Appendix F

Preliminary Project Specific Water Quality Management Plan
Lake Creek Harley-Knox

SDH & Associates

February 2022

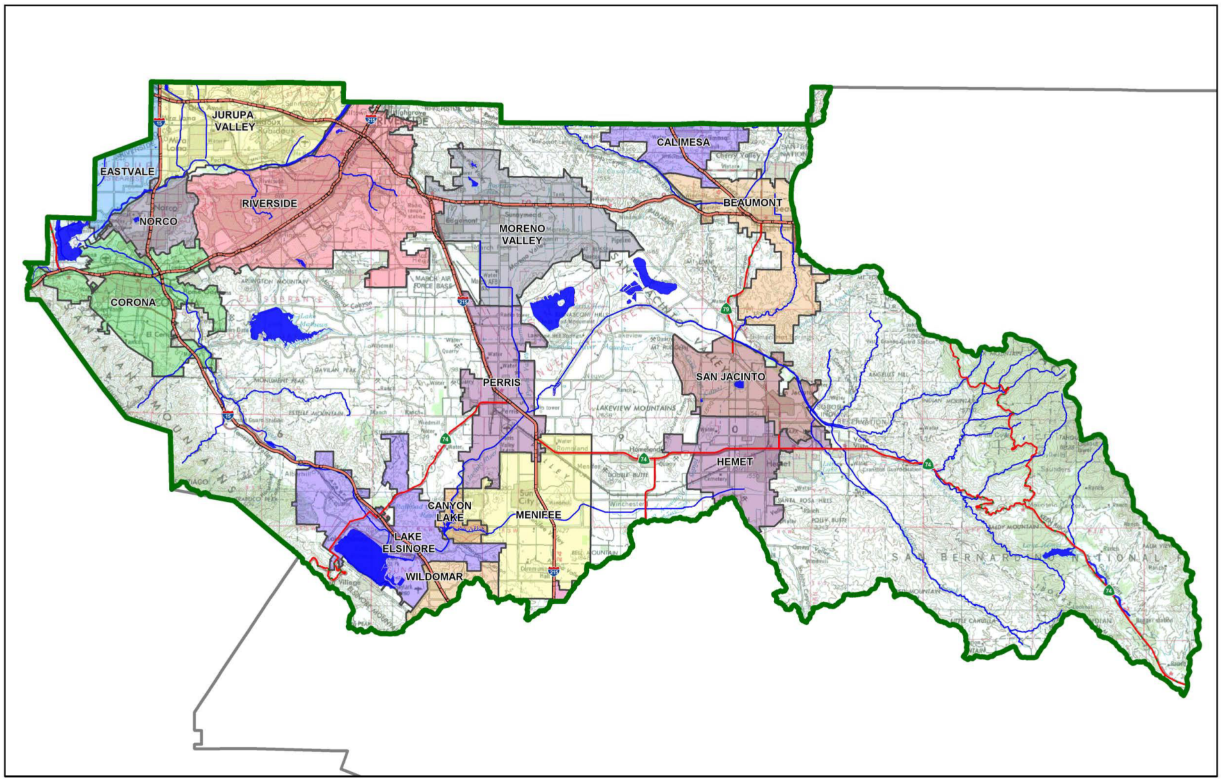
Project Specific Water Quality Management Plan

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

Project Title: Lake Creek-Harley Knox

Development No: TBD

Design Review/Case No: P21-00008



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☒ **Preliminary**
☐ **Final**

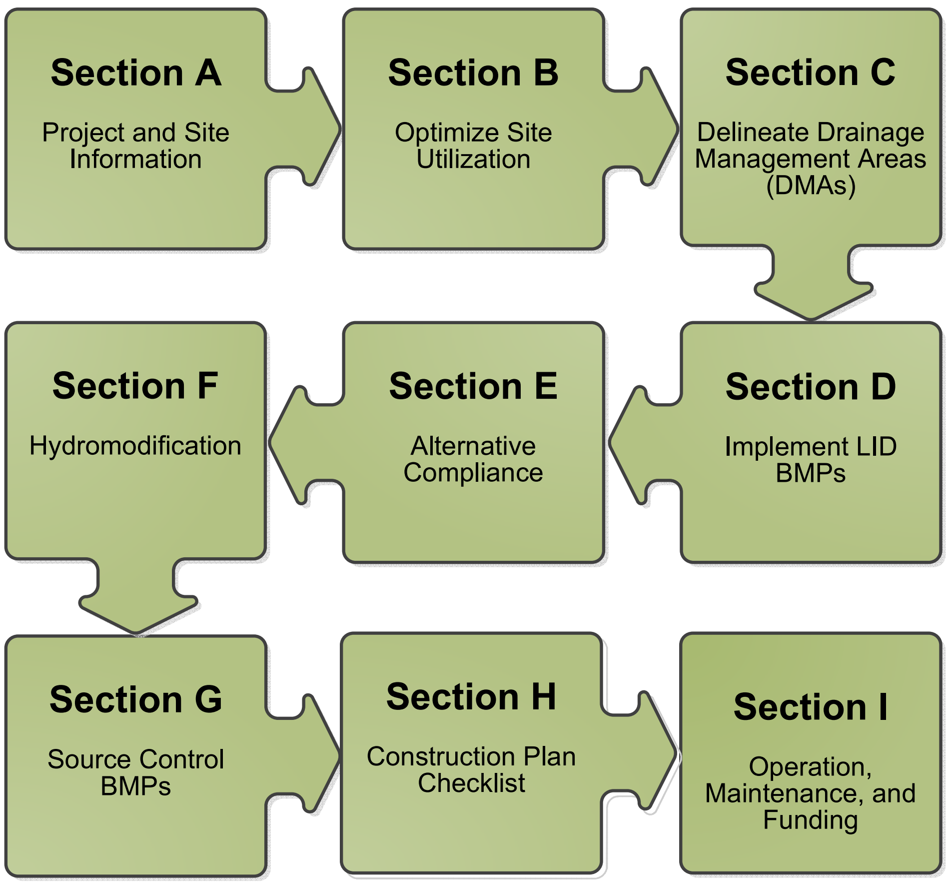
Original Date Prepared: June 9, 2021

Revision Date(s): February 18, 2022

Prepared for Compliance with
Regional Board Order No. **R8-2010-0033**
Template revised June 30, 2016

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 Lake Creek Industrial, LLC for the Lake Creek-Harley Knox project (City Case No. P21-00008), located at the northwest corner of the intersection of Harley Knox Blvd. and Las Palmas in the City of Perris, California.

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 the City of Perris Water Quality Ordinance 1194.

"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

02/18/2022
Date

Michael Johnson
Owner's Printed Name

Sole Member and Principal
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

02/18/2022
Date

Nobu Murakami
Preparer's Printed Name

Water Resources Engineer
Preparer's Title/Position

Preparer's Licensure:

REGISTERED PROFESSIONAL ENGINEER
NOBUYA MURAKAMI
NO. 78149
CIVIL
STATE OF CALIFORNIA

See Attached Certificate

FEB 18 2022

[✓] Acknowledgment
[ ] Jurat
[ ] Copy Certificate

CALIFORNIA ALL-PURPOSE ACKNOWLEDGEMENT

A Notary Public or other officer completing this certificate verifies only the identity of the individual who signed the document to which this certificate is attached, and not the truthfulness, accuracy, or validity of that document.

State of California }

County of Orange

On Feb 18th, 2022 before me Shannon Brown, Notary Public, Notary Public, personally appeared

Michael Johnson

who proved to me on the basis of satisfactory evidence to be the person(s) whose name(s) is/are subscribed to the within instrument and acknowledged to me that he/she/they executed the same in his/her/their authorized capacity(ies), and that by his/her/their signature(s) on the instrument the person(s), or the entity upon behalf of which the person(s) acted, executed the instrument.

I certify under PENALTY OF PERJURY under the laws of State of California that the foregoing paragraph is true and correct.

WITNESS my hand and official seal.

SHANNON BROWN
COMM...2375517
NOTARY PUBLIC-CALIFORNIA
ORANGE COUNTY
My Term Exp. September 18, 2025
BCT3

SIGNATURE Shannon Brown

PLACE NOTARY SEAL ABOVE

Though the information below is not required by law, it may prove valuable to persons relying on the document and could prevent fraudulent removal and reattachment of this form to another document.

Description of attached document

Title or type of document: Owner's Certification

Table of Contents

List of Tables

List of Appendices

Section A: Project and Site Information

PROJECT INFORMATION	
Type of Project:	Industrial
Planning Area:	Perris Valley Commerce Center (PVCC) Specific Plan Area
Community Name:	Perris Valley
Development Name:	Lake Creek-Harley Knox
PROJECT LOCATION	
Latitude & Longitude (DMS): 33°51'29.06"N, 117°13'22.57"W	
Project Watershed and Sub-Watershed: Santa Ana (Watershed) Perris Reservoir (Sub Watershed)	
Gross Acres: ~7.9 acres (parcel); ~7.1 acres for overall project-specific drainage management area	
APN(s): 302-100-002	
Map Book and Page No.: Book 14, Page 668	
PROJECT CHARACTERISTICS	
Proposed or Potential Land Use(s)	Light Industrial
Proposed or Potential SIC Code(s)	1541
Area of Impervious Project Footprint (SF)	253,165 SF
Total Area of proposed Impervious Surfaces within the Project Footprint (SF)/or Replacement	253,165 SF
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 Footprint (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)	See Appendix 3 – NRCS Soil Types B, D, & A/D
What is the Water Quality Design Storm Depth for the project?	0.64 inch

Lake Creek Industrial, LLC is proposing to develop an industrial tilt-up warehouse building and associated parking as part of this project, which is located at the northwest corner of the intersection of Harley Knox Boulevard and Las Palmas in the City of Perris (within Riverside County), California. A vicinity map is provided in Appendix 1 of this report for reference purpose. Applicable Assessor Parcel Numbers (APNs) are 302-030-002 and 302-100-007. The site is approximately 7.9 acres (parcel gross area) with approximately 7.1 acres of drainage management area. The proposed warehouse building footprint is approximately 143,168 square feet and there will be a total of 88 parking spaces to be provided. The proposed impervious and pervious footprints within the drainage management area are approximately 253,165 square feet and 57,522 square feet, respectively. The project also includes minor improvement for the easterly frontage Las Palmas.

In the existing condition, the site consists of open, undeveloped space, draining generally from north to south. There are minor offsite run-on flows to the site from the westerly undeveloped land and a small portion of northerly undeveloped area. Runoff from the project generally drains in a southeasterly direction in a sheet flow manner towards Harley Knox Blvd. Runoff is captured by an inlet along Harley Knox Blvd. and drains into an existing storm drain pipe (i.e. – 24-inch RCP) that eventually connects with the Riverside County Flood Control District's storm drain Line D-3 in Redlands Avenue. For reference purpose, the existing 24-inch RCP was

constructed per the Storm Drain Plans Lateral D-3A (DPR No. 11-12-0004, City File No. P8-1189) and it has a capacity of ~6.5 cfs. Runoff eventually discharges into the existing District's Perris Valley Storm Drain Channel that ultimately discharges to Canyon Lake and then Lake Elsinore.

In the post-project condition, the drainage characteristics will remain similar as compared to the pre-project condition. Regarding the minor run-on mentioned above, the project proposes a swale on the westerly edge of the project to direct the minor westerly run-on towards Harley Knox Blvd.; therefore, there will be no run-on to the project from the westerly offsite area. The minor northerly offsite area will be captured and conveyed to the proposed BMP for treatment. Runoff from the site will be captured via proposed catch basins and conveyed via proposed storm drain pipes towards a proposed underground storage facility (StormTrap – 4'2" SingleTrap) along the southerly edge of the project for the purpose of attenuating the larger peak flow back to the existing condition, prior to connecting into the existing storm drain located along Harley Knox Blvd. The proposed StormTrap facility is a hard-bottom closed system. Select on-site catch basins will have pre-treatment BMPs (i.e. – connector pipe screen) in an effort to help minimize trash/debris into the proposed underground storage facility. Immediately downstream of the underground storage facility will include a proprietary Modular Wetland System (MWS) ("BMP 1"), sized per volume-based approach, to treat the on-site runoff prior to connecting into the outlet location. The overflow from the underground storage facility will be bypassed via a separate storm drain outlet pipe. Additionally, the proposed landscape areas in the southeasterly area of the site will also provide pre-treatment in the form of vegetated swales prior to discharging flows at the proposed atrium grates. This train of storm water management features, including the pre-treatment and structural BMPs, should help address the storm water quality management requirements for the project. Where applicable, runoff from the proposed hardscape area will be directed towards landscape area in an effort to promote incidental infiltration and preserve the infiltration capacity. Additionally, roof runoff through downspouts will be directed to proposed landscape areas where feasible to help slow down the storm water runoff.

In support of the infiltration feasibility for the proposed permanent storm water BMP, the project-specific geotechnical engineer conducted infiltration testing and results showed field infiltration rates of 2.4 in/hr and 6.5 in/hr. These rates are above the infiltration threshold of 1.6 in/hr; however, these infiltration rates were obtained near the northerly portion of the site, which is in the vicinity of the existing Riverside County Flood Control Master Drainage Plan Channel and have a tendency to have slightly more permeable soils. As mentioned above, the existing site generally wants to drain in a southeasterly direction towards Harley Knox Blvd. To be consistent with the existing drainage characteristics, the most suitable location for a proposed BMP would be in the southeasterly area of the site. Based on the boring logs near the southerly edge of the project (i.e. – Boring ID's B-3 and B-4), clayey materials (including clayey fine sand, clayey silt, silty clay) were observed approximately 5 feet below existing surface and deeper. The clayey materials are not generally conducive to infiltration and our understanding is that infiltration is not practicable in most areas within the City of Perris based on the nature of existing soils in the area. Additionally, the geotechnical engineer recommends setback of 25 feet from any structures and retaining walls for infiltration facilities. The project proposes some landscape areas in the southeasterly area of the project but it would not be practicable with the aforementioned constraints. Therefore, infiltration BMPs were not recommended for the site.

A.1 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
- BMP Locations (Lat/Long)

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.2 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	N/A	N/A	San Jacinto River Rach 3 (downstream).
San Jacinto River Reach 3 – Canyon Lake to Nuevo Road (HU#802.11)	None	MUN, AGR, GWR, REC1, REC2, WARM, WILD, RARE	This river reach has existing or potential RARE beneficial use.
Canyon Lake (HU#802.11, 802.12)	Nutrients, Pathogens **TMDL Completed** - Nutrients	MUN, AGR, GWR, REC1, REC2, COMM, WARM, WILD	San Jacinto River Reaches 1 (downstream).
San Jacinto River Rach 1 (HU#802.32, 802.31)	None	MUN, AGR, GWR, REC1, REC2, WARM, WILD, RARE	This river reach has existing or potential RARE beneficial use.
Lake Elsinore (HU#802.31)	Nutrients, Organic Enrichment/Low Dissolved Oxygen, PCBs, Toxicity **TMDL Completed** – Nutrients, Organic Enrichment/Low Dissolved Oxygen	MUN, REC1, REC2, COMM, WARM, WILD, RARE	The lake has existing or potential RARE beneficial use.

Note: Based on the direction from the City, the 2012 impairment listing is referenced.

A.3 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 (dependent on tenant)	☒ Y	☐ N
Western Riverside MSHCP Consistency Approval (e.g., JPR, DBESP)	☐ Y	☒ N
Other *(please list in the space below as required)* City of Perris – Grading Permit & Building 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.

Consideration of "highest and best use" of the discharge should also be considered. For example, Lake Elsinore is evaporating faster than runoff from natural precipitation can recharge it. Requiring infiltration of 85% of runoff events for projects tributary to Lake Elsinore would only exacerbate current water quality problems associated with Pollutant concentration due to lake water evaporation. In cases where rainfall events have low potential to recharge Lake Elsinore (i.e. no hydraulic connection between groundwater to Lake Elsinore, or other factors), requiring infiltration of Urban Runoff from projects is counterproductive to the overall watershed goals. Project proponents, in these cases, would be allowed to discharge Urban Runoff, provided they used equally effective filtration-based BMPs.

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 existing site drains in a southeasterly direction towards Harley Knox Blvd. and the drainage pattern will be maintained in the post-project condition.

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

The site has little or no existing vegetation as it has been graded and consistently cleared over many years.

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

Where applicable, runoff from the proposed hardscape area will be directed towards landscape area in an effort to promote incidental infiltration and preserve the infiltration capacity. Additionally, roof runoff through downspouts will be directed to proposed landscape areas where feasible to help slow

down the storm water runoff. The project-specific geotechnical engineer conducted infiltration testing and results showed field infiltration rates of 2.4 in/hr and 6.5 in/hr. These rates are above the infiltration threshold of 1.6 in/hr; however, these infiltration rates were obtained near the northerly portion of the site, which is in the vicinity of the existing Riverside County Flood Control Master Drainage Plan Channel and have a tendency to have slightly more permeable soils. As mentioned above, the existing site generally wants to drain in a southeasterly direction towards Harley Knox Blvd. To be consistent with the existing drainage characteristics, the most suitable location for a proposed BMP would be in the southeasterly area of the site. Based on the boring logs near the southerly edge of the project (i.e. – Boring ID's B-3 and B-4), clayey materials (including clayey fine sand, clayey silt, silty clay) were observed approximately 5 feet below existing surface and deeper. The clayey materials are not generally conducive to infiltration and our understanding is that infiltration is not practicable in most areas within the City of Perris based on the nature of existing soils in the area. Additionally, the geotechnical engineer recommends setback of 25 feet from any structures and retaining walls for infiltration facilities. The project proposes some landscape areas in the southeasterly area of the project but it would not be practicable with the aforementioned constraints. Therefore, infiltration BMPs were not recommended for the site.

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

Impervious areas are only used where necessary and have been minimized to the extent practicable. Parking spaces are minimized close to the required amount and the landscaped areas have been maximized to the extent practicable.

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

Runoff from impervious surfaces is directed to the pervious areas where possible prior to being directed to the proposed structural BMP for water quality treatment.

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)[1,2]	Area (Sq. Ft.)	DMA Type
DMA 1-1	Ornamental Landscaping	36,586	Type D
DMA 1-2	Concrete or Asphalt	114,227	Type D
DMA 1-3	Roofs	138,938	Type D
DMA 1-4	D.G.	17,320	Type D

[1]*Reference Table 2-1 in the WQMP Guidance Document to populate this column*
[2]*If multi-surface provide back-up*

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

DMA Name or ID	Area (Sq. Ft.)	Stabilization Type	Irrigation Type (if any)
DMA 1-1	36,586	Landscaping	Drip
DMA 1-5 (Self-Treating)	3,616	Landscaping	N/A

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]
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	Impervious fraction [B]	Product [C] = [A] x [B]	DMA name /ID	Area (square feet) [D]	Ratio [C]/[D]
N/A							

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

DMA Name or ID	BMP Name or ID
DMA 1-1	StormTrap (4'2" SingleTrap) / BMP 1-Modular Wetland System (MWS-8-12-5'-0"-V-UG)
DMA 1-2	StormTrap (4'2" SingleTrap) / BMP 1-Modular Wetland System (MWS-8-12-5'-0"-V-UG)
DMA 1-3	StormTrap (4'2" SingleTrap) / BMP 1-Modular Wetland System (MWS-8-12-5'-0"-V-UG)
DMA 1-4	StormTrap (4'2" SingleTrap) / BMP 1-Modular Wetland System (MWS-8-12-5'-0"-V-UG)

Note: 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; proceed to section D.3

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?		✓
If Yes, list affected DMAs:		
…have any DMAs located within 100 feet of a water supply well?		✓
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?		✓
If Yes, list affected DMAs:		
…have measured in-situ infiltration rates of less than 1.6 inches / hour?		✓
If Yes, list affected DMAs:		
…have significant cut and/or fill conditions that would preclude in-situ testing of infiltration rates at the final infiltration surface?		✓
If Yes, list affected DMAs:		
…geotechnical report identify other site-specific factors that would preclude effective and safe infiltration?	✓	
Describe here: Clayey materials observed approximately 5' below existing grade and below and 25' setback would be needed from structures and retaining walls for infiltration facilities.		

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 none 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 2-2 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 2-4 in Chapter 2 to determine the minimum demand for non-potable uses per tributary impervious acre.

Enter the factor from Table 2-4: Enter Value

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 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 projected average daily use (Step 1) to the minimum required non-potable use (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 per Section 3.4.2 of the WQMP Guidance Document.

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.

Note: The proposed site will be treated via a proprietary Modular Wetland System (MWS), which is to be located immediately downstream of the proposed underground storage facility (for detention purpose). Additional discussion is provided in Section D.4 below.

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 1-1	☐	☐	☐	☐	☒
DMA 1-2	☐	☐	☐	☐	☒
DMA 1-3	☐	☐	☐	☐	☒
DMA 1-4	☐	☐	☐	☐	☒

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.

Note: As indicated above, bioretention and biotreatment options were explored in the southeasterly area of the site within the proposed landscape areas. However, by the time the facility accounts for the 4:1 side slopes based on the County of Riverside LID Manual, it would need to take up the provided landscape in the southeasterly area in order to meet the minimum required footprint and there would not be enough setback from the proposed building footing (creating geotechnical concern). Therefore, the proposed site will be treated via a proprietary Modular Wetland System (MWS), which is to be located immediately downstream of the proposed underground storage facility (for detention purpose). The proposed underground storage facility is to store the minimum required design capture volume and slowly release it within acceptable drawdown time (i.e. – within 48 hours) to the proposed MWS for treatment.

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	***StormTrap (4'2" SingleTrap) & BMP 1 / Modular Wetland System (MWS-8-12-5'0"-V-UG) – Volume-based***		
	[A]		[B]	[C]	[A] x [C]	*Design Storm Depth (in)*	*Design Capture Volume,* **V_{BMP}** *(cubic feet)*	*Proposed Volume on Plans (cubic feet)*
DMA 1-1	*36,586*	*Ornamental Landscaping*	*0.1*	*0.11*	*4041.2*			
DMA 1-2	*114,227*	*Concrete or Asphalt*	*1.0*	*0.89*	*101890.5*			
DMA 1-3	*138,933*	*Roofs*	*1.0*	*0.89*	*123928.2*			
DMA 1-4	*17,324*	*Natural (B Soil)*	*0.15*	*0.14*	*2450.4*			
	A_T = Σ[A] = 307,070				Σ= [D] = 232310.3	[E] = 0.64	[F] = $\frac{[D]x[E]}{12}$ = 12389.9	[G] = 25,020

[B], [C] is obtained as described in Section 2.3.1 of the WQMP Guidance Document.
[E] is obtained from Section 2.3.1 in the WQMP Guidance Document.
[G] is obtained from the proprietary BMP manufacturer (i.e. –StormTrap - SingleTrap).

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.

Note: DMA 1 will be treated via proposed proprietary Modular Wetland Systems (MWS), which is to be located downstream of an underground storage facility (i.e. – StormTrap – SingleTrap).

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	*BMP 1 / Proprietary Modular Wetland System (MWS)*			
	[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)*
DMA 1-1	*36,586*	*Ornamental Landscaping*	*0.1*	*0.11*	*4041.2*				
DMA 1-2	*114,227*	*Concrete or Asphalt*	*1.0*	*0.892*	*101890.5*				
DMA 1-3	*138,933*	*Roofs*	*1.0*	*0.892*	*123928.2*				
DMA 1-4	*17,324*	*Natural B Soil*	*0.15*	*0.141*	*2450.4*				
	$A_T = \Sigma[A]$ 307,070				Σ= [D] 232310.3	[E] 0.20	$[F] = \frac{[D]x[E]}{[G]}$ 1.1	[F] X (1-[H]) N/A	[I] 1.1

[B], [C] is obtained as described in Section 2.3.1 from the WQMP Guidance Document

[E] is for Flow-Based Treatment Control BMPs [E] = .2, for Volume-Based Control Treatment BMPs, [E] 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. It is important to note that this Modular Wetland System was sized using the volume-based approach by storing the minimum required design capture volume in a proposed underground storage facility (i.e. – StormTrap – SingleTrap) located upstream of the MWS.

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]
Modular Wetland System (BMP 1)	Metals, Nutrients, Pesticides, Toxic Organic Compounds, Sediments, Trash & Debris, and Oil & Grease	Metal (Medium), Nutrients/Pesticides (Medium), Toxic Organic Compounds (Medium), Sediments (High), Trash & Debris (High), Oil & Grease (High)

[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 Susceptibility 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.

<u>**Note: The project is within the Riverside County WAP HCOC Exemption area approved on April 20, 2017.**</u>

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
On-site storm drain inlets	Mark all inlets with the words "Only Rain Down the Storm Drain" or similar. Catch Basin Markers may be available from the Riverside County Flood Control and Water 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. 3See applicable operational BMPs in Fact Sheet SC-44, "Drainage System Maintenance," in the CASQA Stormwater Quality Handbooks at www.cabmphandbooks.com Include the following in lease agreements: "Tenant 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 drain.”
Interior floor drains	Interior floor drains shall be plumbed to sanitary sewer.	Inspect and maintain drains to prevent blockages and overflow.
Need for future indoor & structural pest control	Building design features including sealants barriers and fully closing windows and doors have been included to discourage entry of pests.	Integrated Pest Management (IPM) information to be provided to owners, lessees, and operators.
Landscape/outdoor pesticide use	Final Landscape Plans will accomplish the following: Preserve existing native trees, shrubs, and ground cover to the maximum extent possible. Design landscaping 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 establishment, select plants appropriate to site soils, slopes, climate, sun, wind, rain, land use, air movement, ecological consistency, and plant interactions.	Maintain landscaping using minimum or no pesticides. Prevent erosion of slopes by planting fast-growing, dense ground covering plants. Plant native vegetation to reduce the amount of water, fertilizers, and pesticides applied to the landscape. Do not overwater. Use irrigation practices such as drip irrigation, soaker hoses or micro-spray systems. Periodically inspect and fix leaks and misdirected sprinklers. Do not rake or blow leaves, clippings, or pruning waste into the street, gutter, or storm drain. Instead, dispose of green waste by composting, hauling it to a permitted landfill, or recycling it through your city’s program. Integrated Pest Management (IPM) information to be provided to owners, lessees, and operators.
Refuse areas	Site design features dumpster enclosures. Signs will be posted on or near dumpsters with the words “Do not dump hazardous materials here” or similar.	Periodic inspections for leaky, overfilled, uncovered, or other problematic conditions will occur. Corrective action will be made upon detection, as circumstances permit. Dumping of liquid or hazardous wastes will be prohibited. Spill control materials will be available on-site. All wastes to properly stored and disposed of in accordance with all applicable Local, State and Federal regulations
Industrial Processes	All process activities to be performed indoors. No processes to drain to exterior or to storm drain system.	All process activities to be performed indoors. No processes to drain to exterior or to storm drain system. See Fact Sheet SC-10, “Non-Stormwater Discharges” in the CASQA Stormwater Quality Handbooks at www.cabmphandbooks.com See the brochure “Industrial & Commercial Facilities Best Management Practices for: Industrial, Commercial Facilities” at http://rcflood.org/stormwater/
Loading Docks	Maintain in a clean and orderly fashion. Loading dock areas draining directly to the sanitary sewer shall be equipped with a spill control valve or equivalent device, which shall be kept closed during periods of operation. Provide a roof overhang over the loading area or	Move loaded and unloaded items indoors as soon as possible. See Fact Sheet SC-30, “Outdoor Loading and Unloading,” in the CASQA Stormwater Quality Handbooks at www.cabmphandbooks.com

	install door skirts (cowling) at each bay that enclose the end of the trailer.	
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, "Building and Grounds Maintenance," in the CASQA Stormwater Quality Handbooks at www.cabmphandbooks.com
Miscellaneous Drain or Wash Water or 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. Rooftop equipment with potential to produce pollutants shall be roofed and/or have secondary. Any drainage sumps on-site shall feature a sediment sump to reduce pumped water. Roofing, gutters, and trim made out of unprotected metals that may leach into runoff shall be avoided.	Inspect periodically to verify that equipment is not leaking or discharging to the storm drain system.
Plazas, Sidewalks, and Parking Lots	Maintain in a clean and orderly fashion.	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 wash water 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)	BMP Location (Lat/Long)
BMP 1	BMP 1 / Modular Wetland System (MWS-8-12-5’0”-V-UG) (Note: to be located downstream of the underground storage facility – StormTrap.)	BMP Site Plan	33°49'49.32"N / 117°13'30.46"W

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.

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: See Appendix 9

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

☐ Y ☒ N

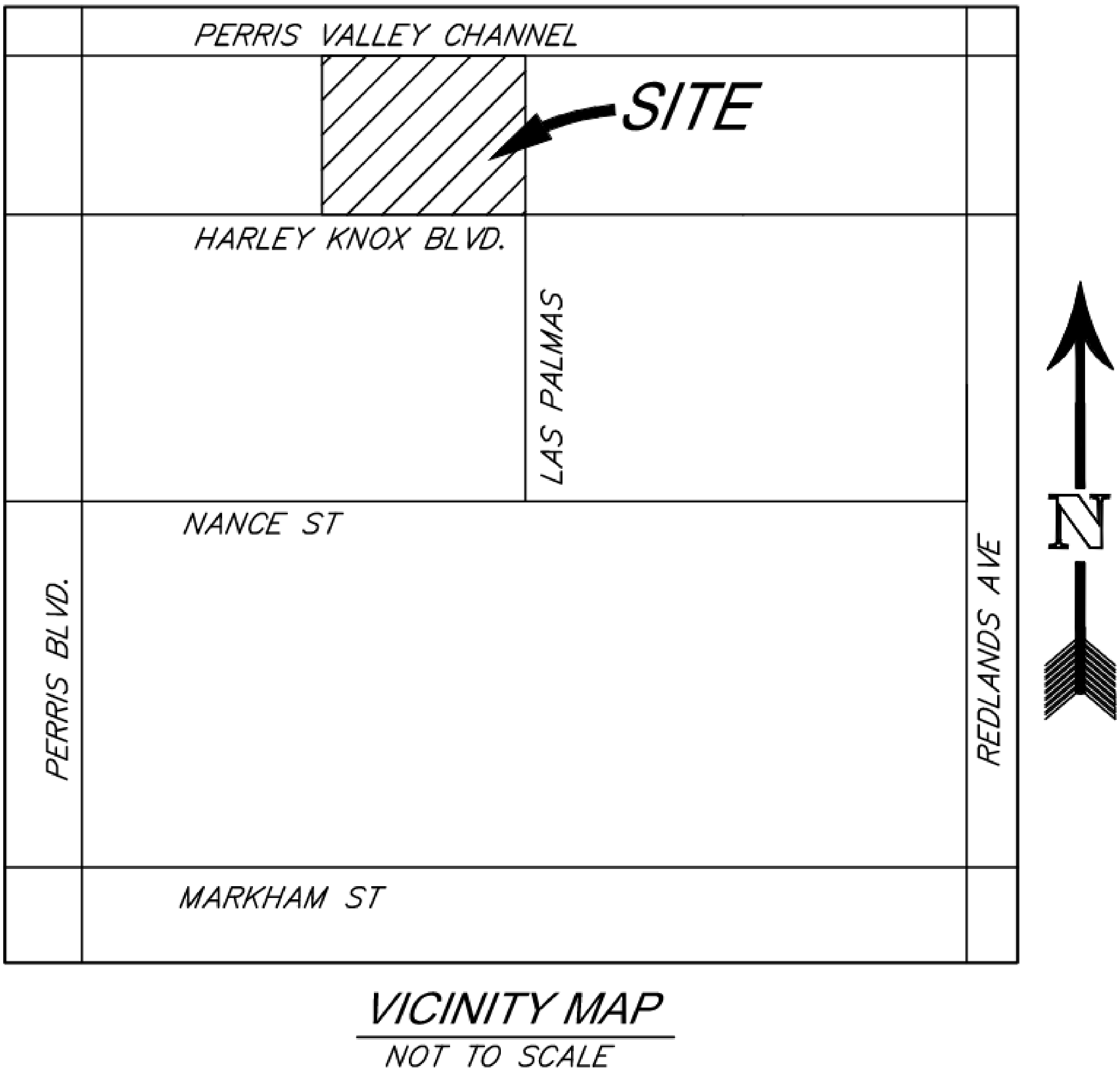
Include your Operation and Maintenance Plan and Maintenance Mechanism in Appendix 9. Additionally, include all pertinent forms of educational materials for those personnel that will be maintaining the proposed BMPs within this Project-Specific WQMP in Appendix 10.

<u>**Note: To be completed at the time of the FWQMP.**</u>

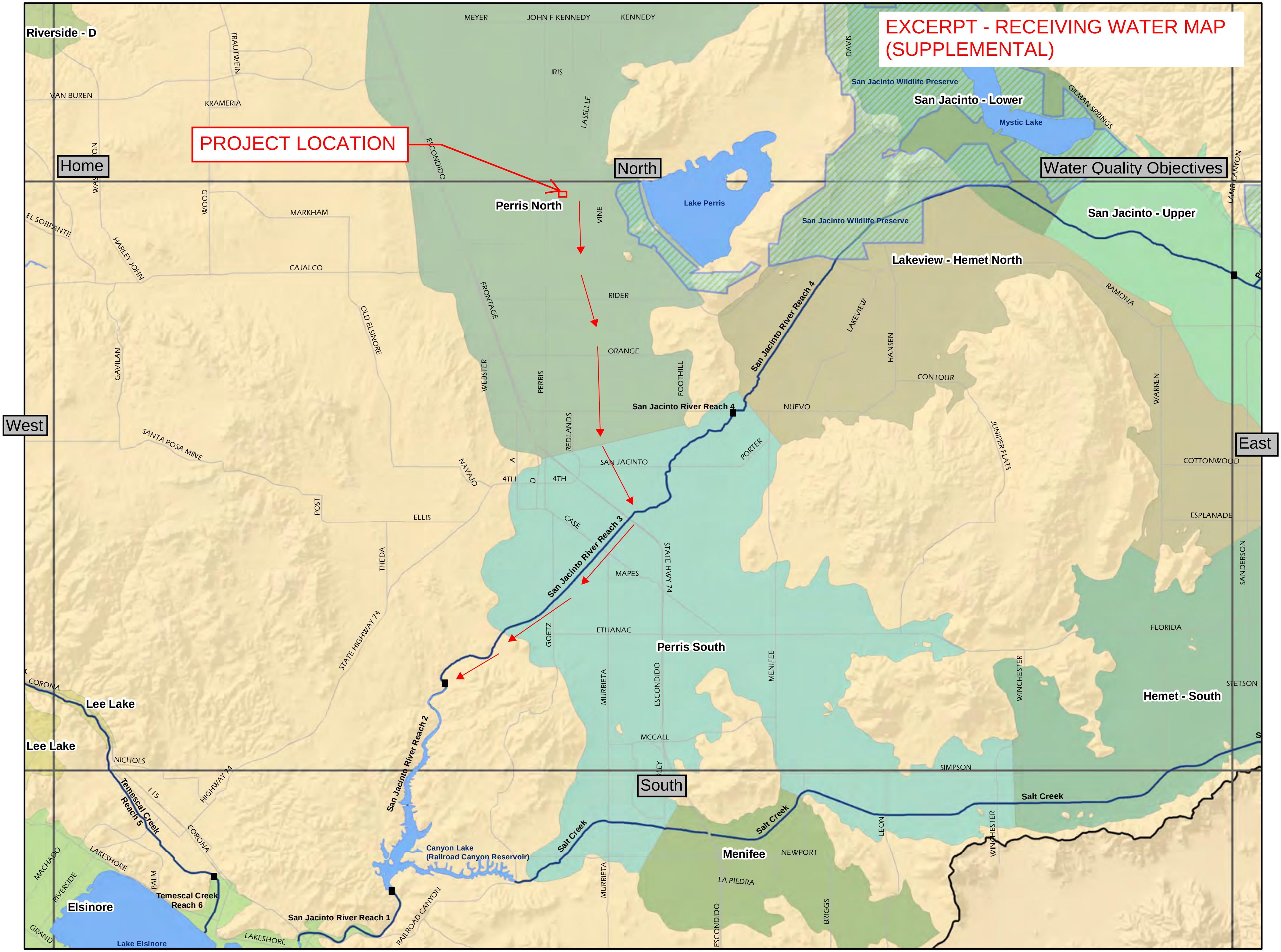
Appendix 1: Maps and Site Plans

Location Map, WQMP Site Plan and Receiving Waters Map

Vicinity Map



The project is located at the northwest corner of the intersection of Harley Knox Blvd. and Las Palmas in the City of Perris, CA.



EXCERPT - RECEIVING WATER MAP (SUPPLEMENTAL)
PROJECT LOCATION
Riverside - D
Home
North
Water Quality Objectives
West
East
South
Perris North
Perris South
Lake Perris
San Jacinto Wildlife Preserve
San Jacinto - Lower
Mystic Lake
San Jacinto - Upper
Lakeview - Hemet North
Hemet - South
San Jacinto River Reach 4
San Jacinto River Reach 3
San Jacinto River Reach 2
San Jacinto River Reach 1
Canyon Lake
(Railroad Canyon Reservoir)
Salt Creek
Menifee
Lee Lake
Elsinore
Lake Elsinore
Temescal Creek
Reach 5
Temescal Creek
Reach 6
MEYER
JOHN F KENNEDY
KENNEDY
IRIS
LASSELLE
ESCONDIDO
VAN BUREN
KRAMERIA
TRAUTWEIN
MARKHAM
WOOD
EL SOBRANTE
HARLEY JOHN
CAJALCO
VINE
RIDER
FRONTAGE
OLD ELSINORE
ORANGE
GAVILAN
WEBSTER
PERRIS
FOOTHILL
LAKEVIEW
HANSEN
CONTOUR
WARREN
RAMONA
GILMAN SPRINGS
DAVIS
LAMB CANYON
NUEVO
REDLANDS
PORTER
JUNIPER FLATS
COTTONWOOD
SANTA ROSA MINE
NAVAJO
SAN JACINTO
4TH
D
A
POST
ELLIS
CASE
ESPLANADE
THEDA
STATE HWY 74
MAPES
SANDERSON
STATE HIGHWAY 74
ETHANAC
GOETZ
FLORIDA
MENIFEE
ESCONDIDO
MURRIETA
WINCHESTER
STETSON
CORONA
NICHOLS
MCCALL
SIMPSON
HIGHWAY 74
I 15
CORONA
LAKESHORE
MACHADO
RIVERSIDE
PALM
LEON
WINCHESTER
NEWPORT
LA PIEDRA
BRIGGS
MURRIETA
ESCONDIDO
RAILROAD CANYON
GRAND
LAKESHORE

DMA PLAN
LAKE CREEK-HARLEY KNOX

DMA LEGEND & AREAS

DRAINAGE MANAGEMENT AREA

DISCHARGE LOCATION

DMA 1 DRAINING TO PERMANENT STRUCTURAL BMP

- DMA 1–1 (ORNAMENTAL LANDSCAPING) – 36,586 S.F.
- DMA 1–2 (CONCRETE OR ASPHALT) – 114,227 S.F.
- DMA 1–3 (ROOFS) – 138,938 S.F.
- DMA 1–4 (NATURAL B SOIL) – 17,320 S.F.

TOTAL AREA = 307,071 S.F.

MISC. DMAs

- DMA 1–5 SELF–TREATING AREA – 3,616 S.F.

