Preliminary Water Quality Management Plan (WQMP)

Project Name:

La Habra Medical Office Medical Building

1201 West Whittier Boulevard

La Habra, CA 90631

Prepared for:

Los Altos, XIX, LP

1201 North Magnolia Avenue

Anaheim, CA 92801

Marc Antonio Arzola

714-778-3784

Prepared by:

Ware Malcomb

Engineer ______Luke Corsbie______ Registration No._72588

10 Edelman

Irvine, CA 92618

949-660-9128

October 20th, 2020

Project Owner's Certification

Permit/Application No.	CUP 16-04, CUP 16-05 & DR16-05	Grading Permit No.	
Tract/Parcel Map No.	Parcels 1 and 2, PMB 92/09	Building Permit No.	
CUP, SUP, and/or APN (Specify Lot Numbers if Portions of Tract)			APN: 017-152-16

This Water Quality Management Plan (WQMP) has been prepared for Los Altos XIX, LP by Ware Malcomb. The WQMP is intended to comply with the requirements of the local NPDES Stormwater Program requiring the preparation of the plan.

The undersigned, while it owns the subject property, is responsible for the implementation of the provisions of this plan and will ensure that this plan is amended as appropriate to reflect up-to-date conditions on the site consistent with the current Orange County Drainage Area Management Plan (DAMP) and the intent of the non-point source NPDES Permit for Waste Discharge Requirements for the County of Orange, Orange County Flood Control District and the incorporated Cities of Orange County within the Santa Ana Region. Once the undersigned transfers its interest in the property, its successors-in-interest shall bear the aforementioned responsibility to implement and amend the WQMP. An appropriate number of approved and signed copies of this document shall be available on the subject site in perpetuity.

Owner: Marc Antonio Arzola			
Title	Director of Construction and Maintenance		
Company	Los Altos, XIX, LP		
Address	1201 North Magnolia Avenue, Anaheim, California 92801		
Email			
Telephone #	714-778-3784		
Signature		Date	

ENGINEER'S CERTIFICATION

PRELIMINARY WATER QUALITY MANAGEMENT PLAN

Preparer (Engineer) Certification			
Preparer (Engineer): Lucas Corsbie			
Title	Director, Civil Engineering	PE Registration #	72588
Company	Ware Malcomb		
Address	10 Edelman, Irvine, CA 92618		
Email	lcorsbie@waremalcomb.com		
Telephone #	(949)-788-4059		
I hereby certify that this Water Quality Management Plan is in compliance with, and meets the requirements set forth in, Order No. R8-2009-0030/NPDES No. CAS618030, of the Santa Ana Regional Water Quality Control Board**.**			
Preparer Signature		Date	
Place Stamp Here			

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Attachments

Section I Discretionary Permit(s) and Water Quality Conditions

Provide discretionary permit and water quality information. *Refer to Section 2.1 in the Technical Guidance Document (TGD) available from the Orange County Stormwater Program (ocwatersheds.com).*

Project Infomation			
Permit/Application No.	CUP 16-05, CUP 16-05, & DR16-05	Tract/Parcel Map No.	Parcels 1 & 2 PMB 92/09
Additional Information/ Comments:	Project includes rehabilitation of an existing site and will affect more than 50% of the existing site area. Therefore, water quality treatment covers the entire property.		
Water Quality Conditions			
Water Quality Conditions (list verbatim)	A WQMP is required by the conditions of approval.		
Watershed-Based Plan Conditions			
Provide applicable conditions from watershed - based plans including WIHMPs and TMDLS.	There are currently no WIHMPS for the San Gabriel River-Coyote Creek Watershed. The San Gabriel River-Coyote Creek watershed has an established Heavy Metals TMDL (technical TMDL(1). (1) This TMDL has been adopted for the Coyote Creek/San Gabriel River by the Los Angeles Regional Water Quality Control Board (Region 4); however it applies to the areas of Orange County that drain to Coyote Creek and the San Gabriel River.		

Section II Project Description

II.1 Project Description

Provide a detailed project description including:

- Project areas;
- Land uses;
- Land cover;
- Design elements;
- A general description not broken down by drainage management areas (DMAs).

