, California 92606

Attention: Mr. Stephen Hollis

Project No.: **17G206-2**

Subject: **Results of Infiltration Testing**
Rider 4 – Proposed Commercial/Industrial Building
SEC Redlands Avenue at Morgan Street
Perris, California

Reference: Geotechnical Investigation, Rider 4 – Proposed Commercial/Industrial Building, SEC Redlands Avenue at Morgan Street, Perris, California, prepared for IDI Gazeley by Southern California Geotechnical, Inc. (SCG), SCG Project No. 17G206-1, dated November 30, 2017.

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. 17P383 dated October 10, 2017. The scope of services included site reconnaissance, subsurface exploration, field testing, and engineering analysis to determine the infiltration rates of the onsite 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 site is located at the southeast corner of Redlands Avenue and Morgan Street in Perris, California. The site is bounded to the north by vacant lot, to the west by Redlands Avenue, to the south by an agricultural field, and to the east by a flood channel. The general location of the site is illustrated on the Site Location Map included as Plate 1 of this report.

The subject site consists of an irregular-shaped parcel, approximately 37.93± acres in size. The site is currently being utilized or was recently utilized as an agricultural field. The current ground surface cover consists of exposed soil and extensive crop stubble. There is an existing water pump station located at the northwest corner of the site.

Detailed topographic information was obtained from a conceptual site plan prepared by Albert A. Webb Associates. This plan indicates that the overall site topography generally slopes downward to the southeast at an estimated gradient of less than 1 percent. The maximum site elevation is

1448± feet mean sea level (msl) located in the northwestern corner of the subject site, and the minimum site elevation is 1443± feet msl in the southeastern corner of the subject site.

Proposed Development

A site plan for the proposed development was provided to our office by the client. The plan indicates that the site will be developed with one (1) new warehouse building. The building will be located in the center of the site and will be 540,913± ft² in size. The building will be constructed in a cross-dock configuration with loading docks along both the east and west sides of the building. It is expected that the building will be surrounded by asphaltic concrete pavements for parking and drive lanes and Portland cement concrete pavements in the loading dock areas. Several landscape planters and concrete flatwork are expected to be included throughout the site.

We understand that the proposed development will include on-site infiltration to dispose of storm water. Based on the site plan provided and conversations with the representatives of Albert A. Webb Associates, the project civil engineer, the proposed infiltration system will consist an infiltration basin located in the southeastern corner of the site. The bottom of the proposed infiltration basin will be 13± feet below the existing site grades.

Concurrent Study

Southern California Geotechnical, Inc. (SCG) recently conducted a geotechnical investigation at the subject site, referenced above. As a part of this study, ten (10) borings were advanced to depths of 5 to 50± feet below existing site grades.

Native alluvial soils were encountered at the ground surface at all of the boring locations. The near-surface alluvium generally consists of loose to medium dense silty fine sands and fine sandy silts, extending to depths of 3 to 12± feet below existing site grades. At greater depths, the alluvium consists of stiff to very stiff silty clays and clayey silts. Interbedded layers of medium dense to dense sandy silts and silty sands as well as stiff to very stiff silty clays and clayey silts extend to at least the maximum depth explored of 50± feet.

Groundwater

Free water was encountered during drilling at a depth of 34± feet. Based on the water level measurements and the moisture contents of the recovered soil samples, the static groundwater table is considered to have existed at a depth of 34± feet below existing site grades at the time of the subsurface investigation. As part of our research, we reviewed available groundwater data in order to determine the historic high groundwater level for the site. The primary reference used to determine the groundwater depths in this area is the California Department of Water Resources website, http://www.water.ca.gov/waterdatalibrary/. Several monitoring wells are located within a mile radius of the subject site with high groundwater level readings ranging from 26 to 108± feet from the ground surface. Therefore, the high groundwater depth of 26± feet (February 2012) reported in a monitoring well located 0.9± miles southeast of the subject site is considered to be conservative with respect to the recent site conditions.

Subsurface Exploration

Scope of Exploration

The subsurface exploration for the infiltration testing consisted of three (3) backhoe excavated trenches, extending to depths of 12½ to 13± feet below existing site grades. The trenches were logged during excavation by a member of our staff. The approximate locations of the infiltration trenches (identified as I-1 through I-3) are indicated on the Infiltration Test Location Plan, enclosed as Plate 2 of this report.

Geotechnical Conditions

Native alluvium was encountered at the ground surface at all of the infiltration trench locations, extending to at least 13± feet below existing site grades. The native alluvial soils generally consist of medium dense to dense fine sandy silts and clayey fine to medium sands, and medium stiff to stiff silty clays and fine to medium sandy clays. Free water was not encountered during the excavation of any of the trenches. 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 testing will be used to prepare a preliminary design for the storm water infiltration system that will be used at the subject site. 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 the test locations, the outer ring was driven 3± inches into the soil at the base of each trench. The inner ring was centered inside the outer ring and subsequently driven 3± inches into the soil at the base of the trenches. The rings were driven into the soil using a ten-pound sledge hammer. The soil surrounding the wall of the infiltration rings was only slightly disturbed during the driving process.

Infiltration Testing Procedure

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

The schedule for readings was determined based on the observed soil type at the base of each backhoe excavated trench. Based on the existing soils at each infiltration test location, the volumetric measurements were made at increments of 20 minutes. The water volume measurements are presented on the spreadsheets enclosed with this report. The infiltration rates for each of the timed intervals are also tabulated on these spreadsheets.

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

Infiltration Test No.	Mean Sea Level (feet)	Soil Description	Infiltration Rate (inches/hour)
I-1	1431	Clayey fine to medium Sand	1.0
I-2	1430.5	Fine to medium Sandy Clay	1.7
I-3	1431	Clayey fine to medium Sand, trace Silt	1.4

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

Three (3) infiltration tests were performed at the subject site. As noted above, the calculated infiltration rates at the infiltration test locations range from 1.0 to 1.7 inches per hour. The primary factors affecting the infiltration rates are the varying relative densities, and the clay and silt content of the encountered soils, which vary at different depths and locations at the subject site. In general, dense clayey sands were encountered at the bottom of Infiltration Test No. I-1, which exhibited a slower infiltration rate.

Based on the infiltration test results, we recommend a design infiltration rate of 1 inch per hour be used for the proposed infiltration basin located in the southeastern corner of the subject site.

The design of the proposed storm water infiltration system should be performed by the project civil engineer, in accordance with the City of Perris and/or County of Riverside 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 system. The presence of such materials would decrease the effective infiltration rate. **It is recommended that the project civil engineer apply an appropriate factor of safety. The infiltration rate recommended above is 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 rate is based on infiltration testing at three (3) discrete locations and the overall infiltration rate of the storm water infiltration system could vary considerably.