PERMANENT STRUCTURAL BMP

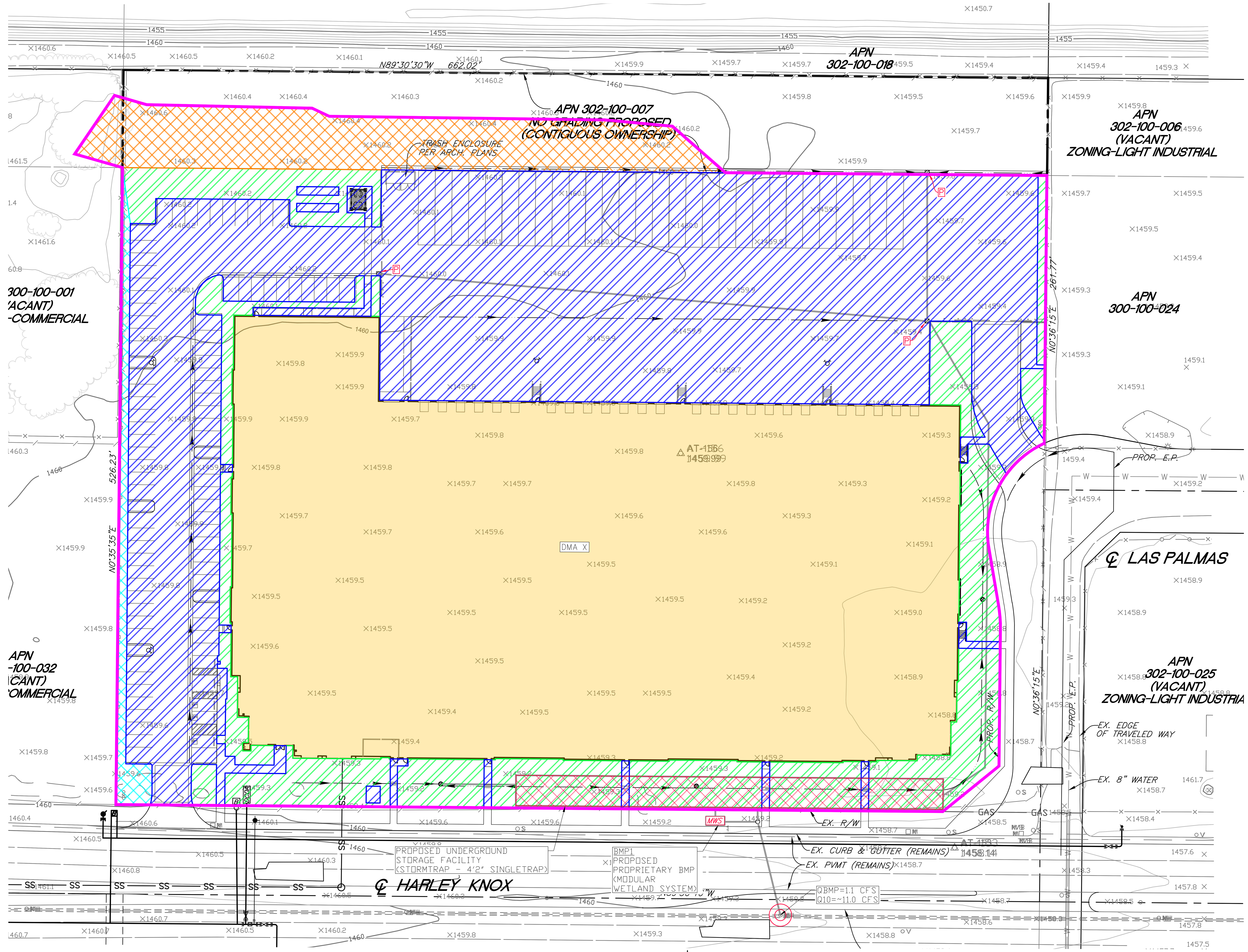
PROPOSED UNDERGROUND STORAGE FACILITY
(STORMTRAP – SINGLETRAP OR EQUALLY ACCEPTABLE PRODUCT)
- OVERALL PROVIDED VOLUME: ~25,020 C.F.
- FOOTPRINT PROVIDED: ~6,005 S.F. (MIN.)
- EFFECTIVE DEPTH: ~4'2"

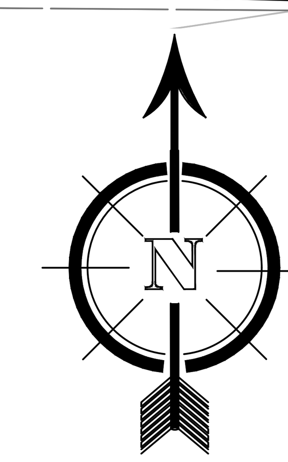
MWS — PROPOSED MODULAR WETLAND SYSTEM (MWS)
- BMP 1 (MWS–L–8–12–5'–0"–V–UG)

PRE-TREATMENT BMP

PROPOSED CONNECTOR PIPE SCREEN (CPS)
- PRE–TREATMENT DEVICE AT SELECT PROPOSED CATCH BASIN

PROPOSED VEGETATED SWALE
- PRE–TREATMENT PRIOR TO REACHING ATRIUM GRATES





SCALE 1"=40'

0 20 40 80 120 160

REVISED: FEBRUARY 2022

CITY OF PERRIS DMA PLAN	1 OF 3 SHEETS
LAKE CREEK-HARLEY KNOX (CITY CASE NO. P21-00008)	

NOT FOR CONSTRUCTION - THIS POST-CONSTRUCTION BMP SITE PLAN IS FOR WQMP REVIEW PURPOSE

POST-CONSTRUCTION BMP SITE PLAN
LAKE CREEK-HARLEY KNOX

GENERAL NOTES

1. THE EXISTING SITE CONSISTS OF OPEN, UNDEVELOPED SPACE, DRAINING GENERALLY FROM NORTH TO SOUTH. THERE ARE MINOR OFFSITE RUN-ON FLOWS TO THE SITE FROM THE WESTERLY UNDEVELOPED LAND AND A SMALL PORTION OF NORTHERLY UNDEVELOPED AREA. RUNOFF FROM THE PROJECT GENERALLY DRAINS IN A SOUTHEASTERLY DIRECTION IN A SHEET FLOW MANNER TOWARDS HARLEY KNOX BLVD. RUNOFF IS CAPTURED BY AN INLET ALONG HARLEY KNOX BLVD. AND DRAINS INTO AN EXISTING STORM DRAIN PIPE (I.E. – 24-INCH RCP) THAT EVENTUALLY CONNECTS WITH THE RCFC'S STORM DRAIN LINE D-3 IN REDLANDS AVENUE. RUNOFF EVENTUALLY DISCHARGES INTO THE EXISTING PERRIS VALLEY STORM DRAIN CHANNEL.
2. THE POST-PROJECT DRAINAGE CHARACTERISTICS WILL BE MAINTAINED SIMILAR AS COMPARED TO THE PRE-PROJECT CONDITION. REGARDING THE MINOR RUN-ON MENTIONED ABOVE, THE PROJECT PROPOSES A SWALE ON THE WESTERLY EDGE OF THE PROJECT TO DIRECT THE MINOR WESTERLY RUN-ON TOWARDS HARLEY KNOX BLVD.; THEREFORE, THERE WILL BE NO RUN-ON TO THE PROJECT FROM THE WESTERLY OFFSITE AREA. THE MINOR NORTHERLY OFFSITE AREA WILL BE CAPTURED AND CONVEYED TO THE PROPOSED BMP FOR TREATMENT. ON-SITE RUNOFF WILL BE DIRECTED VIA ON-SITE STORM DRAIN SYSTEM TO AN UNDERGROUND STORAGE FACILITY (STORMTRAP – 4'2" SINGLETRAP) FOR ATTENUATING LARGER PEAK FLOW BACK TO THE EXISTING CONDITION LEVEL PEAK FLOW PRIOR TO CONNECTING INTO THE EXISTING STORM DRAIN IN HARLEY KNOX BLVD. SELECT ON-SITE CATCH BASINS WILL HAVE PRE-TREATMENT BMP's (I.E. – CONNECTOR PIPE SCREEN) TO HELP MINIMIZE TRASH/DEBRIS GETTING INTO THE UNDERGROUND STORAGE FACILITY. A PROPRIETARY MODULAR WETALND SYSTEM ("BMP 1") IS PROPOSED IMMEDIATELY DOWNSTREAM OF THE UNDERGROUND STORAGE FACILITY TO PROVIDE TREATMENT FOR THE ON-SITE RUNOFF, BASED ON THE VOLUME-BASED APPROACH. ADDITIONALLY, THE PROPOSED LANDSCAPE AREAS IN THE SOUTHEASTERLY AREA OF THE PROJECT WILL PROVIDE PRE-TREATMENT IN THE FORM OF VEGETATED SWALES PRIOR TO CONNECTING INTO PROPOSED ATRIUM GRATES. RUNOFF FROM PAVED SURFACES AND ROOFS WILL BE DIRECTED TOWARDS LANDSCAPE AREAS WHERE POSSIBLE TO HELP PROMOTE INCIDENTAL INFILTRATION AND EVAPORATION.
3. THE PROJECT-SPECIFIC GEOTECHNICAL ENGINEER CONDUCTED INFILTRATION TESTING AND RESULTS SHOWED FIELD INFILTRATION RATES OF 2.4 IN/HR AND 6.5 IN/HR. THESE RATES ARE ABOVE THE INFILTRATION THRESHOLD OF 1.6 IN/HR; HOWEVER, THESE INFILTRATION RATES WERE OBTAINED NEAR THE NORTHERLY PORTION OF THE SITE, WHICH IS IN THE VICINITY OF THE EXISTING RIVERSIDE COUNTY FLOOD CONTROL MASTER DRAINAGE PLAN CHANNEL AND HAVE A TENDENCY TO HAVE SLIGHTLY MORE PERMEABLE SOILS. AS MENTIONED ABOVE, THE EXISTING SITE GENERALLY WANTS TO DRAIN IN A SOUTHEASTERLY DIRECTION TOWARDS HARLEY KNOX BLVD. TO BE CONSISTENT WITH THE EXISTING DRAINAGE CHARACTERISTICS, THE MOST SUITABLE LOCATION FOR A PROPOSED BMP WOULD BE IN THE SOUTHEASTERLY AREA OF THE SITE. BASED ON THE BORING LOGS NEAR THE SOUTHERLY EDGE OF THE PROJECT (I.E. – BORING ID'S B-3 AND B-4), CLAYEY MATERIALS (INCLUDING CLAYEY FINE SAND, CLAYEY SILT, SILTY CLAY) WERE OBSERVED APPROXIMATELY 5 FEET BELOW EXISTING SURFACE AND DEEPER. THE CLAYEY MATERIALS ARE NOT GENERALLY CONDUCIVE TO INFILTRATION AND OUR UNDERSTANDING IS THAT INFILTRATION IS NOT PRACTICABLE IN MOST AREAS WITHIN THE CITY OF PERRIS BASED ON THE NATURE OF EXISTING SOILS IN THE AREA. ADDITIONALLY, THE GEOTECHNICAL ENGINEER RECOMMENDS SETBACK OF 25 FEET FROM ANY STRUCTURES AND RETAINING WALLS FOR INFILTRATION FACILITIES. THE PROJECT PROPOSES SOME LANDSCAPE AREAS IN THE SOUTHEASTERLY AREA OF THE PROJECT BUT IT WOULD NOT BE PRACTICABLE WITH THE AFOREMENTIONED CONSTRAINTS. THEREFORE, INFILTRATION BMPS WERE NOT RECOMMENDED FOR THE SITE.
4. THE PROJECT IS SITUATED WITHIN THE ZONE X BASED ON THE FEMA FLOOD INSURANCE RATE MAP NUMBER 0065C1430H, EFFECTIVE AUGUST 18, 2014. THEREFORE, FEMA SUBMITTALS/PROCESSING SHOULD NOT BE NEEDED FOR THIS PROJECT.
5. PRELIMINARY DETAILS FOR TRASH ENCLOSURE WITH COVER, STENCIL, AND/OR ROOF DRAIN OUTLET LOCATIONS ARE PROVIDED ON THIS EXHIBIT OR BMP DETAIL SHEET; HOWEVER, THOSE DETAILS COULD BE REFINED FURTHER AT THE TIME OF FINAL WQMP.

PERMANENT SOURCE CONTROL BMPs

1. MARK ALL INLETS WITH THE WORDS "ONLY RAIN DOWN THE STORM DRAIN" OR SIMILAR
2. ENCLOSED REFUSE AREA WITH SIGNS POSTED NEARBY STATING "DO NOT DUMP HAZARDOUS MATERIALS HERE" OR SIMILAR

- LANDSCAPING DESIGNED 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.

OPERATIONAL SOURCE CONTROL BMPs

- MAINTAIN LANDCAPING USING MINIMUM OR NO PESTICIDES
- PREVENT EROSION OF SLOPES BY PLANTING FAST-GROWING, DENSE GROUND COVERING PLANTS
- PLANT NATIVE VEGETATION TO REDUCE THE AMOUNT OF WATER, FERTILIZERS, AND PESTICIDES APPLIED TO THE LANDSCAPE
- DO NOT OVERWATER
- USE IRRIGATION PRACTICES SUCH AS DRIP IRRIGATION, SOAKER HOSES OR MICRO-SPRAY SYSTEMS
- PERIODICALLY INSPECT AND FIX LEAKS AND MISDIRECTED SPRINKLERS.
- DO NOT RAKE OR BLOW LEAVES, CLIPPINGS, OR PRUNING WASTE INTO THE STREET, GUTTER OR STORM DRAIN
- DISPOSE OF GREEN WASTE BY COMPOSTING, HAULING IT TO A PERMITTED LANDFILL, OR RECYCLING IT THROUGH YOUR CITY'S PROGRAM
- PROVIDE IPM INFORMATION TO NEW OWNERS, LESSEES AND OPERATORS
- PERIODIC INSPECTIONS FOR LEAKY, OVERFILLED, UNCOVERED, OR OTHER PROBLEMATIC CONDITIONS WILL OCCUR
- CORRECTIVE ACTION WILL BE MADE UPON DETECTION, AS CIRCUMSTANCES PERMIT
- DUMPING OF LIQUID OR HAZARDOUS WASTES WILL BE PROHIBITED
- SPILL CONTROL MATERIALS WILL BE AVAILABLE ON-SITE
- MOVE LOADED AND UNLOADED ITEMS INDOORS AS SOON AS POSSIBLE
- 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 THE STORM DRAIN)

LID OPPORTUNITIES

1. PRESERVE EXISTING PERVIOUS AREA WHERE POSSIBLE.
2. LANDSCAPED AREAS DESIGNED TO BE SELF-RETAINING WHERE FEASIBLE.

DMA LEGEND & AREAS

DMA 1 DRAINING TO PERMANENT STRUCTURAL BMP

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TOTAL AREA = 307,071 S.F.

MISC. DMAs

DMA 1-5 SELF-TREATING AREA – 3,616 S.F.

LEGEND

DRAINAGE MANAGEMENT AREA

TRACT BOUNDARY

CENTERLINE

CURB AND GUTTER

1280 EXISTING CONTOUR LINE

SLOPE

ROOF DRAIN LOCATION (TBD)

DISCHARGE LOCATION

PROPOSED STORM DRAIN

GENERAL SURFACE FLOW PATH

PERMANENT STRUCTURAL BMP

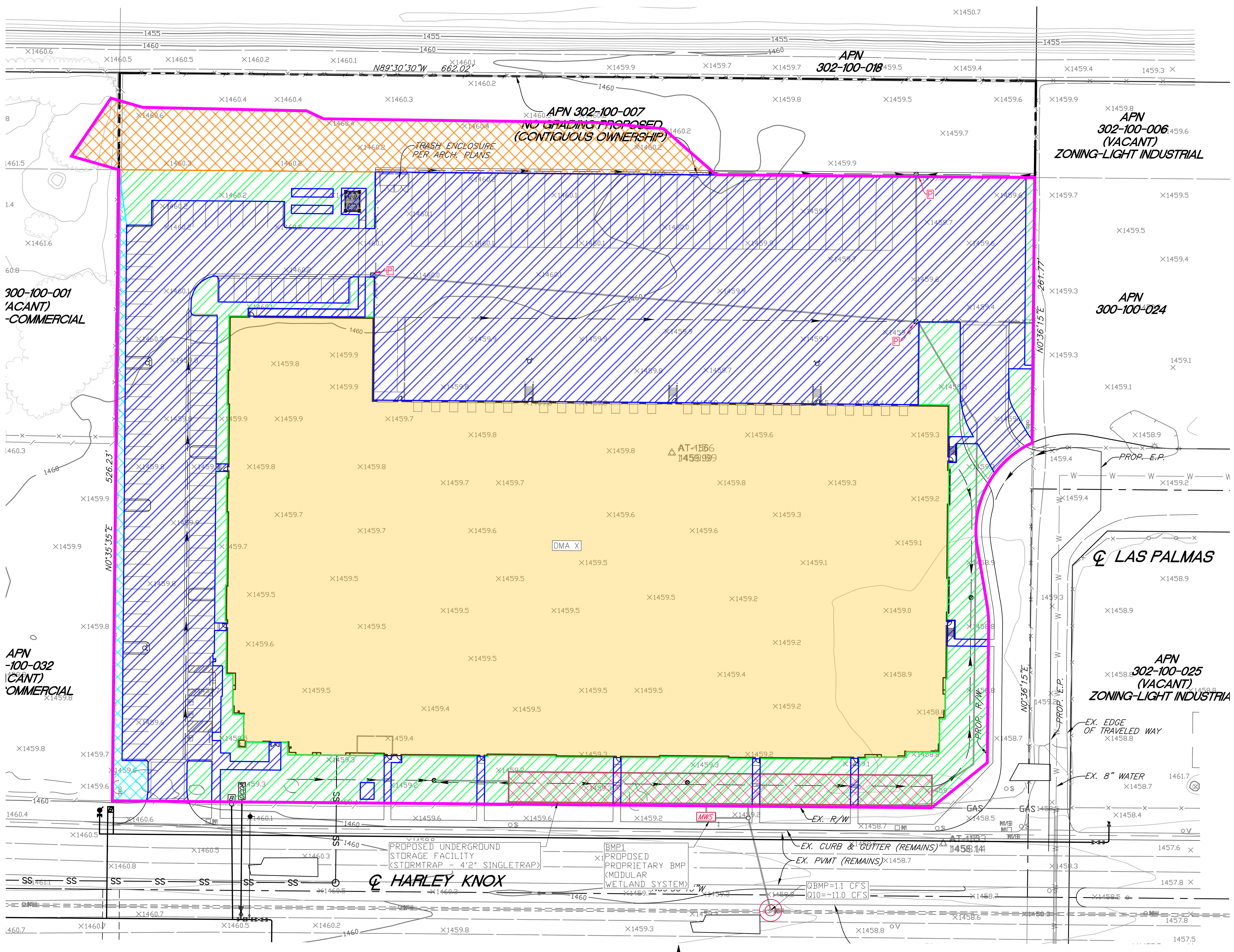
PROPOSED UNDERGROUND STORAGE FACILITY (STORMTRAP – SINGLETRAP OR EQUALLY ACCEPTABLE PRODUCT)
- OVERALL PROVIDED VOLUME: ~25,020 C.F.
- FOOTPRINT PROVIDED: ~6,005 S.F. (MIN.)
- EFFECTIVE DEPTH: ~4'2"

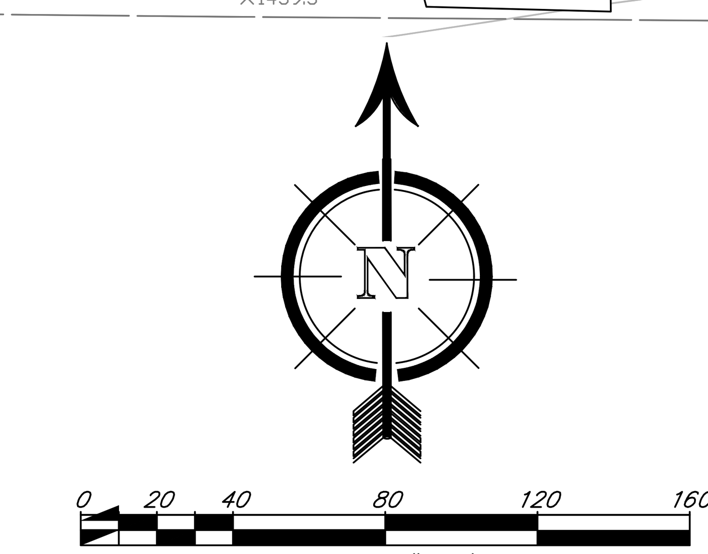
MWS PROPOSED MODULAR WETLAND SYSTEM (MWS)
- BMP 1 (MWS-L-8-12-5'-0"-V-UG)

PRE-TREATMENT BMP

P PROPOSED CONNECTOR PIPE SCREEN (CPS)
- PRE-TREATMENT DEVICE AT SELECT PROPOSED CATCH BASIN

PROPOSED VEGETATED SWALE
- PRE-TREATMENT PRIOR TO REACHING ATRIUM GRATES





REVISED: FEBRUARY 2022

CITY OF PERRIS	
POST-CONSTRUCTION BMP SITE PLAN	2 OF 3 SHEETS
LAKE CREEK-HARLEY KNOX	
(CITY CASE NO. P21-00008)	

POST-CONSTRUCTION BMP SECTION DETAILS
LAKE CREEK-HARLEY KNOX

SITE SPECIFIC DATA			
PROJECT NUMBER		13291	
PROJECT NAME		LAKE CREEK – HARLEY KNOX	
PROJECT LOCATION		PERRIS, CA	
STRUCTURE ID		BMP 1	
TREATMENT REQUIRED			
VOLUME BASED (CF)		FLOW BASED (CFS)	
12,490		N/A	
TREATMENT HGL AVAILABLE (FT)			2.2
PEAK BYPASS REQUIRED (CFS) – IF APPLICABLE			N/A
PIPE DATA	I.E.	MATERIAL	DIAMETER
INLET PIPE 1	49.80	PVC	8"
INLET PIPE 2	N/A	N/A	N/A
OUTLET PIPE	48.10	PVC	8"
	PRETREATMENT	BIOFILTRATION	DISCHARGE
RIM ELEVATION	60.7	60.7	60.7
SURFACE LOAD	PEDESTRIAN	PEDESTRIAN	PEDESTRIAN
FRAME & COVER	2EA ø30"	30"X48" & ø30"	ø30"
WETLANDMEDIA VOLUME (CY)			5.51
ORIFICE SIZE (DIA. INCHES)			ø1.34"
NOTES: BYPASS PIPE IN DETENTION SYSTEM SET AT 51.50'.			

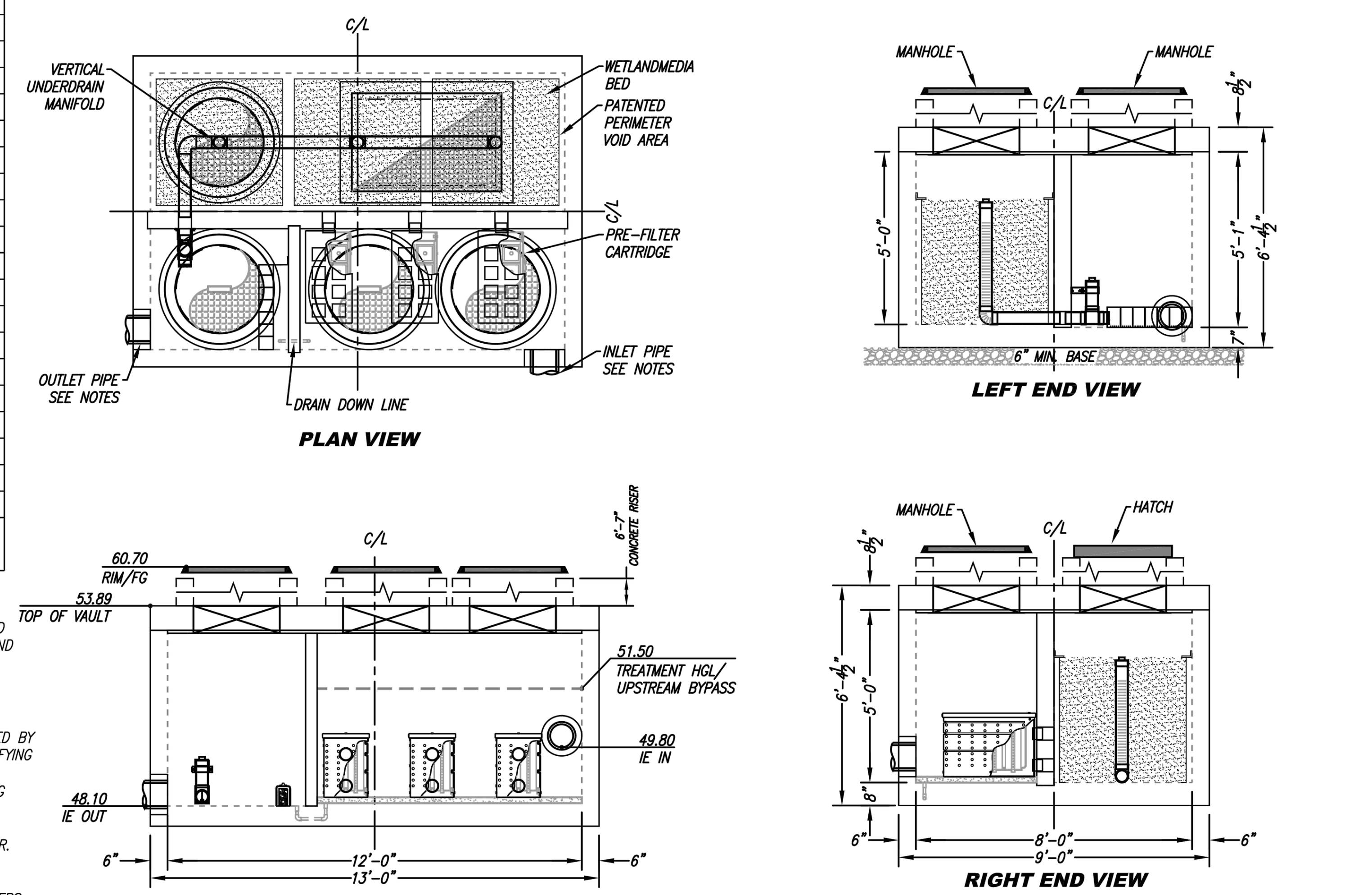
INSTALLATION NOTES

1. CONTRACTOR TO PROVIDE ALL LABOR, EQUIPMENT, MATERIALS AND INCIDENTALS REQUIRED TO OFFLOAD AND INSTALL THE SYSTEM AND APPURTENANCES IN ACCORDANCE WITH THIS DRAWING AND THE MANUFACTURERS' SPECIFICATIONS, UNLESS OTHERWISE STATED IN MANUFACTURER'S CONTRACT.
2. UNIT MUST BE INSTALLED ON LEVEL BASE. MANUFACTURER RECOMMENDS A MINIMUM 6" LEVEL ROCK BASE UNLESS SPECIFIED BY THE PROJECT ENGINEER. CONTRACTOR IS RESPONSIBLE FOR VERIFYING PROJECT ENGINEER'S RECOMMENDED BASE SPECIFICATIONS.
4. CONTRACTOR TO SUPPLY AND INSTALL ALL EXTERNAL CONNECTING PIPES. ALL PIPES MUST BE FLUSH WITH INSIDE SURFACE OF CONCRETE (PIPES CANNOT INTRUDE BEYOND FLUSH). INVERT OF OUTFLOW PIPE MUST BE FLUSH WITH DISCHARGE CHAMBER FLOOR. ALL PIPES SHALL BE SEALED WATERTIGHT PER MANUFACTURER'S STANDARD CONNECTION DETAIL.
5. CONTRACTOR RESPONSIBLE FOR INSTALLATION OF ALL PIPES, RISERS, MANHOLES, AND HATCHES. CONTRACTOR TO GROUT ALL MANHOLES AND HATCHES TO MATCH FINISHED SURFACE UNLESS SPECIFIED OTHERWISE.
6. VEGETATION SUPPLIED AND INSTALLED BY OTHERS. ALL UNITS WITH VEGETATION MUST HAVE DRIP OR SPRAY IRRIGATION SUPPLIED AND INSTALLED BY OTHERS.
7. CONTRACTOR RESPONSIBLE FOR CONTACTING BIO CLEAN FOR ACTIVATION OF UNIT. MANUFACTURER'S WARRANTY IS VOID WITHOUT PROPER ACTIVATION BY A BIO CLEAN REPRESENTATIVE.

GENERAL NOTES

1. MANUFACTURER TO PROVIDE ALL MATERIALS UNLESS OTHERWISE NOTED.
2. ALL DIMENSIONS, ELEVATIONS, SPECIFICATIONS AND CAPACITIES ARE SUBJECT TO CHANGE. FOR PROJECT SPECIFIC DRAWINGS DETAILING EXACT DIMENSIONS, WEIGHTS AND ACCESSORIES PLEASE CONTACT BIO CLEAN.

6/9/21BREEVE



REQUIRED TREATMENT VOLUME (CF)	12,490
DRAINDOWN DURATION (HOURS)	47
AVERAGE DISCHARGE RATE PER MWS UNIT(GPM)	33.21
OPERATING HEAD (FT)	3.40
WETLANDMEDIA INFILTRATION RATE (IN/HR)	22
OR	
WETLANDMEDIA LOADING RATE (GPM/SF)	0.22

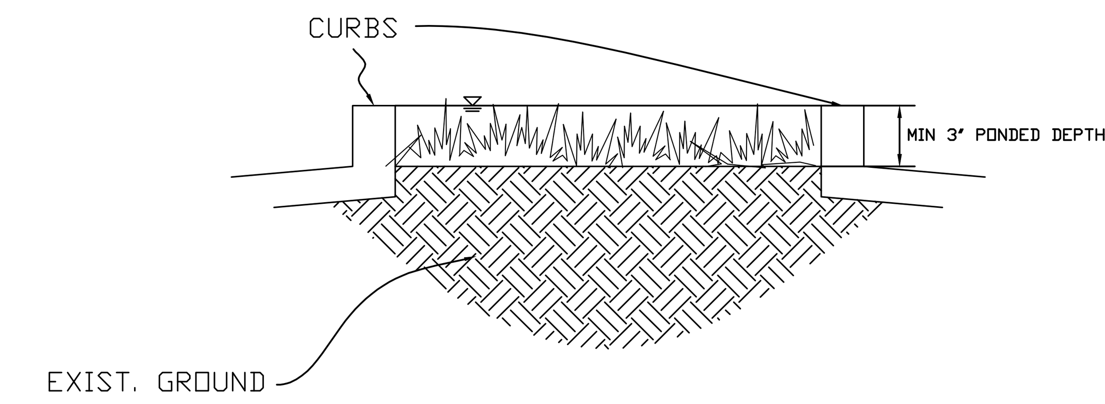
PROPRIETARY AND CONFIDENTIAL:

Bio Clean A Forterra Company

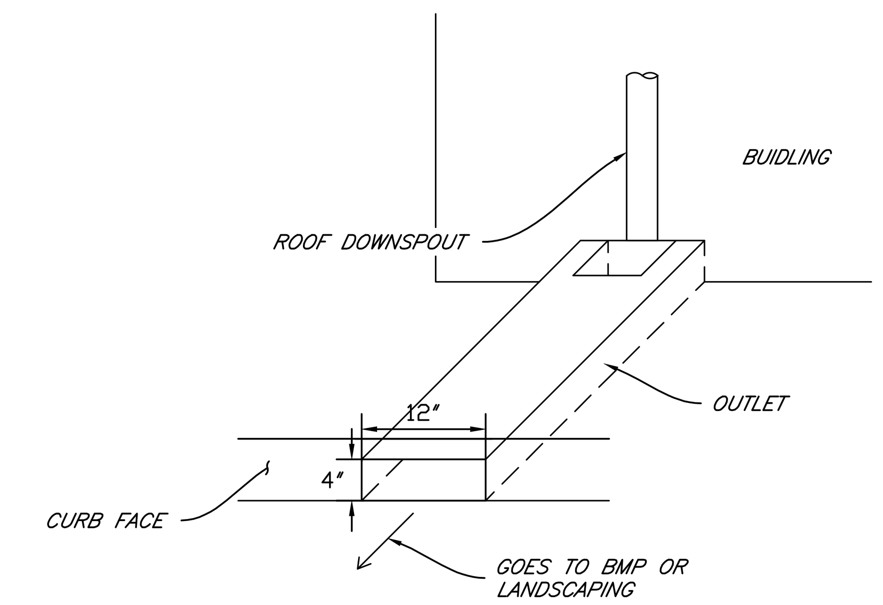
MWS-L-8-12-5'-0"-V-UG
STORMWATER BIOFILTRATION SYSTEM
STANDARD DETAIL

BMP 1 - PROPRIETARY MODULAR WETLAND SYSTEM (MWS)
NOT TO SCALE

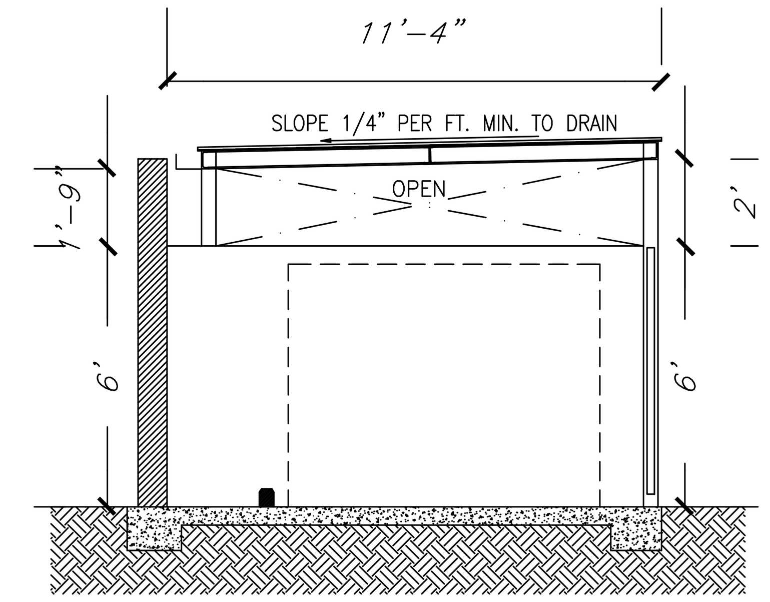
INLET PLACARD DETAIL (TYP.)
NOT TO SCALE



LANDSCAPED ISLAND DETAIL (TYP.)
NOT TO SCALE



ROOF DRAIN CURB OUTLET STRUCTURE DETAIL (TYP.)
NOT TO SCALE

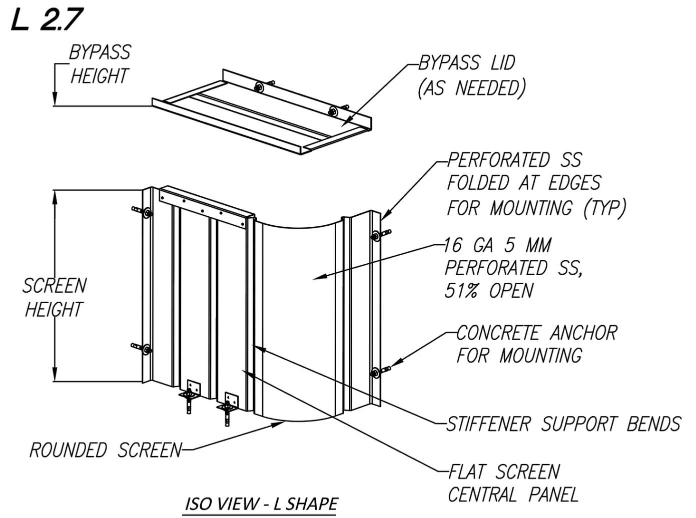


TRASH ENCLOSURE STRUCTURE DETAIL (TYP.)
NOT TO SCALE

GENERAL NOTES

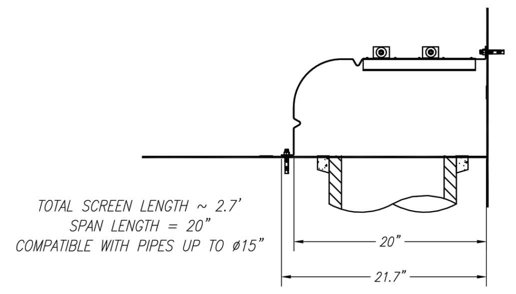
1. THE PROPOSED UNDERGROUND STORAGE FACILITY ("STORMTRAP – 4'2 SINGLETRAP") SECTION DETAIL IS PROVIDED ON THIS SHEET. ALL THE OTHER DETAILS ASSOCIATED WITH THE PROPOSED STORMTRAP ARE PROVIDED FOLLOWING THIS SHEET FOR REFERENCE PURPOSE.
2. THE PROPOSED MODULAR WETLAND SYSTEM (MWS) (DETAILED SHOWN ON THIS SHEET) WILL BE PROVIDED IMMEDIATELY DOWNSTREAM OF THE PROPOSED UNDERGROUND FACILITY AND HAS BEEN SIZED USING THE VOLUME-BASED APPROACH.
3. THE PROPOSED LANDSCAPING/PLANTING (PLANT PALETTE) IS TO BE PROVIDED SEPARATELY BY THE PROJECT LANDSCAPE ARCHITECT.
4. ADDITIONAL PRE-TREATMENT WILL BE PROVIDED IN THE SOUTHEASTERLY LANDSCAPE AREAS IN THE FORM OF VEGETATED SWALES PRIOR TO CONNECTING THE RUNOFF TO PROPOSED ATRIUM GRATES.

CONNECTOR PIPE SCREEN (CPS)
L 2.7



ISO VIEW - L SHAPE

CPS L WITH 2.7 FT SCREEN LENGTH	
CPS HEIGHT (IN)	SCREEN FLOW (CFS)
12	3.84
18	7.06
24	10.88
30	15.20
36	19.99
NOTE: BYPASS FLOW RATES VARY WITH VAULT DEPTH AND BYPASS HEIGHT. CONTACT BIO CLEAN FOR ADDITIONAL INFORMATION.	



TOP SECTION VIEW - L SHAPE

GENERAL NOTES

1. BIO CLEAN TO PROVIDE ALL MATERIALS UNLESS OTHERWISE NOTED.
2. ALL DIMENSIONS, ELEVATIONS, SPECIFICATIONS, AND CAPACITIES ARE SUBJECT TO CHANGE.
3. THIS CPS UNIT IS DESIGNED FOR TREATMENT FLOWS THROUGH THE SCREEN. FLOWS GREATER THAN THE TREATMENT FLOW RATE WILL BYPASS OVER THE SCREEN.
4. A BYPASS LID IS REQUIRED WHEN THE OUTLET PIPE IS DIRECTLY BELOW THE CURB OPENING.
5. CPS IS COMPRISED OF 304 STAINLESS STEEL. THICKNESS IS 16 GAUGE. SCREEN PERFORATIONS ARE 5 MILLIMETERS IN DIAMETER. THE SCREEN AREA IS 51% OPEN SPACE.

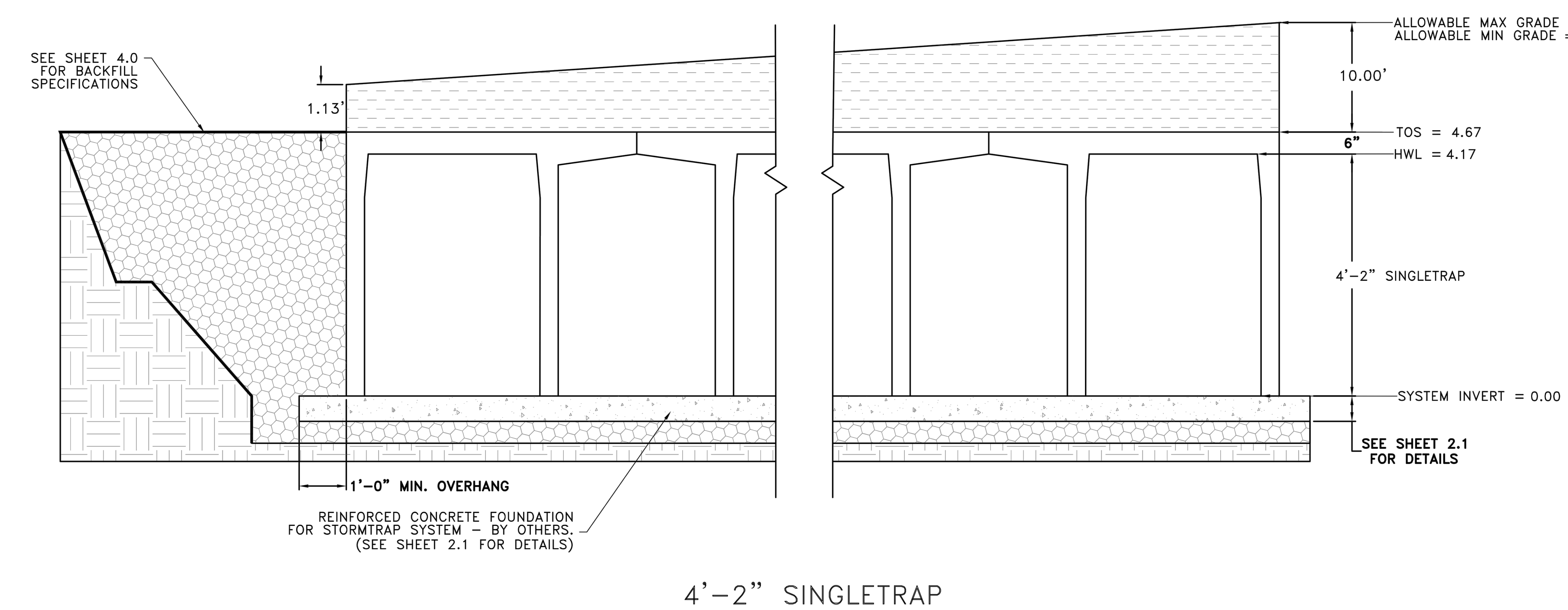
INSTALLATION NOTES

1. CONTRACTOR TO PROVIDE ALL LABOR, EQUIPMENT, MATERIALS, AND INCIDENTALS REQUIRED TO INSTALL THE CPS UNIT AND APPURTENANCES IN ACCORDANCE WITH THIS DRAWING AND THE MANUFACTURER'S SPECIFICATIONS, UNLESS OTHERWISE STATED IN MANUFACTURER'S CONTRACT.
2. POSITION THE CPS SO IT IS EVENLY SPACED AROUND THE CONNECTOR PIPE, ENSURING A MIN. OF 4" SPACING AWAY FROM ANY CORNERS. SCREEN BOTTOM SHALL BE FLUSH WITH THE CATCH BASIN FLOOR, OR WITH GAPS NO GREATER THAN 5 MM.
3. IF A BYPASS LID IS REQUIRED, VERIFY THE BYPASS HEIGHT NEEDED AND MARK THAT LOCATION ON THE WALL DIRECTLY ABOVE THE BASE UPRIGHTS. LIFT THE LID IN PLACE AND MARK THE HOLE LOCATIONS FOR THE LID MOUNTING BRACKETS. SECURE THE LID WITH STAINLESS STEEL NUTS.