Include attributes relevant to determining applicable source controls. *Refer to Section 2.2 in the TGD for information that must be included in the project description.*

Description of Proposed Project		
Development Category (Verbatim from WQMP):	Category 8: All significant redevelopment projects, where significant redevelopment is defined as the addition or replacement or 5000 of more square feet of impervious surface on an already developed site. Redevelopment does not include routine maintenance activities that area conducted to maintain line and grade, hydraulic capacity, original purpose of the facility, or emergency redevelopment activity required to project public health and safety.	
Project Area (ft^2): 37,111	Number of Dwelling Units: N/A	SIC Code: 8011
Narrative Project Description:	The proposed project includes rehabilitation of an existing retail shopping center located on Whittier Boulevard in the City of La Habra. The site consists of one existing retail building located on 37,111 square feet (0.852 acres) of land area. The existing building is 10,079 sf of floor space with adjacent parking fields. The proposed project will remove the existing building and build a proposed medical office building The project will also replace and add landscaping within the parking fields for the site. The project will provide improvement to compliance with current ADA requirements along the front of the northerly building faces and access to Whittier Boulevard.	

Project Area	Pervious		Impervious	
	Area (acres or sq ft)	Percentage	Area (acres or sq ft)	Percentage
Pre-Project Conditions	1,727	5%	35,384	95%
Post-Project Conditions	5,636	15%	31,475	85%
Drainage Patterns/Connections	The existing site drainage is generally from the north to the south towards Whittier Blvd. The existing public storm drain system extends from Russell Street on the north property line and bisects the site as it extends to the south to Whittier Blvd. The site drains to this storm drain either by overland flow onto Whittier Blvd and then to the existing catch basin in Whittier Blvd. or into the onsite catch basins through the onsite private storm drain lines that connect to the existing public storm drain line. The public storm drain lines extend approximately 1.25 miles southerly from Whittier Blvd and the site to Coyote Creek. Coyote Creek drains to the southwest to the San Gabriel River and then to the Pacific Ocean just south of the Long Beach Harbor.			

II.2 Potential Stormwater Pollutants

Determine and list expected stormwater pollutants based on land uses and site activities. *Refer to Section 2.2.2 and Table 2.1 in the TGD for guidance.*

Pollutants of Concern			
Pollutant	Circle One: E=Expected to be of concern N=Not Expected to be of concern		Additional Information and Comments
Suspended-Solid/ Sediment	E	N	
Nutrients	E	N	
Heavy Metals	E	N	
Pathogens (Bacteria/Virus)	E	N	
Pesticides	E	N	

Oil and Grease	E	N	
Toxic Organic Compounds	E	N	
Trash and Debris	E	N	

II.3 Hydrologic Conditions of Concern

Determine if streams located downstream from the project area are determined to be potentially susceptible to hydromodification impacts. *Refer to Section 2.2.3.1 in the TGD for NOC or Section 2.2.3.2 for <SOC>.*

☐ No – Show map

☒ Yes – Describe applicable hydrologic conditions of concern below. *Refer to Section 2.2.3 in the TGD.*

In North County, a project does not have an HCOC if either of the following conditions is met:

- The volumes and time of concentration of stormwater runoff for the post-development condition do not significantly exceed those of the development condition for a two-year frequency storm event (a difference of five percent or less is considered significant).
- The site infiltrations at least the runoff from a two-year storm event.

The proposed site area being redeveloped is DMA 1 shown on the site plan. The existing 2-year 24-hour runoff volume is 2,140 cubic feet. The proposed 2-year 24-hour runoff volume is 1,867 cubic feet. See Worksheet C in Attachment D.

The time of concentration from the existing condition is 4.75 minutes and the time of concentration from the proposed condition is 6 minutes. See nomograph

The total runoff volume in the proposed condition is less than the existing condition because there is a large amount of landscaping introduced in the proposed condition. The volume calculation is attached. This meets the first requirement for no HCOC.

The Tc is greater in the proposed (redevelopment) condition from the DMA because existing flow is being rerouted via a proposed concrete valley gutter. This meets second requirement for no HCOC. The nomograph is attached.

II.4 Post Development Drainage Characteristics

Describe post development drainage characteristics. *Refer to Section 2.2.4 in the TGD.*

The proposed project will not alter the overall existing drainage patterns within the site. The project will decrease the overall impervious area by 12.4 percent by the addition of landscape areas. The drainage area where the existing building is being replaced with a new medical office building as well as the area within the property with Filterra tree boxes. There is no offsite run-on into the proposed Filterra unit.