Infiltration versus Permeability

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

Location of Infiltration Systems

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

General Comments

This report has been prepared as an instrument of service for use by the client in order to aid in the evaluation of this property and to assist the architects and engineers in the design and preparation of the project plans and specifications. This report may be provided to the contractor(s) and other design consultants to disclose information relative to the project. However, this report is not intended to be utilized as a specification in and of itself, without appropriate interpretation by the project architect, structural engineer, and/or civil engineer. The design of the infiltration system is the responsibility of the civil engineer. The role of the geotechnical engineer is limited to determination of infiltration rate only. By using the design infiltration rates contained herein, the civil engineer agrees to indemnify, defend, and hold harmless the geotechnical engineer for all aspects of the design and performance of the infiltration system. The reproduction and distribution of this report must be authorized by the client and Southern California Geotechnical, Inc. Furthermore, any reliance on this report by an unauthorized third party is at such party's sole risk, and we accept no responsibility for damage or loss which may occur.

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

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

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

Closure

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

Respectfully Submitted,

SOUTHERN CALIFORNIA GEOTECHNICAL, INC.

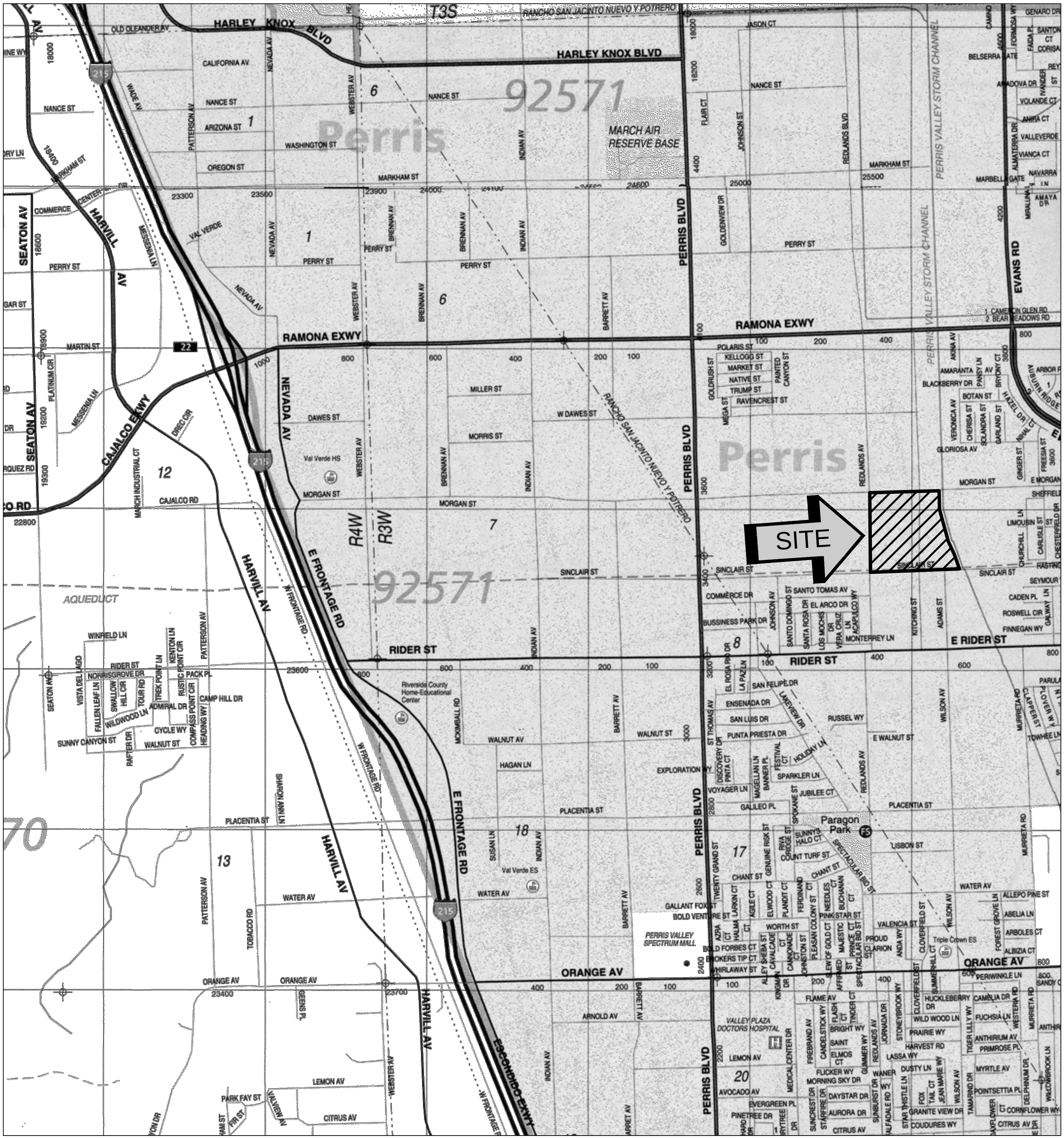
Scott McCann
Staff Scientist

Gregory K. Mitchell, GE 2364
Principal Engineer

REGISTERED PROFESSIONAL ENGINEER
GREGORY K. MITCHELL
No. 2364
GEOTECHNICAL
STATE OF CALIFORNIA

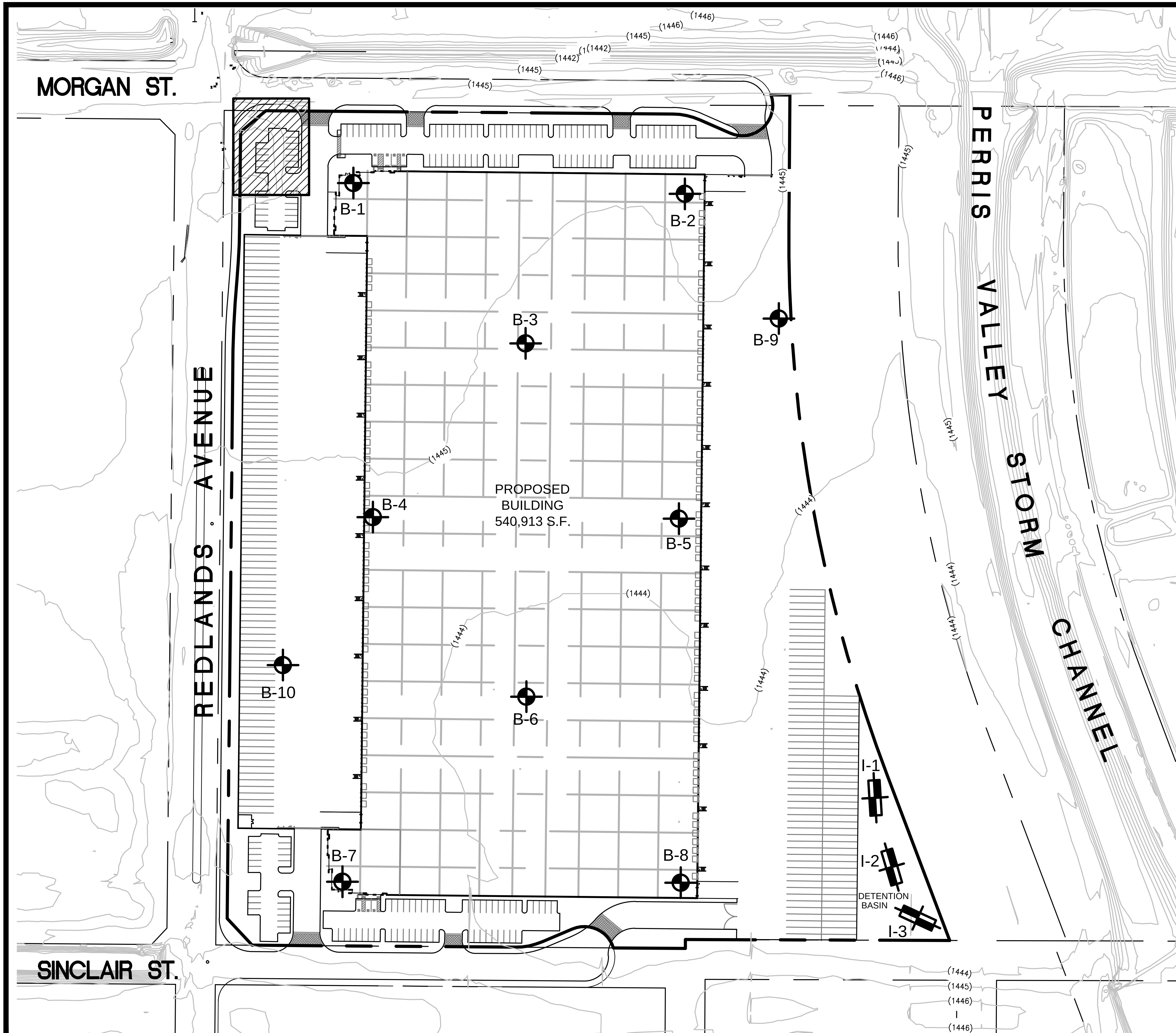
Distribution: (1) Addressee

Enclosures: Plate 1 - Site Location Map
Plate 2 - Infiltration Test Location Plan
Trench Logs (3 pages)
Infiltration Test Results Spreadsheets (3 pages)
Grain Size Distribution Graphs (3 pages)



SOURCE: RIVERSIDE COUNTY THOUGHT
THOMAS GUIDE, 2013

SITE LOCATION MAP	
RIDER 4 - PROPOSED COMMERCIAL/INDUSTRIAL BUILDING	
PERRIS, CALIFORNIA	
SCALE: 1" = 2400'	SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
DRAWN: SAM CHKD: GKM	
SCG PROJECT 17G206-2	
PLATE 1	



GEOTECHNICAL LEGEND

APPROXIMATE INFILTRATION TEST LOCATION

APPROXIMATE BORING LOCATION (SCG PROJECT NO. 17G206-1)

EXISTING PUMP STATION

NOTE: SITE PLAN PREPARED BY ALBERT A. WEBB ASSOCIATES.

INFILTRATION TEST LOCATION PLAN

RIDER 4 - PROPOSED COMMERCIAL/INDUSTRIAL BUILDING

PERRIS, CALIFORNIA

SCALE: 1" = 150'

DRAWN: JLL
CHKD: GKM

SCG PROJECT 17G206-2

PLATE 2

SOUTHERN CALIFORNIA GEOTECHNICAL

SOUTHERN CALIFORNIA GEOTECHNICAL

TRENCH NO.
I-1

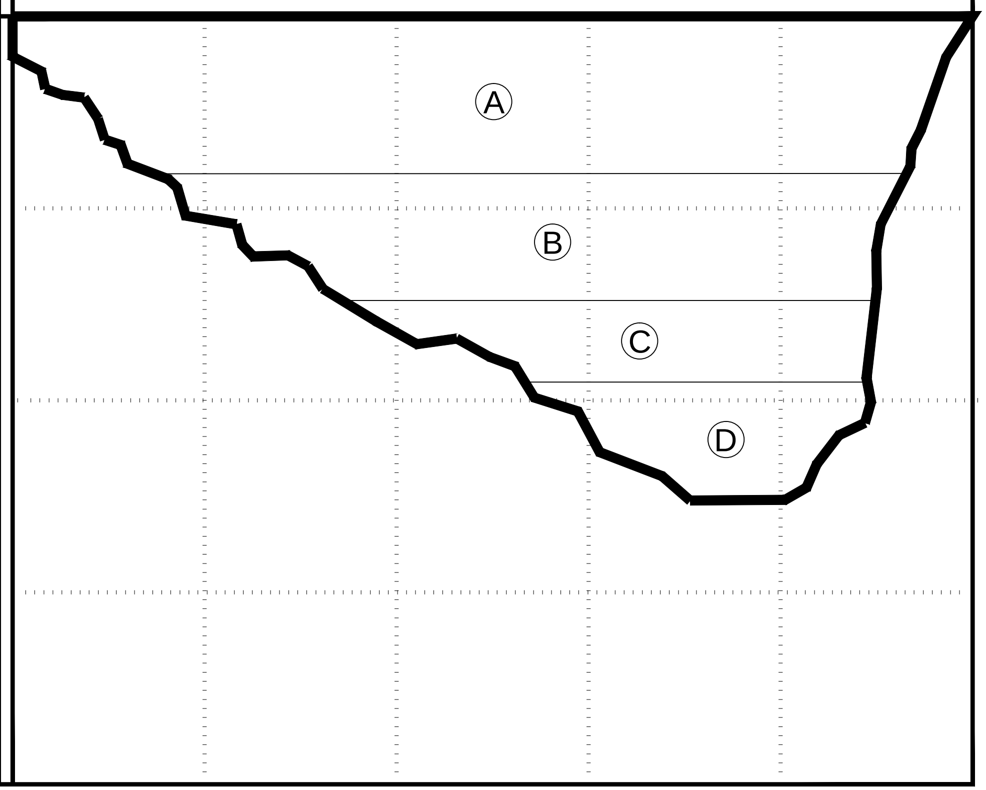
JOB NO.: 17G206-2	EQUIPMENT USED: Backhoe	WATER DEPTH: Dry
PROJECT: Rider 4 - Proposed C/I Building	LOGGED BY: Scott McCann	SEEPAGE DEPTH: Dry
LOCATION: Perris, CA	ORIENTATION: N 4 W	READINGS TAKEN: At Completion
DATE: 11-9-2017	TOP OF TRENCH ELEVATION: 1443.5 feet msl	