WARRANTY: 3 YEAR MANUFACTURER'S		MEETS FULL CAPTURE REQUIREMENTS	
BIO CLEAN ENVIRONMENTAL SERVICES, INC. 398 VIA EL CENTRO, OCEANSIDE CA 92058 PHONE: 760-433-7640		REVISIONS:	DATE:
		REVISIONS:	DATE:
DATE: 1/17/2020	SCALE: NTS	REVISIONS:	DATE:
DRAFTER: G.M.S.	UNITS = INCHES	REVISIONS:	DATE:

Bio Clean A Forterra Company

PRE-TREATMENT: PROPRIETARY CONNECTOR PIPE SCREEN (CPS) - TYP.
NOT TO SCALE



STORMTRAP - SINGLETRAP - TYP.
NOT TO SCALE

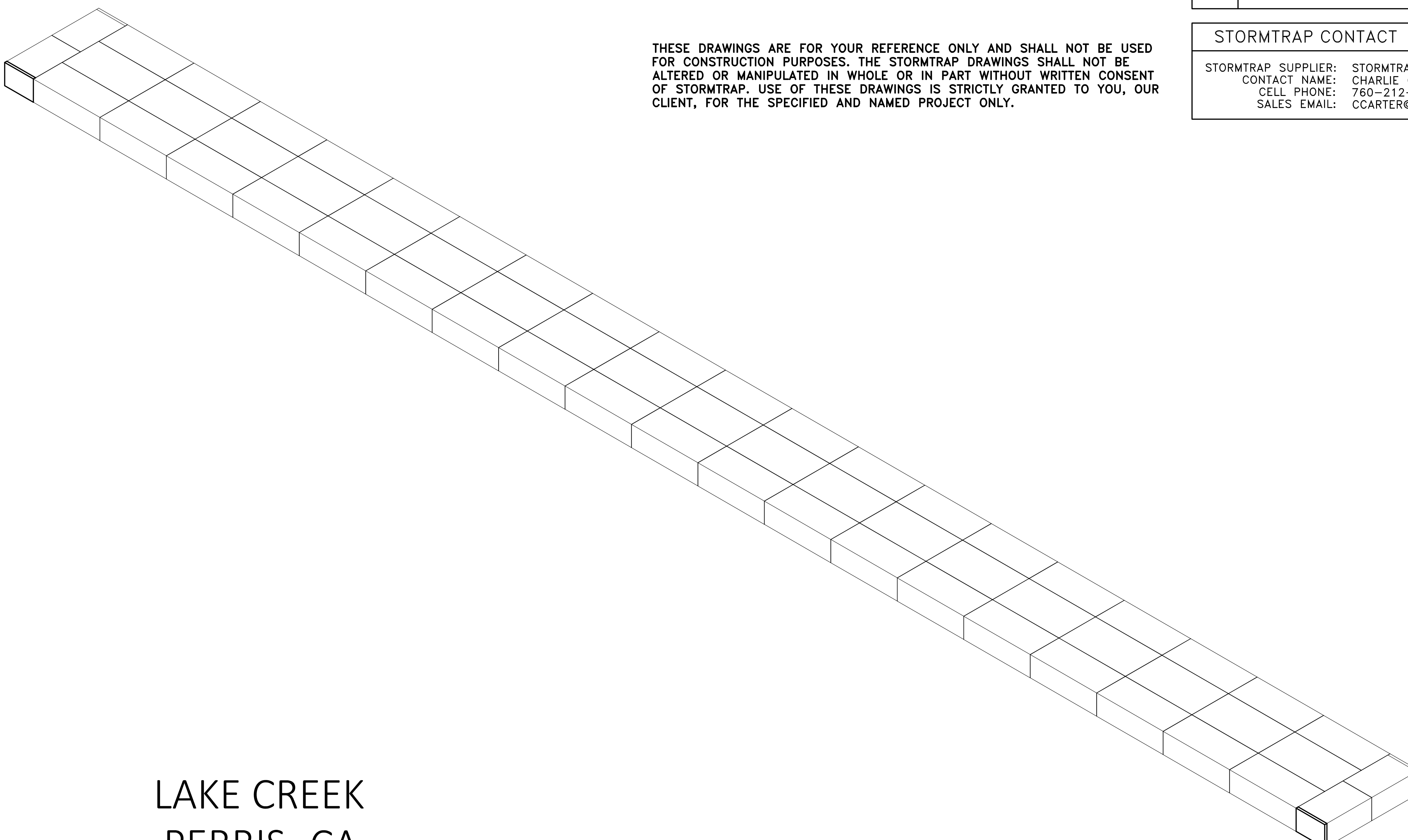
REVISED: FEBRUARY 2022

CITY OF PERRIS	
POST-CONSTRUCTION BMP SECTION DETAILS	3 OF 3 SHEETS
LAKE CREEK-HARLEY KNOX	
(CITY CASE NO. P21-00008)	

NOT FOR CONSTRUCTION – THIS POST-CONSTRUCTION BMP SITE PLAN IS FOR WQMP REVIEW PURPOSE

StormTrap®

MODULAR CONCRETE
STORMWATER MANAGEMENT



THESE DRAWINGS ARE FOR YOUR REFERENCE ONLY AND SHALL NOT BE USED FOR CONSTRUCTION PURPOSES. THE STORMTRAP DRAWINGS SHALL NOT BE ALTERED OR MANIPULATED IN WHOLE OR IN PART WITHOUT WRITTEN CONSENT OF STORMTRAP. USE OF THESE DRAWINGS IS STRICTLY GRANTED TO YOU, OUR CLIENT, FOR THE SPECIFIED AND NAMED PROJECT ONLY.

LAKE CREEK
PERRIS, CA

SHEET INDEX

STORMTRAP CONTACT INFORMATION

STORMTRAP SUPPLIER: STORMTRAP
CONTACT NAME: CHARLIE CARTER
CELL PHONE: 760-212-5628
SALES EMAIL: CCARTER@STORMTRAP.COM

StormTrap®

PATENTS LISTED AT: [HTTP://STORMTRAP.COM/PATENT]

1287 WINDHAM PARKWAY
ROMEOVILLE, IL 60446
P:815-941-4549 / F:331-318-5347

ENGINEER INFORMATION:

SDH AND ASSOCIATES

27363 VIA INDUSTRIA
RIVERSIDE, CA
951-683-3691

PROJECT INFORMATION:

LAKE CREEK

PERRIS, CA

CURRENT ISSUE DATE:

6/8/2021

ISSUED FOR:

PRELIMINARY

REV.	DATE:	ISSUED FOR:	DWN BY:
⚠	6/8/2021	PRELIMINARY	RJL

SCALE:

NTS

SHEET TITLE:

COVER SHEET

SHEET NUMBER:

0.0

STRUCTURAL DESIGN LOADING CRITERIA

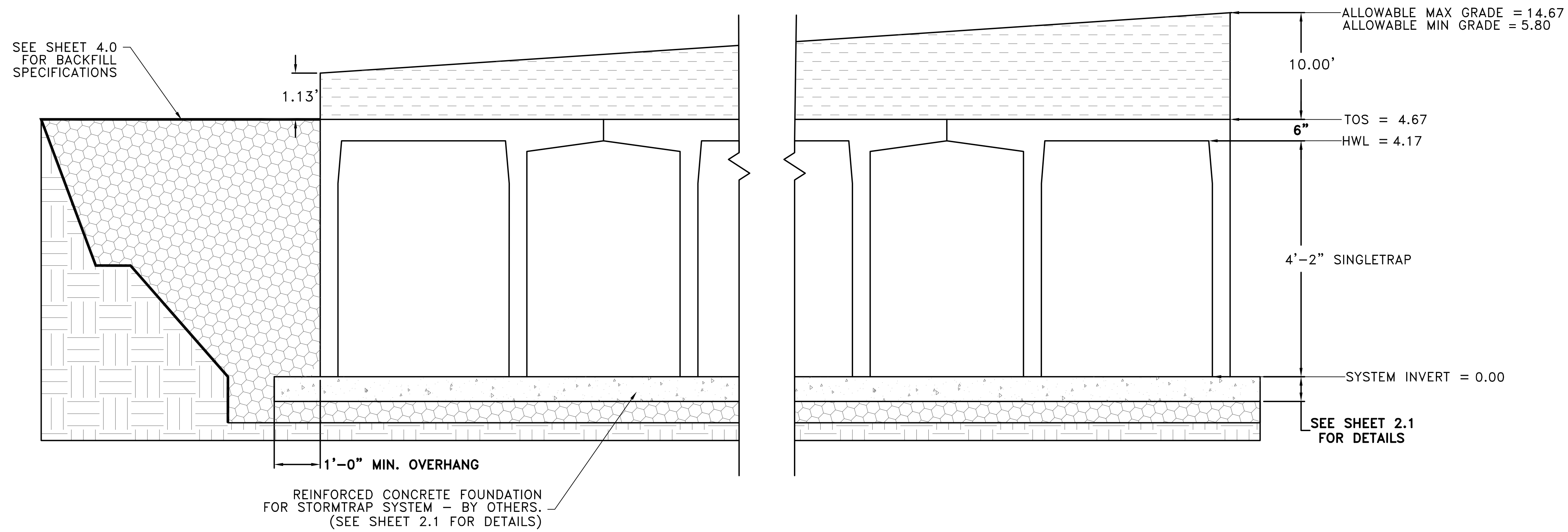
LIVE LOADING:	AASHTO HS-20 HIGHWAY LOADING
GROUND WATER TABLE:	BELOW INVERT OF SYSTEM
SOIL BEARING PRESSURE:	3000PSF
SOIL DENSITY:	120 PCF
EQUIVALENT UNSATURATED LATERAL ACTIVE EARTH PRESSURE:	35 PSF / FT.
EQUIVALENT SATURATED LATERAL ACTIVE EARTH PRESSURE:	80 PSF/FT. (IF WATER TABLE PRESENT)
APPLICABLE CODES:	ASTM C857 ACI-318
BACKFILL TYPE:	SEE SHEET 4.0 FOR BACKFILL OPTIONS

STORMTRAP SYSTEM INFORMATION

WATER STORAGE PROV:	25,020.04 CUBIC FEET
UNIT HEADROOM:	4'-2" SINGLETRAP
UNIT QUANTITY:	61 TOTAL PIECES

SITE SPECIFIC DESIGN CRITERIA

1. STORMTRAP UNITS SHALL BE MANUFACTURED AND INSTALLED ACCORDING TO SHOP DRAWINGS APPROVED BY THE INSTALLING CONTRACTOR AND ENGINEER OF RECORD. THE SHOP DRAWINGS SHALL INDICATE SIZE AND LOCATION OF ROOF OPENINGS AND INLET/ OUTLET PIPE TYPES, SIZES, INVERT ELEVATIONS AND SIZE OF OPENINGS.
2. COVER RANGE: MIN. 1.13' MAX. 10.00' CONSULT STORMTRAP FOR ADDITIONAL COVER OPTIONS.
3. ALL DIMENSIONS AND SOIL CONDITIONS, INCLUDING BUT NOT LIMITED TO GROUNDWATER AND SOIL BEARING CAPACITY ARE REQUIRED TO BE VERIFIED IN THE FIELD BY OTHERS PRIOR TO STORMTRAP INSTALLATION.
4. FOR STRUCTURAL CALCULATIONS THE GROUND WATER TABLE IS ASSUMED TO BE BELOW INVERT OF SYSTEM IF WATER TABLE IS DIFFERENT THAN ASSUMED, CONTACT STORMTRAP.
5. SYSTEM DESIGN MAY ALLOW FOR INCIDENTAL LEAKAGE AND WILL NOT BE SUBJECT TO LEAKAGE TESTING.



4'-2" SINGLETRAP

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PROJECT INFORMATION:

LAKE CREEK
PERRIS, CA

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SCALE:

NTS

SHEET TITLE:

SINGLETRAP DESIGN CRITERIA

SHEET NUMBER:

1.0

BILL OF MATERIALS			
QTY.	UNIT TYPE	DESCRIPTION	WEIGHT
19	I	4'–2" SINGLETRAP	14984
0	II	4'–2" SINGLETRAP	0
38	III	4'–2" SINGLETRAP	14358
0	IV	4'–2" SINGLETRAP	0
0	VII	4'–2" SINGLETRAP	0
4	SPIV	4'–2" SINGLETRAP	VARIES
0	T2 PANEL	6" THICK PANEL	0
4	T4 PANEL	6" THICK PANEL	2319
0	T7 PANEL	6" THICK PANEL	0
12	JOINTWRAP	150' PER ROLL	
56	JOINTTAPE	14.5' PER ROLL	

LOADING DISCLAIMER:

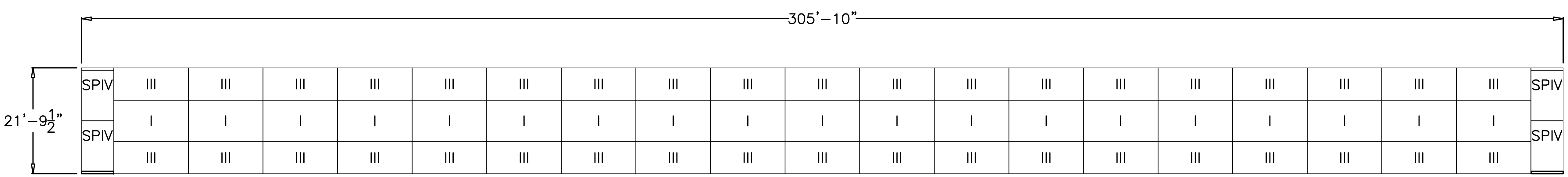
STORMTRAP IS NOT DESIGNED TO ACCEPT ANY ADDITIONAL LOADINGS FROM NEARBY STRUCTURES NEXT TO OR OVER THE TOP OF STORMTRAP. IF ADDITIONAL LOADING CONSIDERATIONS ARE REQUIRED FOR STRUCTURAL DESIGN OF STORMTRAP, PLEASE CONTACT STORMTRAP IMMEDIATELY.

THE STORMTRAP SYSTEM HAS NOT BEEN DESIGNED TO SUPPORT THE ADDITIONAL WEIGHT OF ANY TREES. FURTHERMORE, THE ROOTS OF THE TREES MUST BE CONTAINED TO PREVENT FUTURE DAMAGE TO THE STORMTRAP SYSTEM. STORMTRAP ACCEPTS NO LIABILITY FOR DAMAGES CAUSED BY TREES OR OTHER VEGETATION PLACE AROUND OR ON TOP OF THE SYSTEM.

N

DESIGN CRITERIA
ALLOWABLE MAX GRADE = 14.67
ALLOWABLE MIN GRADE = 5.80
INSIDE HEIGHT ELEVATION = 4.17
SYSTEM INVERT = 0.00

NOTES:
1. DIMENSIONING OF STORMTRAP SYSTEM SHOWN BELOW ALLOW FOR A 3/4" GAP BETWEEN EACH MODULE.
2. ALL DIMENSIONS TO BE VERIFIED IN THE FIELD BY OTHERS.
3. SEE SHEET 3.0 FOR INSTALLATION SPECIFICATIONS.
4. SP – INDICATES A MODULE WITH MODIFICATIONS.
5. P – INDICATES A MODULE WITH A PANEL ATTACHMENT.
6. CONTRACTORS RESPONSIBILITY TO ENSURE CONSISTENCY/ACCURACY TO FINAL ENGINEER OF RECORD PLAN SET.



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SINGLETRAP SYSTEM LAYOUT

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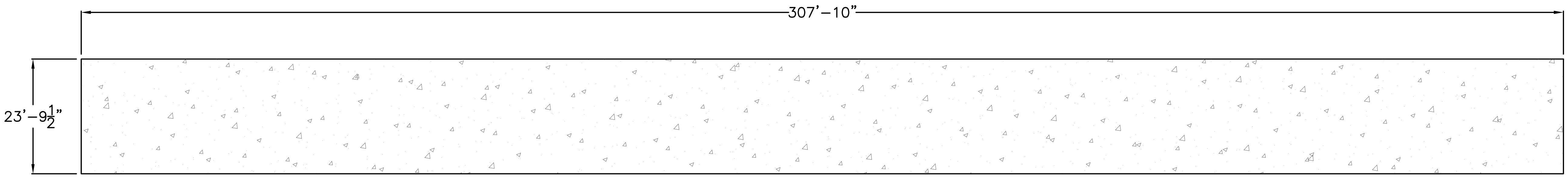
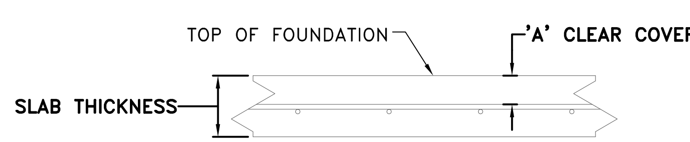
2.0



CONCRETE FOUNDATION NOTES:

1. CONCRETE FOUNDATION TO BE SUPPLIED AND INSTALLED BY OTHERS.
2. CONCRETE STRENGTH @ 28 DAYS, 5%–8% ENTRAINED AIR, 4" MAX SLUMP.
3. NET ALLOWABLE SOIL PRESSURE AS INDICATED ON SHEET 1.0.
4. SOIL CONDITIONS TO BE VERIFIED ON SITE BY OTHERS.
5. REBAR: ASTM A615 GRADE 60, BLACK BAR.
6. DIMENSION OF FOUNDATION MUST HAVE 1'–0" OVERHANG BEYOND EXTERNAL FACE OF MODULE.
7. DIMENSION OF STORMTRAP SYSTEM ALLOW FOR A 3/4" GAP BETWEEN EACH MODULE.
8. ALL DIMENSIONS TO BE VERIFIED IN THE FIELD BY OTHERS.
9. SEE SHEET 3.0 FOR INSTALLATION SPECIFICATIONS.



STORMTRAP FOUNDATION DETAIL

MAXIMUM SYSTEM COVER	SLAB THICKNESS	CONCRETE STRENGTH	REINFORCEMENT (BOTH DIRECTIONS)	'A' CLEAR COVER
6" – 12"	0'–8"	4000 PSI	#4 @ 18" O.C.	3.5"
>1'–0" – 2'–0"	0'–8"	4000 PSI	#4 @ 16" O.C.	3.5"
>2'–0" – 3'–0"	0'–8"	4000 PSI	#4 @ 12" O.C.	3.5"
>3'–0" – 4'–0"	0'–8"	4000 PSI	#4 @ 12" O.C.	3.5"
>4'–0" – 5'–0"	0'–8"	4000 PSI	#5 @ 18" O.C.	3.375"
>5'–0" – 6'–0"	0'–8"	4000 PSI	#5 @ 16" O.C.	3.375"
>6'–0" – 7'–0"	0'–8"	4000 PSI	#5 @ 16" O.C.	3.375"
>7'–0" – 8'–0"	0'–9"	4000 PSI	#5 @ 12" O.C.	3.875"
>8'–0" – 9'–0"	0'–10"	4000 PSI	#5 @ 12" O.C.	4.375"
>9'–0" – 10'–0"	0'–10"	4500 PSI	#5 @ 12" O.C.	4.375"

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SHEET NUMBER:

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STORMTRAP INSTALLATION SPECIFICATIONS

1. STORMTRAP SHALL BE INSTALLED IN ACCORDANCE WITH ASTM C891, STANDARD FOR INSTALLATION OF UNDERGROUND PRECAST CONCRETE UTILITY STRUCTURES, THE FOLLOWING ADDITIONS AND/OR EXCEPTIONS SHALL APPLY:

2. IT IS THE RESPONSIBILITY OF THE INSTALLING CONTRACTOR TO ENSURE THAT PROPER/ADEQUATE EQUIPMENT IS USED TO SET/INSTALL THE MODULES.

3. STORMTRAP MODULES SHALL BE PLACED ON A LEVEL CONCRETE FOUNDATION (SEE SHEET 2.1) WITH A 1'-0" OVERHANG ON ALL SIDES THAT SHALL BE POURED IN PLACE BY INSTALLING CONTRACTOR. A QUALIFIED GEOTECHNICAL ENGINEER WILL BE EMPLOYED, BY OWNER, TO PROVIDE ASSISTANCE IN EVALUATING THE EXISTING SOIL CONDITIONS TO ENSURE THAT THE SOIL BEARING PRESSURE MEETS OR EXCEEDS THE STRUCTURAL DESIGN LOADING CRITERIA AS SPECIFIED ON SHEET 1.0.

4. THE STORMTRAP MODULES SHALL BE PLACED SUCH THAT THE MAXIMUM SPACE BETWEEN ADJACENT MODULES DOES NOT EXCEED $\frac{3}{4}$" (SEE DETAIL 2). IF THE SPACE EXCEEDS $\frac{3}{4}$", THE MODULES SHALL BE RESET WITH APPROPRIATE ADJUSTMENT MADE TO LINE AND GRADE TO BRING THE SPACE INTO SPECIFICATION.

5. THE PERIMETER HORIZONTAL JOINT BETWEEN THE STORMTRAP MODULES AND THE CONCRETE FOUNDATION SHALL BE SEALED TO THE FOUNDATION WITH PRE-FORMED MASTIC JOINT SEALER ACCORDING TO ASTM C891, 8.8 AND 8.12 (SEE DETAIL 1). THE MASTIC JOINT TAPE DOES NOT PROVIDE A WATERTIGHT SEAL.

6. ALL EXTERIOR JOINTS BETWEEN ADJACENT STORMTRAP MODULES SHALL BE SEALED WITH 8" WIDE PRE-FORMED, COLD-APPLIED, SELF-ADHERING ELASTOMERIC RESIN, BONDED TO A WOVEN , HIGHLY PUNCTURE RESISTANT POLYMER WRAP, CONFORMING TO ASTM C891 AND SHALL BE INTEGRATED WITH PRIMER SEALANT AS APPROVED BY STORMTRAP (SEE DETAILS 3 & 4). THE JOINT WRAP DOES NOT PROVIDE A WATERTIGHT SEAL. THE SOLE PURPOSE OF THE JOINT WRAP IS TO PROVIDE A SILT AND SOIL TIGHT SYSTEM. THE ADHESIVE EXTERIOR JOINT WRAP SHALL BE INSTALLED ACCORDING TO THE FOLLOWING INSTALLATION INSTRUCTIONS:

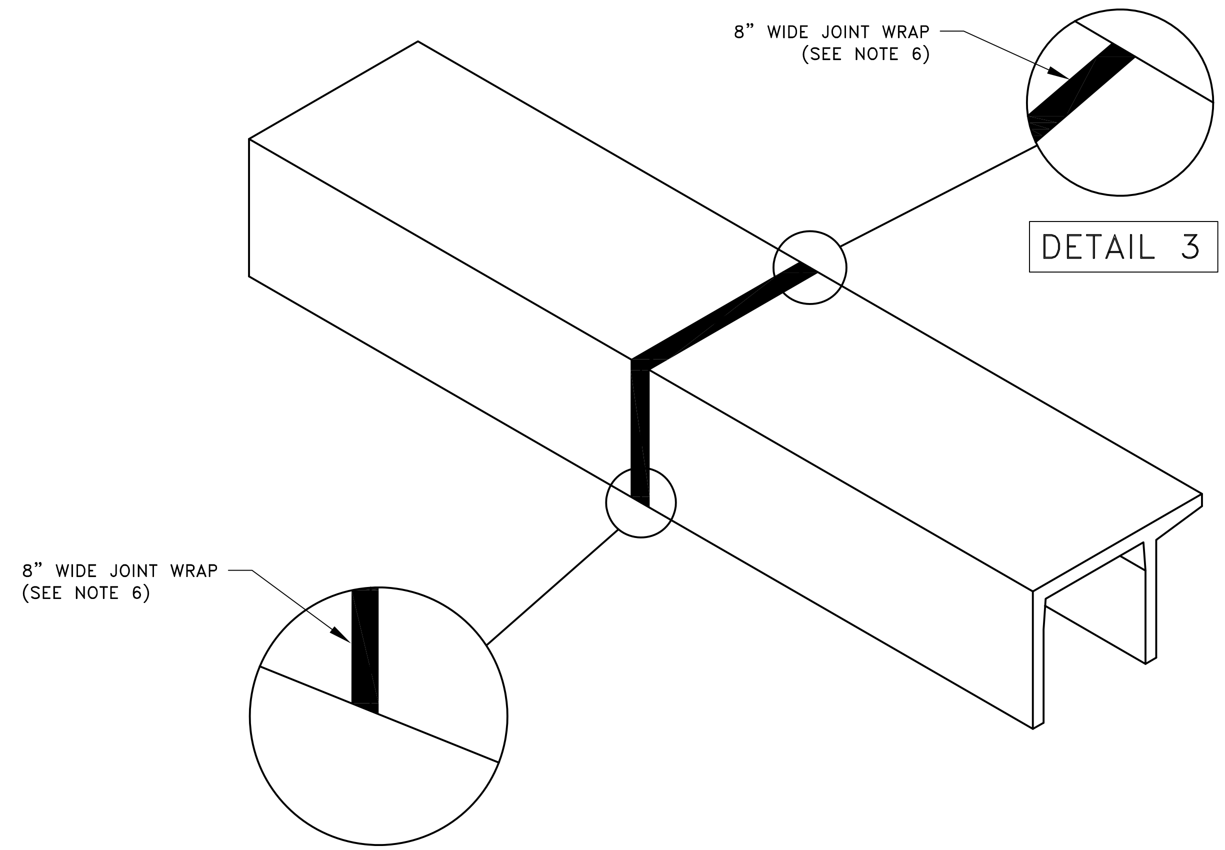
   6.1. USE A BRUSH OR WET CLOTH TO THOROUGHLY CLEAN THE OUTSIDE SURFACE AT THE POINT WHERE JOINT WRAP IS TO BE APPLIED.

   6.2. A RELEASE PAPER PROTECTS THE ADHESIVE SIDE OF THE JOINT WRAP. PLACE THE ADHESIVE TAPE (ADHESIVE SIDE DOWN) AROUND THE STRUCTURE, REMOVING THE RELEASE PAPER AS YOU GO. PRESS THE JOINT WRAP FIRMLY AGAINST THE STORMTRAP MODULE SURFACE WHEN APPLYING.

7. IF THE CONTRACTOR NEEDS TO CANCEL ANY SHIPMENTS, THEY MUST DO SO 48 HOURS PRIOR TO THEIR SCHEDULED ARRIVAL AT THE JOB SITE. IF CANCELED AFTER THAT TIME, PLEASE CONTACT THE PROJECT MANAGER.

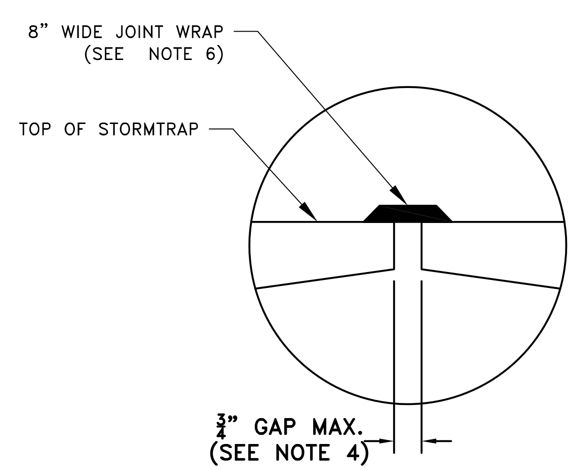
8. IF THE STORMTRAP MODULE(S) IS DAMAGED IN ANY WAY PRIOR, DURING, OR AFTER INSTALL, STORMTRAP MUST BE CONTACTED IMMEDIATELY TO ASSESS THE DAMAGE AND DETERMINE WHETHER OR NOT THE MODULE(S) WILL NEED TO BE REPLACED. IF ANY MODULE ARRIVES AT THE JOBSITE DAMAGED DO NOT UNLOAD IT; CONTACT STORMTRAP IMMEDIATELY. ANY DAMAGE NOT REPORTED BEFORE THE TRUCK IS UNLOADED WILL BE THE CONTRACTOR'S RESPONSIBILITY.

9. STORMTRAP MODULES CANNOT BE ALTERED IN ANY WAY AFTER MANUFACTURING WITHOUT WRITTEN CONSENT FROM STORMTRAP.

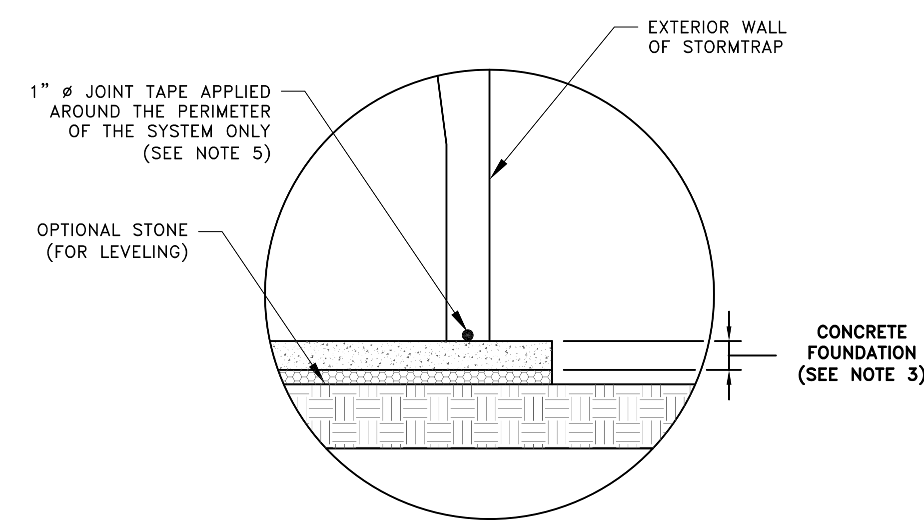


DETAIL 3

DETAIL 4



DETAIL 2



DETAIL 1

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SINGLETRAP INSTALLATION SPECIFICATIONS

SHEET NUMBER:

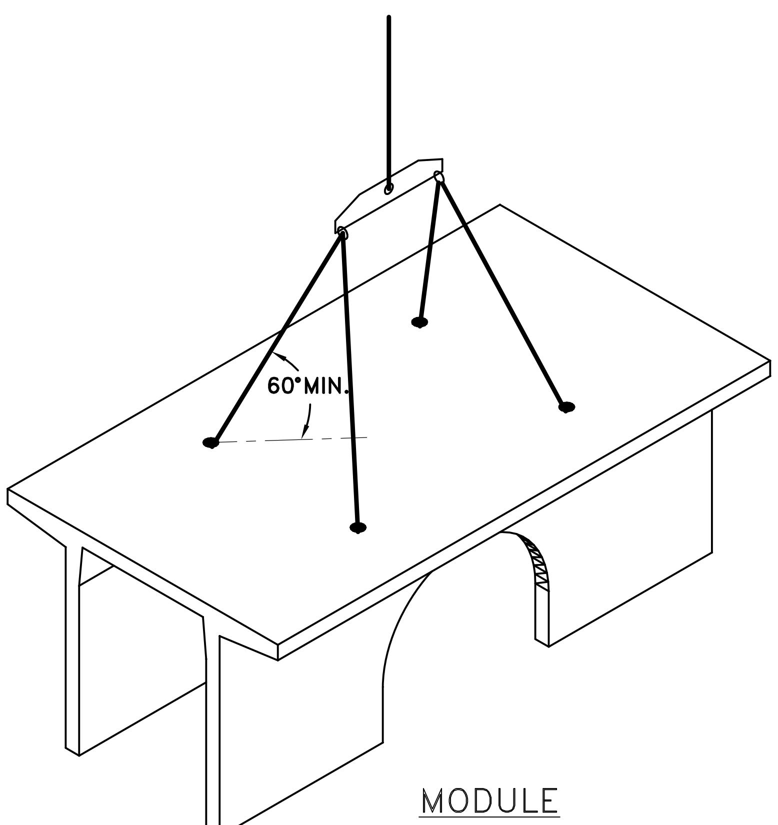
3.0

STORMTRAP MODULE LIFTING INSTALLATION NOTES

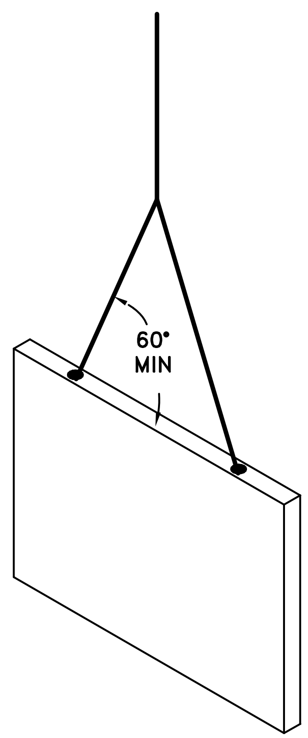
1. IT IS THE CONTRACTOR'S RESPONSIBILITY TO ENSURE THAT ALL (4) CHAINS/CABLES ARE SECURED PROPERLY TO THE LIFTING ANCHORS AND IN EQUAL TENSION WHEN LIFTING THE STORMTRAP MODULE (SEE RECOMMENDATIONS 2 & 3).

2. MINIMUM 7'-0" CHAIN/CABLE LENGTH TO BE USED TO LIFT STORMTRAP MODULES (SUPPLIED BY CONTRACTOR).

3. CONTRACTOR TO ENSURE MINIMUM LIFTING ANGLE IS 60° FROM TOP SURFACE OF STORMTRAP MODULE. SEE DETAIL.



MODULE
LIFTING DETAIL



END PANEL
LIFTING DETAIL

END PANEL ERECTION/INSTALLATION NOTES

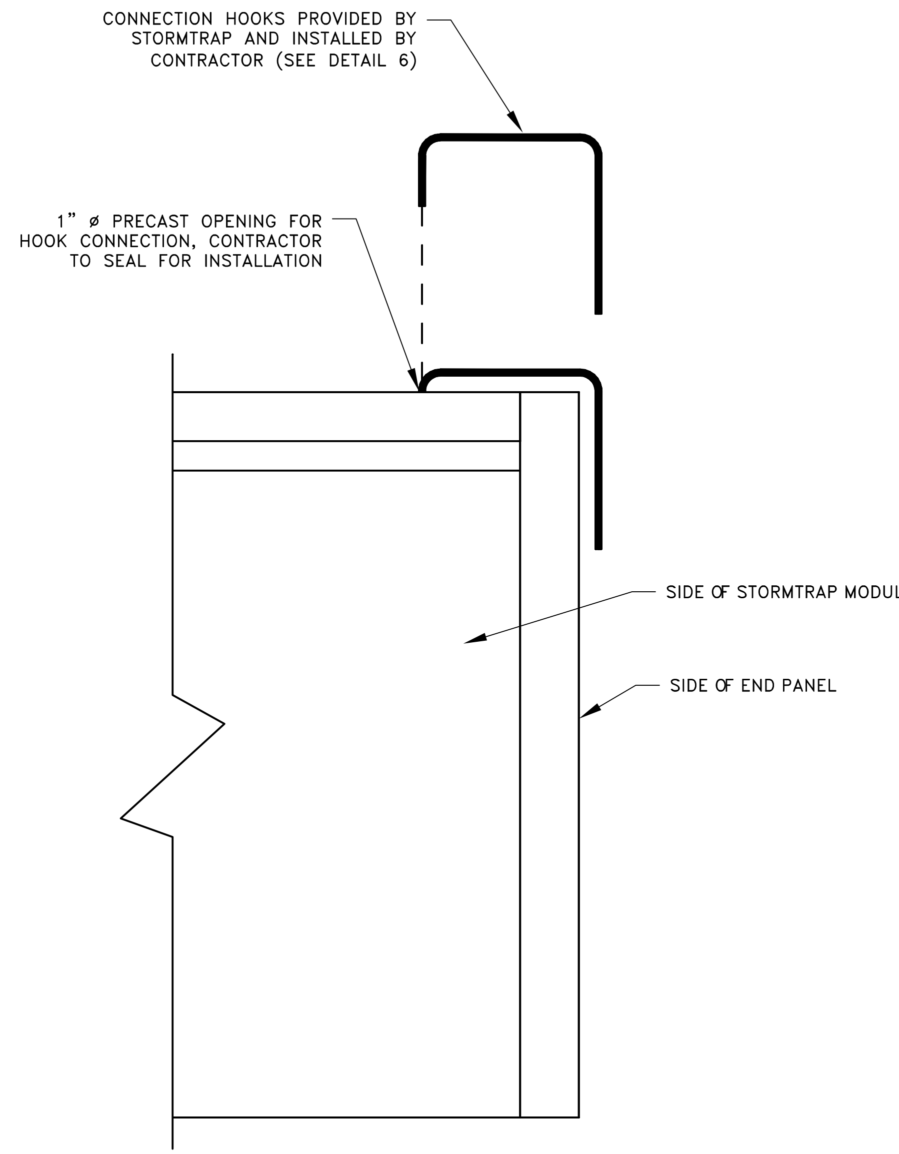
1. END PANELS WILL BE SUPPLIED TO CLOSE OFF OPEN ENDS OF ROWS.

2. PANELS SHALL BE INSTALLED IN A TILT UP FASHION DIRECTLY ADJACENT TO OPEN END OF MODULE (REFER TO SHEET 2.0 FOR END PANEL LOCATIONS).

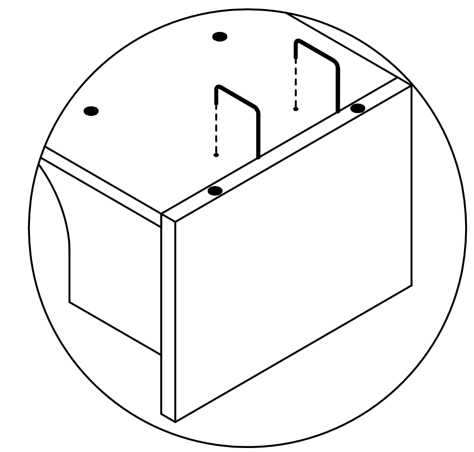
3. CONNECTION HOOKS WILL BE SUPPLIED WITH END PANELS TO SECURELY CONNECT PANEL TO ADJACENT STORMTRAP MODULE (SEE PANEL CONNECTION ELEVATION VIEW).

4. ONCE CONNECTION HOOK IS ATTACHED, LIFTING CLUTCHES MAY BE REMOVED.

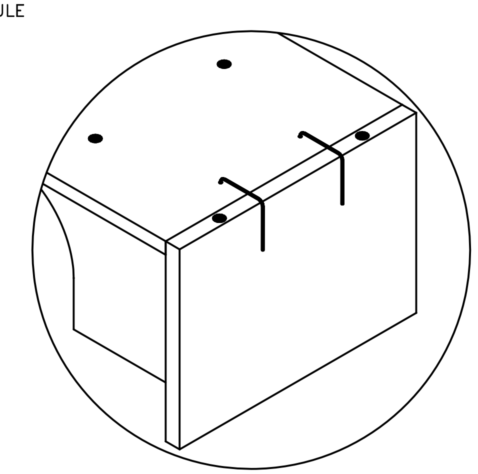
5. JOINT WRAP SHALL BE PLACED AROUND PERIMETER JOINT PANEL (SEE SHEET 3.0).



PANEL CONNECTION
ELEVATION VIEW



STEP 1



STEP 2

DETAIL 6

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SINGLETRAP
INSTALLATION
SPECIFICATIONS

SHEET NUMBER:

3.1

ZONE CHART

ZONES	ZONE DESCRIPTIONS	REMARKS
ZONE 1	FOUNDATION AGGREGATE	#5 (¾") STONE AGGREGATE (SEE NOTE 4 FOR DESCRIPTION)
ZONE 2	BACKFILL	UNIFIED SOILS CLASSIFICATION (GW, GP, SW, SP) OR SEE BELOW FOR APPROVED BACKFILL OPTIONS
ZONE 3	FINAL COVER OVERTOP	MATERIALS NOT TO EXCEED 120 PCF

FILL DEPTH	TRACK WIDTH	MAX VEHICLE WEIGHT (KIPS)	MAX GROUND PRESSURE
12"	12"	51.8	1690 psf
	18"	56.1	1219 psf
	24"	68.1	1111 psf
	30"	76.7	1000 psf
	36"	85.0	924 psf

NOTE:
TRACK LENGTH NOT TO EXCEED 15'-4".
ONLY TWO TRACKS PER VEHICLE.

APPROVED ZONE 2 BACKFILL OPTIONS

OPTION	REMARKS
¾" STONE AGGREGATE	THE STONE AGGREGATE SHALL CONSIST OF CLEAN AND FREE DRAINING ANGULAR MATERIAL. THE SIZE OF THIS MATERIAL SHALL HAVE 100% PASSING THE 1" SIEVE WITH 0% TO 5% PASSING THE #8 SIEVE. THIS MATERIAL SHALL BE SEPARATED FROM NATIVE MATERIAL USING GEOFABRIC AROUND THE PERIMETER OF THE BACKFILL (ASTM SIZE #57) AS DETERMINED BY THE GEOTECHNICAL ENGINEER.
SAND	IMPORTED PURE SAND IS PERMITTED TO BE USED AS BACKFILL IF IT IS CLEAN AND FREE DRAINING. THE SAND USED FOR BACKFILLING SHALL HAVE LESS THAN 40% PASSING #40 SIEVE AND LESS THAN 5% PASSING #200 SIEVE. THIS MATERIAL SHALL BE SEPARATED FROM NATIVE MATERIAL USING GEOFABRIC AROUND THE PERIMETER OF THE SAND BACKFILL.
CRUSHED CONCRETE AGGREGATE	CLEAN, FREE DRAINING CRUSHED CONCRETE AGGREGATE MATERIAL CAN BE USED AS BACKFILL FOR STORMTRAP'S MODULES. THE SIZE OF THIS MATERIAL SHALL HAVE 100% PASSING THE 1" SIEVE WITH 0% TO 5% PASSING THE #8 SIEVE. THIS MATERIAL SHALL BE SEPARATED FROM NATIVE MATERIAL USING GEOFABRIC AROUND THE PERIMETER OF THE BACKFILL.
ROAD PACK	STONE AGGREGATE 100% PASSING THE 1-1/2" SIEVE WITH LESS THAN 12% PASSING THE #200 SIEVE (ASTM SIZE #467). GEOFABRIC AS PER GEOTECHNICAL ENGINEER RECOMMENDATION.