The treatment flow rate is 0.174 cfs and the capacity of the Filterra unit is 0.21 cfs.

The project is also improving a number of the existing landscape islands within the site. Each of these islands will contain the irrigation and storm water within each island by keeping landscape material three to four inches below the tops of island curbs and providing soil amendments and gravel drains to improve water absorption into soil. There will be a number of islands that will be expanded in size as well as a number of islands will be added within the parking fields. The islands being added and expanded are distributed throughout the parking fields.

II.5 Property Ownership/Management

Describe property ownership/management. *Refer to Section 2.2.5 in the TGD.*

The property will be managed by a Property Management Company under the supervision of Los Altos, XIX, LP.

Section III Site Description

III.1 Physical Setting

Fill out table with relevant information. *Refer to Section 2.3.1 in the TGD.*

Planning Area/ Community Name	City of La Habra
Location/Address	1201 West Whittier Boulevard
	La Habra, California 90631
Land Use	Retail
Zoning	C2 Commercial
Acreage	0.852 acres
Predominant Soil Type	HSG D (See attached map)

III.2 Site Characteristics

Fill out table with relevant information and include information regarding BMP sizing, suitability, and feasibility, as applicable. Refer to Section 2.3.2 in the TGD.

Precipitation Zone	*0.95 inches*
Topography	*Graded and paved site drainage gradient is from north to south at an average rate of grade 3%.*
Drainage Patterns/Connections	*Onsite drainage patterns are from the north property line northerly of Whittier Boulevard. The existing public storm drain line runs from the north property line across the site to Whittier Boulevard and then southerly to Coyote Creek. The onsite generated storm flows are collected within the property in various catch basins within the site and conveyed to the public storm drain system.*

Soil Type, Geology, and Infiltration Properties	*The soil type is HSG D with very low infiltration rates (less than 0.30 inches/hour, see Soils Report and Infiltration Report).*

Site Characteristics (continued)	
Hydrogeologic (Groundwater) Conditions	*Soil sampling and testing exports soils to a depth of 25 feet and did not encounter groundwater.*
Geotechnical Conditions (relevant to infiltration)	*Existing site soil does not have an effective infiltration rate and therefore, infiltration BMPs cannot be used (see Soils Report and Infiltration Report).*
Off-Site Drainage	*There is no significant surface offsite drainage flow into the site.*
Utility and Infrastructure Information	*All perimeter sewer water and storm drain facilities that feed the site are owned by the City of La Habra.*

III.3 Watershed Description

Fill out table with relevant information and include information regarding BMP sizing, suitability, and feasibility, as applicable. *Refer to Section 2.3.3 in the TGD.*

Receiving Waters	San Gabriel River
303(d) Listed Impairments	Coliform, Bacteria, Copper, Dioxin, Lead, PH, Toxicity, and Zinc
Applicable TMDLs	Copper, Lead, and Selenium
Pollutants of Concern for the Project	Suspended solids/sediments, Nutrients, Pathogens, Pesticides, Oils and Grease, Toxic Organic Compounds, Trash and Debris
Environmentally Sensitive and Special Biological Significant Areas	None

Section IV Best Management Practices (BMPs)

IV. 1 Project Performance Criteria

Describe project performance criteria. Several steps must be followed in order to determine what performance criteria will apply to a project. These steps include:

- If the project has an approved WIHMP or equivalent, then any watershed specific criteria must be used and the project can evaluate participation in the approved regional or sub-regional opportunities. The local Permittee planning or NPDES staff should be consulted regarding the existence of an approved WIHMP or equivalent.
- Determine applicable hydromodification control performance criteria. *Refer to Section 7.II-2.4.2.2 of the Model WQMP.*
- Determine applicable LID performance criteria. *Refer to Section 7.II-2.4.3 of the Model WQMP.*
- Determine applicable treatment control BMP performance criteria. *Refer to Section 7.II-3.2.2 of the Model WQMP.*
- Calculate the LID design storm capture volume for the project. *Refer to Section 7.II-2.4.3 of the Model WQMP.*