DEPTH	SAMPLE	DRY DENSITY (PCF)	MOISTURE (%)	EARTH MATERIALS DESCRIPTION
				A: ALLUVIUM: Light Brown fine Sandy Silt, little medium Sand, trace to abundant fine root fibers, slightly porous, medium dense to dense - damp to moist
5				B: ALLUVIUM: Light Gray Brown Silty Clay, little fine to medium Sand, little calcareous veining, stiff - damp to moist
				C: ALLUVIUM: Light Gray Brown Clayey fine to coarse Sand, trace fine Gravel, trace Silt, little calcareous veining, dense - moist
10				D: ALLUVIUM: Brown Clayey fine to medium Sand, trace calcareous veining, dense - moist
				Trench Terminated @ 12.5 feet Bottom of Trench Elevation: 1431 feet msl
15				

GRAPHIC REPRESENTATION

N 4 W

SCALE: 1" = 5'



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

TRENCH LOG

PLATE B-1

SOUTHERN CALIFORNIA GEOTECHNICAL

TRENCH NO.
I-2

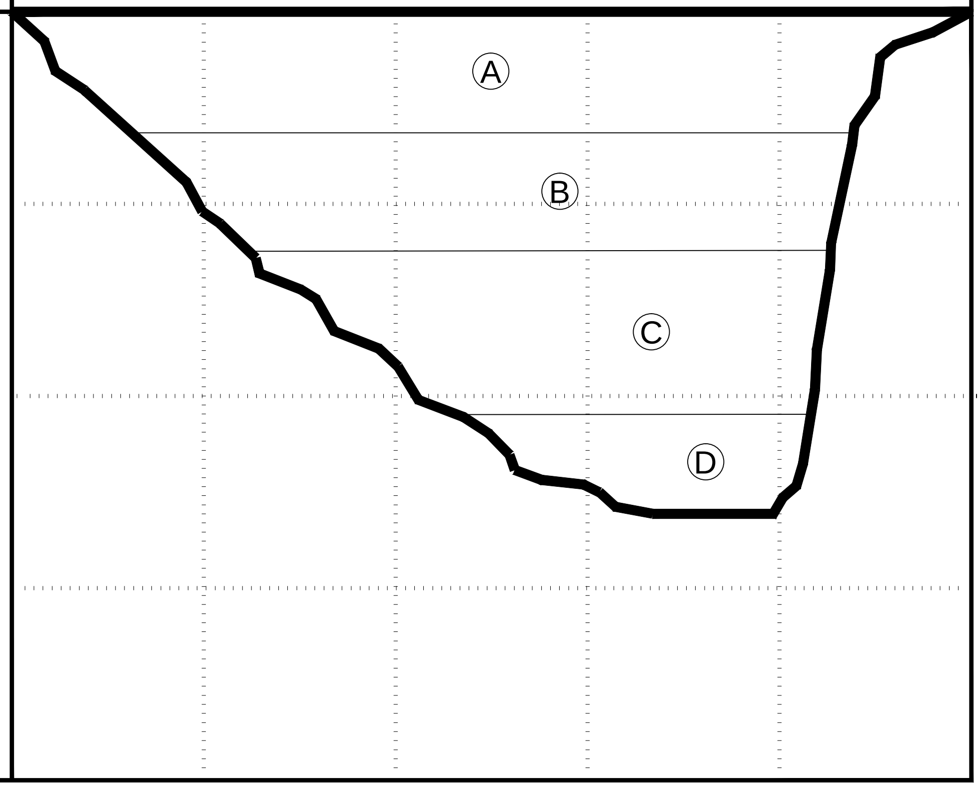
JOB NO.: 17G206-2	EQUIPMENT USED: Backhoe	WATER DEPTH: Dry
PROJECT: Rider 4 - Proposed C/I Building	LOGGED BY: Scott McCann	SEEPAGE DEPTH: Dry
LOCATION: Perris, CA	ORIENTATION: N 17 W	READINGS TAKEN: At Completion
DATE: 11-13-2017	TOP OF TRENCH ELEVATION: 1443.5 feet msl	

DEPTH	SAMPLE	DRY DENSITY (PCF)	MOISTURE (%)	EARTH MATERIALS DESCRIPTION
				A: ALLUVIUM: Light Gray Brown fine Sandy Silt, trace to abundant fine root fibers, slightly porous, medium dense to dense - damp
				B: ALLUVIUM: White to Light Gray Brown Silty Clay, trace fine Sand, trace calcareous nodules, stiff - damp to moist
5				
				C: ALLUVIUM: Light Gray fine Sandy Clay, trace medium Sand, trace Silt, little calcareous nodules, medium stiff - moist
10				
				D: ALLUVIUM: Gray Brown fine to medium Sandy Clay, medium stiff - moist
				Trench Terminated @ 13 feet Bottom of Trench Elevation: 1430.5 feet msl
15				

GRAPHIC REPRESENTATION

N 17 W

SCALE: 1" = 5'



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

TRENCH LOG

PLATE B-2

SOUTHERN CALIFORNIA GEOTECHNICAL

TRENCH NO.
I-3

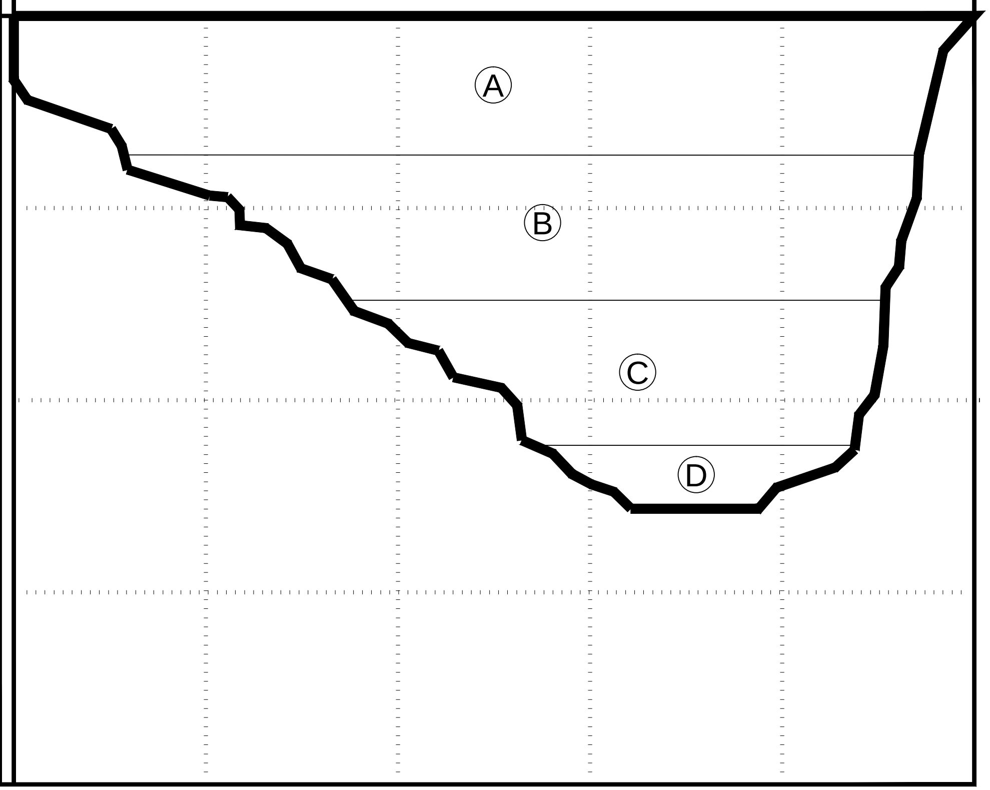
JOB NO.: 17G206-2	EQUIPMENT USED: Backhoe	WATER DEPTH: Dry
PROJECT: Rider 4 - Proposed C/I Building	LOGGED BY: Scott McCann	SEEPAGE DEPTH: Dry
LOCATION: Perris, CA	ORIENTATION: S 66 E	READINGS TAKEN: At Completion
DATE: 11-9-2017	TOP OF TRENCH ELEVATION: 1444 feet msl	