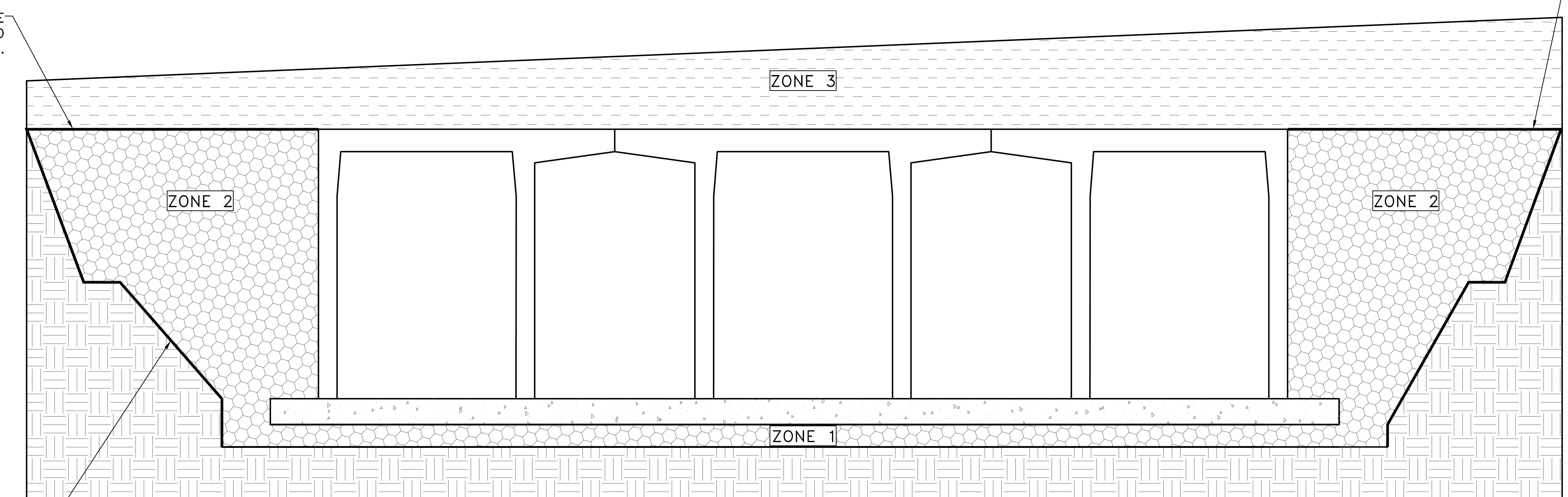
STORMTRAP ZONE INSTALLATION SPECIFICATIONS/PROCEDURES

1. THE FILL PLACED AROUND THE STORMTRAP MODULES MUST DEPOSITED ON BOTH SIDES AT THE SAME TIME AND TO APPROXIMATELY THE SAME ELEVATION. AT NO TIME SHALL THE FILL BEHIND ONE SIDE WALL BE MORE THAN 2'-0" HIGHER THAN THE FILL ON THE OPPOSITE SIDE. BACKFILL SHALL EITHER BE COMPACTED AND/OR VIBRATED TO ENSURE THAT BACKFILL AGGREGATE/STONE MATERIAL IS WELL SEATED AND PROPERLY INTER LOCKED. CARE SHALL BE TAKEN TO PREVENT ANY WEDGING ACTION AGAINST THE STRUCTURE, AND ALL SLOPES WITHIN THE AREA TO BE BACKFILLED MUST BE STEPPED OR SERRATED TO PREVENT WEDGING ACTION. CARE SHALL ALSO BE TAKEN AS NOT TO DISRUPT THE JOINT WRAP FROM THE JOINT DURING THE BACKFILL PROCESS. BACKFILL MUST BE FREE-DRAINING MATERIAL. SEE ZONE 2 BACKFILL CHART ON THIS PAGE FOR APPROVED BACKFILL OPTIONS. IF NATIVE EARTH IS SUSCEPTIBLE TO MIGRATION, CONFIRM WITH GEOTECHNICAL ENGINEER AND PROVIDE PROTECTION AS REQUIRED (PROVIDED BY OTHERS).

2. DURING PLACEMENT OF MATERIAL OVERTOP THE SYSTEM, AT NO TIME SHALL MACHINERY BE USED OVERTOP THAT EXCEEDS THE DESIGN LIMITATIONS OF THE SYSTEM. WHEN PLACEMENT OF MATERIAL OVERTOP, MATERIAL SHALL BE PLACED SUCH THAT THE DIRECTION OF PLACEMENT IS PARALLEL WITH THE OVERALL LONGITUDINAL DIRECTION OF THE SYSTEM WHENEVER POSSIBLE.

3. THE FILL PLACED OVERTOP THE SYSTEM SHALL BE PLACED AT A MINIMUM OF 6" LIFTS. AT NO TIME SHALL MACHINERY OR VEHICLES GREATER THAN THE DESIGN HS-20 LOADING CRITERIA TRAVEL OVERTOP THE SYSTEM WITHOUT THE MINIMUM DESIGN COVERAGE. IF TRAVEL IS NECESSARY OVERTOP THE SYSTEM PRIOR TO ACHIEVING THE MINIMUM DESIGN COVER, IT MAY BE NECESSARY TO REDUCE THE ULTIMATE LOAD/BURDEN OF THE OPERATING MACHINERY SO AS TO NOT EXCEED THE DESIGN CAPACITY OF THE SYSTEM. IN SOME CASES, IN ORDER TO ACHIEVE REQUIRED COMPACTION, HAND COMPACTION MAY BE NECESSARY IN ORDER NOT TO EXCEED THE ALLOTTED DESIGN LOADING. SEE CHART FOR TRACKED VEHICLE WIDTH AND ALLOWABLE MAXIMUM PRESSURE PER TRACK.

4. STONE AGGREGATE FOUNDATION IN ZONE 1 IS RECOMMENDED FOR LEVELING PURPOSES ONLY (OPTIONAL).



BACKFILL DETAIL

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4.0

RECOMMENDED ACCESS OPENING SPECIFICATION

1. A TYPICAL ACCESS OPENING FOR THE STORMTRAP SYSTEM ARE 2'-0" IN DIAMETER. ACCESS OPENINGS LARGER THAN 3'-0" IN DIAMETER NEED TO BE APPROVED BY STORMTRAP. ALL OPENINGS MUST RETAIN AT LEAST 1'-0" OF CLEARANCE FROM THE END OF THE STORMTRAP MODULE UNLESS NOTED OTHERWISE. ALL ACCESS OPENINGS TO BE LOCATED ON INSIDE LEG UNLESS OTHERWISE SPECIFIED.
2. PLASTIC COATED STEEL STEPS PRODUCED BY M.A. INDUSTRIES PART #PS3-PFC OR APPROVED EQUAL (SEE STEP DETAIL) ARE PROVIDED INSIDE ANY MODULE WHERE DEEMED NECESSARY. THE HIGHEST STEP IN THE MODULE IS TO BE PLACED A DISTANCE OF 1'-0" FROM THE INSIDE EDGE OF THE STORMTRAP MODULES. ALL ENSUING STEPS SHALL BE PLACED AT A DISTANCE BETWEEN 10" MIN AND 14" MAX BETWEEN THEM. STEPS MAY BE MOVED OR ALTERED TO AVOID OPENINGS OR OTHER IRREGULARITIES IN THE MODULE.
3. STORMTRAP LIFTING INSERTS MAY BE RELOCATED TO AVOID INTERFERENCE WITH ACCESS OPENINGS OR THE CENTER OF GRAVITY OF THE MODULE AS NEEDED.
4. STORMTRAP ACCESS OPENINGS MAY BE RELOCATED TO AVOID INTERFERENCE WITH INLET AND/OR OUTLET PIPE OPENINGS SO PLACEMENT OF STEPS IS ATTAINABLE.
5. ACCESS OPENINGS SHOULD BE LOCATED IN ORDER TO MEET THE APPROPRIATE MUNICIPAL REQUIREMENTS. STORMTRAP RECOMMENDS AT LEAST TWO ACCESS OPENINGS PER SYSTEM FOR ACCESS AND INSPECTION.
6. USE PRECAST ADJUSTING RINGS AS NEEDED TO MEET GRADE. STORMTRAP RECOMMENDS FOR COVER OVER 2' TO USE PRECAST BARREL OR CONE SECTIONS. (PROVIDED BY OTHERS)

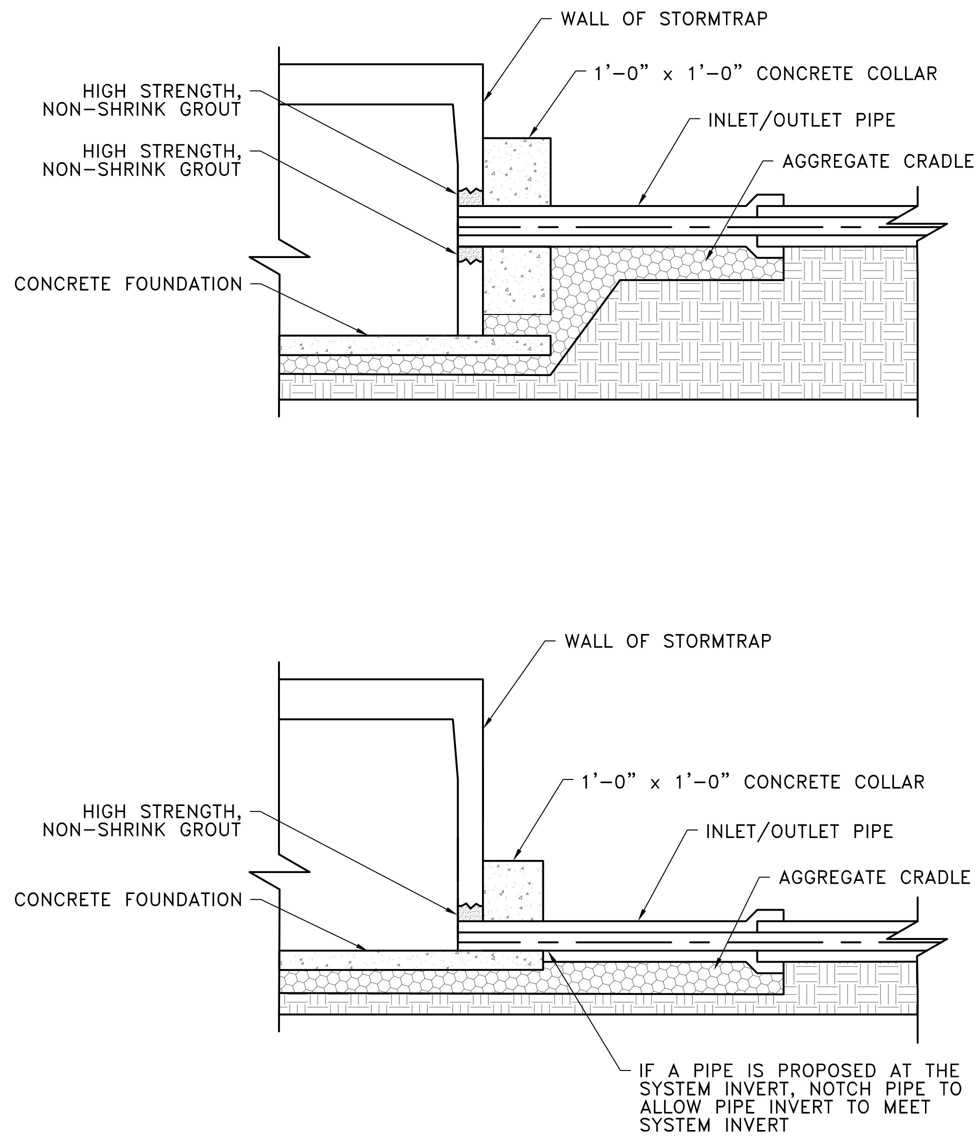
RECOMMENDED PIPE OPENING SPECIFICATION

1. MINIMUM EDGE DISTANCE FOR AN OPENING ON THE OUTSIDE WALL SHALL BE NO LESS THAN 1'-0".
2. MAXIMUM OPENING SIZE TO BE DETERMINED BY THE MODULE HEIGHT. PREFERRED OPENING SIZE ø 36" OR LESS. ANY OPENING NEEDED THAT DOES NOT FIT THIS CRITERIA SHALL BE BROUGHT TO THE ATTENTION OF STORMTRAP FOR REVIEW.
3. CONNECTING PIPES SHALL BE INSTALLED WITH A 1'-0" CONCRETE COLLAR, AND AN AGGREGATE CRADLE FOR AT LEAST ONE PIPE LENGTH (SEE PIPE CONNECTION DETAIL). A STRUCTURAL GRADE CONCRETE OR HIGH STRENGTH, NON-SHRINK GROUT WITH A MINIMUM 28 DAY COMPRESSIVE STRENGTH OF 3000 PSI SHALL BE USED.
4. THE ANNULAR SPACE BETWEEN THE PIPE AND THE HOLE SHALL BE FILLED WITH HIGH STRENGTH NON-SHRINK GROUT.

RECOMMENDED PIPE INSTALLATION INSTRUCTIONS

1. CLEAN AND LIGHTLY LUBRICATE ALL OF THE PIPE TO BE INSERTED INTO STORMTRAP.
2. IF PIPE IS CUT, CARE SHOULD BE TAKEN TO ALLOW NO SHARP EDGES. BEVEL AND LUBRICATE LEAD END OF PIPE.
3. ALIGN CENTER OF PIPE TO CORRECT ELEVATION AND INSERT INTO OPENING.

NOTE: ALL ANCILLARY PRODUCTS/SPECIFICATIONS RECOMMENDED AND SHOWN ON THIS SHEET ARE RECOMMENDATIONS ONLY AND SUBJECT TO CHANGE PER THE INSTALLING CONTRACTOR AND/OR PER LOCAL MUNICIPAL CODE/REQUIREMENTS.



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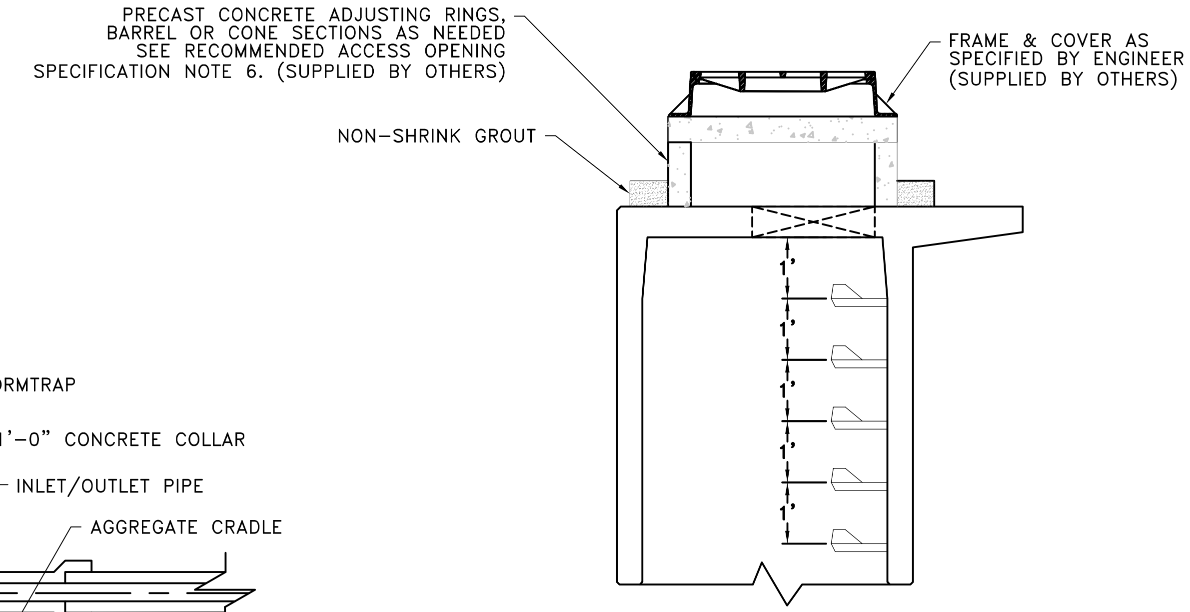
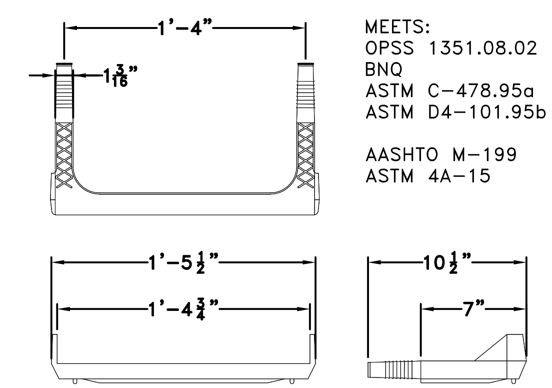


RISER/STAIR DETAIL



STEP DETAIL

StormTrap®

PATENTS LISTED AT: [HTTP://STORMTRAP.COM/PATENT]

1287 WINDHAM PARKWAY
ROMEOVILLE, IL 60446
P:815-941-4549 / F:331-318-5347

ENGINEER INFORMATION:

SDH AND ASSOCIATES

27363 VIA INDUSTRIA
RIVERSIDE, CA
951-683-3691

PROJECT INFORMATION:

LAKE CREEK

PERRIS, CA

CURRENT ISSUE DATE:

6/8/2021

ISSUED FOR:

PRELIMINARY

REV.	DATE:	ISSUED FOR:	DWN BY:
1	6/8/2021	PRELIMINARY	RJL

SCALE:

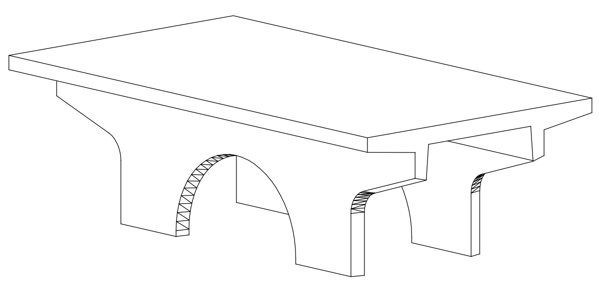
NTS

SHEET TITLE:

RECOMMENDED PIPE / ACCESS OPENING SPECIFICATIONS

SHEET NUMBER:

5.0



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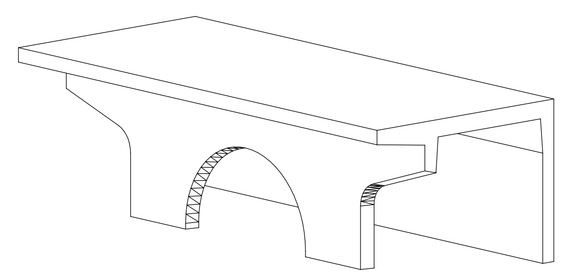

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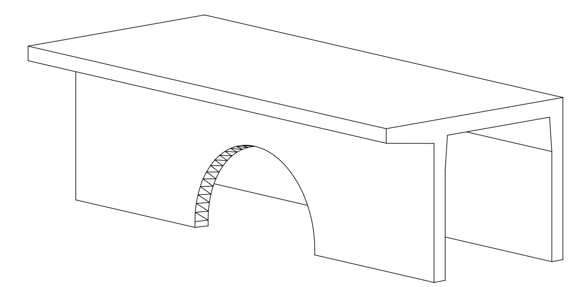
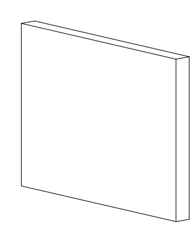


TYPE IV



TYPE IV
END PANEL

NOTES:
1. OPENING LOCATIONS AND SHAPES MAY VARY.
2. SP – INDICATES A MODULE WITH MODIFICATIONS.
3. P – INDICATES A MODULE WITH A PANEL ATTACHMENT.
4. POCKET WINDOW OPENINGS ARE OPTIONAL.

StormTrap®
PATENTS LISTED AT: [HTTP://STORMTRAP.COM/PATENT]

1287 WINDHAM PARKWAY
ROMEOVILLE, IL 60446
P:815-941-4549 / F:331-318-5347

ENGINEER INFORMATION:

SDH AND ASSOCIATES

27363 VIA INDUSTRIA
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951-683-3691

PROJECT INFORMATION:

LAKE CREEK

PERRIS, CA

CURRENT ISSUE DATE:

6/8/2021

ISSUED FOR:

PRELIMINARY

REV.	DATE:	ISSUED FOR:	DWN BY:
1	6/8/2021	PRELIMINARY	RJL

SCALE:

NTS

SHEET TITLE:

SINGLETRAP
MODULE TYPES

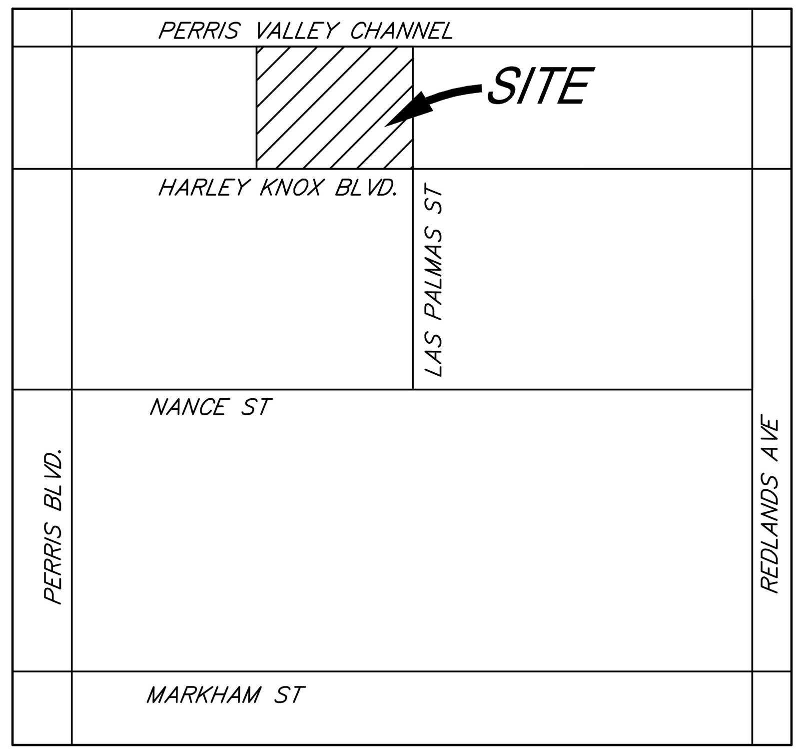
SHEET NUMBER:

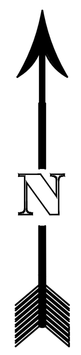
6.0

Appendix 2: Construction Plans

Grading and Drainage Plans

Note: Preliminary site plans are provided.



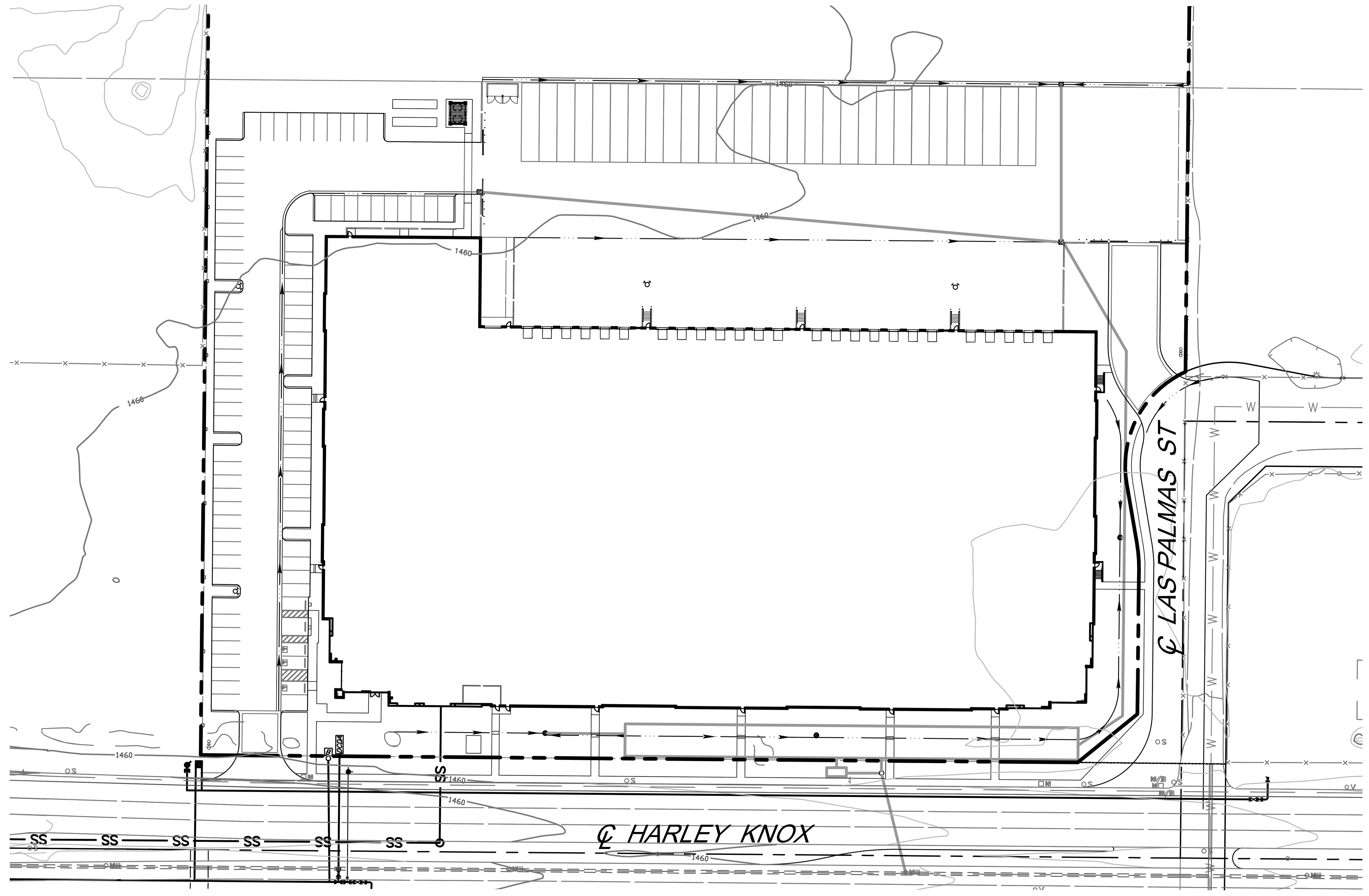


VICINITY MAP
NOT TO SCALE

CITY OF PERRIS

PRELIMINARY GRADING PLAN

SDH & ASSOCIATES, INC.
JUNE 2021



INDEX MAP
SCALE 1"=60'

OWNER/ DEVELOPER
LAKE CREEK INDUSTRIAL, LLC
1302 BRITTANY CROSS CIRCLE
SANTA ANA, CA 90705
PHONE: (949) 910-4616

ENGINEER
SDH & ASSOCIATES, INC
27363 VIA INDUSTRIA
TEMECULA, CA 92590
VOICE: (951) 683-3691
FAX: (951) 788-2314

ARCHITECT
RGA OFFICE OF ARCHITECTURAL DESIGN
15231 ALTON PARKWAY, SUITE 100
IRVINE CA. 92612
PHONE: (949) 341-0920

EARTHWORK
CUT: 7,500 C.Y.
FILL: 7,500 C.Y.

SOURCE OF TOPO
ARROWHEAD MAPPING CORP.
1887 BUSINESS
CENTER DR SUITE 5A
SAN BERNADINO CA. 92408
VOICE: (909) 889-2420
FLOWN: 11-16-19

UTILITY PURVEYORS
WATER............................ E.M.W.D.
GAS................................ SO. CALIF. GAS
ELECTRICAL..................... EDISON
TELEPHONE..................... VERIZON
SEWER............................ CITY OF PERRIS/EMWD
CABLE............................. VERIZON

PROJECT DATA
SITE AREA: 292,606 S.F. (6.71 AC.)
BUILDING AREA: 143,168 S.F.

PARKING INFO
PARKING REQUIRED: 56 SPACES
PARKING PROVIDED: 88 SPACES

HAZARDOUS MATERIALS
NOT IN A FIRE HAZARD ZONE

FEMA FLOOD ZONE DESIGNATION
ZONE D

ZONING AND LAND USE
EXISTING ZONING............ PVCC SP
EXISTING LAND USE........VACANT
PROPOSED ZONING.........PVCC SP
PROPOSED LAND USE.....INDUSTRIAL

LEGEND
T.C.B. - TOP CATCH BASIN
F.G. - FINISHED GRADE
F.L. - FLOW LINE
H.P. - HIGH POINT
EXIST. - EXISTING
P.E. - PAD ELEVATION
G.B. - GRADE BREAK

TRACT BOUNDARY
CENTERLINE
CURB AND GUTTER
1280 EXISTING CONTOUR LINE
LOT LINE
SLOPE

THOMAS BROTHERS INFO.
PAGE: 747, GRID: E7

WATER QUALITY
A PROJECT SPECIFIC WQMP
HAS BEEN PREPARED
FOR THIS PROJECT

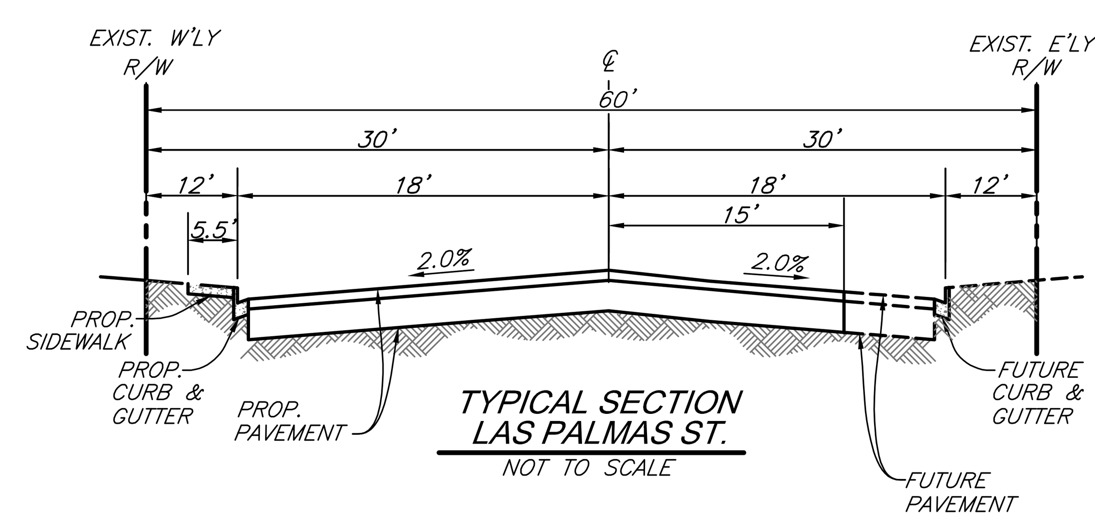
ZONING DISTRIC
NOT IN A ZONING DISTRICT/AREA

APN
302-030-002, 302-100-007

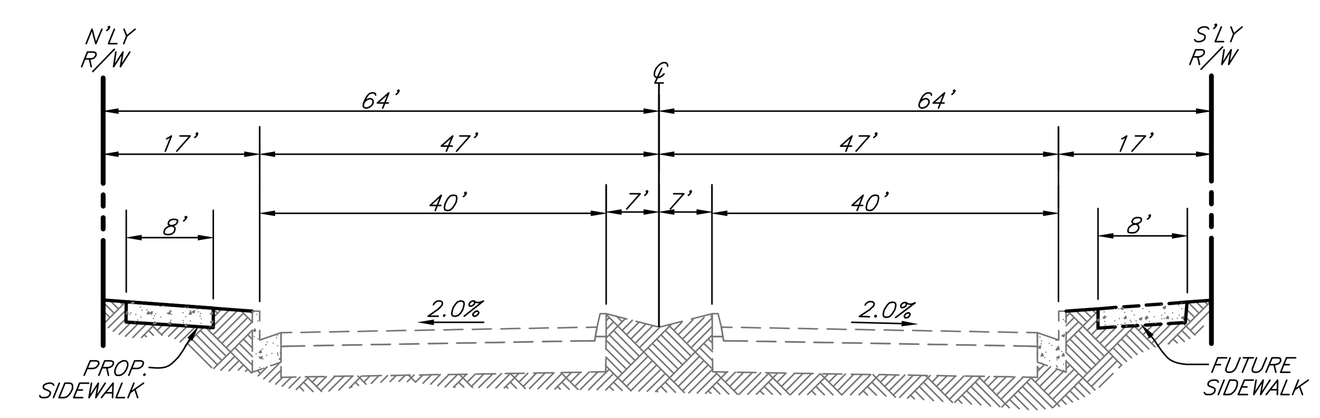
SCHOOL DISTRICT
VAL VERDE UNIFIED

SHEET INDEX
SHEET 1: TITLE SHEET
SHEET 2: PRELIMINARY GRADING PLAN
SHEET 3: SECTIONS AND DETAILS

CONSTRUCTION NOTES
1. CONSTRUCT P.C.C./A.C. 15LE & PARKING AREAS
2. CONSTRUCT 6" CURB ONLY
3. CONSTRUCT 6" CURB AND AND GUTTER
4. CONSTRUCT 3" WIDE CONCRETE RIBBON GUTTER
5. CONSTRUCT P.C.C. SIDEWALK (ONSTE) FINISHES PER ARCH. PLANS
6. CONSTRUCT 6" CURB AND AND GUTTER (OFFSITE) PER COUNTY STDS.
7. CONSTRUCT P.C.C. SIDEWALK (OFFSITE) PER COUNTY STDS.
8. CONSTRUCT A.D.A. COMPLIANT RAMP.
9. CONSTRUCT COMMERCIAL DRIVEWAY APPROACH PER COUNTY STDS.
10. CONSTRUCT 24" CATCH BASIN PER BROOKS STD 2424CB OR APP'D EQUAL
11. CONSTRUCT HDPE STORM DRAIN
12. CONSTRUCT A.C. PAVING (OFFSITE) PER COUNTY/CITY STDS.
13. CONSTRUCT DUAL 3" PVC PIPES UNDER SIDEWALK
14. CONSTRUCT LANDSCAPE DRAIN W/ATRIUM GRATE
15. CONSTRUCT JUNCTION STRUCTURE @ CHANNEL PER RCFC STDS.
16. CONSTRUCT UNDERGROUND DETENTION FACILITY-STORMTRAP SINGLE TRAP (OR EQUIVALENT)
17. CONSTRUCT MODULAR WETLAND SYSTEM (MWS-L-8-12)
18. CONSTRUCT VEGETATED SWALE (PRE-TREATMENT)
19. CONSTRUCT 18" WIDE CONCRETE DITCH
20. CONSTRUCT PRE-TREATMENT CONNECTOR PIPE SCREEN (CPS)



TYPICAL SECTION
LAS PALMAS ST.
NOT TO SCALE



TYPICAL SECTION
HARLEY KNOX BLVD.
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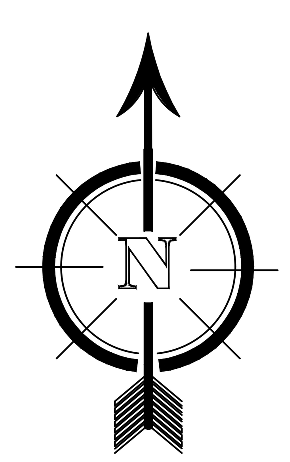

0 30 60 120 180 240
SCALE 1"=60'

PRELIMINARY
NOT FOR CONSTRUCTION

REVISIONS				
MARK	DESCRIPTION	BY	APPR	DATE

DESIGNED BY: S.S. DRAWN BY: S.J.S.
CHECKED BY: R.V.Z. PROJECT MANAGER: S.S.

PLANNING DIVISION: DATE:

PREPARED BY: DATE:
DANE SOMMERS
R.C.E. NO.: 90433 EXP. 9-30-21

SEAL:
REGISTERED PROFESSIONAL ENGINEER
DANE A. SOMMERS
No. 90433
Exp. 9-30-21
CIVIL
STATE OF CALIFORNIA

PREPARED BY:
SDH & ASSOCIATES INCORPORATED
SDH AND ASSOCIATES INC.
27363 VIA INDUSTRIA
TEMECULA, CA 92590
TEL: (951) 683-3691 FAX (951) 788-2314

SCALE: 1" =60'
DATE: JUNE 2021
BENCHMARK

CITY OF PERRIS
LAKE CREEK - HARLEY KNOX
PRELIMINARY GRADING PLAN

1 OF 3 SHEETS

FOR: W.O. CITY FILE NO.

CONSTRUCTION NOTES

1. CONSTRUCT P.C.C./A.C. ISLE & PARKING AREAS
2. CONSTRUCT 6" CURB ONLY
3. CONSTRUCT 6" CURB AND AND GUTTER
4. CONSTRUCT 3" WIDE CONCRETE RIBBON GUTTER
5. CONSTRUCT P.C.C. SIDEWALK (ONSTE) FINISHES PER ARCH. PLANS
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19. CONSTRUCT 18" CONCRETE DITCH
20. CONSTRUCT PRE-TREATMENT CONNECTOR PIPE SCREEN (CPS)

PRELIMINARY
NOT FOR CONSTRUCTION

REVISIONS				
MARK	DESCRIPTION	BY	APPR	DATE

DESIGNED BY: S.S. DRAWN BY: S.J.S.
CHECKED BY: R.V.Z. PROJECT MANAGER: S.S.

PLANNING DIVISION: DATE:

PREPARED BY: DATE:
DANE SOMMERS
R.C.E. NO.: 90433 EXP. 9-30-21

SEAL: REGISTERED PROFESSIONAL ENGINEER DANE A. SOMMERS No. 90433 Exp. 9-30-21 CIVIL STATE OF CALIFORNIA

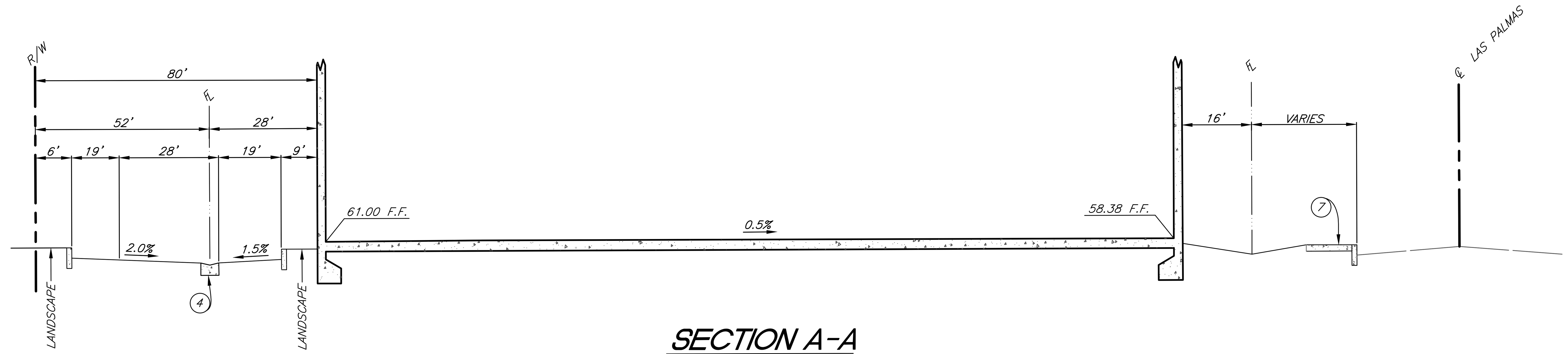
PREPARED BY:
SDH AND ASSOCIATES INC.
27363 VIA INDUSTRIA
TEMECULA, CA 92590
TEL: (951) 683-3691 FAX (951) 788-2314

SCALE: 1"=40'
DATE: JUNE 2021
BENCHMARK

CITY OF PERRIS
LAKE CREEK HARLEY KNOX
PRELIMINARY GRADING PLAN

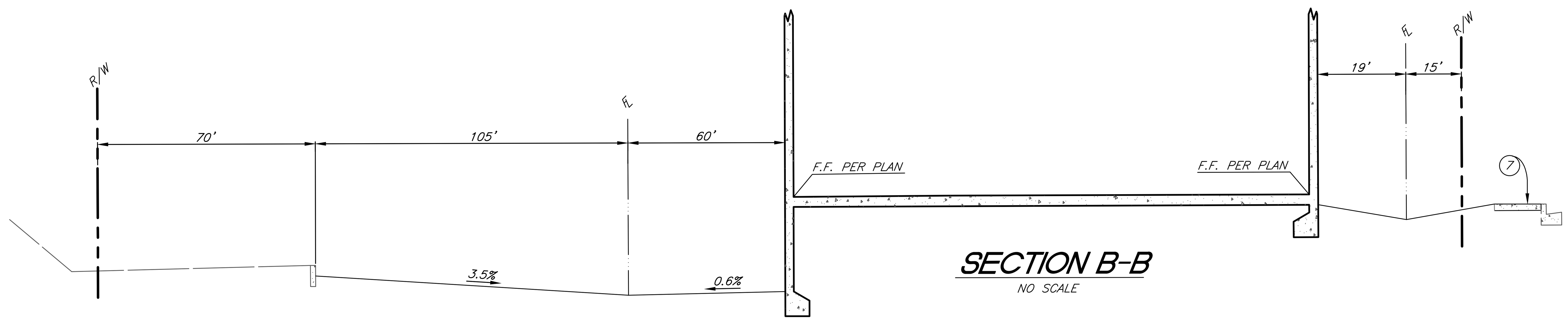
2 OF 3 SHEETS

FOR: W.O. CITY FILE NO.