<table>
<tr><td colspan="2">(NOC Permit Area only) Is there an approved WIHMP or equivalent for the project area that includes more stringent LID feasibility criteria or if there are opportunities identified for implementing LID on regional or sub-regional basis?</td><td>YES ☐</td><td>NO ☒</td></tr>
<tr><td>If yes, describe WIHMP feasibility criteria or regional/sub-regional LID opportunities.</td><td colspan="3"></td></tr>
</table>

Project Performance Criteria (continued)	
If HCOC exists, list applicable hydromodification control performance criteria (Section 7.II-2.4.2.2 in MWQMP)	The post development 2-year 24-hour storm flows and time of concentration do not exceed the pre-development 2-year 24-hour storm flows and time of concentration by more than 5%.
List applicable LID performance criteria (Section 7.II-2.4.3 from MWQMP)	The project must infiltrate, harvest and use, evaporatranspire, or bio treat/biofilter the 85th percentile 24-hour storm event.
List applicable treatment control BMP performance criteria (Section 7.II-3.2.2 from MWQMP)	If it is not feasible to meet LID performance criteria through retention and/or biotreatment provided on-site or at a sub-regional/regional scale, then treatment control BMPs shall be provided on-site or offsite prior to discharge to waters of the US. Sizing of treatment control BMP(s) shall be based on either the unmet volume after claiming applicable water quality credits, if appropriate (See Section 7.II-3.1 Water Quality Credits) and as calculated in TGD Appendix VI. If treatment control BMPs can treat all of the remaining unmet volume and have a medium to high effectiveness for reducing the primary POCs, the project is considered to be in compliance; a waiver application and participation in an alternative program is not required.
Calculate LID design storm capture volume for Project.	DMA 1 C = 0.75 X 0.85 + 0.15 = 0.786 D = 0.95 A = 0.852 DCV = 0.786 X 0.95 X 0.852 X 43560 SF/AC X 1/12 IN/FT = 2,217 CU-FT Due to low infiltration rates, infiltration and detention are not feasible. A Filterra unit is proposed instead to treat via flow based system.

	Q = (C X DI X DA X 43560/ (12 X 3600) Q = flow DA = drainage area (acres) DI = design intensity (in/hr) C = runoff coefficient (unitless) Q = 0.786 x 0.26 x 0.852 x 43560 / (12 x 3600) Q = 0.174 cfs Proposed 7x13 Filterra unit, capacity 0.21 cfs.

IV.2. SITE DESIGN AND DRAINAGE PLAN

Describe site design and drainage plan including

- A narrative of site design practices utilized or rationale for not using practices;
- A narrative of how site is designed to allow BMPs to be incorporated to the MEP
- A table of DMA characteristics and list of LID BMPs proposed in each DMA.
- Reference to the WQMP plot plan.
- Calculation of Design Capture Volume (DCV) for each drainage area.
- A listing of GIS coordinates for LID and Treatment Control BMPs (unless not required by local jurisdiction).

Refer to Section 2.4.2 in the TGD.

The project is more than 50% of the property. As such, the runoff from the entire project area will need to be treated. As the site's existing drainage areas and patterns were not altered, treatment methodology had to be able to accommodate localized drainage areas and flows in different areas. There is a proposed valley gutter that runs along the north and west of the site to capture runoff and convey it to a proposed Filterra tree box. The Filterra tree boxes fit into the site as independent systems capable of treating form flows from the localized areas. The WQMP indicates the location of Filterra tree box as well as the size, make, treatment capacity, and required capacity for the project site. The Filterra is located at the southwest of the site near the driveway.

IV.3 LID BMP SELECTION AND PROJECT CONFORMANCE ANALYSIS

Each sub-section below documents that the proposed design features conform to the applicable project performance criteria via check boxes, tables, calculations, narratives, and/or references to worksheets. *Refer to Section 2.4.2.3 in the TGD for selecting LID BMPs and Section 2.4.3 in the TGD for conducting conformance analysis with project performance criteria.*

IV.3.1 Hydrologic Source Controls

If required HSCs are included, fill out applicable check box forms. If the retention criteria are otherwise met with other LID BMPs, include a statement indicating HSCs not required.

Name	Included?
Localized on-lot infiltration	☐
Impervious area dispersion (e.g. roof top disconnection)	☐
Street trees (canopy interception)	☐
Residential rain barrels (not actively managed)	☐
Green roofs/Brown roofs	☐
Blue roofs	☐
Impervious area reduction (e.g. permeable pavers, site design)	☐
Other:	☐
Other:	☐
Other:	☐
Other:	☐
Other:	☐
Other:	☐
Other:	☐
Other:	☐

Hydrologic Source Controls are not required.