DEPTH	SAMPLE	DRY DENSITY (PCF)	MOISTURE (%)	EARTH MATERIALS DESCRIPTION
				A: ALLUVIUM: Light Brown fine Sandy Silt, trace medium Sand, trace to abundant fine root fibers, slightly porous, medium dense to dense - damp
5				B: ALLUVIUM: Light Gray Brown Silty Clay, little fine Sand, little calcareous veining, stiff - damp to moist
				C: ALLUVIUM: Gray Brown fine Sandy Clay, little Silt, trace medium Sand, trace calcareous veining, medium stiff - moist
10				D: ALLUVIUM: Brown Clayey fine to medium Sand, trace Silt, dense - moist
				Trench Terminated @ 13 feet Bottom of Trench Elevation: 1431 feet msl
15				

GRAPHIC REPRESENTATION

S 66 E

SCALE: 1" = 5'



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

TRENCH LOG

PLATE B-3

INFILTRATION CALCULATIONS

Project Name	Rider 4 - Proposed Commercial/Industrial Building
Project Location	Perris, CA
Project Number	17G206-2
Engineer	Scott McCann

Infiltration Test No: I-1

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

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

Test Interval		Time (hr)	Interval **Elapsed** (min)	Flow Readings: Inner Ring (ml)	Ring Flow (cm^3)	Annular Ring (ml)	Space Flow (cm^3)	Infiltration Rates: Inner Ring* (cm/hr)	Annular Space* (cm/hr)	Inner Ring* (in/hr)	Annular Space* (in/hr)
1	Initial	1:40 PM	20	100	1100	500	3600	4.52	4.93	1.78	1.94
	Final	2:00 PM	**20**	1200		4100					
2	Initial	2:01 PM	20	1250	850	100	2600	3.49	3.56	1.38	1.40
	Final	2:21 PM	**41**	2100		2700					
3	Initial	2:22 PM	20	100	625	0	2400	2.57	3.29	1.01	1.30
	Final	2:42 PM	**62**	725		2400					
4	Initial	2:43 PM	20	0	600	0	2250	2.47	3.08	0.97	1.21
	Final	3:03 PM	**83**	600		2250					
5	Initial	3:04 PM	20	50	600	200	2200	2.47	3.02	0.97	1.19
	Final	3:24 PM	**104**	650		2400					
6	Initial	3:25 PM	20	200	600	900	2100	2.47	2.88	0.97	1.13
	Final	3:45 PM	**125**	800		3000					
7	Initial	3:46 PM	20	100	600	600	2100	2.47	2.88	0.97	1.13
	Final	4:06 PM	**145**	700		2700					

INFILTRATION CALCULATIONS

Project Name	Rider 4 - Proposed Commercial/Industrial Building
Project Location	Perris, CA
Project Number	17G206-2
Engineer	Scott McCann

Infiltration Test No I-2

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

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

Test Interval		Time (hr)	Interval **Elapsed** (min)	Flow Readings: Inner Ring (ml)	Flow Readings: Ring Flow (cm^3)	Flow Readings: Annular Ring (ml)	Flow Readings: Space Flow (cm^3)	Infiltration Rates: Inner Ring* (cm/hr)	Infiltration Rates: Annular Space* (cm/hr)	Infiltration Rates: Inner Ring* (in/hr)	Infiltration Rates: Annular Space* (in/hr)
1	Initial	10:30 AM	20	200	1800	200	6500	7.40	8.91	2.91	3.51
	Final	10:50 AM	**20**	2000		6700					
2	Initial	10:51 AM	20	50	1650	300	6000	6.78	8.22	2.67	3.24
	Final	11:11 AM	**41**	1700		6300					
3	Initial	11:12 AM	20	150	1500	150	5600	6.17	7.68	2.43	3.02
	Final	11:32 AM	**62**	1650		5750					
4	Initial	11:33 AM	20	2100	1250	4400	4800	5.14	6.58	2.02	2.59
	Final	11:53 AM	**83**	3350		9200					
5	Initial	11:54 AM	20	900	1150	1700	4500	4.73	6.17	1.86	2.43
	Final	12:14 PM	**104**	2050		6200					
6	Initial	12:15 PM	20	550	1075	1200	4400	4.42	6.03	1.74	2.37
	Final	12:35 PM	**125**	1625		5600					
7	Initial	12:36 PM	20	700	1050	1500	4400	4.32	6.03	1.70	2.37
	Final	12:56 PM	**145**	1750		5900					
8	Initial	12:57 PM	20	2750	1050	200	4400	4.32	6.03	1.70	2.37
	Final	1:17 PM	**166**	3800		4600					

INFILTRATION CALCULATIONS

Project Name	Rider 4 - Proposed Commercial/Industrial Building
Project Location	Perris, CA
Project Number	17G206-2
Engineer	Scott McCann