SECTION A-A

NO SCALE



SECTION B-B

NO SCALE

PRELIMINARY
NOT FOR CONSTRUCTION

REVISIONS				
MARK	DESCRIPTION	BY	APPR	DATE

DESIGNED BY: S.S. DRAWN BY: S.J.S.
CHECKED BY: R.V.Z. PROJECT MANAGER: S.S.

PLANNING DIVISION: DATE: ______

PREPARED BY: DATE: ______

DANE SOMMERS
R.C.E. NO.: 90433 EXP. 9-30-21

SEAL: REGISTERED PROFESSIONAL ENGINEER DANE A. SOMMERS No. 90433 Exp. 9-30-21 CIVIL STATE OF CALIFORNIA

PREPARED BY:
SDH & ASSOCIATES INCORPORATED
SDH AND ASSOCIATES INC.
27363 VIA INDUSTRIA
TEMECULA, CA 92590
TEL: (951) 683-3691 FAX (951) 788-2314

SCALE: NOT TO SCALE
DATE: JUNE 2021
BENCHMARK

CITY OF PERRIS
LAKE CREEK HARLEY KNOX
SECTIONS AND DETAILS

3 OF 3 SHEETS

FOR: W.O. CITY FILE NO.

Appendix 3: Soils Information

Geotechnical Study and Other Infiltration Testing Data

April 19, 2021

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

Lake Creek Industrial, LLC
1302 Brittany Cross Road
Santa Ana, California 92705

Attention: Mr. Eric Mendelson
Senior Associate

Project No.: **21G151-2**

Subject: **Results of Infiltration Testing**
Proposed Warehouse
150 Harley Knox Boulevard
Perris, California

Reference: Geotechnical Investigation, Proposed Warehouse, 150 Harley Knox Boulevard, Perris, California, prepared for Lake Creek Industrial, LLC, by Southern California Geotechnical, Inc. (SCG), SCG Project No. 21G151-1, dated April 19, 2021.

Mr. Mendelson:

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. 20P411R2, dated March 11, 2021. 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 subject site is located on the north side of Harley Knox Boulevard, approximately 500 feet east of Perris Boulevard in Perris, California. The site is also referenced by the street address 150 Harley Knox Boulevard. The site is bounded to the north by a portion of the Perris Valley Storm Drain Channel, to the west by a vacant lot and a commercial property, to the south by Harley Knox Boulevard, and to the east by a single-family residence and a trailer storage lot. The general location of the site is illustrated on the Site Location Map, enclosed as Plate 1 in Appendix A of this report.

The site consists of a rectangular-shaped parcel, 9.32± acres in size. The site is presently vacant and undeveloped. The ground surface consists of exposed soil with moderate native grass and weed growth.

Detailed topographic information was not available at the time of this report. Based on elevations obtained from Google Earth and visual observations made at the time of the subsurface investigation, the site is relatively level with localized undulations of 1 to 2± feet.

Proposed Development

Two preliminary site plans (Scheme A1-07 and A1-2.r) prepared by RGA were provided to our office by the client.

Scheme A1-07

Based on this plan, the site will be developed with one (1) new warehouse, 133,529± ft² in size, located in the south-central area of the site. Dock-high doors will be constructed along a portion of the north building wall. The building is expected to be surrounded by asphaltic concrete (AC) pavements in the parking and drive areas, Portland cement concrete (PCC) pavements in the truck court area, and limited areas of concrete flatwork and landscaped planters throughout.

Detailed structural information has not been provided. It is assumed that the new building will be a single-story structure of tilt-up concrete construction, supported on a conventional shallow foundation system 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.

No significant amounts of below grade construction, such as basements or crawl spaces, are expected to be included in the proposed development. Based on the assumed topography, cuts and fills of up to 1 to 2± feet are expected to be necessary to achieve the proposed site grades.

Scheme A1-2.r

Based on this plan, the site will be developed with one (1) new warehouse, 143,000± ft² in size, located in the south-central area of the site. Dock-high doors will be constructed along a portion of the north building wall. The building is expected to be surrounded by asphaltic concrete pavements in the parking and drive areas, Portland cement concrete pavements in the truck court area, and limited areas of concrete flatwork and landscaped planters throughout.

Detailed structural information has not been provided. It is assumed that the new building will be a single-story structure of tilt-up concrete construction, supported on a conventional shallow foundation system 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.

No significant amounts of below grade construction, such as basements or crawl spaces, are expected to be included in the proposed development. Based on the assumed topography, cuts and fills of up to 1 to 2± feet are expected to be necessary to achieve the proposed site grades.

Infiltration

We understand that the proposed development will include on-site storm water infiltration. The infiltration system will consist of below grade chambers located in the northern area. The bottom of the infiltration system will be 8± feet below the existing site grades.

Concurrent Study

SCG concurrently conducted a geotechnical investigation at the subject site, which is referenced above. As part of this study, five (5) borings were advanced to depths of 20 to 55± feet below existing site grades. Two of these borings were used as a part of a liquefaction evaluation. Artificial fill soils were encountered at the ground surface extending to depths of 2½ to 4½± feet below the existing site grades at all of the boring locations. The fill soils consist of medium dense to dense silty fine sand to fine sandy silts and stiff to very stiff clayey silts and fine sandy clays. Native alluvium was encountered beneath the artificial fill soil at all the boring locations, extending to at least the maximum depth explored of 55± feet below the existing site grades. The alluvium generally consists of interbedded strata of medium dense to dense silty sands, sandy silts and clayey sands, stiff to very stiff clayey silts, and stiff silty clays, with occasional strata of medium dense sands, and stiff to hard sandy clays. Boring No. B-4 encountered a stratum consisting of loose sandy silts at a depth of 12 to 17± feet.

Groundwater

Free water was encountered during drilling at Boring Nos. B-1, B-2, and B-4 at a depth 22± feet below the ground surface. Delayed groundwater level readings were taken at Borings Nos. B-1 and B-4. These groundwater levels were taken after 4 to 6 hours after the drilling was completed and the augers removed. These readings indicated that the groundwater was at a depth of 18 and 20± feet, respectively. Therefore, the static groundwater table is considered to have been present at depths of 18 to 20± feet below the existing site grades at the time of subsurface exploration.

As part of our research, we reviewed available groundwater data in order to determine the historic high groundwater level for the site. The primary reference used to determine the groundwater depths in this area is the California Department of Water Resources website, http://www.water.ca.gov/waterdatalibrary/. The nearest monitoring well is located approximately 2,000 feet east of the site. Water level readings within this monitoring well indicate a high groundwater level of 9± feet below the ground surface in March 2020.

Subsurface Exploration

Scope of Exploration

The subsurface exploration for the infiltration testing consisted of two (2) backhoe-excavated trenches, extending to a depth of 8± 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-2) are indicated on the Infiltration Test Location Plans, enclosed as Plate 2A and Plate 2B of this report.

Geotechnical Conditions

Native alluvium was encountered at the ground surface at all of the infiltration testing locations, extending to at least the maximum explored depth of 8± feet below existing site grades. The alluvial soils consist of medium dense silty fine sands. Variable medium to coarse sand, fine gravel, and clay content were encountered in the alluvial strata. At depth of the proposed infiltration system, some calcareous nodules and veining were also observed in the strata. The Trench Logs, which illustrate the conditions encountered at the infiltration test locations, are presented in 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. As previously mentioned, the infiltration testing was performed in general accordance with ASTM Test Method D-3385-03, Standard Test Method for Infiltration Rate of Soils in Field Using Double Ring Infiltrometer.

Two stainless steel infiltration rings were used for the infiltration testing. The outer infiltration ring is 2 feet in diameter and 20 inches in height. The inner infiltration ring is 1 foot in diameter and 20 inches in height. At 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 trench. The rings were driven into the soil using a ten-pound sledge hammer. The soil surrounding the wall of the infiltration rings was only slightly disturbed during the driving process.

Infiltration Testing Procedure

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

The schedule for readings was determined based on the observed soil type at the base of each backhoe-excavated trench. Based on the existing soils at the trench locations, the volumetric measurements were made at 10-minute increments. The water volume measurements are presented on the spreadsheets enclosed with this report. The infiltration rates for each of the timed intervals are also tabulated on these spreadsheets.

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

Infiltration Test No.	Depth (feet)	Soil Description	Infiltration Rate (inches/hour)
I-1	8	Silty fine Sand, trace Clay, trace medium to coarse Sand, trace fine Gravel	2.4
I-2	8	Silty fine Sand, trace Clay, trace medium to coarse Sand, trace fine Gravel	6.5

Design Recommendations

Two (2) infiltration tests were performed at the subject site. As note above, the calculated infiltration rates at the infiltration test locations range from 2.4 to 6.5 inches per hour. **Based on the results of infiltration testing, we recommend an infiltration rate of 2.4 inches per hour to be used for the design of the proposed infiltration system located in the northern region of the subject site, if the bottom of the infiltration system extends to 10± feet below the existing site grades.**

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

The design of the 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. 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 systems. The presence of such materials would decrease the effective infiltration rates. **It is recommended that the project civil engineer apply an appropriate factor of safety. The infiltration rates recommended above 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 rates are based on infiltration testing at two (2) discrete locations and that the overall infiltration rates of the proposed infiltration systems could vary considerably.

Infiltration Rate Considerations

The infiltration rates presented herein was determined in accordance with the Riverside County guidelines and are considered valid only for the time and place of the actual test. Varying subsurface conditions will exist in other areas of the site, which could alter the recommended infiltration rates presented above. The infiltration rates will decline over time between maintenance cycles as silt or clay particles accumulate on the BMP surface. The infiltration rate is highly dependent upon a number of factors, including density, silt and clay content, grainsize distribution throughout the range of particle sizes, and particle shape. Small changes in these factors can cause large changes in the infiltration rates.

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. Changes in soil moisture content will affect the infiltration rate. 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.

Construction Considerations

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

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

We recommend that scrapers and other rubber-tired heavy equipment not be operated on the basin bottom, or at levels lower than 2 feet above the bottom of the system, particularly within basins. As such, the bottom 24 inches of the infiltration systems should be excavated with non-rubber-tired equipment, such as excavators.

Basin Maintenance

The proposed project may include infiltration basins. Water flowing into these basins will carry some level of sediment. Wind-blown sediments and erosion of the basin side walls will also contribute to sediment deposition at the bottom of the basin. This layer has the potential to significantly reduce the infiltration rate of the basin subgrade soils. Therefore, a formal basin maintenance program should be established to ensure that these silt and clay deposits are removed from the basin on a regular basis. Appropriate vegetation on the basin sidewalls and bottom may reduce erosion and sediment deposition.

Basin maintenance should also include measures to prevent animal burrows, and to repair any burrows or damage caused by such. Animal burrows in the basin sidewalls can significantly increase the risk of erosion and piping failures.

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 area could potentially be damaged due to saturation of the subgrade soils. **The proposed infiltration systems for this site should be located at least 25 feet away from any structures, including retaining walls.** Even with this provision of locating the infiltration system at least 25 feet from the building(s), it is possible that infiltrating water into the subsurface soils could have an adverse effect on the proposed or existing structures. It should also be noted that utility trenches which happen to collect storm water can also serve as conduits to transmit storm water toward the structure, depending on the slope of the utility trench. Therefore, consideration should also be given to the proposed locations of underground utilities which may pass near the proposed infiltration system.

The infiltration system designer should also give special consideration to the effect that the proposed infiltration systems may have on nearby subterranean structures, open excavations, or descending slopes. In particular, infiltration systems should not be located near the crest of descending slopes, particularly where the slopes are comprised of granular soils. Such systems will require specialized design and analysis to evaluate the potential for slope instability, piping failures and other phenomena that typically apply to earthen dam design. This type of analysis is beyond the scope of this infiltration test report, but these factors should be considered by the infiltration system designer when locating the infiltration systems.

Closure

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

Respectfully Submitted,

SOUTHERN CALIFORNIA GEOTECHNICAL, INC.

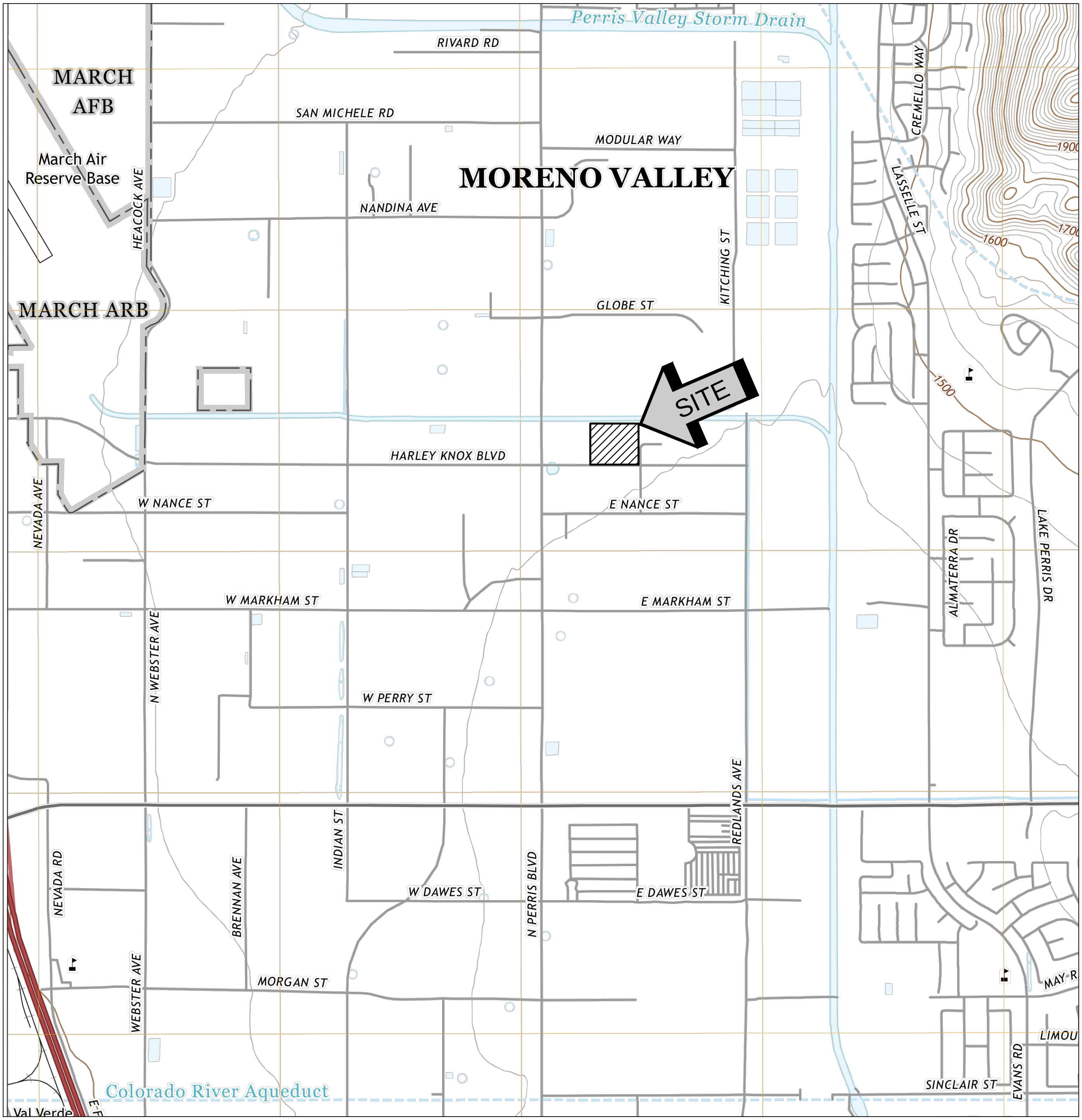
Ryan Bremer
Staff Geologist

Robert G. Trazo, GE 2655
Principal Engineer

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

Distribution: (1) Addressee

Enclosures: Plate 1 - Site Location Map
Plate 2A: Boring Location Plan – Scheme A1-07
Plate 2B: Boring Location Plan – Scheme A1-2.r
Trench Log Legend and Logs (4 pages)
Infiltration Test Results Spreadsheets (2 pages)



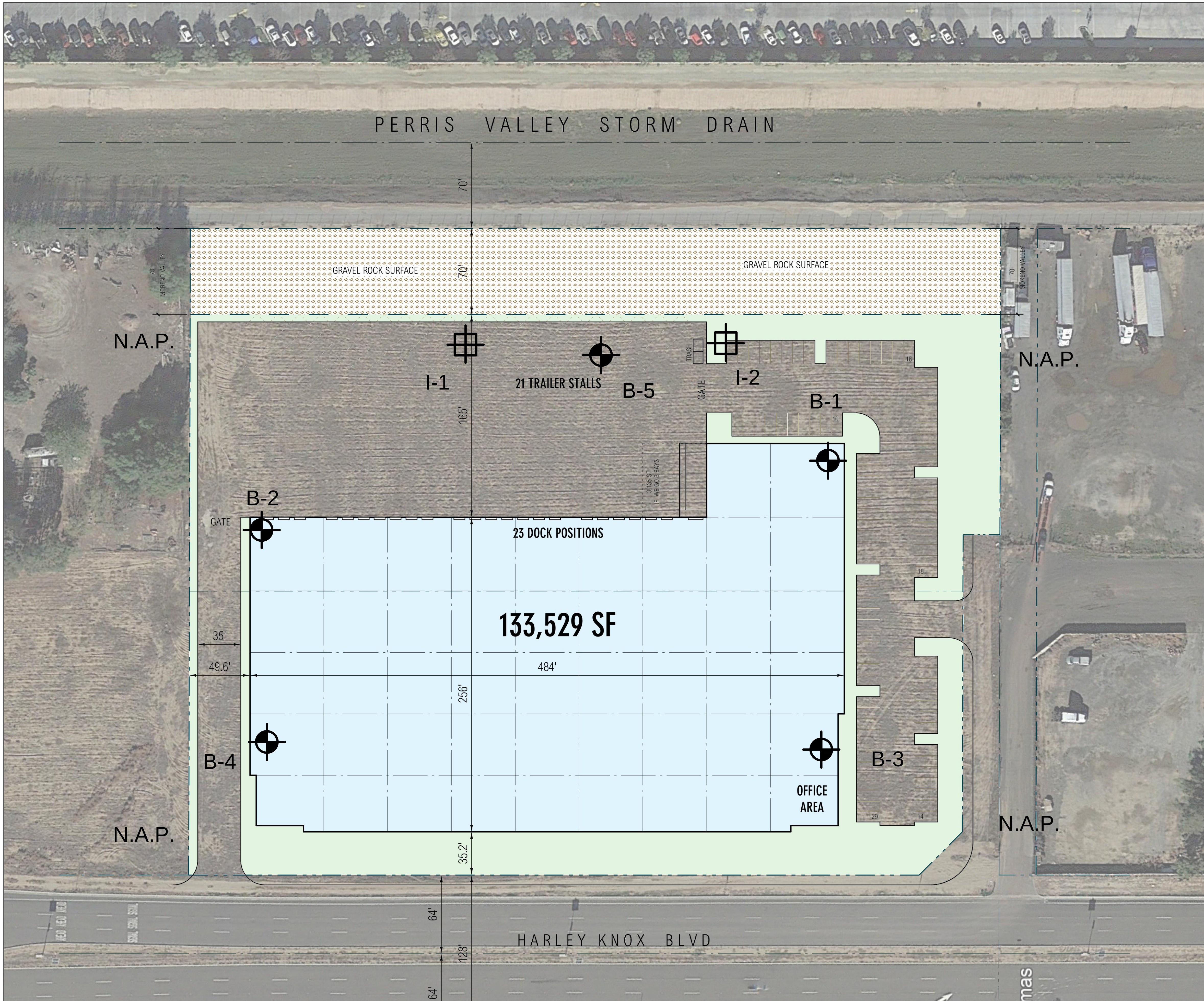
SOURCE: USGS TOPOGRAPHIC MAP OF THE PERRIS QUADRANGLE, RIVERSIDE COUNTY, CALIFORNIA, 2018

SITE LOCATION MAP

PROPOSED WAREHOUSE

PERRIS, CALIFORNIA

SCALE: 1" = 2000'	SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
DRAWN: JLL CHKD: RGT	
SCG PROJECT 21G151-2	
PLATE 1	



NORTH
SoCalGeo

GEOTECHNICAL LEGEND

- APPROXIMATE INFILTRATION TEST LOCATION
- APPROXIMATE BORING LOCATION FROM CONCURRENT STUDY (SCG PROJECT NO. 21G151-1)

NOTE: PRELIMINARY SITE PLAN PREPARED BY RGA.
AERIAL PHOTOGRAPH OBTAINED FROM GOOGLE EARTH.

INFILTRATION TEST LOCATION PLAN - SCHEME A1-07

PROPOSED WAREHOUSE

PERRIS, CALIFORNIA

SCALE: 1" = 80'

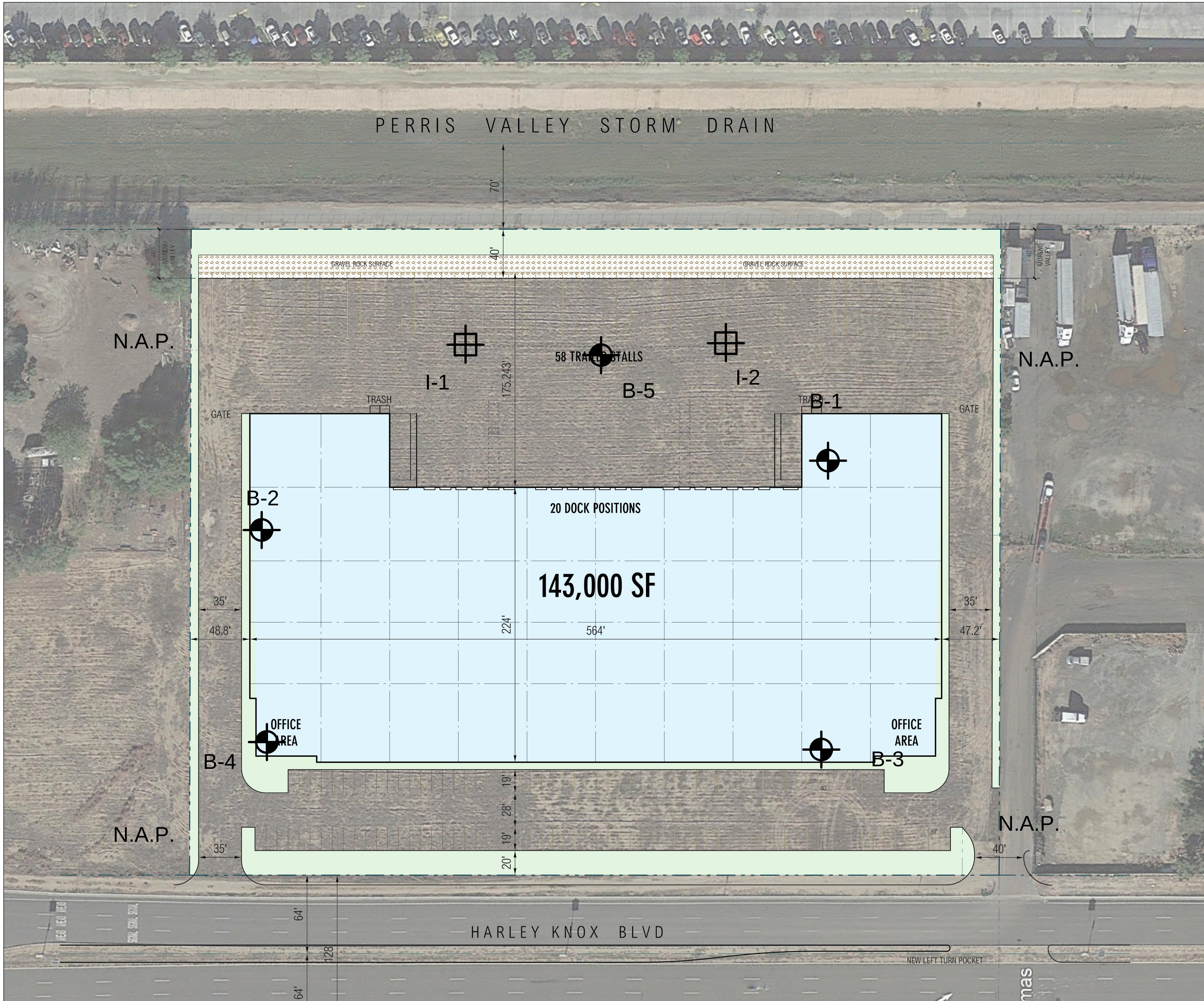
DRAWN: JLL
CHKD: RGT

SCG PROJECT
21G151-2

PLATE 2A

SoCalGeo

SOUTHERN CALIFORNIA GEOTECHNICAL



NORTH
SoCalGeo

GEOTECHNICAL LEGEND

- APPROXIMATE INFILTRATION TEST LOCATION
- APPROXIMATE BORING LOCATION FROM CONCURRENT STUDY (SCG PROJECT NO. 21G151-1)

NOTE: PRELIMINARY SITE PLAN PREPARED BY RGA.
AERIAL PHOTOGRAPH OBTAINED FROM GOOGLE EARTH.

INFILTRATION TEST LOCATION PLAN - SCHEME A1-2.r

PROPOSED WAREHOUSE

PERRIS, CALIFORNIA

SCALE: 1" = 80'

DRAWN: JLL
CHKD: RGT

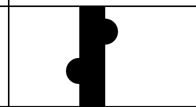
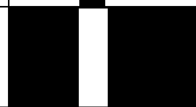
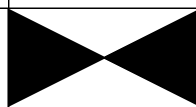
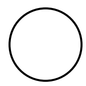
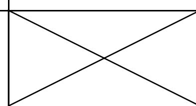
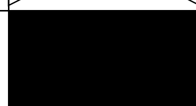
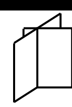
SCG PROJECT
21G151-2

PLATE 2B

SoCalGeo

SOUTHERN CALIFORNIA GEOTECHNICAL

TRENCH 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	GRAVEL AND GRAVELLY SOILS	CLEAN GRAVELS		GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
		(LITTLE OR NO FINES)		GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
	MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	GRAVELS WITH FINES		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
		(APPRECIABLE AMOUNT OF FINES)		GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	SAND AND SANDY SOILS	CLEAN SANDS		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
		(LITTLE OR NO FINES)		SP	POORLY-GRADED SANDS, GRAVELLY SAND, LITTLE OR NO FINES
	MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	SANDS WITH FINES		SM	SILTY SANDS, SAND - SILT MIXTURES
		(APPRECIABLE AMOUNT OF FINES)		SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS	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
MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	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

TRENCH NO. I-1

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

JOB NO.: 21G151-2
PROJECT: Proposed Warehouse
LOCATION: Perris, California

DRILLING DATE: 3/26/21
EXCAVATION METHOD: Hollow Stem Auger
LOGGED BY: Ryan Bremer

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

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					ALLUVIUM: Light Brown Silty fine Sand, trace medium to coarse Sand, trace fine root fibers, medium dense-dry							
5					Light Brown Silty fine Sand, trace Clay, trace medium to coarse Sand, trace fine Gravel, some Calcareous nodules/veining, medium dense-damp							
					Trench Terminated at 8'							

TBL 21G151-2.GPJ SOCALGEO.GDT 4/20/21

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

TRENCH NO. I-2

JOB NO.: 21G151-2
PROJECT: Proposed Warehouse
LOCATION: Perris, California

DRILLING DATE: 3/26/21
EXCAVATION METHOD: Hollow Stem Auger
LOGGED BY: Ryan Bremer

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

FIELD RESULTS					DESCRIPTION	LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					ALLUVIUM: Light Brown Silty fine Sand to fine Sandy Silt, trace medium to coarse Sand, trace fine root fibers, some porosity, medium dense-damp							
5					Light Brown Silty fine Sand, trace Clay, trace medium to coarse Sand, trace fine Gravel, some Calcareous veining/nodules, medium dense-damp							
					Trench Terminated at 8'							

TBL 21G151-2.GPJ SOCALGEO.GDT 4/20/21

TEST TRENCH LOG

INFILTRATION CALCULATIONS

Project Name	Proposed Warehouse
Project Location	Perris, California
Project Number	21G151-2
Engineer	Ryan Bremer

Infiltration Test No: I-1

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

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

Test Interval		Time (hr)	Interval **Elapsed** (min)	Flow Readings: Inner Ring (ml)	Ring Flow (cm^3)	Annular Ring (ml)	Space Flow (cm^3)	Infiltration Rates: Inner Ring* (cm/hr)	Annular Space* (cm/hr)	Inner Ring* (in/hr)	Annular Space* (in/hr)
1	Initial	9:41 AM	10	1000	1400	1400	5200	11.51	14.25	4.53	5.61
	Final	9:51 AM	**10**	2400		6600					
2	Initial	9:54 AM	10	1200	1500	2800	4600	12.33	12.61	4.86	4.96
	Final	10:04 AM	**20**	2700		7400					
3	Initial	10:07 AM	10	900	1050	3700	4300	8.63	11.79	3.40	4.64
	Final	10:17 AM	**30**	1950		8000					
4	Initial	10:35 AM	10	500	800	4800	3300	6.58	9.05	2.59	3.56
	Final	10:45 AM	**40**	1300		8100					
5	Initial	10:47 AM	10	400	750	1500	3800	6.17	10.42	2.43	4.10
	Final	10:57 AM	**50**	1150		5300					
6	Initial	10:59 AM	10	1150	750	2600	4000	6.17	10.96	2.43	4.32
	Final	11:09 AM	**60**	1900		6600					
7	Initial	11:11 AM	10	450	750	2000	3300	6.17	9.05	2.43	3.56
	Final	11:21 AM	**70**	1200		5300					
8	Initial	11:23 AM	10	550	750	1900	3700	6.17	10.14	2.43	3.99
	Final	11:33 AM	**80**	1300		5600					
9	Initial	11:35 AM	10	500	750	1600	3300	6.17	9.05	2.43	3.56
	Final	11:45 AM	**90**	1250		4900					

INFILTRATION CALCULATIONS

Project Name	Proposed Warehouse
Project Location	Perris, California
Project Number	21G151-2
Engineer	Ryan Bremer

Infiltration Test No: I-2

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

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

Test Interval		Time (hr)	Interval **Elapsed** (min)	Flow Readings: Inner Ring (ml)	Ring Flow (cm^3)	Annular Ring (ml)	Space Flow (cm^3)	Infiltration Rates: Inner Ring* (cm/hr)	Annular Space* (cm/hr)	Inner Ring* (in/hr)	Annular Space* (in/hr)
1	Initial	11:58 AM	10	250	2700	1000	8100	22.20	22.20	8.74	8.74
	Final	12:08 PM	**10**	2950		9100					
2	Initial	12:10 PM	10	300	2600	300	7500	21.38	20.56	8.42	8.09
	Final	12:20 PM	**20**	2900		7800					
3	Initial	12:22 PM	10	700	2500	0	7000	20.56	19.19	8.09	7.55
	Final	12:32 PM	**30**	3200		7000					
4	Initial	12:34 PM	10	450	2500	0	6800	20.56	18.64	8.09	7.34
	Final	12:44 PM	**40**	2950		6800					
5	Initial	12:46 PM	10	1050	2550	3100	7400	20.97	20.28	8.26	7.99
	Final	12:56 PM	**50**	3600		10500					
6	Initial	12:58 PM	10	250	2400	1500	7300	19.74	20.01	7.77	7.88
	Final	1:08 PM	**60**	2650		8800					
7	Initial	1:10 PM	10	300	2450	900	7100	20.15	19.46	7.93	7.66
	Final	1:20 PM	**70**	2750		8000					
8	Initial	1:22 PM	10	700	2250	1800	6500	18.50	17.82	7.28	7.01
	Final	1:32 PM	**80**	2950		8300					
9	Initial	1:34 PM	10	450	2000	700	6300	16.45	17.27	6.48	6.80
	Final	1:44 PM	**90**	2450		7000					
10	Initial	1:46 PM	10	500	2000	1200	6400	16.45	17.54	6.48	6.91
	Final	1:56 PM	**100**	2500		7600					

April 22, 2021

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

Lake Creek Industrial, LLC
1302 Brittany Cross Road
Santa Ana, California 92705

Attention: Mr. Eric Mendelson
Senior Associate

Project No.: **21G151-3**

Subject: **Results of Laboratory Testing**
Proposed Warehouse
150 Harley Knox Boulevard
Perris, California

Reference: Geotechnical Investigation, Proposed Warehouse, 150 Harley Knox Boulevard, Perris, California, prepared for Lake Creek Industrial, LLC, by Southern California Geotechnical, Inc. (SCG), SCG Project No. 21G151-1, dated April 19, 2021.

Dear Mr. Mendelson:

As discussed in the referenced report, a representative sample of the near-surface soils was submitted to a subcontracted analytical laboratory for determination of soluble sulfate, electrical resistivity, pH, chloride and nitrate concentrations of the on-site soils. These test results were not available at the time of the referenced report, and are therefore presented in this addendum:

Soluble Sulfates

Soluble sulfates are naturally present in soils, and if the concentration is high enough, can result in degradation of concrete which comes into contact with these soils. The result of the soluble sulfate testing is presented below:

Sample Identification	**Soluble Sulfates (%)**	**ACI Classification**
B-4 @ 0 to 5 feet	0.008	Not applicable (S0)

The results of the soluble sulfate testing indicate that the selected sample of the on-site soils contains a sulfate concentration that corresponds to Class S0 with respect to the American Concrete Institute (ACI) Publication 318-05 Building Code Requirements for Structural Concrete and Commentary, Section 4.3. Therefore, specialized concrete mix designs are not considered to be necessary, with regard to sulfate protection purposes. It is, however, recommended that additional soluble sulfate testing be conducted at the completion of rough grading to verify the soluble sulfate concentrations of the soils which are present at pad grade within the building area.

Corrosivity Testing

The corrosivity testing included a determination of the electrical resistivity, pH, chloride and nitrate concentrations of the on-site soils, as well as other tests. The results of some of these tests are presented below:

Sample Identification	Saturated Resistivity (ohm-cm)	pH	Chlorides (mg/kg)	Nitrates (mg/kg)
B-4 @ 0 to 5 feet	1,560	8.0	54	46

The results of laboratory testing indicate that the tested sample of the on-site soils possesses a saturated resistivity value of 1,560 ohm-cm, and a pH value of 8.0. These test results have been evaluated in accordance with guidelines published by the Ductile Iron Pipe Research Association (DIPRA). The DIPRA guidelines consist of a point system by which characteristics of the soils are used to quantify the corrosivity characteristics of the site. Resistivity and pH are two of the five factors that enter into the evaluation procedure. Redox potential, relative soil moisture content and sulfides are also included. Although sulfide testing was not part of the scope of services for this project, we have evaluated the corrosivity characteristics of the on-site soils using resistivity, pH and moisture content. **Based on these factors, and utilizing the DIPRA procedure, the on-site soils are considered to be severely corrosive to ductile iron pipe. Therefore, polyethylene protection is expected to be required for cast iron or ductile iron pipes.** It should be noted that SCG does not practice in the field of corrosion engineering. **Therefore, the client may also wish to contact a corrosion engineer to provide a more thorough evaluation.**

A relatively low concentration (54 mg/kg) of chlorides was detected in the sample submitted for corrosivity testing. In general, soils possessing chloride concentrations in excess of 500 parts per million (ppm) are considered to be corrosive with respect to steel reinforcement within reinforced concrete. Based on the lack of any significant chlorides in the tested sample, the site is considered to have a C1 chloride exposure in accordance with the American Concrete Institute (ACI) Publication 318 Building Code Requirements for Structural Concrete and Commentary. Therefore, a specialized concrete mix design for reinforced concrete for protection against chloride exposure is not considered warranted.

Nitrates present in soil can be corrosive to copper tubing at concentrations greater than 50 mg/kg. The tested sample possesses a nitrate concentration of 46 mg/kg. Based on this test result, the on-site soils are not considered to be corrosive to copper pipe.

It should be noted that SCG does not practice in the field of corrosion engineering. Therefore, the client may wish to contact a corrosion engineer to provide a more thorough evaluation.

Closure

We sincerely appreciate the opportunity to be of service on this project. If there are any questions concerning this matter, please contact our office at your convenience.

Respectfully Submitted,

SOUTHERN CALIFORNIA GEOTECHNICAL, INC.

Joseph Lozano Leon
Staff Engineer

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

Robert G. Trazo, GE 2655
Principal Engineer

Distribution: (1) Addressee

GEOTECHNICAL INVESTIGATION
PROPOSED WAREHOUSE

150 Harley Knox Boulevard
Perris, California
for
Lake Creek Industrial, LLC

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

April 19, 2021

Lake Creek Industrial, LLC
1302 Brittany Cross Road
Santa Ana, California 92705

Attention: Mr. Eric Mendelson
Senior Associate

Project No.: **21G151-1**

Subject: **Geotechnical Investigation**
Proposed Warehouse
150 Harley Knox Boulevard
Perris, California

Dear Mr. Mendelson:

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.

Joseph Lozano Leon
Staff Engineer

Robert G. Trazo, GE 2655
Principal Engineer

REGISTERED PROFESSIONAL ENGINEER
ROBERT TRAZO
No. 2655
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STATE OF CALIFORNIA

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TABLE OF CONTENTS

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 a zone of high liquefaction susceptibility as mapped by the county of Riverside.
- Our site-specific liquefaction evaluation included two borings extended to depths of 50 to 55± feet. Three (3) potentially liquefiable soil strata were encountered at Boring No. B-1 between depths of 22 and 54½± feet, and four (4) potentially liquefiable soil strata were encountered at depths between 9 and 32± feet at Boring No. B-4. The potential total dynamic settlement at these boing locations is estimated to be 2.63 to 3.46± 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 of this report.
- All of the boring locations encountered artificial fill materials, extending from the ground surface to depths of 2½ to 4½± feet. The fill soils possess varying strengths and densities, and are considered to represent undocumented fill. These soils, in their present condition, are not considered suitable for support of the foundation loads of the new structure.
- These fill soils are underlain by native alluvium which possesses varying strengths and densities. The results of laboratory testing indicate that the near-surface soils within the upper 5 to 6± feet possess a potential for moderate to severe 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.
- Some of the near-surface soils at this site possess a medium expansion potential. Additional design considerations related to expansive soils are presented in this report.

Site Preparation

- Initial site preparation should include stripping of any surficial vegetation. The surficial vegetation, and any organic soils should be properly disposed of off-site.
- Demolition should include utilities and any other subsurface improvements that will not remain in place with the new development. Debris resultant from demolition should be disposed of off-site.
- Remedial grading is recommended to be performed within the proposed building area in order to remove all of the undocumented fill soils in their entirety, the upper portion of the near-surface native alluvial soils, and any soils disturbed during the demolition process. The proposed building area should be overexcavated to a depth of at least 5 feet below existing grade and to a depth of 4 feet below proposed building pad subgrade elevation, whichever is greater. Within the foundation influence zones, the overexcavation should extend to a depth of at least 4 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. The resulting subgrade should then be scarified to a depth of 12 inches and moisture

conditioned (or air dried) 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 potentially liquefiable and medium expansive native alluvial soils. 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 potentially liquefiable and medium expansive native alluvial soils. The actual floor slab reinforcement should be determined by the structural engineer, based upon the imposed loading.

Pavement Design Recommendations

ASPHALT PAVEMENTS (R = 20)					
Materials	**Thickness (inches)**				
	Parking Stalls (TI = 4.0)	Auto Drive Lanes (TI = 5.0)	Truck Traffic (TI = 6.0)	Truck Traffic (TI = 7.0)	Truck Traffic (TI = 8.0)
Asphalt Concrete	3	3	3½	4	5
Aggregate Base	6	8	10	12	14
Compacted Subgrade (90% minimum compaction)	12	12	12	12	12

PORTLAND CEMENT CONCRETE PAVEMENTS (R = 20)				
Materials	**Thickness (inches)**			
	Automobile Parking and Drive Areas (TI = 5.0)	Truck Traffic (TI =6.0)	Truck Traffic (TI =7.0)	Truck Traffic (TI =8.0)
PCC	5	5	5½	7
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. 20P411R2, dated March 11, 2021. 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 subject site is located on the north side of Harley Knox Boulevard, approximately 500 feet east of Perris Boulevard in Perris, California. The site is also referenced by the street address 150 Harley Knox Boulevard. The site is bounded to the north by a portion of the Perris Valley Storm Drain Channel, to the west by a vacant lot and a commercial property, to the south by Harley Knox Boulevard, and to the east by a single-family residence and a trailer storage lot. The general location of the site is illustrated on the Site Location Map, enclosed as Plate 1 in Appendix A of this report.