IV.3.2 Infiltration BMPs

Identify infiltration BMPs to be used in project. If design volume cannot be met state why BMPs cannot be met

Name	Included?
Bioretention without underdrains	☐
Rain gardens	☐
Porous landscaping	☐
Infiltration planters	☐
Retention swales	☐
Infiltration trenches	☐
Infiltration basins	☐
Drywells	☐
Subsurface infiltration galleries	☐
French drains	☐
Permeable asphalt	☐
Permeable concrete	☐
Permeable concrete pavers	☐
Other:	☐
Other:	☐

Show calculations below to demonstrate if the LID Design Strom Capture Volume can be met with infiltration BMPs. If not document how much can be met with infiltration and document why it is not feasible to meet the full volume with infiltration BMPs.

Onsite soil will not support infiltration. Soil testing determined that the rate of infiltration was less than 0.30 inches per house (see Soils Report and Infiltration Report).

IV.3.3 Evapotranspiration, Rainwater Harvesting BMPs

If the full Design Storm Capture Volume cannot be met with infiltration BMPs, describe any evapotranspiration, rainwater harvesting BMPs.

Name	Included?
All HSCs; *See Section IV.3.1*	☐
Surface-based infiltration BMPs	☐
Biotreatment BMPs	☐
Above-ground cisterns and basins	☐
Underground detention	☐
Other:	☐
Other:	☐
Other:	☐

Show calculations below to demonstrate if the LID Design Strom Capture Volume can be met with evapotranspiration, rainwater harvesting BMPs in combination with infiltration BMPs. If not document how much can be met with either infiltration BMPs, evapotranspiration, rainwater harvesting BMPs, or a combination, and document why it is not feasible to meet the full volume with either of these BMPs categories.

Existing site being redeveloped. The existing building will be demolished for a new proposed building. The building footprint and parking requirements do not allow for areas of surface ponding for evapotranspiration. The existing surface grading averages over 3 percent from north to south, which concentrates the surface flows along the street frontage with Whittier Boulevard and grade differential allows for only a slope down to a right of way. The parking requirements for the site do not allow for removal of any parking areas for BMP facilities. The rainwater harvesting BMP would be limited to the areas being disturbed, which are in different locations on the site that does not allow for an effective integration into the irrigation system.

IV.3.4 Biotreatment BMPs

If the full Design Storm Capture Volume cannot be met with infiltration BMPs, and/or evapotranspiration and rainwater harvesting BMPs, describe biotreatment BMPs. Include sections for selection, suitability, sizing, and infeasibility, as applicable.

Name	Included?
Bioretention with underdrains	☐
Stormwater planter boxes with underdrains	☐
Rain gardens with underdrains	☐
Constructed wetlands	☐
Vegetated swales	☐
Vegetated filter strips	☐
Proprietary vegetated biotreatment systems	☒
Wet extended detention basin	☐
Dry extended detention basins	☐
Other:	☐
Other:	☐

Show calculations below to demonstrate if the LID Design Storm Capture Volume can be met with infiltration, evapotranspiration, rainwater harvesting and/or biotreatment BMPs. If not document how much can be met with either infiltration BMPs, evapotranspiration, rainwater harvesting BMPs, or a combination, and document why it is not feasible to meet the full volume with either of these BMPs categories.

Existing site being redeveloped, with a new proposed building and small adjustments to the parking fields. The proposed building footprints and parking requirements to not provide sufficient area for large biotreatment BMPs. The existing surface grading averages over 3 percent from north to south, which concentrates the surface flows along the street frontage with Whittier Boulevard and the grade differential allows for only a slope down to right of way. The parking requirements for the site do not allow for removal of any parking areas for BMP facilities. The Filterra tree box does fit into the site plan and conforms to the recommendations of the TGD BIO-7.

IV.3.5 Hydromodification Control BMPs

Describe hydromodification control BMPs. See Section 5 TGD. Include sections for selection, suitability, sizing, and infeasibility, as applicable. Detail compliance with Prior Conditions of Approval. <Delete or leave blank if not used>

Hydromodification Control BMPs	
BMP Name	**BMP Description**

IV.3.6 Regional/Sub-Regional LID BMPs

Describe regional/sub-regional LID BMPs in which the project will participate. *Refer to Section 7.II-2.4.3.2 of the Model WQMP.*

Regional/Sub-Regional LID BMPs
Not applicable.