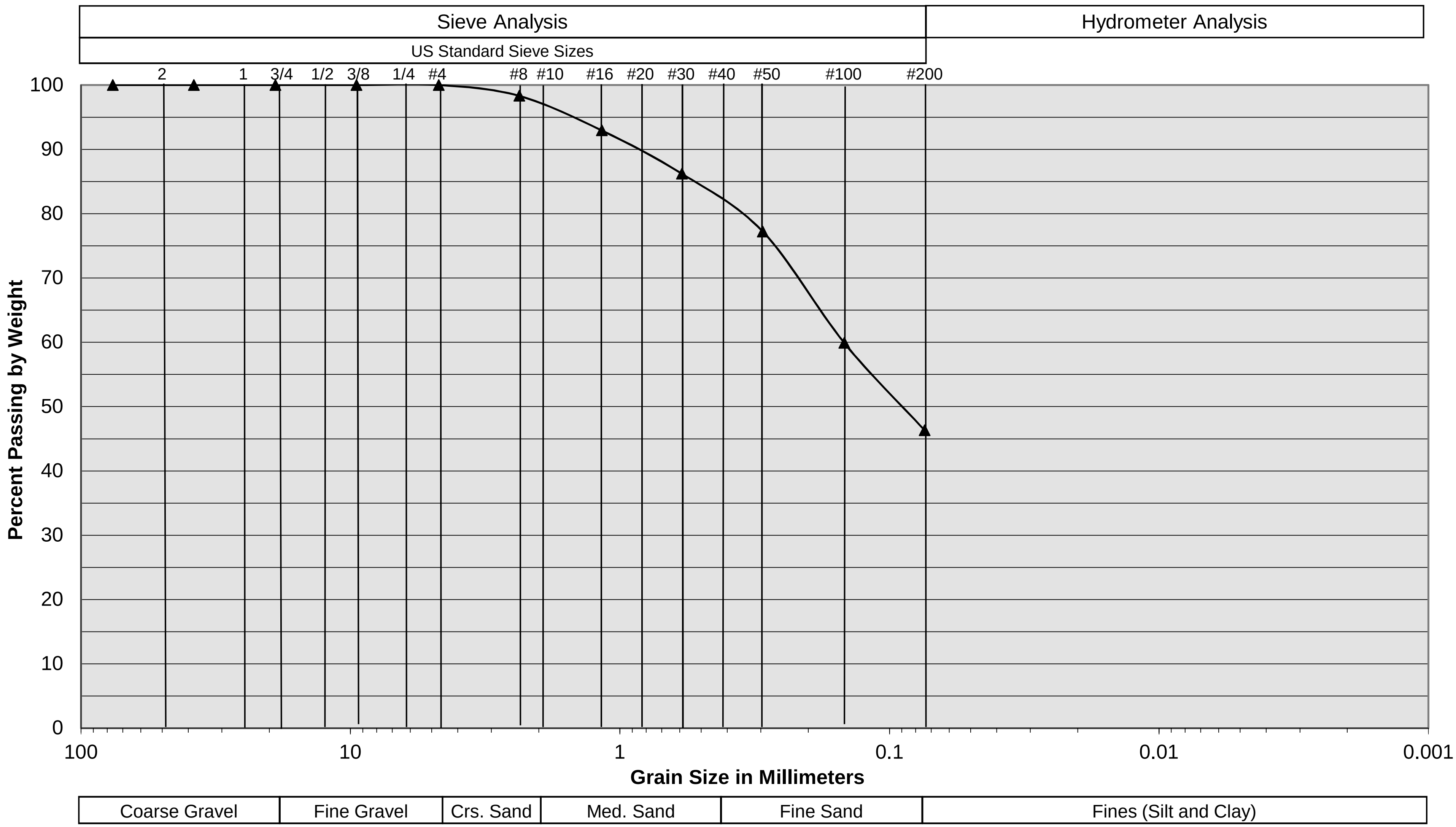
Infiltration Test No: I-3

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

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

Test Interval		Time (hr)	Interval **Elapsed** (min)	Flow Readings: Inner Ring (ml)	Ring Flow (cm^3)	Annular Ring (ml)	Space Flow (cm^3)	Infiltration Rates: Inner Ring* (cm/hr)	Annular Space* (cm/hr)	Inner Ring* (in/hr)	Annular Space* (in/hr)
1	Initial	11:00 AM	20	50	1500	200	5100	6.17	6.99	2.43	2.75
	Final	11:20 AM	**20**	1550		5300					
2	Initial	11:21 AM	20	0	1100	0	4500	4.52	6.17	1.78	2.43
	Final	11:41 AM	**41**	1100		4500					
3	Initial	11:42 AM	20	150	975	600	4350	4.01	5.96	1.58	2.35
	Final	12:02 PM	**62**	1125		4950					
4	Initial	12:03 PM	20	50	925	900	4200	3.80	5.76	1.50	2.27
	Final	12:23 PM	**83**	975		5100					
5	Initial	12:24 PM	20	50	900	200	4100	3.70	5.62	1.46	2.21
	Final	12:44 PM	**104**	950		4300					
6	Initial	12:45 PM	20	200	850	500	4100	3.49	5.62	1.38	2.21
	Final	1:05 PM	**125**	1050		4600					
7	Initial	1:06 PM	20	100	850	400	4100	3.49	5.62	1.38	2.21
	Final	1:26 PM	**145**	950		4500					
8	Initial	1:27 PM	20	150	850	100	4100	3.49	5.62	1.38	2.21
	Final	1:47 PM	**166**	1000		4200					

Grain Size Distribution



Sample Description	I-1 @ 12.5 feet
Soil Classification	Brown Clayey fine to medium Sand

Rider 4 - Proposed Commercial/Industrial Building
Perris, California
Project No. 17G206-2
PLATE C-1