The site consists of a rectangular-shaped parcel, 9.32± acres in size. The site is presently vacant and undeveloped. The ground surface consists of exposed soil with moderate native grass and weed growth.

Detailed topographic information was not available at the time of this report. Based on elevations obtained from Google Earth and visual observations made at the time of the subsurface investigation, the site is relatively level with localized undulations of 1 to 2± feet.

3.2 Proposed Development

Two preliminary site plans (Scheme A1-07 and A1-2.r) prepared by RGA were provided to our office by the client.

Scheme A1-07

Based on this plan, the site will be developed with one (1) new warehouse, 133,529± ft² in size, located in the south-central area of the site. Dock-high doors will be constructed along a portion of the north building wall. The building is expected to be surrounded by asphaltic concrete (AC) pavements in the parking and drive areas, Portland cement concrete (PCC) pavements in the truck court area, and limited areas of concrete flatwork and landscaped planters throughout.

Detailed structural information has not been provided. It is assumed that the new building will be a single-story structure of tilt-up concrete construction, supported on a conventional shallow foundation system 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.

No significant amounts of below grade construction, such as basements or crawl spaces, are expected to be included in the proposed development. Based on the assumed topography, cuts and fills of up to 1 to 2± feet are expected to be necessary to achieve the proposed site grades.

Scheme A1-2.r

Based on this plan, the site will be developed with one (1) new warehouse, 143,000± ft² in size, located in the south-central area of the site. Dock-high doors will be constructed along a portion of the north building wall. The building is expected to be surrounded by asphaltic concrete pavements in the parking and drive areas, Portland cement concrete pavements in the truck court area, and limited areas of concrete flatwork and landscaped planters throughout.

Detailed structural information has not been provided. It is assumed that the new building will be a single-story structure of tilt-up concrete construction, supported on a conventional shallow foundation system 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.

No significant amounts of below grade construction, such as basements or crawl spaces, are expected to be included in the proposed development. Based on the assumed topography, cuts and fills of up to 1 to 2± feet are expected to be necessary to achieve the proposed site grades.

4.0 SUBSURFACE EXPLORATION

4.1 Scope of Exploration/Sampling Methods

The subsurface exploration conducted for this project consisted of five (5) borings (identified as Boring Nos. B-1 through B-5) advanced to depths of 20 to 55± feet below the existing site grades. Two of these borings were advanced to depths of 50 and 55± feet as a part of the liquefaction evaluation. All of the borings were logged during drilling by a member of our staff.

The borings were advanced with hollow-stem augers, by a conventional truck-mounted drilling rig. Representative bulk and relatively undisturbed soil samples were taken during drilling. Relatively undisturbed soil 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 Plans, included as Plate 2A and Plate 2B in Appendix A of this report. The Boring Logs, which illustrate the conditions encountered at the boring locations, as well as the results of some of the laboratory testing, are included in Appendix B.

4.2 Geotechnical Conditions

Artificial Fill

Artificial fill soils were encountered at the ground surface extending to depths of 2½ to 4½± feet below the existing site grades at all of the boring locations. The fill soils consist of medium dense to dense silty fine sands to fine sandy silts and stiff to very stiff clayey silts and fine sandy clays. The fill soils possessed a disturbed appearance and varying strengths resulting in their classification of artificial fill.

Alluvium

Native alluvium was encountered beneath the artificial fill soil at all the boring locations, extending to at least the maximum depth explored of 55± feet below the existing site grades. The alluvium generally consists of interbedded strata of medium dense to dense silty sands, sandy silts and clayey sands, stiff to very stiff clayey silts, and stiff silty clays, with occasional strata of medium dense sands, and stiff to hard sandy clays. Boring No. B-4 encountered a stratum consisting of loose sandy silts at a depth of 12 to 17± feet.

Groundwater

Free water was encountered during drilling at Boring Nos. B-1, B-2, and B-4 at a depth 22± feet below the ground surface. Delayed groundwater level readings were taken at Borings Nos. B-1 and B-4. These groundwater levels were taken after 4 to 6 hours after the drilling was completed and the augers removed. These readings indicated that the groundwater was at a depth of 18 and 20± feet, respectively. Therefore, the static groundwater table is considered to have been present at depths of 18 to 20± feet below the existing site grades at the time of subsurface exploration.

As part of our research, we reviewed available groundwater data in order to determine the historic high groundwater level for the site. The primary reference used to determine the groundwater depths in this area is the California Department of Water Resources website, http://www.water.ca.gov/waterdatalibrary/. The nearest monitoring well is located approximately 2,000 feet east of the site. Water level readings within this monitoring well indicate a high groundwater level of 9± feet below the ground surface in March 2020.

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. The 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.

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 were 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-4 in Appendix C of this report.

Maximum Dry Density and Optimum Moisture Content

One 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-5 in Appendix C of this report. This test is generally used to compare the in-situ densities of undisturbed field samples, and for later compaction testing. Additional testing of other soil types or soil mixes may be necessary at a later date.

Expansion Index (EI)

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-4 @ 0 to 5 feet	69	Medium

Soluble Sulfates

A representative sample of the near-surface soil was submitted to a subcontracted analytical laboratory for determination of soluble sulfate content. Soluble sulfates are naturally present in soils, and if the concentration is high enough, can result in degradation of concrete which comes into contact with these soils. The result of the soluble sulfate testing is not yet available. This result, along with recommendations for any appropriate sulfate-resistant concrete mix designs will be presented in an addendum report.

Corrosivity Testing

A representative bulk sample of the near-surface soils was submitted to a subcontracted corrosion engineering laboratory to determine if the near-surface soils possess corrosive characteristics with respect to common construction materials. The corrosivity testing included a determination of the electrical resistivity, pH, and chloride concentrations of the soils, as well as other tests. The results of these tests are not yet available. These results, along with any appropriate corrosion protection recommendations will be presented in an addendum report.

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.

Atterberg Limits

Atterberg Limits testing (ASTM D-4318) was performed on selected samples of various soil strata encountered at the site. This test is used to determine the Liquid Limit and Plastic Limit of the soil. The Plasticity Index (PI) is the difference between the two limits. Plasticity Index is a general indicator of the expansive potential of the soil, with higher numbers indicating higher expansive potential. Soils with a PI greater than 25 are considered to have a high plasticity, and a high expansion potential. Soils with a PI greater than 18 are not considered to be susceptible to liquefaction. Soils with a PI between 12 and 18 may possess a moderate susceptibility to liquefaction. The results of the Atterberg Limits testing are presented on the Boring Logs.

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

The potential for other geologic hazards such as seismically induced settlement, lateral spreading, tsunamis, inundation, seiches, flooding, and subsidence affecting the site is considered low.

Seismic Design Parameters

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

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

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

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

2019 CBC SEISMIC DESIGN PARAMETERS

Parameter		Value
Mapped Spectral Acceleration at 0.2 sec Period	S_S	1.500
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}	1.020
Design Spectral Acceleration at 0.2 sec Period	S_{DS}	1.000
Design Spectral Acceleration at 1.0 sec Period	S_{D1}	0.680

*The 2019 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 7-16. However, Section 20.3.1 of ASCE 7-16 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 will be required and additional subsurface exploration will be necessary.

It should be noted that the site coefficient F_v and the parameters S_{M1} and S_{D1} were not included in the SEAOC/OSHPD Seismic Design Maps Tool output for the 2019 CBC. We calculated these parameters-based on Table 1613.2.3(2) in Section 16.4.4 of the 2019 CBC using the value of S_1 obtained from the Seismic Design Maps Tool, assuming that a site-specific ground motion hazards analysis is not required for the proposed building at this site.

Ground Motion Parameters

For the purposes of the liquefaction analysis performed for this study, we utilized a site acceleration consistent with maximum considered earthquake ground motions, as required by the 2019 CBC. The peak ground acceleration (PGA) was determined in accordance with Section 11.8.3 of ASCE 7-16. 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 7-16. The web-based software application SEAOC/OSHPD Seismic Design Maps Tool (described in the previous section) was used to determine PGA_M, which is 0.589g. A portion of the program output is included as Plate E-1 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 mean magnitude is 7.09, 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 high 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, 2014). 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-4 were extended to depths of 55 and 50± feet, respectively. 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 0.589g for a magnitude 7.09 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 9 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.

<u>Conclusions and Recommendations</u>

Potentially liquefiable soils were encountered at both of the 50 to 55±-foot deep boring locations. Three (3) potentially liquefiable soil strata were encountered at Boring No. B-1 between depths of 22 and 54½± feet, and four (4) potentially liquefiable soil strata were encountered at depths between 9 and 32± feet at Boring No. B-4. The remaining soil strata encountered below the historic high groundwater table either possess factors of safety in excess of 1.3, 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 performed for the potentially liquefiable strata. The results of the settlement analyses indicate the following total deformations:

- Boring No. B-1: 2.63 inches
- Boring No. B-4: 3.46 inches

Based on the results of the settlement analyses, differential settlements are expected to be on the order of 1½± inches or less. The estimated differential settlement can be assumed to occur across a distance of 100 feet, indicating a maximum angular distortion of less than 0.002 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 structure would not catastrophically fail. Designing the proposed structure to remain completely undamaged during a major seismic event 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 by 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 techniques or mat foundations.

6.2 Geotechnical Design Considerations

General

All of the boring locations encountered artificial fill materials consisting of medium dense to dense silty fine sands to fine sandy silts and stiff to very stiff clayey silts and fine sandy clays, extending to depths of 2½ to 4½± feet below the existing site grades. Based on a lack of documentation regarding the placement and compaction of the existing fill materials, these soils are considered to consist of undocumented fill, and are not suitable for the support of the foundation loads of the proposed building. These fill soils are underlain by native alluvium which possesses varying strengths and densities. The results of laboratory testing indicate that the near-surface soils within the upper 5 to 6± feet possess a potential for moderate to severe 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 the proposed building pad area.

Settlement

The recommended remedial grading will remove the compressible/collapsible fill soils and near-surface alluvium from the proposed building area, 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

The near-surface soils at this site range from silty sands and sandy silts to clayey silts and sandy clays. Laboratory testing performed on a representative sample of the near surface soils indicates that these materials possess a medium expansion potential (EI = 69). 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. Civil and structural design considerations are presented in Section 6.4 of this report.

Shrinkage/Subsidence

Removal and recompaction of the artificial fill and near-surface native soils is estimated to result in an average shrinkage of 6 to 16 percent. Shrinkage estimates for the individual samples range between 2 and 26 percent based on the results of density testing and the assumption that the onsite soils will be compacted to about 92 percent of the ASTM D-1557 maximum dry density. It should be noted that the shrinkage estimate is based on the results of dry density testing performed on small-diameter samples of the existing soils 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 0.1 feet. This estimate may be used for grading in areas that are underlain by native alluvial soils.

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

Grading and Foundation Plan Review

Grading and foundation plans were not available at the time of this report. It is therefore recommended that we be provided with copies of the preliminary 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 and Demolition

Initial site preparation should include stripping of any surficial vegetation. This includes the removal of native grass and weeds at the site as well as any trees that will not remain with the proposed development. The removal of any trees should also include their associated root masses. These materials should be disposed of off-site. The actual extent of site stripping should be determined in the field by the geotechnical engineer, based on the organic content and stability of the materials encountered.

Demolition of any improvements that will not remain in place for use with the new development will be required at this site. Debris resultant from demolition should be disposed of off-site. All applicable federal, state and local specifications and regulations should be followed in demolition, abandonment, and disposal of the resulting debris.

Treatment of Existing Soils: Building Pad

Remedial grading should be performed within the proposed building area in order to remove the existing undocumented fill soils, and the existing potentially compressible/collapsible native alluvium. It is recommended that the overexcavation extend to a depth of at least 5 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 4 feet below proposed foundation bearing grade.

The overexcavation areas should extend at least 5 feet beyond the building and foundation perimeters, and to an extent equal to the depth of fill placed below the foundation bearing grade, whichever is greater. If the proposed structure incorporates any exterior columns (such as for a canopy or overhang) the area of overexcavation should also encompass these areas.

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

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. 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 area may then be raised to grade with previously excavated soils or imported structural fill.

Treatment of Existing Soils: Retaining Walls and Site Walls

The existing soils within the areas of any proposed retaining walls and site walls should be overexcavated to a depth of 4 feet below foundation bearing grade and replaced as compacted structural fill as discussed above for the proposed building pad. Any undocumented fill soils or disturbed native alluvium within any of these foundation areas should be removed in their entirety. The overexcavation areas should extend at least 4 feet beyond the foundation perimeters, and to an extent equal to the depth of fill below the new foundations. Any erection pads for tilt-up concrete walls are considered to be part of the foundation system. Therefore, these overexcavation recommendations are applicable to erection pads. The overexcavation subgrade soils should be evaluated by the geotechnical engineer prior to scarifying, moisture conditioning to within 2 to 4 percent above the optimum moisture content, and recompacting the upper 12 inches of exposed subgrade soils. The previously excavated soils may then be replaced as compacted structural fill.

If the full lateral recommended remedial grading cannot be completed for the proposed retaining walls and site walls located along property lines, the foundations for those walls should be designed using a reduced allowable bearing pressure. Furthermore, the contractor should take necessary precautions to protect the adjacent improvements during rough grading. Specialized grading techniques, such as A-B-C slot cuts, will likely be required during remedial grading. The geotechnical engineer of record should be contacted if additional recommendations, such as shoring design recommendations, are required during grading.

Treatment of Existing Soils: Parking Areas

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

The geotechnical engineer should then evaluate the subgrade to identify any areas of additional unsuitable soils. Any such materials should be removed to a level of firm and unyielding soil. The exposed subgrade soils should then be scarified to a depth of 12± inches, moisture conditioned to 2 to 4 percent above the optimum moisture content, and recompacted to at least 90 percent of the ASTM D-1557 maximum dry density. Based on the presence of variable strength surficial soils throughout the site, it is expected that some isolated areas of additional overexcavation may be required to remove zones of lower strength, unsuitable soils.

The grading recommendations presented above for the proposed flatwork, parking and drive areas assume that the owner and/or developer can tolerate minor amounts of settlement within these areas. The grading recommendations presented above do not mitigate the extent of undocumented fill or compressible/collapsible native alluvium in the flatwork, parking and drive areas. As such, some 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 flatwork, 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 possible 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 or air dried 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 subject site, it is expected that some isolated areas of additional overexcavation may be required to remove zones of lower strength, unsuitable soils.

As noted previously, the subject site is underlain by medium expansive soils. Support of new flatwork on medium expansive soils carries additional risk with respect to flatwork movement and potential distress. This report provides recommendations for moisture conditioning and additional steel reinforcement in the flatwork areas in order to minimize the potential effects of the expansive soils. However, if additional protection is desired, the client should consider the placement of a 1 to 2-foot thick layer of non-expansive soil beneath all flatwork.

Fill Placement

- Fill soils should be placed in thin (6± inches), near-horizontal lifts, moisture conditioned (or air dried) 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 2019 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 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 (horizontal to vertical) 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.

Any soils used to backfill voids around subsurface utility structures, such as manholes or vaults, should be placed as compacted structural fill. If it is not practical to place compacted fill in these areas, then such void spaces may be backfilled with lean concrete slurry. Uncompacted pea gravel or sand is not recommended for backfilling these voids since these materials have a potential to settle and thereby cause distress of pavements placed around these subterranean structures.

6.4 Construction Considerations

Excavation Considerations

The near-surface soils generally consist of moderate strength silty fine sands to fine sandy silts, clayey silts and fine sandy clays. Some of 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 within on-site clayey soils should not exceed 1.5h: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

The near-surface soils within the subject site have been determined to possess a medium expansion potential. Therefore, care should be given to proper moisture conditioning of all 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 (EI < 50) characteristics. **In**

addition to adequately moisture conditioning the subgrade soils and fill soils during grading, special care must be taken to maintain the 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 new 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 proposed building. If landscaped planters around the building 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 structure. 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 structure 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 off-site.
- 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.

Moisture Sensitive Subgrade Soils

As discussed in Section 6.3 of this report, unstable subgrade soils may be encountered at the base of the overexcavations within the proposed building area. The extent of unstable subgrade soils will, to a large degree. depend on methods used by the contractor to avoid adding additional moisture to these soils or disturbing soils which already possess high moisture contents. If grading occurs during a period of relatively wet weather, an increase in subgrade instability should also be expected. **If unstable subgrade conditions are encountered, it is recommended that only tracked vehicles be used for fill placement and compaction.**

If the construction schedule dictates that site grading will occur during a period of wet weather, allowances should be made for costs and delays associated with drying the on-site soils or import of a drier, less moisture sensitive fill material. Grading during wet or cool weather may also increase the depth of overexcavation in the pad area as well as the need for a stabilization layer.

Groundwater

The groundwater table is considered to exist at a depth between 18 and 20± feet below existing grades. 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 4 feet below foundation bearing grade, underlain by 1± foot of additional soil that has been densified and moisture conditioned in place. Based on this subsurface profile, the proposed structure may be supported on conventional shallow foundations.

Foundation Design Parameters

New square and rectangular footings may be designed as follows:

- Maximum, net allowable soil bearing pressure: 2,500 lbs/ft^2.
- Maximum, net allowable soil bearing pressure: 1,500 lbs/ft^2 if the full recommended lateral extent of remedial grading cannot be achieved.
- 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 medium expansive and potentially liquefiable soils.

- 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. **However, the allowable bearing pressures presented above may not be increased when considering 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 to at least 90 percent of the ASTM D-1557 maximum dry density. Any unsuitable materials should be removed to a depth of suitable bearing compacted structural fill or suitable native alluvium (where reduced bearing pressures are utilized), 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 psi/in.

- Minimum slab reinforcement: No. 3 bars at 18-inches on-center, in both directions, due to presence of medium expansive and potentially liquefiable soils. 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. 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 vary in composition and include silty fine sands, fine sandy silts, clayey silts and fine sandy clays. Based on their composition, the on-site soils have been assigned a friction angle of 28 degrees. It is recommended that the medium expansive soils be excluded from use as retaining wall backfill, where possible.

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

RETAINING WALL DESIGN PARAMETERS

Design Parameter		**Soil Type**
		On-site Soils
Internal Friction Angle (ϕ)		28°
Unit Weight		124 lbs/ft^3
Equivalent Fluid Pressure:	Active Condition (level backfill)	45 lbs/ft^3
	Active Condition (2h:1v backfill)	78 lbs/ft^3
	At-Rest Condition (level backfill)	66 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 2019 CBC, any retaining walls more than 6 feet in height must be designed for seismic lateral earth pressures. If walls 6 feet or more are required for this site, the geotechnical engineer should be contacted for supplementary seismic lateral earth pressure recommendations.

Retaining Wall Foundation Design

The retaining wall foundations should be underlain by at least 4 feet of newly placed structural fill. Foundations to support new retaining walls should be designed in accordance with the general Foundation Design Parameters presented in a previous section of this report.

Backfill Material

On-site soils may be used to backfill the retaining walls, 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 silty sand, clayey sands, sandy silts, clayey silts and sandy clays. These soils are generally considered to possess poor to fair pavement support characteristics with estimated R-values ranging from 15 to 30. The subsequent pavement design is therefore based upon an assumed R-value of 20. 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

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 = 20)					
Materials	**Thickness (inches)**				
	Parking Stalls (TI = 4.0)	Auto Drive Lanes (TI = 5.0)	Truck Traffic		
			(TI = 6.0)	(TI = 7.0)	(TI = 8.0)
Asphalt Concrete	3	3	3½	4	5
Aggregate Base	6	8	10	12	14
Compacted Subgrade (90% minimum compaction)	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 batch plant-reported maximum density. 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 = 20)				
Materials	**Thickness (inches)**			
	Automobile Parking and Drive Areas (TI = 5.0)	Truck Traffic		
		(TI =6.0)	(TI =7.0)	(TI =8.0)
PCC	5	5	5½	7
Compacted Subgrade (95% minimum compaction)	12	12	12	12

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

7.0 GENERAL COMMENTS

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

The analysis of this site was based on a subsurface profile interpolated from limited discrete soil samples. While the materials encountered in the project area are considered to be representative of the total area, some variations should be expected between 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

California Division of Mines and Geology (CDMG), "Guidelines for Evaluating and Mitigating Seismic Hazards in California," State of California, Department of Conservation, Division of Mines and Geology, Special Publication 117A, 2008.

Idriss, I. M. and Boulanger, R. W., "Soil Liquefaction During Earthquakes," Earthquake Engineering Research Institute, 2008.

Idriss, I. M. and Boulanger, R. W., "SPT-Based Liquefaction Triggering Procedures," Earthquake Engineering Research Institute, 2010.

Boulanger, R. W. and Idriss, I. M., "CPT and SPT Based Liquefaction Triggering Procedures," Earthquake Engineering Research Institute, 2014.

National Research Council (NRC), "Liquefaction of Soils During Earthquakes," Committee on Earthquake Engineering, National Research Council, Washington D. C., Report No. CETS-EE-001, 1985.

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.

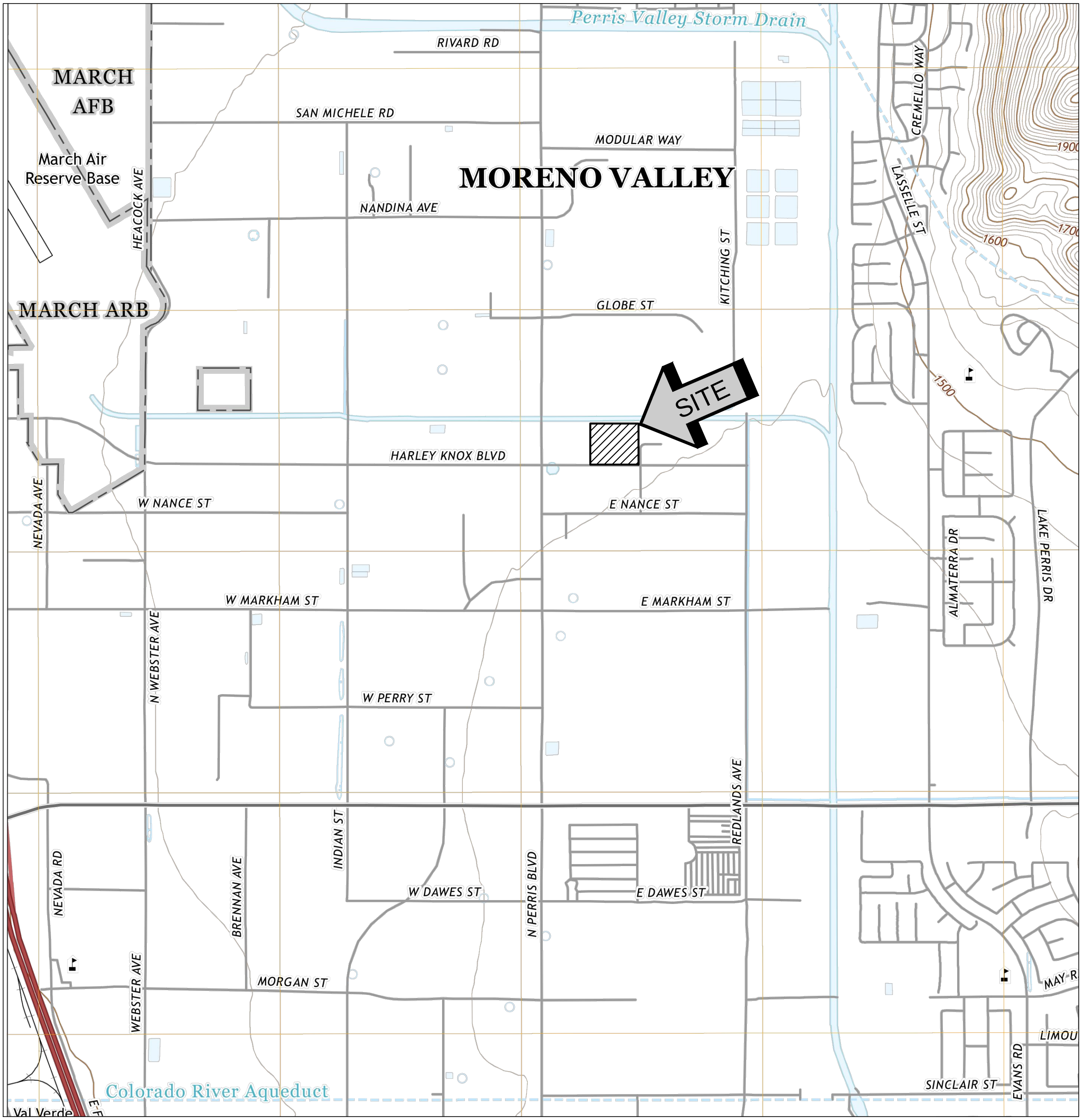
Southern California Earthquake Center (SCEC), University of Southern California, "Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Liquefaction in California," Committee formed 1997.

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.

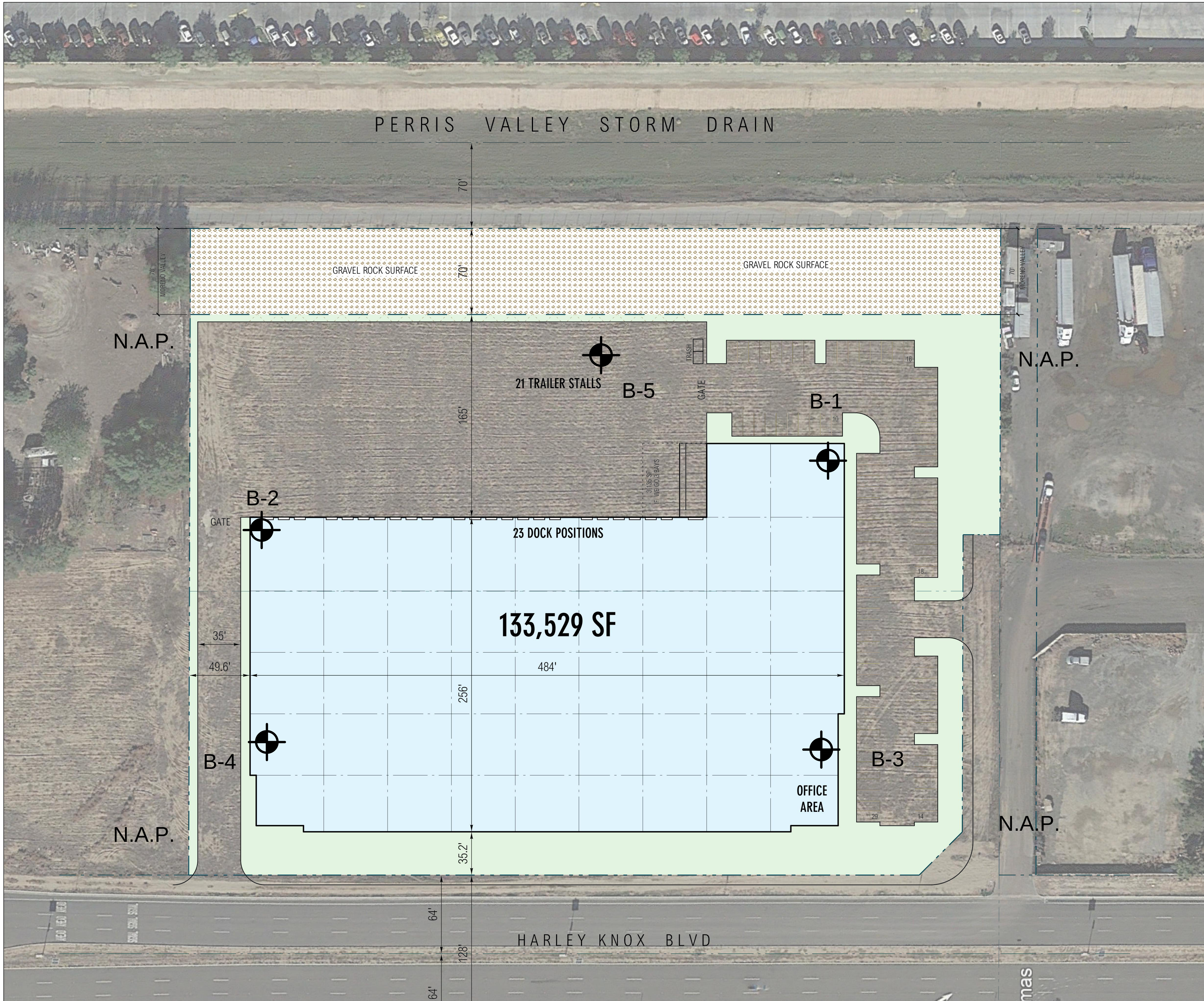
Youd, T. L. and Idriss, I. M. (Editors), "Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils," Salt Lake City, UT, January 5-6 1996, NCEER Technical Report NCEER-97-0022, Buffalo, NY.

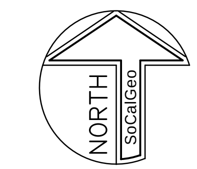
APPENDIX A



SOURCE: USGS TOPOGRAPHIC MAP OF THE PERRIS QUADRANGLE, RIVERSIDE COUNTY, CALIFORNIA, 2018

SITE LOCATION MAP	
PROPOSED WAREHOUSE	
PERRIS, CALIFORNIA	
SCALE: 1" = 2000'	SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
DRAWN: JLL CHKD: RGT	
SCG PROJECT 21G151-1	
PLATE 1	



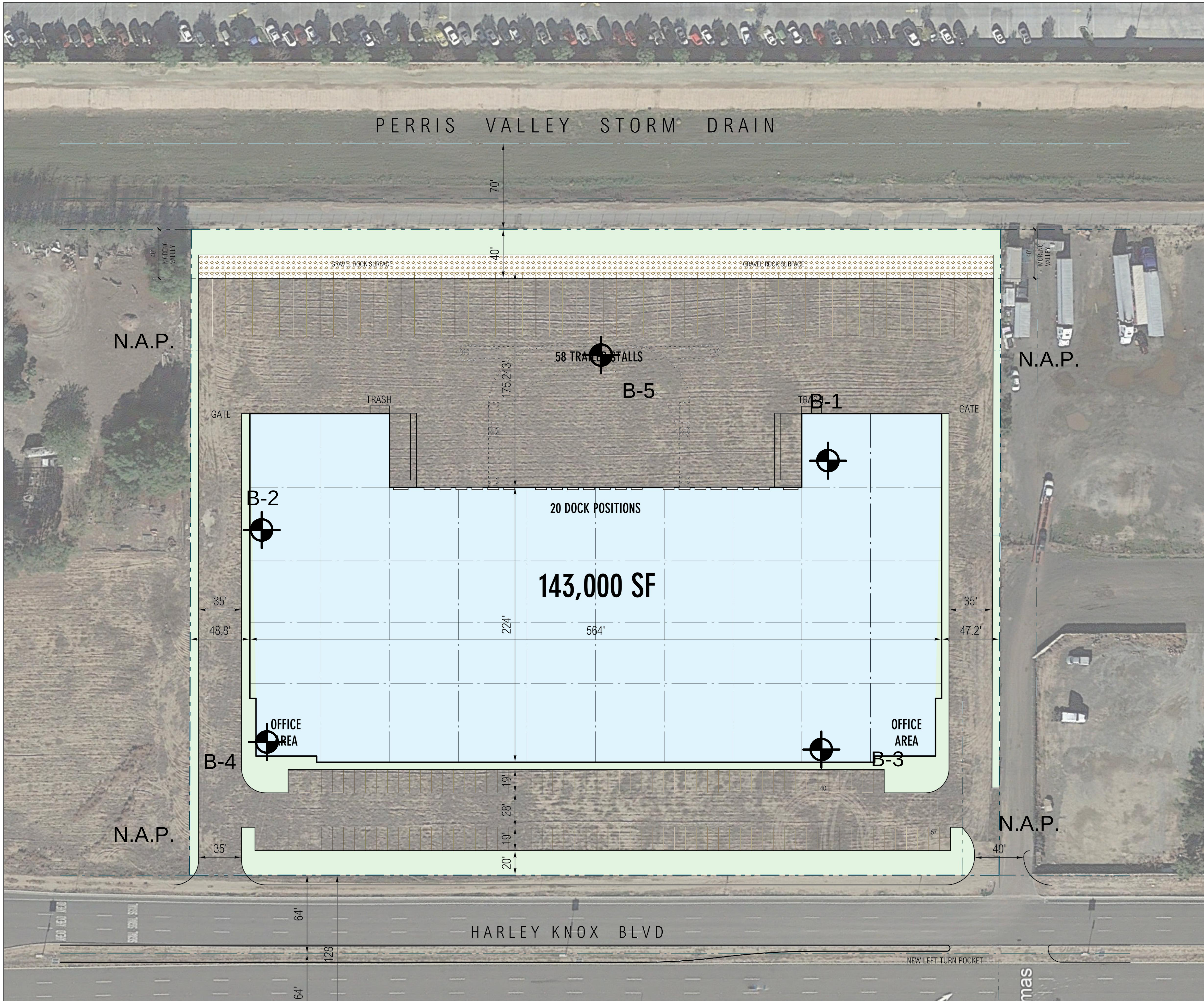


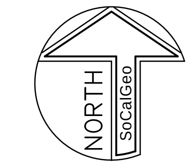
GEOTECHNICAL LEGEND

APPROXIMATE BORING LOCATION

NOTE: PRELIMINARY SITE PLAN PREPARED BY RGA.
AERIAL PHOTOGRAPH OBTAINED FROM GOOGLE EARTH.

BORING LOCATION PLAN - SCHEME A1-07	
PROPOSED WAREHOUSE	
PERRIS, CALIFORNIA	
SCALE: 1" = 80'	SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
DRAWN: JLL CHKD: RGT	
SCG PROJECT 21G151-1	
PLATE 2A	





GEOTECHNICAL LEGEND

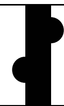
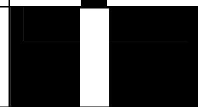
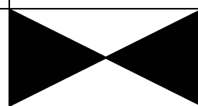
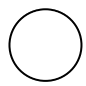
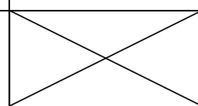
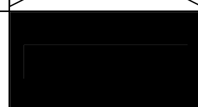
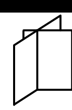
APPROXIMATE BORING LOCATION

NOTE: PRELIMINARY SITE PLAN PREPARED BY RGA.
AERIAL PHOTOGRAPH OBTAINED FROM GOOGLE EARTH.

BORING LOCATION PLAN - SCHEME A1-2.r	
PROPOSED WAREHOUSE	
PERRIS, CALIFORNIA	
SCALE: 1" = 80'	SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
DRAWN: JLL CHKD: RGT	
SCG PROJECT 21G151-1	
PLATE 2B	

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

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-1

JOB NO.: 21G151-1	DRILLING DATE: 3/24/21	WATER DEPTH: 18 feet
PROJECT: Proposed Warehouse	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 20 feet
LOCATION: Perris, California	LOGGED BY: Daryl Kas	READING TAKEN: 6 hrs After Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					FILL: Brown Silty fine Sand, very dense-damp							
	X	72					6					
					ALLUVIUM: Brown Silty fine to medium Sand, trace coarse Sand, medium dense-damp							
	X	17					4					
5					Brown fine to medium Sand, little coarse Sand, trace Silt, medium dense-damp		3					
					Gray Brown Clayey Silt, trace fine Sand, trace Calcareous veining/nodules, very stiff-moist							
	X	23	4.5				19					
					Gray Brown Silty Clay, little fine Sand, abundant Calcareous nodules, stiff-very moist to wet							
	X	11	3.0				43			84		
10												
	X	11	3.5				43	63	37			
15												
					Brown Clayey fine Sand, medium dense-wet							
	X	28					15			44		
20												
					Brown Silty fine Sand, trace Clay, medium dense-wet							
	X	11					22			36		
25												
					Brown fine Sandy Silt, trace Clay, medium dense-wet							
	X	22					23			58		
30												
					Brown Silty fine Sand, trace Clay, medium dense-wet							
	X	15					19			40		

TBL 21G151-1.GPJ SOCALGEO.GDT 4/20/21

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-1

JOB NO.: 21G151-1
PROJECT: Proposed Warehouse
LOCATION: Perris, California

DRILLING DATE: 3/24/21
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Daryl Kas

WATER DEPTH: 18 feet
CAVE DEPTH: 20 feet
READING TAKEN: 6 hrs After Completion

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 (%)	ORGANIC CONTENT (%)	COMMENTS
					Gray Brown Silty Clay, little fine Sand, abundant Calcareous nodules, stiff-wet							
	X	16	2.0				42	53	31	82		
40												
	X	13	2.0				39	51	28	85		
45												
					Gray Brown Silty Clay, little fine Sand, stiff-wet							
	X	20	2.0				26	30	17	79		
50												
					Light Brown fine to coarse Sand, trace Silt, medium dense-wet							
	X	20					19			10		
					Gray Brown fine Sandy Silt to Silty fine Sand, trace Clay, medium dense-wet		18			50		
55					Boring Terminated at 55'							

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SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-2

JOB NO.: 21G151-1
PROJECT: Proposed Warehouse
LOCATION: Perris, California

DRILLING DATE: 3/24/21
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Daryl Kas

WATER DEPTH: 22 feet
CAVE DEPTH: 23 feet
READING TAKEN: At Completion

SURFACE ELEVATION: --- MSL

DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	DESCRIPTION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
				FILL: Light Gray Brown fine Sandy Silt, medium dense to very dense-damp to very moist							
	X	39			97	10					
	X	93			82	16					
				ALLUVIUM: Light Gray Brown fine Sandy Silt, abundant Calcareous nodules/veining, dense-moist to very moist							
5	X	52			78	15					
				Light Gray Brown Clayey fine Sand, trace Silt, medium dense-moist to very moist							
	X	22			78	18					
				Light Gray Brown fine Sandy Silt, trace Calcareous nodules/veining, medium dense-moist							
	X	34			106	12					
10				Brown Clayey fine Sand, little medium Sand, medium dense-damp							
				Gray Brown fine Sandy Clay, little Calcareous nodules, hard-very moist							
	X	56	4.5		96	30					
15											
				Gray Brown fine to coarse Sand, little Silt, medium dense to very dense-very moist to wet							
	X	39			122	11					
20											
	X	47			119	12					
25											

Boring Terminated at 25'

TBL 21G151-1.GPJ SOCALGEO.GDT 4/20/21

TEST BORING LOG

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-3

JOB NO.: 21G151-1
PROJECT: Proposed Warehouse
LOCATION: Perris, California

DRILLING DATE: 3/24/21
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Daryl Kas

WATER DEPTH: Dry
CAVE DEPTH: 21 feet
READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					FILL: Light Gray Brown fine Sandy Silt, dense-damp							
	X	74				106	7					
					ALLUVIUM: Brown Silty fine Sand, dense-damp							
	X	46				110	6					
5					Brown Clayey fine Sand, trace Silt, trace Calcareous veining/nodules, medium dense-damp							
	X	29				107	7					
					Brown Clayey Silt, trace fine Sand, trace Calcareous nodules/veining, very stiff-moist to very moist							
	X	22	4.5			102	18					
	X	33	4.5			96	27					
10												
					Brown Silty Clay, little fine Sand, abundant Calcareous nodules/veining, very stiff-very moist to wet							
	X	27	4.5			74	51					
15												
					Gray Brown Silty fine to medium Sand, trace Clay, medium dense-very moist							
	X	20				112	14					
20												
					Gray Brown fine Sandy Silt, trace to little Clay, medium dense-very moist							
	X	20				101	23					
25												
					Boring Terminated at 25'							

TBL 21G151-1.GPJ SOCALGEO.GDT 4/20/21

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-4

JOB NO.: 21G151-1
PROJECT: Proposed Warehouse
LOCATION: Perris, California

DRILLING DATE: 3/24/21
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Daryl Kas

WATER DEPTH: 20 feet
CAVE DEPTH: 22 feet
READING TAKEN: 4 hrs After Completion

SURFACE ELEVATION: --- MSL

FIELD RESULTS: DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION	LABORATORY RESULTS: DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					FILL: Gray Brown Clayey Silt, little fine Sand, stiff-damp							
		27					12					EI = 69 @ 0 to 5 feet
					ALLUVIUM: Light Gray Brown Silty fine Sand to fine Sandy Silt, trace to little Clay, abundant Calcareous nodules/veining, medium dense-moist to very moist							
		20					15					
5												
		10					28					
		14					26			50		
10												
					Gray Brown fine Sandy Silt, trace Clay, abundant Calcareous veining/nodules, loose-very moist							
		9					28			64		
15												
					Brown fine Sandy Clay to Clayey fine Sand, little Silt, very stiff-moist to wet							
		25	4.0				13			53		
20												
					Brown Silty fine Sand, trace Clay, medium dense-wet							
		15					15	25	19	35		
25												
					Gray Brown Clayey fine Sand, little Silt, medium dense-wet							
		15					16			41		
30												
					Brown Clayey fine Sand to fine Sandy Clay, little Silt, medium dense to stiff-wet							
		18	4.0				16 23	23	15	52 51		
					Brown Silty fine Sand to fine Sandy Silt, little Clay, trace							

TBL 21G151-1.GPJ SOCALGEO.GDT 4/20/21

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-4

JOB NO.: 21G151-1
PROJECT: Proposed Warehouse
LOCATION: Perris, California

DRILLING DATE: 3/24/21
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Daryl Kas