IV.3.7 Treatment Control BMPs

Treatment control BMPs can only be considered if the project conformance analysis indicates that it is not feasible to retain the full design capture volume with LID BMPs. Describe treatment control BMPs including sections for selection, sizing, and infeasibility, as applicable. <Delete or leave blank if not used>

Treatment Control BMPs	
BMP Name	**BMP Description**

IV.3.8 Non-structural Source Control BMPs

Fill out non-structural source control check box forms or provide a brief narrative explaining if non-structural source controls were not used.

Non-Structural Source Control BMPs				
Identifier	**Name**	**Check One**		**If not applicable, state brief reason**
		Included	**Not Applicable**	
N1	Education for Property Owners, Tenants and Occupants	☒	☐	
N2	Activity Restrictions	☒	☐	
N3	Common Area Landscape Management	☒	☐	
N4	BMP Maintenance	☒	☐	
N5	Title 22 CCR Compliance (How development will comply)	☒	☐	
N6	Local Industrial Permit Compliance	☐	☒	No industrial activities proposed.
N7	Spill Contingency Plan	☒	☐	
N8	Underground Storage Tank Compliance	☐	☒	No underground tanks proposed.
N9	Hazardous Materials Disclosure Compliance	☒	☐	
N10	Uniform Fire Code Implementation	☒	☐	
N11	Common Area Litter Control	☒	☐	
N12	Employee Training	☒	☐	
N13	Housekeeping of Loading Docks	☒	☐	
N14	Common Area Catch Basin Inspection	☒	☐	
N15	Street Sweeping Private Streets and Parking Lots	☒	☐	
N16	Retail Gasoline Outlets	☐	☒	No gasoline facilities proposed.

IV.3.9 Structural Source Control BMPs

Fill out structural source control check box forms or provide a brief narrative explaining if Structural source controls were not used.

Structural Source Control BMPs				
Identifier	**Name**	**Check One**		**If not applicable, state brief reason**
		Included	**Not Applicable**	
S1	Provide storm drain system stenciling and signage	☒	☐	
S2	Design and construct outdoor material storage areas to reduce pollution introduction	☒	☐	
S3	Design and construct trash and waste storage areas to reduce pollution introduction	☒	☐	
S4	Use efficient irrigation systems & landscape design, water conservation, smart controllers, and source control	☒	☐	
S5	Protect slopes and channels and provide energy dissipation	☐	☒	No onsite open channels.
	Incorporate requirements applicable to individual priority project categories (from SDRWQCB NPDES Permit)	☒	☐	
S6	Dock areas	☐	☒	Not proposed within project.
S7	Maintenance bays	☐	☒	Not proposed within project.
S8	Vehicle wash areas	☐	☒	Not proposed within project.
S9	Outdoor processing areas	☐	☒	Not proposed within project.
S10	Equipment wash areas	☐	☒	Not proposed within project.
S11	Fueling areas	☐	☒	Not proposed within project.
S12	Hillside landscaping	☐	☒	Not proposed within project.
S13	Wash water control for food preparation areas	☐	☒	Not proposed within project.
S14	Community car wash racks	☐	☒	Not proposed within project.

IV.4 ALTERNATIVE COMPLIANCE PLAN (IF APPLICABLE)

IV.4.1 Water Quality Credits

Determine if water quality credits are applicable for the project. *Refer to Section 3.1 of the Model WQMP for description of credits and Appendix VI of the TGD for calculation methods for applying water quality credits.*