SoCalGeo
SOUTHERN
CALIFORNIA
GEOTECHNICAL
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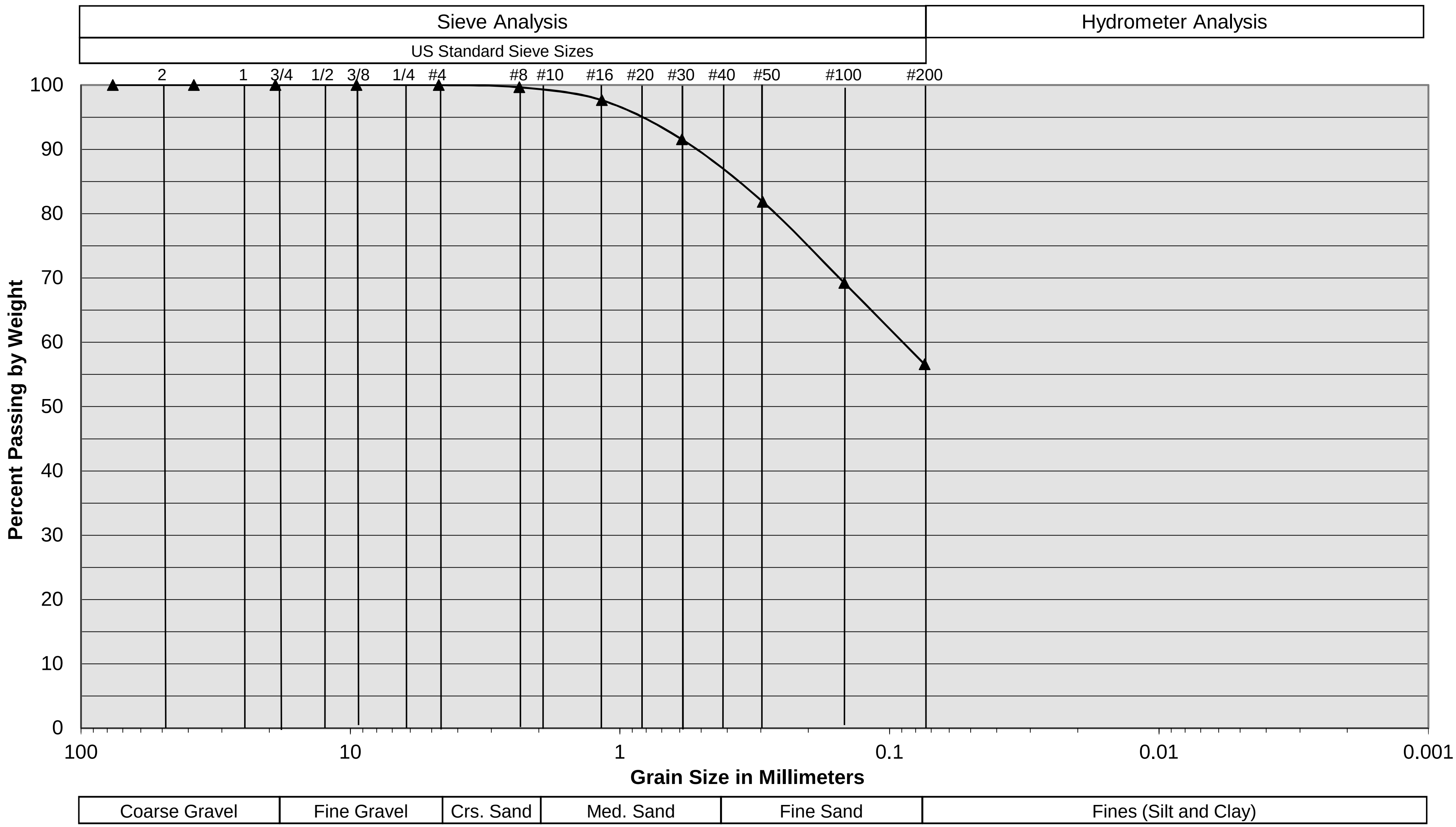
Grain Size Distribution



Sample Description	I-2 @ 13 feet
Soil Classification	Gray Brown fine to medium Sandy Clay

Rider 4 - Proposed Commercial/Industrial Building
Perris, California
Project No. 17G206-2
PLATE C-2

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
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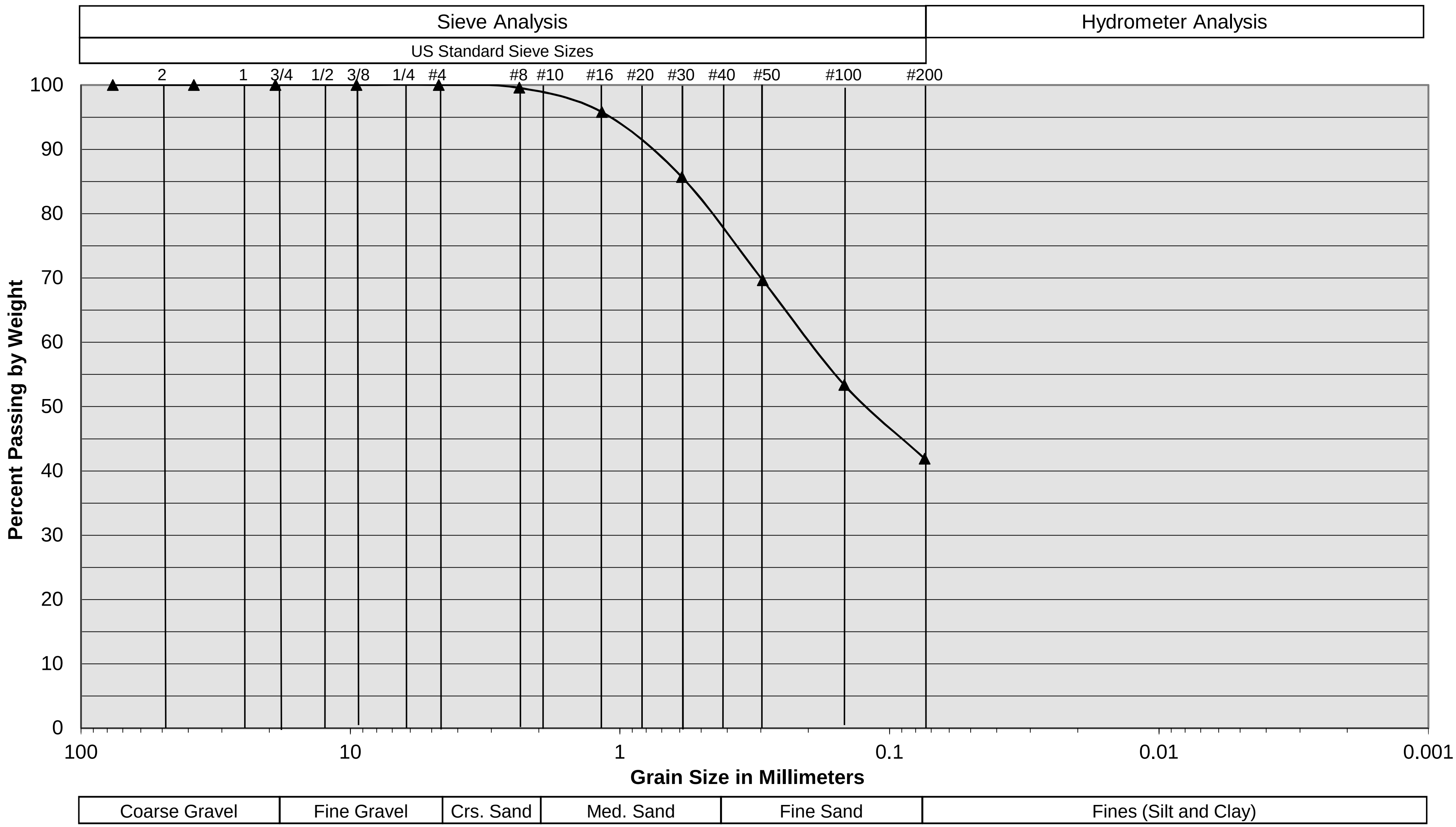
Grain Size Distribution