WATER DEPTH: 20 feet
CAVE DEPTH: 22 feet
READING TAKEN: 4 hrs After Completion

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 (%)	ORGANIC CONTENT (%)	COMMENTS
					medium Sand, medium dense-wet							
					Gray Brown Clayey fine Sand, abundant Calcareous nodules/veining, dense-wet							
	X	41			Brown Silty fine Sand, trace Clay, dense-wet		19 14			45 33		
40												
					Gray Brown Silty Clay, trace fine Sand, trace Calcareous nodules/veining, stiff-wet							
	X	21	2.5				30			95		
45												
					Gray Brown fine Sandy Clay, little Silt, stiff-very moist							
	X	12	2.5				36	52	24	66		
50												
					Boring Terminated at 50'							

TBL 21G151-1.GPJ SOCALGEO.GDT 4/20/21

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-5

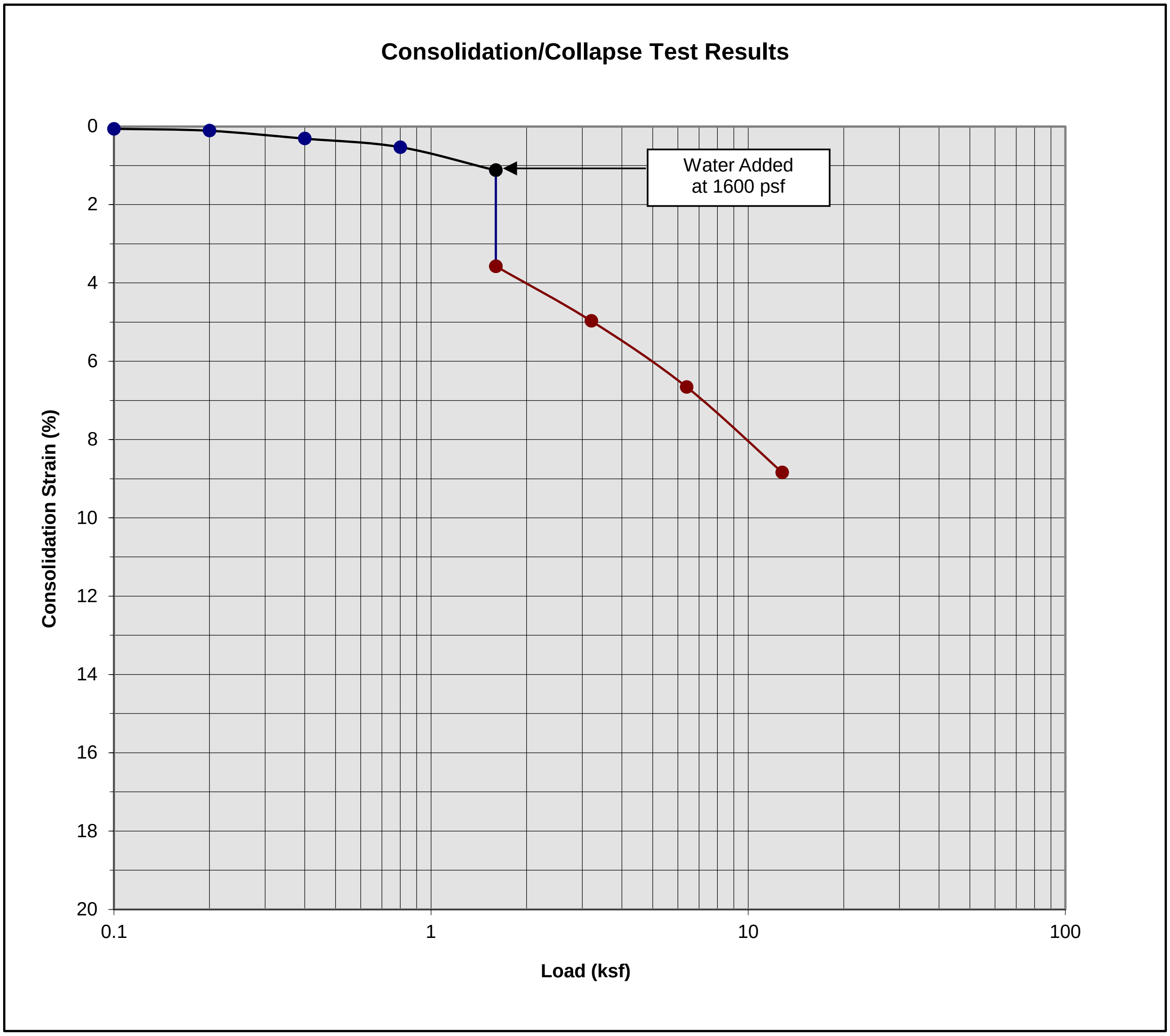
JOB NO.: 21G151-1	DRILLING DATE: 3/24/21	WATER DEPTH: Dry
PROJECT: Proposed Warehouse	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 17 feet
LOCATION: Perris, California	LOGGED BY: Daryl Kas	READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					FILL: Gray Brown fine Sandy Clay, trace Silt, very stiff-moist							
	X	36	2.5				13					
					ALLUVIUM: Light Gray Brown fine Sandy Silt, little Calcareous nodules/veining, medium dense-damp							
	X	19					8					
5												
					Gray Brown Clayey Silt, little fine Sand, very stiff-damp							
	X	24	4.5				10					
					Brown Silty fine to medium Sand, medium dense-damp							
	X	17					7					
					Gray Brown fine Sandy Silt, trace Clay, medium dense-very moist		18					
10												
					Gray Brown Clayey Silt, little fine Sand, abundant Calcareous veining/nodules, stiff-very moist							
	X	9	3.0				33					
15												
					Brown fine Sandy Clay, stiff-moist							
	X	19	2.0				15					
20												
					Boring Terminated at 20'							

TBL 21G151-1.GPJ SOCALGEO.GDT 4/20/21

TEST BORING LOG

APPENDIX C

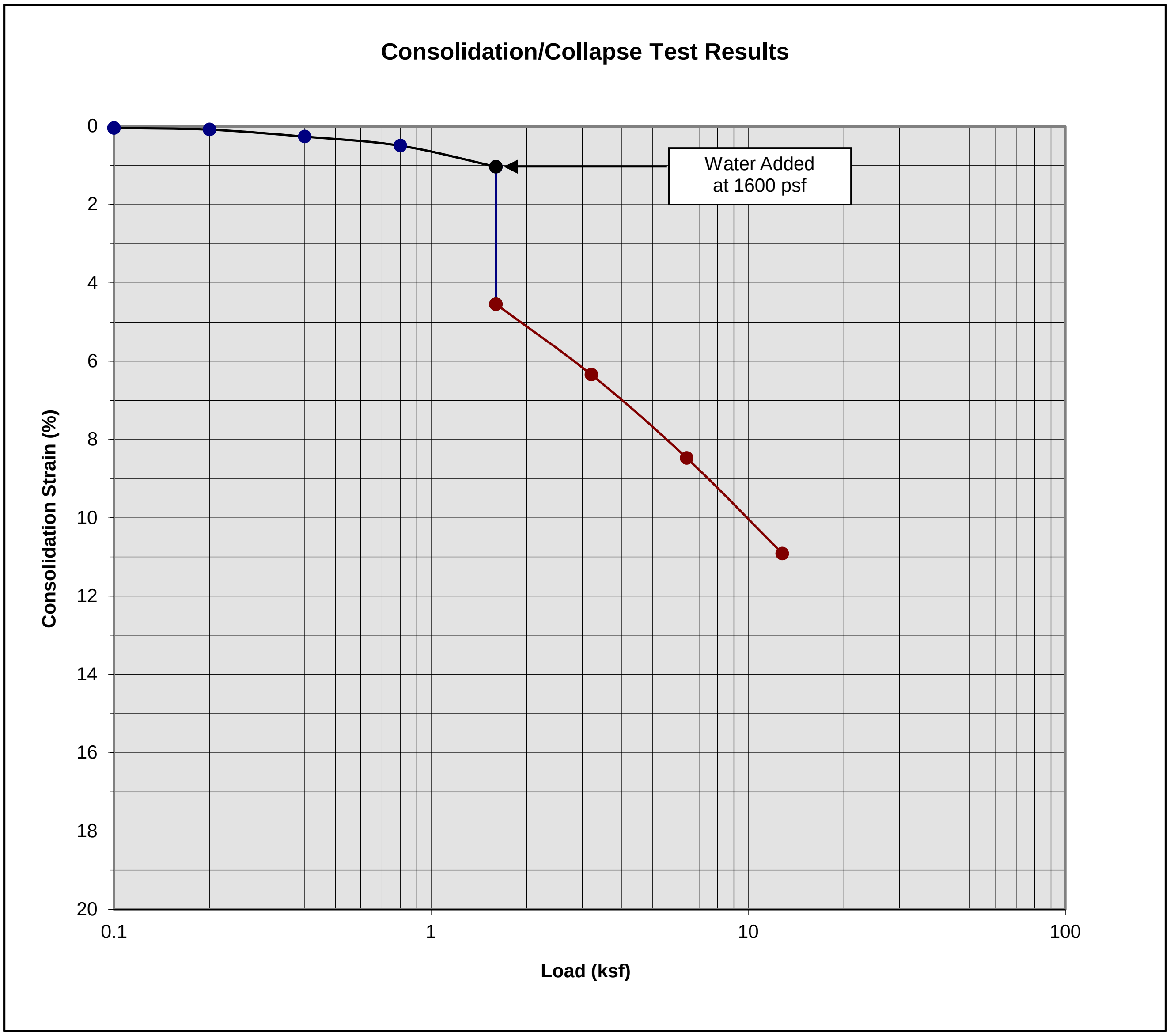


Classification: FILL: Light Gray Brown fine Sandy Silt

Boring Number:	B-3	Initial Moisture Content (%)	7
Sample Number:	---	Final Moisture Content (%)	18
Depth (ft)	1 to 2	Initial Dry Density (pcf)	104.6
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	115.2
Specimen Thickness (in)	1.0	Percent Collapse (%)	2.45

Proposed Warehouse
Perris, California
Project No. 21G151-1
PLATE C-1

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

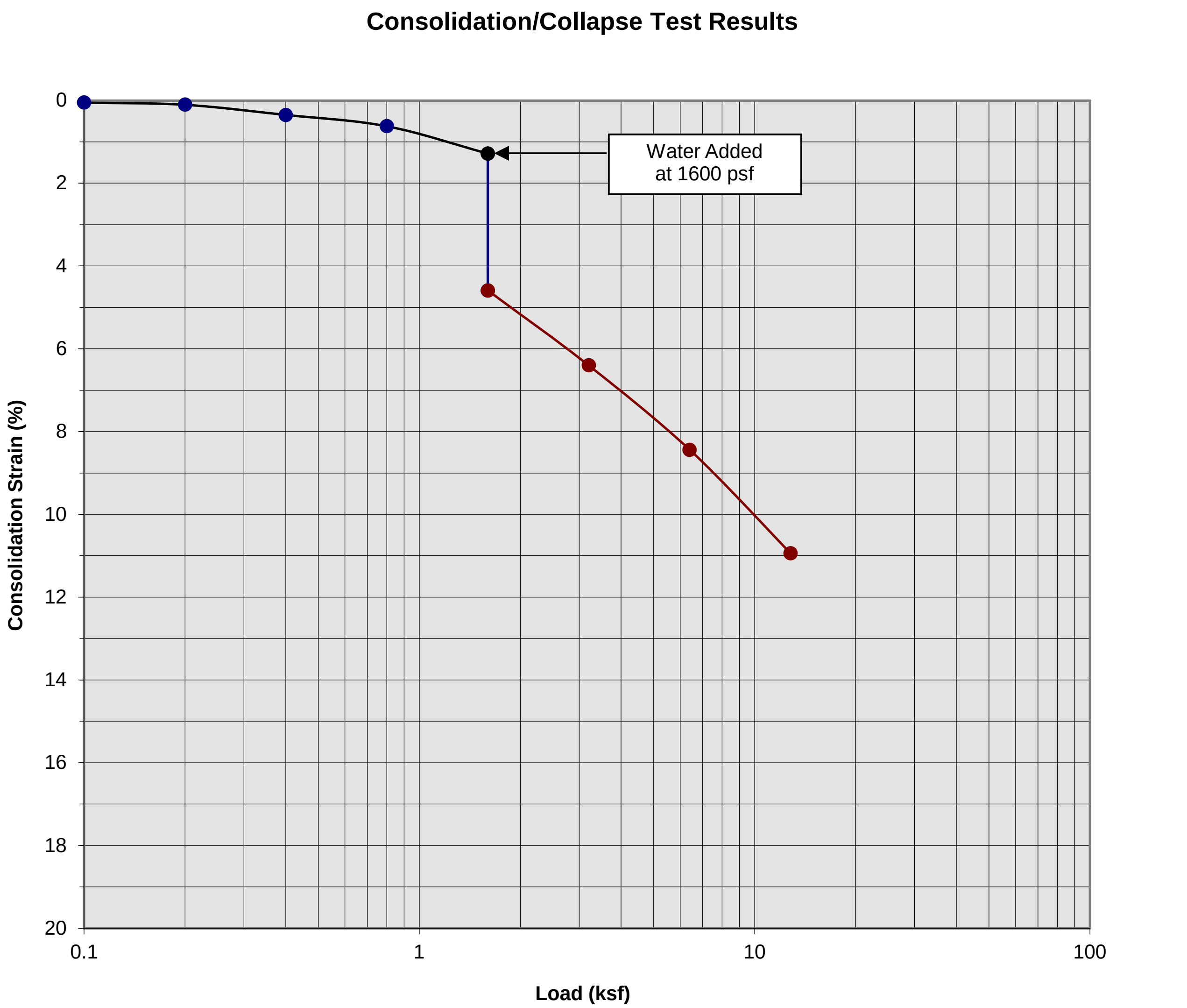


Classification: Brown Sity fine Sand

Boring Number:	B-3	Initial Moisture Content (%)	6
Sample Number:	---	Final Moisture Content (%)	15
Depth (ft)	3 to 4	Initial Dry Density (pcf)	110.0
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	123.3
Specimen Thickness (in)	1.0	Percent Collapse (%)	3.51

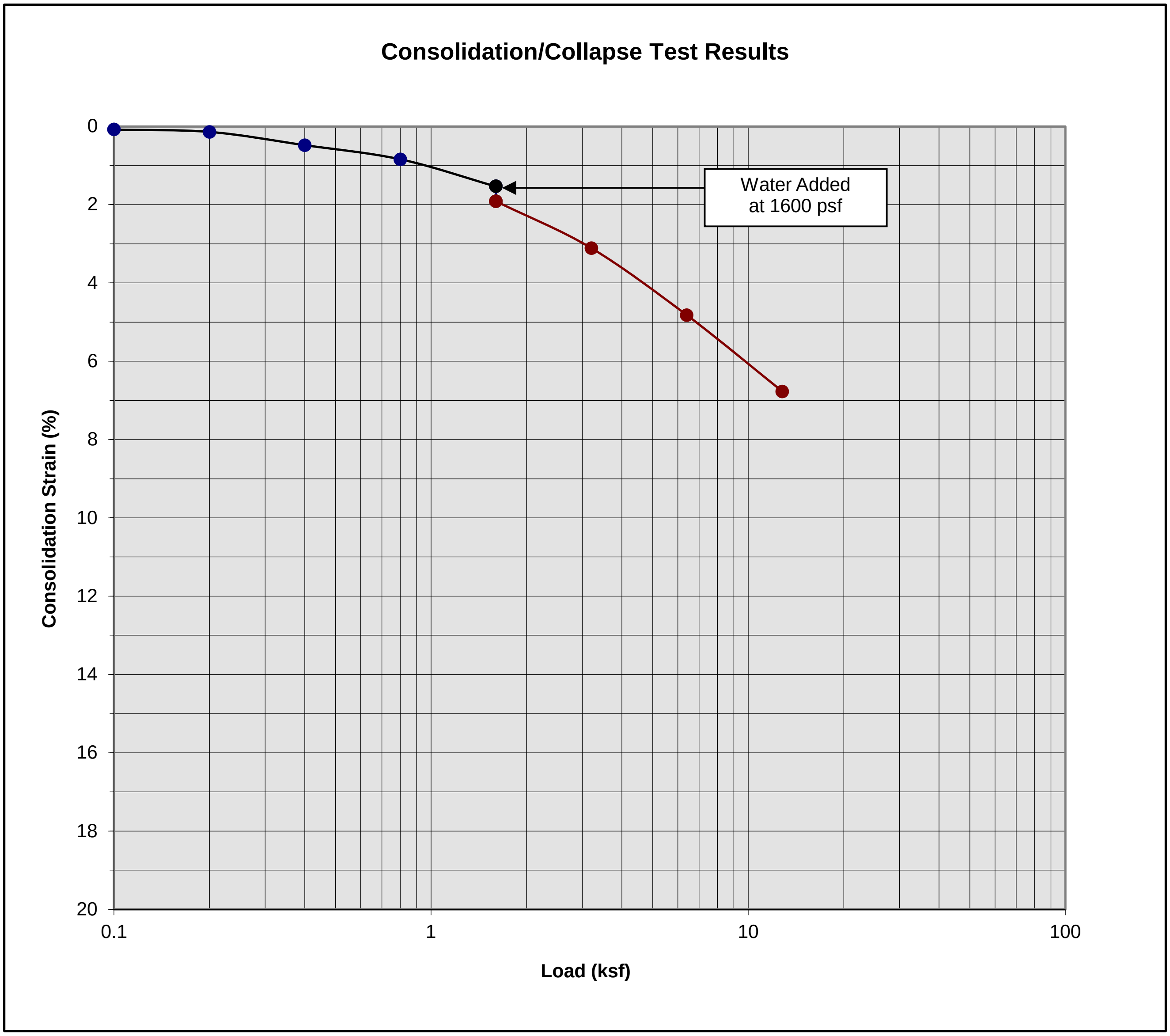
Proposed Warehouse
Perris, California
Project No. 21G151-1
PLATE C-2

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation



Classification: Brown Clayey fine Sand, trace Silt

Boring Number:	B-3	Initial Moisture Content (%)	7
Sample Number:	---	Final Moisture Content (%)	16
Depth (ft)	5 to 6	Initial Dry Density (pcf)	107.5
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	120.4
Specimen Thickness (in)	1.0	Percent Collapse (%)	3.31



Classification: Brown Clayey Silt, trace fine Sand

Boring Number:	B-3	Initial Moisture Content (%)	19
Sample Number:	---	Final Moisture Content (%)	23
Depth (ft)	7 to 8	Initial Dry Density (pcf)	101.5
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	108.6
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.38

Proposed Warehouse
Perris, California
Project No. 21G151-1
PLATE C-4

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
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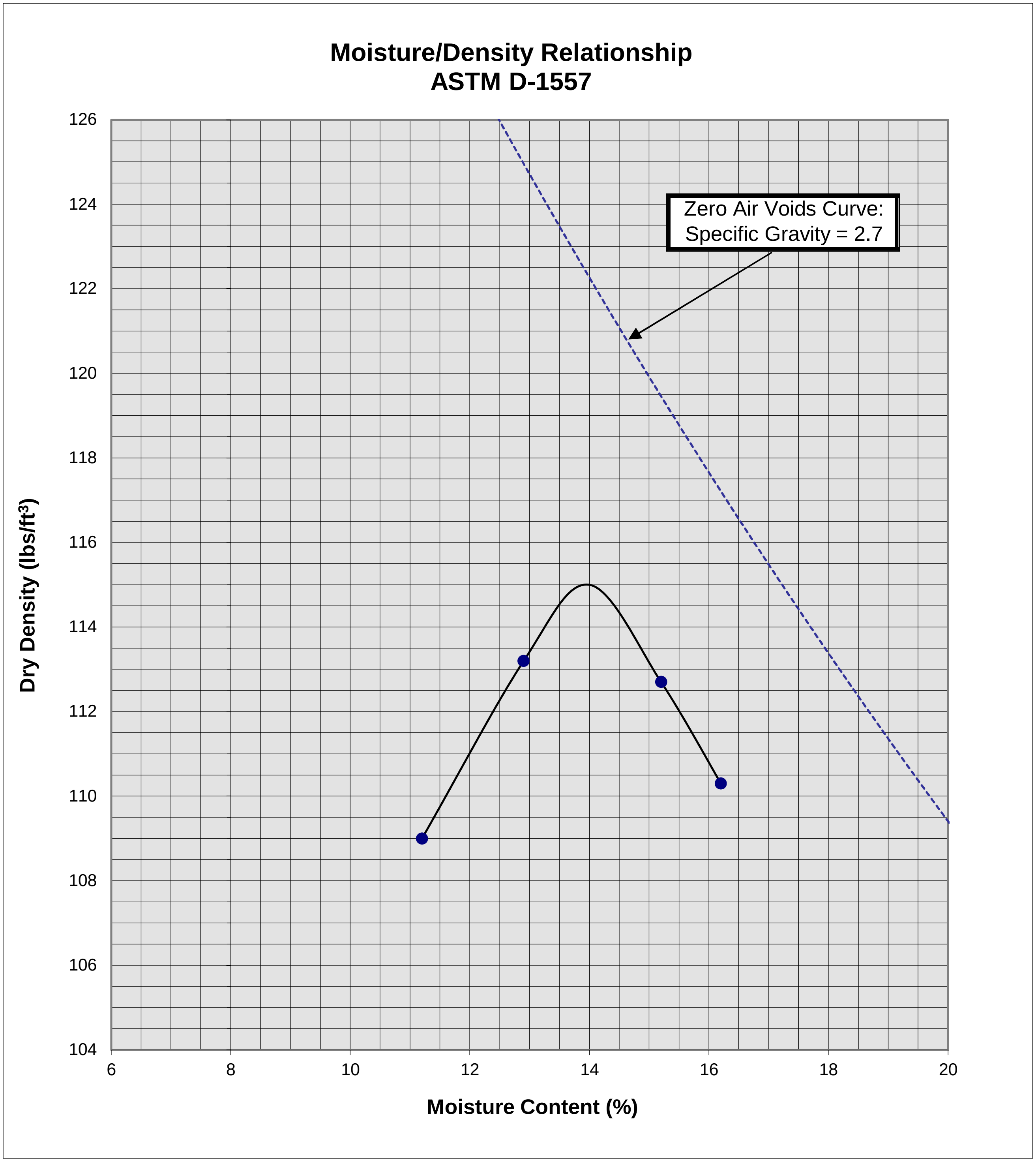


Soil ID Number	B-4 @ 0-5'
Optimum Moisture (%)	14
Maximum Dry Density (pcf)	115
Soil Classification	Gray Brown fine Sandy Silt, little Clay, abundant Calcareous nodules/veining

Proposed Warehouse
Perris, California
Project No. 21G151-1
PLATE C-5

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.

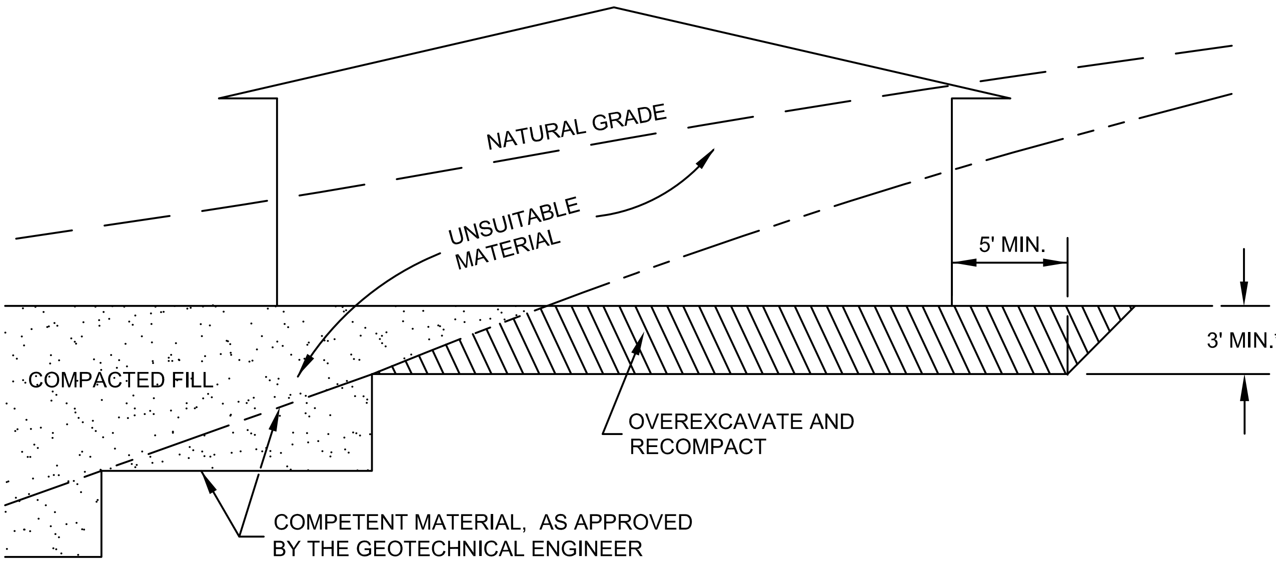
Subdrains

- 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.

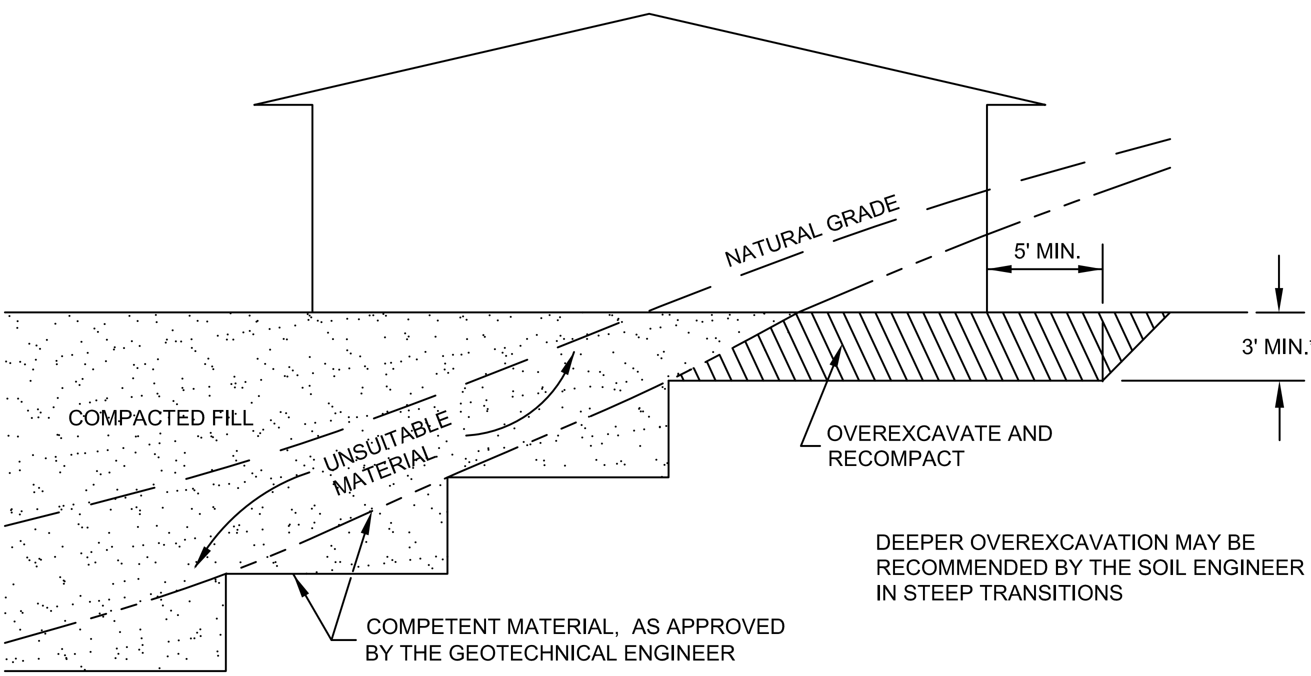
- Plastic pipe may be used for subdrains provided it is Schedule 40 or SDR 35 or equivalent. Pipe should be protected against breakage, typically by placement in a square-cut (backhoe) trench or as recommended by the manufacturer.

- 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

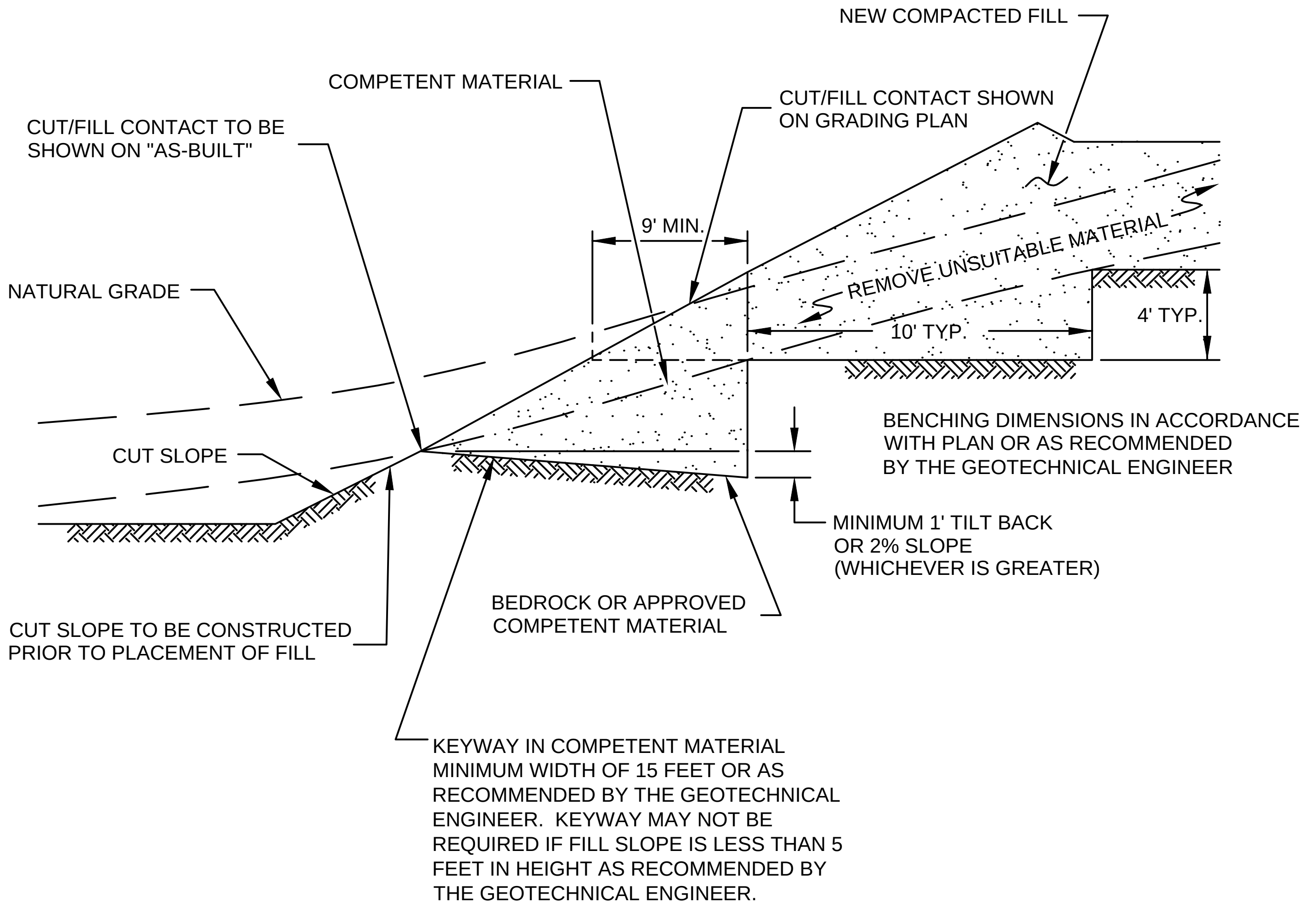
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-1

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL



FILL ABOVE CUT SLOPE DETAIL

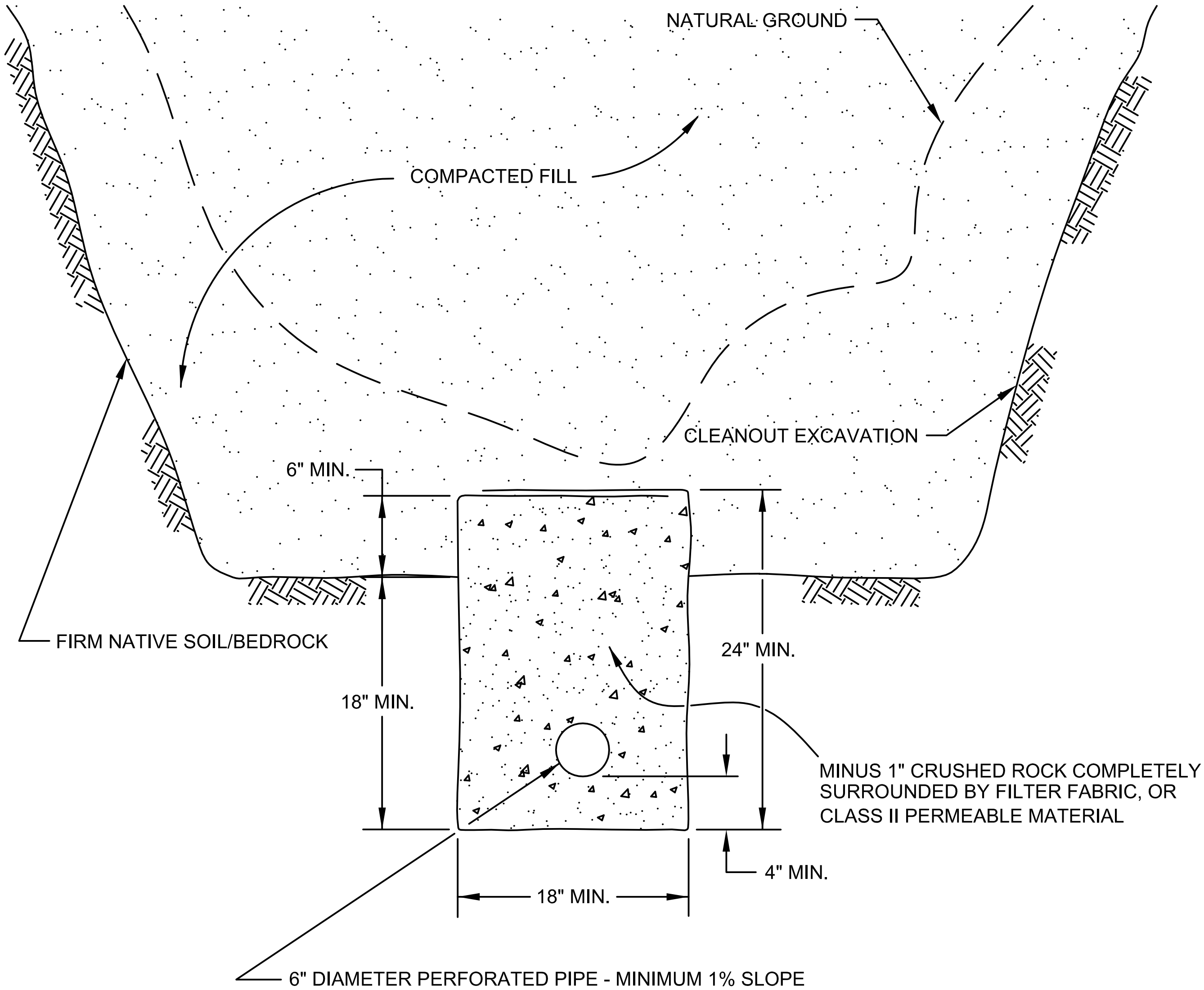
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-2

SOUTHERN CALIFORNIA GEOTECHNICAL



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

SCHEMATIC ONLY
NOT TO SCALE

CANYON SUBDRAIN DETAIL

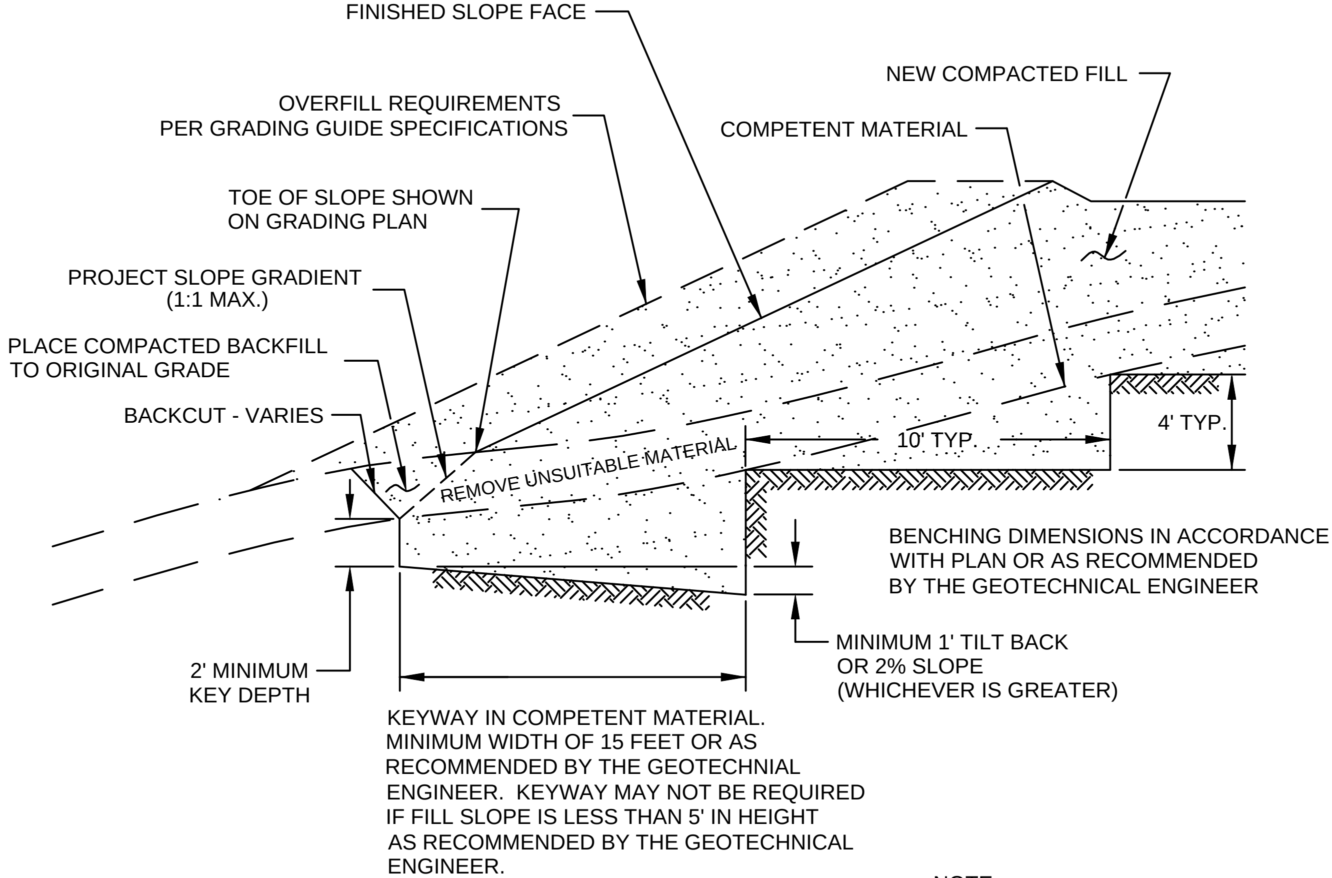
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-3

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL



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

FILL ABOVE NATURAL SLOPE DETAIL

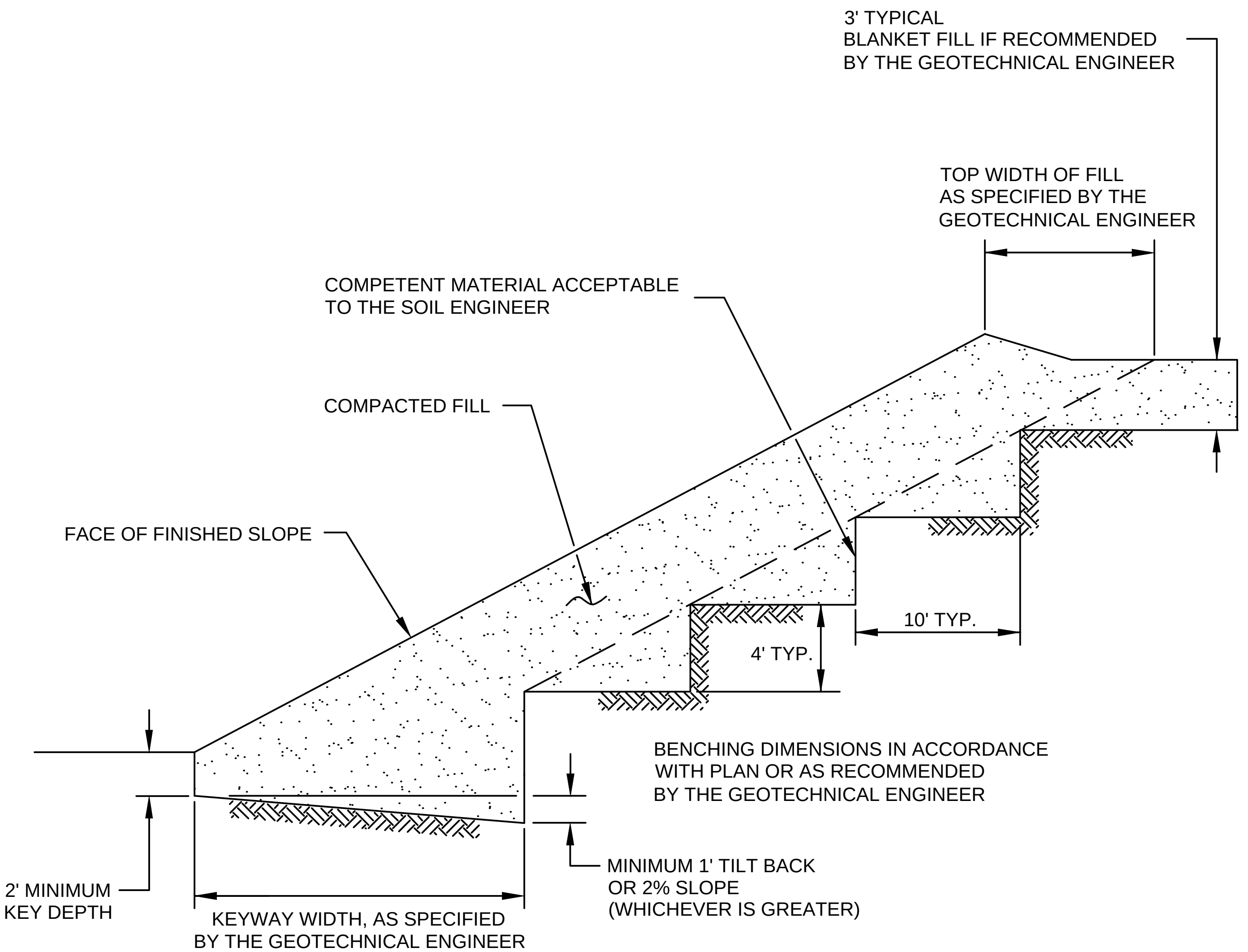
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-4

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL



STABILIZATION FILL DETAIL

GRADING GUIDE SPECIFICATIONS

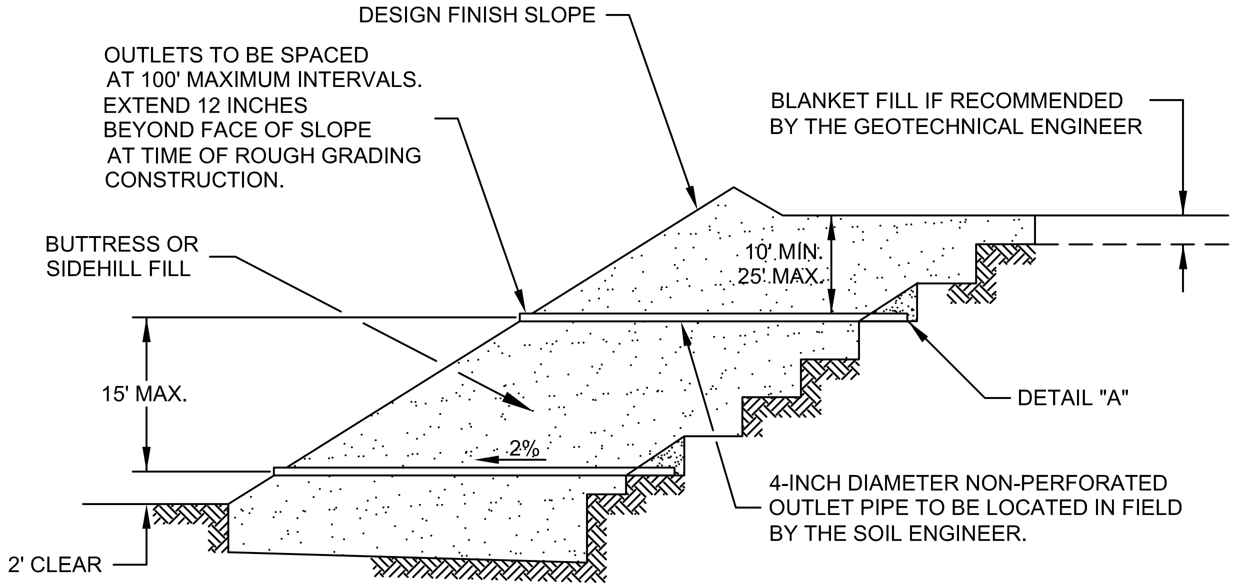
NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-5

SoCalGeo

SOUTHERN CALIFORNIA GEOTECHNICAL



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

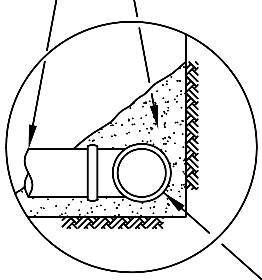
SIEVE SIZE	PERCENTAGE PASSING
1"	100
3/4"	90-100
3/8"	40-100
NO. 4	25-40
NO. 8	18-33
NO. 30	5-15
NO. 50	0-7
NO. 200	0-3

"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

OUTLET PIPE TO BE CONNECTED TO SUBDRAIN PIPE WITH TEE OR ELBOW



FILTER MATERIAL - MINIMUM OF FIVE CUBIC FEET PER FOOT OF PIPE. SEE ABOVE FOR FILTER MATERIAL SPECIFICATION.