Sample Description	I-3 @ 13 feet
Soil Classification	Brown Clayey fine to medium Sand, trace Silt

Rider 4 - Proposed Commercial/Industrial Building
Perris, California
Project No. 17G206-2
PLATE C-3

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Appendix 4: Historical Site Conditions

Phase I Environmental Site Assessment or Other Information on Past Site Use

N/A

Appendix 5: LID Infeasibility

LID Technical Infeasibility Analysis

N/A

Appendix 6: BMP Design Details

BMP Sizing, Design Details and other Supporting Documentation

<u>**Santa Ana Watershed**</u> - BMP Design Volume, V_{BMP}

(Rev. 10-2011)

Legend: Required Entries / Calculated Cells

*(Note this worksheet shall **only** be used in conjunction with BMP designs from the **LID BMP Design Handbook**)*

Company Name: Albert A. Webb Associates — Date: 9/19/2019
Designed by: TSW — Case No: 19-00006
Company Project Number/Name: Rider IV

BMP Identification

BMP NAME / ID: BMP-A

Must match Name/ID used on BMP Design Calculation Sheet

Design Rainfall Depth

85th Percentile, 24-hour Rainfall Depth,
from the Isohyetal Map in Handbook Appendix E — D_{85}= 0.63 inches

Drainage Management Area Tabulation

Insert additional rows if needed to accommodate all DMAs draining to the BMP

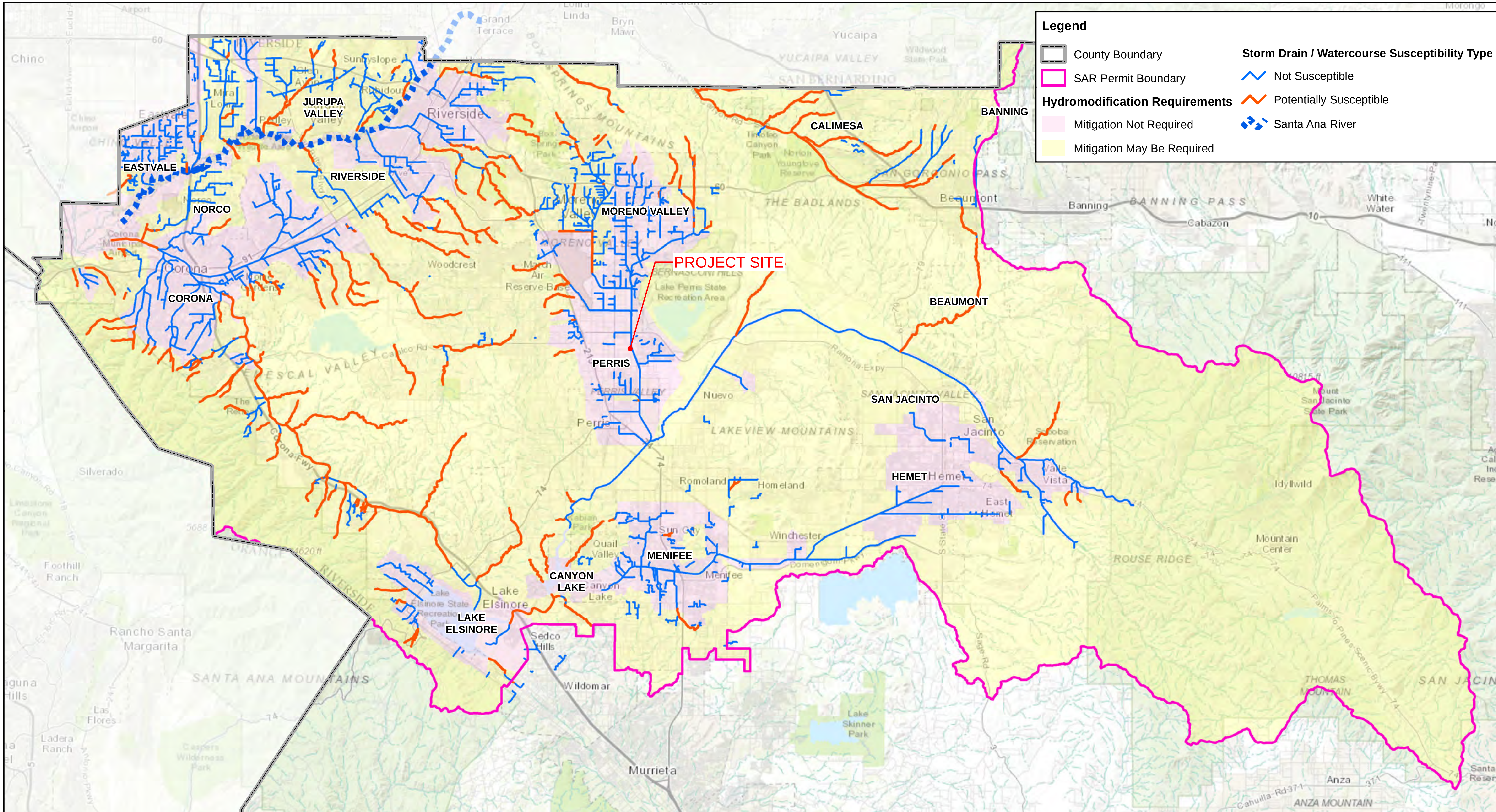
DMA Type/ID	DMA Area (square feet)	Post-Project Surface Type	Effective Imperivous Fraction, I_f	DMA Runoff Factor	DMA Areas x Runoff Factor	*Design Storm Depth (in)*	*Design Capture Volume, V_{BMP} (cubic feet)*	*Proposed Volume on Plans (cubic feet)*
L-A	*20506*	*Ornamental Landscaping*	*0.1*	*0.11*	*2265.1*			
R-A	*567098*	*Roofs*	*1*	*0.89*	*505851.4*			
H-A	*451119*	*Concrete or Asphalt*	*1*	*0.89*	*402398.1*			
BMP-A	*4800*	*Ornamental Landscaping*	*0.1*	*0.11*	*530.2*			
SR-A	*105229*	*Ornamental Landscaping*						
	1148752	***Total***			**911044.8**	**0.63**	***47829.9***	***47850***

Notes:

Appendix 7: Hydromodification

Supporting Detail Relating to Hydrologic Conditions of Concern

N/A



N

0 2.5 5 10 Miles

Updated February 2017

HCOC Applicability Map
SAR Permittees

Map 2

Appendix 8: Source Control

Pollutant Sources/Source Control Checklist

***To be provided during final engineering**

Appendix 9: O&M

Operation and Maintenance Plan and Documentation of Finance, Maintenance and Recording Mechanisms

***To be provided during final engineering**

Appendix 10: Educational Materials

BMP Fact Sheets, Maintenance Guidelines and Other End-User BMP Information

***To be provided during final engineering**