ALTERNATIVE: IN LIEU OF FILTER MATERIAL FIVE CUBIC FEET OF GRAVEL PER FOOT OF PIPE MAY BE ENCASED IN FILTER FABRIC. SEE ABOVE FOR GRAVEL SPECIFICATION.

FILTER FABRIC SHALL BE MIRAFI 140 OR EQUIVALENT. FILTER FABRIC SHALL BE LAPPED A MINIMUM OF 12 INCHES ON ALL JOINTS.

MINIMUM 4-INCH DIAMETER PVC SCH 40 OR ABS CLASS SDR 35 WITH A CRUSHING STRENGTH OF AT LEAST 1,000 POUNDS, WITH A MINIMUM OF 8 UNIFORMLY SPACED PERFORATIONS PER FOOT OF PIPE INSTALLED WITH PERFORATIONS ON BOTTOM OF PIPE. PROVIDE CAP AT UPSTREAM END OF PIPE. SLOPE AT 2 PERCENT TO OUTLET PIPE.

<u>DETAIL "A"</u>

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

SLOPE FILL SUBDRAINS

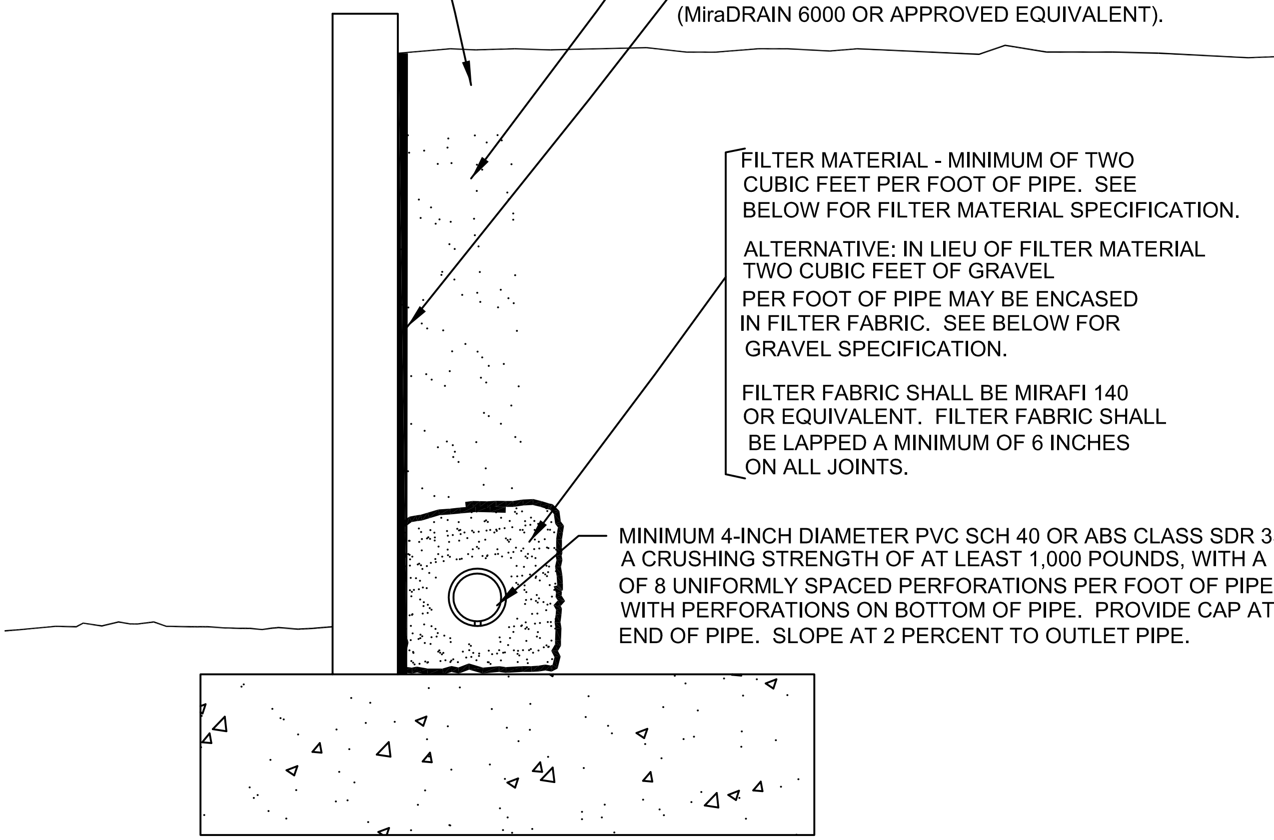
GRADING GUIDE SPECIFICATIONS

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

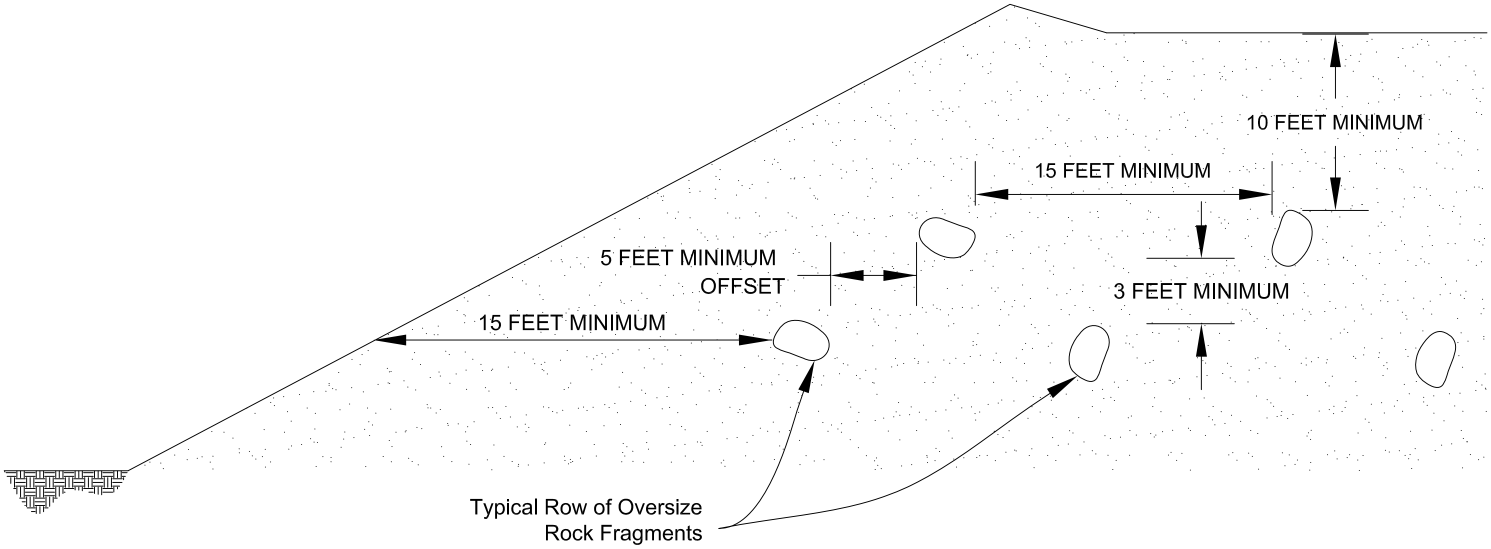
NOT TO SCALE

DRAWN: JAS
CHKD: GKM

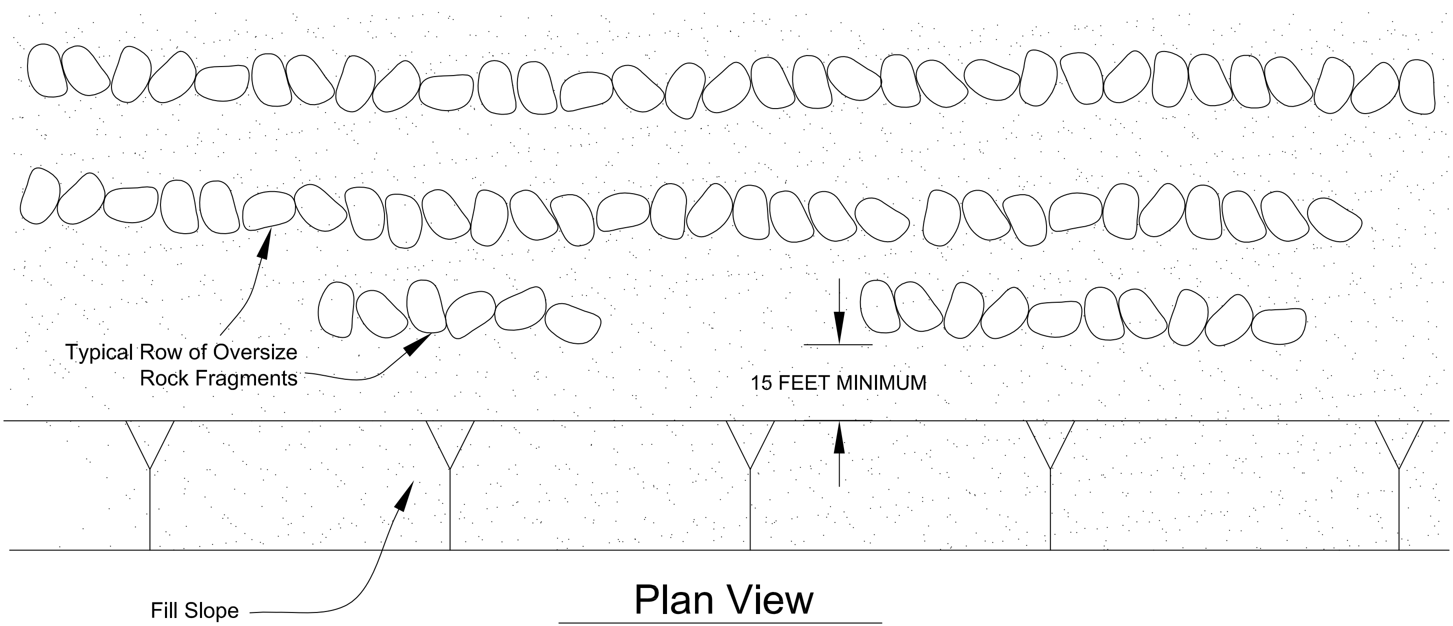
PLATE D-7

SoCalGeo

SOUTHERN CALIFORNIA GEOTECHNICAL



Section View



Plan View

PLACEMENT OF OVERSIZED MATERIAL

GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: PM
CHKD: GKM

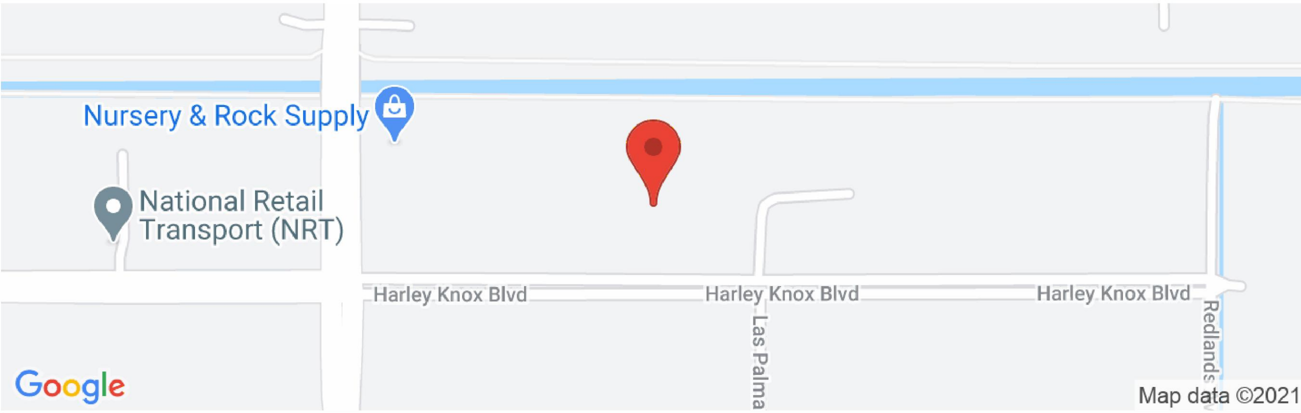
PLATE D-8

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL

APPENDIX E

150 Harley Knox Blvd, Perris, CA 92571, USA

Latitude, Longitude: 33.857993, -117.222795



Date	4/11/2021, 2:54:22 PM
Design Code Reference Document	ASCE7-16
Risk Category	III
Site Class	D - Stiff Soil

Type	Value	Description
S_S	1.5	MCE_R ground motion. (for 0.2 second period)
S_1	0.6	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.5	Site-modified spectral acceleration value
S_{M1}	null -See Section 11.4.8	Site-modified spectral acceleration value
S_{DS}	1	Numeric seismic design value at 0.2 second SA
S_{D1}	null -See Section 11.4.8	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	null -See Section 11.4.8	Seismic design category
F_a	1	Site amplification factor at 0.2 second
F_v	null -See Section 11.4.8	Site amplification factor at 1.0 second
PGA	0.535	MCE_G peak ground acceleration
F_{PGA}	1.1	Site amplification factor at PGA
PGA_M	0.589	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
SsRT	1.593	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	1.711	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	1.5	Factored deterministic acceleration value. (0.2 second)
S1RT	0.6	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.661	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.6	Factored deterministic acceleration value. (1.0 second)
PGAd	0.535	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.931	Mapped value of the risk coefficient at short periods

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

SEISMIC DESIGN PARAMETERS - 2019 CBC

PROPOSED WAREHOUSE

PERRIS, CALIFORNIA

DRAWN: JLL
CHKD: RGT

SCG PROJECT
21G151-1

PLATE E-1

SOUTHERN CALIFORNIA GEOTECHNICAL

APPENDIX F

LIQUEFACTION EVALUATION

Project Name	Proposed Warehouse	Design PGA	0.589 (g)
Project Location	Perris, CA	Design Magnitude	7.09
Project Number	21G151-1	Historic High Depth to Groundwater	9 (ft)
Engineer	JLL	Depth to Groundwater at Time of Drilling	18 (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=7.09)	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	9	4.5		120		1.3	1.05	1.1	1.70	0.75	0.0	0.0	540	540	540	0.99	1.01	1.07	0.06	0.07	N/A	N/A	Above Water Table
9.5	9	12	9.5	11	120	84	1.3	1.05	1.171	1.30	0.75	17.1	22.6	1140	1109	1140	0.98	1.09	1.09	N/A	N/A	N/A	N/A	Non-Liq: PI>18
14.5	12	17	14.5	11	120		1.3	1.05	1.163	1.10	0.85	16.3	16.3	1740	1397	1740	0.96	1.05	1.05	N/A	N/A	N/A	N/A	Non-Liq: PI>18
19.5	17	22	19.5	28	120	44	1.3	1.05	1.3	0.99	0.95	46.6	52.2	2340	1685	2246	0.94	1.17	1.07	2.00	2.00	0.50	4.02	Nonliquefiable
24.5	22	27	24.5	11	120	36	1.3	1.05	1.152	0.92	0.95	15.2	20.7	2940	1973	2534	0.91	1.07	1.01	0.21	0.23	0.52	0.45	**Liquefiable**
29.5	27	32	29.5	22	120	58	1.3	1.05	1.3	0.92	0.95	34.0	39.6	3540	2261	2822	0.89	1.17	0.98	2.00	2.00	0.53	3.76	Nonliquefiable
34.5	32	37	34.5	15	120	40	1.3	1.05	1.214	0.86	1	21.4	27.0	4140	2549	3110	0.86	1.12	0.97	0.35	0.37	0.53	0.70	**Liquefiable**
39.5	37	42	39.5	16	120	82	1.3	1.05	1.223	0.84	1	22.3	27.9	4740	2837	3398	0.83	1.12	0.94	N/A	N/A	N/A	N/A	Non-Liq: PI>18
44.5	42	47	44.5	13	120	85	1.3	1.05	1.163	0.79	1	16.3	21.8	5340	3125	3686	0.81	1.08	0.94	N/A	N/A	N/A	N/A	Non-Liq: PI>18
49.5	47	52	49.5	20	120	79	1.3	1.05	1.283	0.81	1	28.3	33.9	5940	3413	3974	0.78	1.17	0.88	0.89	0.92	0.52	1.76	Nonliquefiable
54.5	52	54.5	53.3	20	120	10	1.3	1.05	1.267	0.77	1	26.7	27.9	6390	3629	4190	0.76	1.12	0.9	0.38	0.38	0.51	0.75	**Lliquefiable**
54.5	54.5	55	54.8	20	120	50	1.3	1.05	1.273	0.79	1	27.3	32.9	6570	3715	4277	0.75	1.17	0.87	0.75	0.76	0.51	1.49	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	Proposed Warehouse
Project Location	Perris, CA
Project Number	21G151-1
Engineer	JLL

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	9	4.5	0.0	0.0	0.0	N/A	0.50	0.95	0.00	9.00		0.000		0.00	Above Water Table
9.5	9	12	9.5	17.1	5.5	22.6	N/A	0.12	0.37	0.00	3.00		0.000		0.00	Non-Liq: PI>18
14.5	12	17	14.5	16.3	0.0	16.3	N/A	0.24	0.70	0.00	5.00		0.000		0.00	Non-Liq: PI>18
19.5	17	22	19.5	46.6	5.6	52.2	4.02	0.00	-1.77	0.00	5.00		0.000		0.00	Nonliquefiable
24.5	22	27	24.5	15.2	5.5	20.7	0.45	0.15	0.48	0.15	5.00		0.022		1.34	**Liquefiable**
29.5	27	32	29.5	34.0	5.6	39.6	3.76	0.01	-0.78	0.00	5.00		0.000		0.00	Nonliquefiable
34.5	32	37	34.5	21.4	5.6	27.0	0.70	0.07	0.11	0.07	5.00		0.015		0.90	**Liquefiable**
39.5	37	42	39.5	22.3	5.5	27.9	N/A	0.06	0.05	0.00	5.00		0.000		0.00	Non-Liq: PI>18
44.5	42	47	44.5	16.3	5.5	21.8	N/A	0.13	0.42	0.00	5.00		0.000		0.00	Non-Liq: PI>18
49.5	47	52	49.5	28.3	5.5	33.9	1.76	0.03	-0.36	0.00	5.00		0.000		0.00	Nonliquefiable
54.5	52	54.5	53.3	26.7	1.1	27.9	0.75	0.06	0.05	0.06	2.50		0.013		0.38	**Lliquefiable**
54.5	54.5	55	54.8	27.3	5.6	32.9	1.49	0.03	-0.29	0.00	0.50		0.000		0.00	Nonliquefiable
											Total Deformation (in)				2.63	

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	Proposed Warehouse	Design PGA	0.589 (g)
Project Location	Perris, CA	Design Magnitude	7.09
Project Number	21G151-1	Historic High Depth to Groundwater	9 (ft)
Engineer	JLL	Depth to Groundwater at Time of Drilling	20 (ft)
		Borehole Diameter	6 (in)
Boring No.	B-4		

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 (1)	C_B (2)	C_S (3)	C_N (4)	Rod Length Correction (5)	$(N_1)_{60}$ (6)	$(N_1)_{60CS}$ (7)	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) (8)	MSF (9)	KS (10)	Cyclic Resistance Ratio (M=7.5) (11)	Cyclic Resistance Ratio (M=7.09) (12)	Cyclic Stress Ratio Induced by Design Earthquake (13)	Factor of Safety	Comments
7	0	9	4.5		120		1.3	1.05	1.1	1.70	0.75	0.0	0.0	540	540	540	0.99	1.01	1.07	0.06	0.07	N/A	N/A	Above Water Table
9.5	9	12	9.5	14	120	50	1.3	1.05	1.221	1.26	0.75	22.1	27.7	1140	1109	1140	0.98	1.12	1.1	0.37	0.46	0.38	1.20	**Liquefiable**
14.5	12	17	14.5	9	120	64	1.3	1.05	1.129	1.09	0.85	12.9	18.5	1740	1397	1740	0.96	1.06	1.05	0.19	0.21	0.46	0.46	**Liquefiable**
19.5	17	22	19.5	25	120	53	1.3	1.05	1.3	0.97	0.95	41.1	46.7	2340	1685	2340	0.94	1.17	1.07	2.00	2.00	0.50	4.02	Nonliquefiable
24.5	22	27	24.5	15	120	35	1.3	1.05	1.217	0.92	0.95	21.7	27.2	2940	1973	2659	0.91	1.12	1.01	0.35	0.40	0.52	0.77	**Liquefiable**
29.5	27	32	29.5	15	120	41	1.3	1.05	1.206	0.88	0.95	20.6	26.2	3540	2261	2947	0.89	1.11	0.99	0.32	0.35	0.53	0.66	**Liquefiable**
34	32	34	33	18	120	52	1.3	1.05	1.273	0.87	1	27.3	32.9	3960	2462	3149	0.87	1.17	0.96	0.75	0.84	0.53	1.57	Nonliquefiable
34.5	34	37	35.5	18	120	51	1.3	1.05	1.267	0.86	1	26.7	32.3	4260	2606	3293	0.85	1.16	0.95	0.68	0.75	0.53	1.40	Nonliquefiable
39	37	39	38	41	120	45	1.3	1.05	1.3	0.94	1	68.6	74.2	4560	2750	3437	0.84	1.17	0.92	2.00	2.00	0.53	3.75	Nonliquefiable
39.5	39	42	40.5	41	120	33	1.3	1.05	1.3	0.94	1	68.1	73.6	4860	2894	3581	0.83	1.17	0.91	2.00	2.00	0.53	3.76	Nonliquefiable
44.5	42	47	44.5	21	120	95	1.3	1.05	1.3	0.83	1	30.9	36.4	5340	3125	3811	0.81	1.17	0.89	1.49	1.55	0.53	2.95	Nonliquefiable
49.5	47	50	48.5	12	120	66	1.3	1.05	1.14	0.75	1	14.0	19.6	5820	3355	4042	0.78	1.07	0.94	N/A	N/A	N/A	N/A	Non-Liq: PI>18

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	Proposed Warehouse
Project Location	Perris, CA
Project Number	21G151-1
Engineer	JLL

Boring No. B-4

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	9	4.5	0.0	0.0	0.0	N/A	0.50	0.95	0.00	9.00		0.000		0.00	Above Water Table
9.5	9	12	9.5	22.1	5.6	27.7	1.20	0.06	0.06	0.02	3.00		0.005		0.18	**Liquefiable**
14.5	12	17	14.5	12.9	5.6	18.5	0.46	0.19	0.60	0.19	5.00		0.025		1.47	**Liquefiable**
19.5	17	22	19.5	41.1	5.6	46.7	4.02	0.00	-1.32	0.00	5.00		0.000		0.00	Nonliquefiable
24.5	22	27	24.5	21.7	5.5	27.2	0.77	0.07	0.10	0.06	5.00		0.013		0.76	**Liquefiable**
29.5	27	32	29.5	20.6	5.6	26.2	0.66	0.08	0.16	0.08	5.00		0.017		1.04	**Liquefiable**
34	32	34	33	27.3	5.6	32.9	1.57	0.00	-0.29	0.00	2.00		0.000		0.00	Nonliquefiable
34.5	34	37	35.5	26.7	5.6	32.3	1.40	0.00	-0.25	0.00	3.00		0.000		0.00	Nonliquefiable
39	37	39	38	68.6	5.6	74.2	3.75	0.00	-3.67	0.00	2.00		0.000		0.00	Nonliquefiable
39.5	39	42	40.5	68.1	5.5	73.6	3.76	0.00	-3.61	0.00	3.00		0.000		0.00	Nonliquefiable
44.5	42	47	44.5	30.9	5.5	36.4	2.95	0.02	-0.53	0.00	5.00		0.000		0.00	Nonliquefiable
49.5	47	50	48.5	14.0	5.6	19.6	N/A	0.17	0.54	0.00	3.00		0.000		0.00	Non-Liq: PI>18
											Total Deformation (in)				3.46	

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)

Appendix 4: Historical Site Conditions

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

Not included.

Appendix 5: LID Infeasibility

LID Technical Infeasibility Analysis

N/A – Runoff from the project is directed to Canon Lake, which ultimately drains to Lake Elsinore. Based on the infiltration investigation from the geotechnical engineer, infiltration is not technically feasible for this project. A proprietary Modular Wetland System (MWS) is proposed immediately downstream of an underground storage facility to treat runoff from the site.

Appendix 6: BMP Design Details

BMP Sizing, Design Details and other Supporting Documentation

Santa Ana Watershed - 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: SDH & Associates, Inc. — Date: 2/18/2022
Designed by: NM — Case No: P21-00008
Company Project Number/Name: 2014 / Lake Creek-Harley Knox

BMP Identification

BMP NAME / ID: MWS (Volume-based) / BMP 1

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}= 0.64 inches

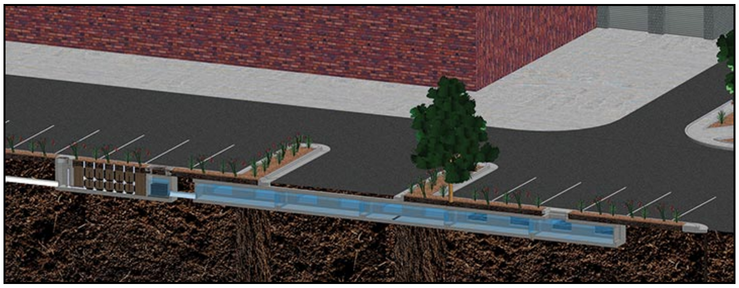
Drainage Management Area Tabulation

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)*
DMA 1-1	*36,586*	*Ornamental Landscaping*	*0.1*	*0.11*	*4041.2*			
DMA 1-2	*114,227*	*Concrete or Asphalt*	*1*	*0.89*	*101890.5*			
DMA 1-3	*138,933*	*Roofs*	*1*	*0.89*	*123928.2*			
DMA 1-4	*17324*	*Natural (B Soil)*	*0.15*	*0.14*	*2450.4*			
	307070	***Total***			**232310.3**	**0.64**	***12389.9***	***25020***

Notes: **THE PROPOSED STORMTRAP UNDERGOUND STORAGE FACILITY WILL PROVIDE APPROXIMATELY 25,020 CUBIC FEET OF VOLUME TO ACCOUNT FOR BOTH WATER QUALITY DESIGN CAPTURE VOLUME AND FLOOD CONTROL DETENTION. THIS VOLUME IS MORE THAN THE MINIMUM REQUIRED DCV OF 12,389.9 C.F. THE PROPOSED MWS (VOLUME-BASED) WILL PROVIDE TREATMENT VOLUME OF UP TO ~15,109 C.F. BASED ON A 48-HOUR DRAWDOWN. THEREFORE, THE PROPOSED BMPs ARE ADEQUATE TO MEET THE WATER QUALITY MANAGEMENT REQUIREMENTS.**

SUPPORTING MATERIALS FOR MWS SIZING (PER VOLUME-BASED APPROACH WITH ~48-HR DRAWDOWN)



Volume Based Sizing

Many states require treatment of a water quality volume and do not offer the option of flow based design. The MWS Linear and its unique horizontal flow makes it the only biofilter that can be used in volume based design installed downstream of ponds, detention basins, and underground storage systems.

Model #	Treatment Capacity (cu. ft.) @ 24-Hour Drain Down	Treatment Capacity (cu. ft.) @ 48-Hour Drain Down
MWS-L-4-4	1140	2280
MWS-L-4-6	1600	3200
MWS-L-4-8	2518	5036
MWS-L-4-13	3131	6261
MWS-L-4-15	3811	7623
MWS-L-4-17	4492	8984
MWS-L-4-19	5172	10345
MWS-L-4-21	5853	11706
MWS-L-6-8	3191	6382
MWS-L-8-8	5036	10072
$\boxed{\text{MWS-L-8-12}}$	7554	$\boxed{15109}$
MWS-L-8-16	10073	20145
MWS-L-8-20	12560	25120
MWS-L-8-24	15108	30216

MWS-L-8-12 WILL PROVIDE TREATMENT VOLUME UP TO ~15,109 C.F. > 12,389 C.F. THEREFORE, OK.

June 8, 2021

LAKE CREEK - PERRIS, CA
STAGE STORAGE BREAKDOWN
4'-2" SingleTrap
TOTAL VOLUME: 25020.04 (C.F.)

Storage	Type I QTY	Type II QTY	Type III QTY	Type IV QTY	Type V QTY	Type VII QTY	SPIV 1 QTY	SPIV 2 QTY	Total Units	Stage Storage Elevation
	19	0	38	0	0	0	4	0	61	
										System Invert 0.00
0.25	572.07	0.00	861.65	0.00	0.00	0.00	60.28	0.00	1,500.00	0.25
0.50	1,144.14	0.00	1,723.31	0.00	0.00	0.00	120.57	0.00	3,000.00	0.50
0.75	1,716.21	0.00	2,584.96	0.00	0.00	0.00	180.85	0.00	4,500.01	0.75
1.00	2,288.28	0.00	3,446.61	0.00	0.00	0.00	241.14	0.00	6,000.01	1.00
1.25	2,860.35	0.00	4,308.27	0.00	0.00	0.00	301.42	0.00	7,500.01	1.25
1.50	3,432.42	0.00	5,169.92	0.00	0.00	0.00	361.70	0.00	9,000.01	1.50
1.75	4,004.49	0.00	6,031.58	0.00	0.00	0.00	421.99	0.00	10,500.02	1.75
2.00	4,576.56	0.00	6,893.23	0.00	0.00	0.00	482.27	0.00	12,000.02	2.00
2.25	5,148.63	0.00	7,754.88	0.00	0.00	0.00	542.55	0.00	13,500.02	2.25
2.50	5,720.70	0.00	8,616.54	0.00	0.00	0.00	602.84	0.00	15,000.02	2.50
2.75	6,292.77	0.00	9,478.19	0.00	0.00	0.00	663.12	0.00	16,500.02	2.75
3.00	6,864.83	0.00	10,339.84	0.00	0.00	0.00	723.41	0.00	18,000.03	3.00
3.25	7,436.90	0.00	11,201.50	0.00	0.00	0.00	783.69	0.00	19,500.03	3.25
3.50	8,008.97	0.00	12,063.15	0.00	0.00	0.00	843.97	0.00	21,000.03	3.50
3.75	8,581.04	0.00	12,924.80	0.00	0.00	0.00	904.26	0.00	22,500.03	3.75
4.00	9,153.11	0.00	13,786.46	0.00	0.00	0.00	964.54	0.00	24,000.04	4.00
4.17	9,542.12	0.00	14,372.38	0.00	0.00	0.00	1,005.54	0.00	25,020.04	4.17

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EMAIL info@stormtrap.com
1287 Windham Parkway
Romeoville, Illinois 60446

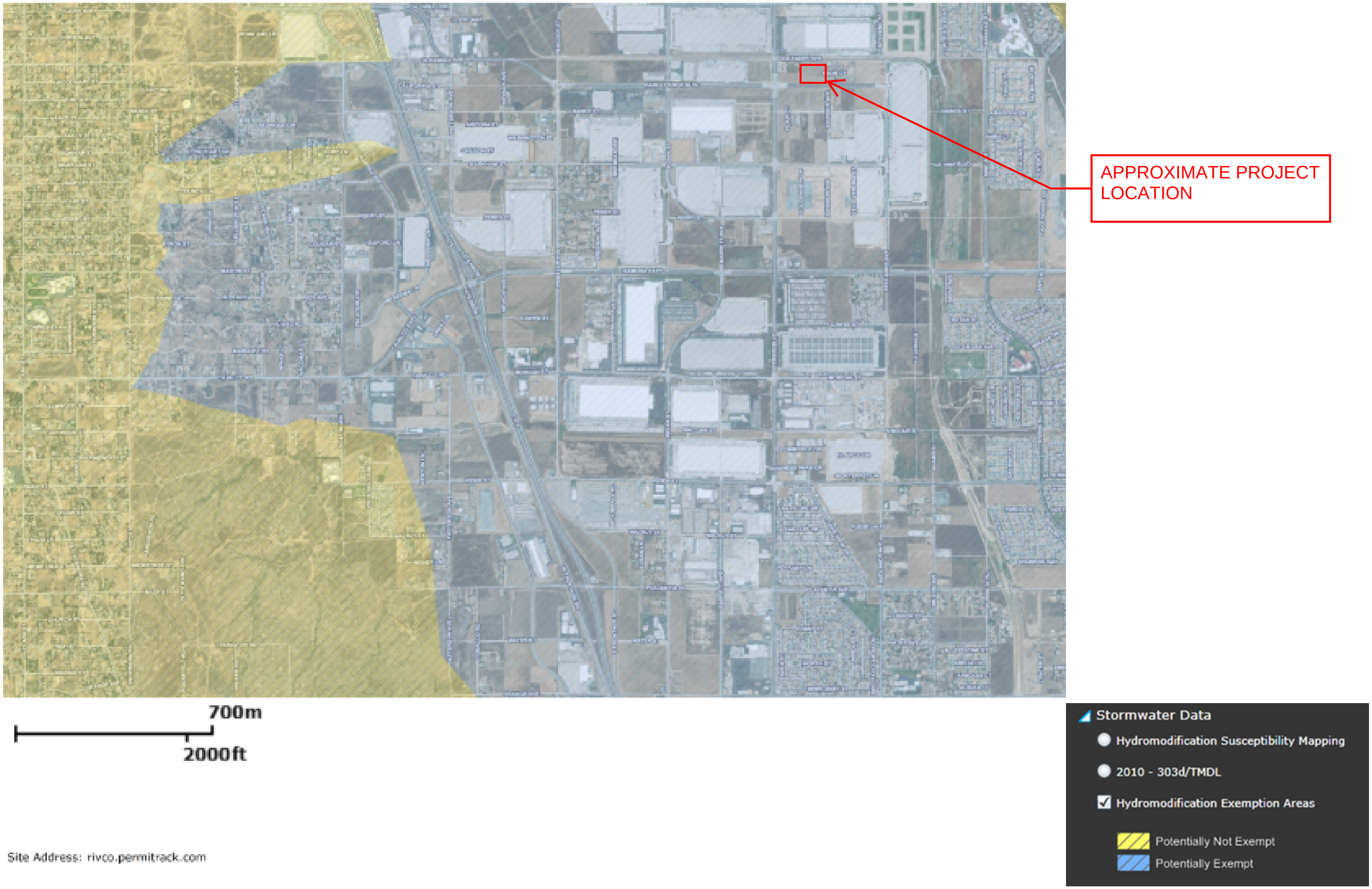
Appendix 7: Hydromodification

Supporting Detail Relating to Hydrologic Conditions of Concern

Note: The project is within the Riverside County WAP HCOC Exemption area approved on April 20, 2017. Therefore, the project is exempt from the HCOC requirements.

SCREEN CAPTURE - RIVERSIDE COUTY STORM WATER & WATER CONSERVATION TRACKING TOOL

HCOC EXEMPTION AREAS



NOTE: THE PROJECT IS WITHIN THE RIVERSIDE COUNTY WAP HCOC EXEMPTION AREA APPROVED ON APRIL 20, 2017. THEREFORE, THE PROJECT SHOULD BE EXEMPT FROM THE HCOC REQUIREMENTS.

Appendix 8: Source Control

Pollutant Sources/Source Control Checklist

Note: The Source Control checklist will be prepared during final engineering (construction document) stage at the time of the final WQMP.

Appendix 9: O&M

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

Note: The O&M Plan will be prepared during final engineering (construction document) stage at the time of the final WQMP.

Appendix 10: Educational Materials

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

Note: The following reference materials are anticipated to be included in this Appendix during final engineering stage at the time of the final WQMP.

- **SC-10 – Non-Stormwater Discharges**
- **SC-11 – Spill Prevention, Control & Cleanup**
- **SC-30 – Outdoor Loading/Unloading**
- **SC-34 – Waste Handling and Disposal**
- **SC-41 – Building & Grounds Maintenance**
- **SC-43 – Parking/Storage Area Maintenance**
- **SC-60 – Housekeeping Practices**
- **SD-10 – Site Design and Landscape Planning**
- **SD-11 – Roof Runoff Controls**
- **SD-12 – Efficient Irrigation**
- **SD-13 – Storm Drain Signage**
- **SD-32 – Trash Storage Areas**