<table>
<tr><th colspan="7">Description of Proposed Project</th></tr>
<tr><td colspan="7">Project Types that Qualify for Water Quality Credits (Select all that apply):</td></tr>
<tr><td colspan="2">☐Redevelopment projects that reduce the overall impervious footprint of the project site.</td><td colspan="3">☐Brownfield redevelopment, meaning redevelopment, expansion, or reuse of real property which may be complicated by the presence or potential presence of hazardous substances, pollutants or contaminants, and which have the potential to contribute to adverse ground or surface WQ if not redeveloped.</td><td colspan="2">☐ Higher density development projects which include two distinct categories (credits can only be taken for one category): those with more than seven units per acre of development (lower credit allowance); vertical density developments, for example, those with a Floor to Area Ratio (FAR) of 2 or those having more than 18 units per acre (greater credit allowance).</td></tr>
<tr><td colspan="2">☐ Mixed use development, such as a combination of residential, commercial, industrial, office, institutional, or other land uses which incorporate design principles that can demonstrate environmental benefits that would not be realized through single use projects (e.g. reduced vehicle trip traffic with the potential to reduce sources of water or air pollution).</td><td colspan="4">☐ Transit-oriented developments, such as a mixed use residential or commercial area designed to maximize access to public transportation; similar to above criterion, but where the development center is within one half mile of a mass transit center (e.g. bus, rail, light rail or commuter train station). Such projects would not be able to take credit for both categories, but may have greater credit assigned</td><td>☐ Redevelopment projects in an established historic district, historic preservation area, or similar significant city area including core City Center areas (to be defined through mapping).</td></tr>
<tr><td>☐Developments with dedication of undeveloped portions to parks, preservation areas and other pervious uses.</td><td colspan="2">☐ Developments in a city center area.</td><td>☐ Developments in historic districts or historic preservation areas.</td><td colspan="2">☐ Live-work developments, a variety of developments designed to support residential and vocational needs together – similar to criteria to mixed use development; would not be able to take credit for both categories.</td><td>☐In-fill projects, the conversion of empty lots and other underused spaces into more beneficially used spaces, such as residential or commercial areas.</td></tr>
</table>

Calculation of Water Quality Credits (if applicable)	Not applicable.

IV.4.2 Alternative Compliance Plan Information

Describe an alternative compliance plan (if applicable). Include alternative compliance obligations (i.e., gallons, pounds) and describe proposed alternative compliance measures. *Refer to Section 7.II 3.0 in the WQMP.*

Alternative compliance plans are not being proposed.

Section V Inspection/Maintenance Responsibility for BMPs

Fill out information in table below. Prepare and attach an Operation and Maintenance Plan. Identify the mechanism through which BMPs will be maintained. Inspection and maintenance records must be kept for a minimum of five years for inspection by the regulatory agencies. *Refer to Section 7.II 4.0 in the Model WQMP.*

BMP Inspection/Maintenance			
BMP	**Reponsible Party(s)**	**Inspection/ Maintenance Activities Required**	**Minimum Frequency of Activities**
Landscape and Irrigation	Property Management	Inspection of irrigation system and landscaping	Weekly
Parking Lot Sweeping	Property Management	Removal of trash, debris, and standing water	Bi-weekly
Trash Enclosures	Property Management	Inspection and removal of any debris or trash within enclosure	Weekly
Catch basins	Property Management	Inspect and remove any debris within basin	Monthly and prior to rain
Property Owner and employee training	Property management and individual tenants	Review the WQMP document and applicable attachments	Ongoing with every new tenant and new hire

Filterra Tree box	Property Management	Inspect and remove any trash and debris within the box and any additional maintenance recommendations of the manufacture.	Bi-weekly and prior to rain or in compliance with manufacturer's recommendation.

Section VI Site Plan and Drainage Plan

VI.1 SITE PLAN AND DRAINAGE PLAN

Include a site plan and drainage plan sheet set containing the following minimum information:

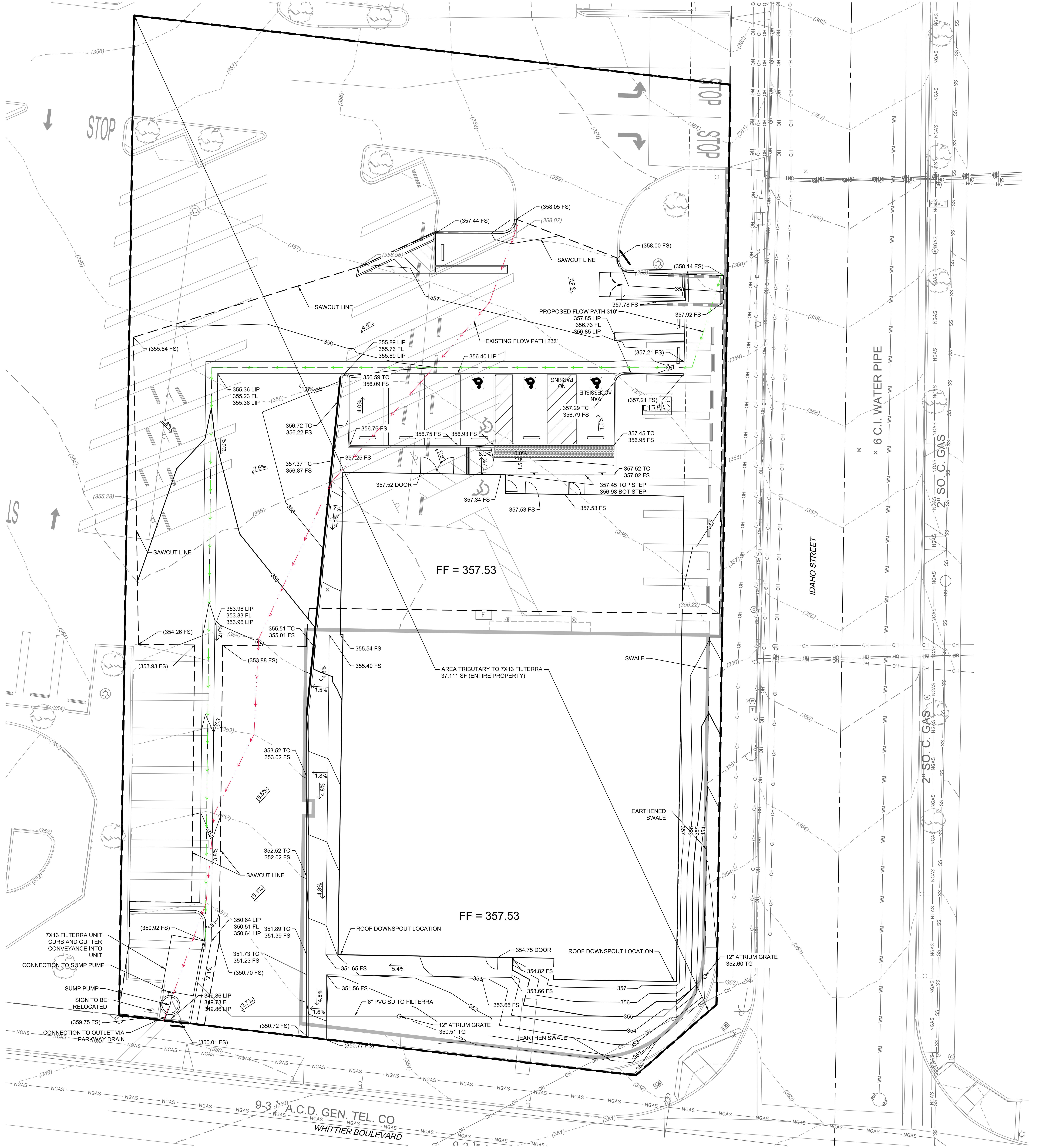
- Project location
- Site boundary
- Land uses and land covers, as applicable
- Suitability/feasibility constraints
- Structural BMP locations
- Drainage delineations and flow information
- Drainage connections
- BMP details

VI.2 ELECTRONIC DATA SUBMITTAL

The minimum requirement is to provide submittal of PDF exhibits in addition to hard copies. Format must not require specialized software to open.

If the local jurisdiction requires specialized electronic document formats (CAD, GIS) to be submitted, this section will be used to describe the contents (e.g., layering, nomenclature, georeferencing, etc.) of these documents so that they may be interpreted efficiently and accurately.

CAUTION: IF THIS SHEET IS NOT 30"x42" IT IS A REDUCED PRINT



LEGEND:

	PROPERTY LINE	BW	BACK OF WALK
	CENTERLINE	FF	FINISHED FLOOR
100	EXISTING 1' CONTOUR	FL	FLOW LINE
	PROPOSED EARTHEN SWALE	FS	FINISHED SURFACE
XX.XX	PROPOSED SPOT ELEVATION	GB	GRADE BREAK
X.XX%	PROPOSED SLOPE AND DIRECTION	LP	LOW POINT
	SAWCUT LINE	PA	PLANTER AREA
	DMA BOUNDARY	TC	TOP OF CURB
	PROPOSED LONGEST FLOW PATH		
	EXISTING LONGEST FLOW PATH		

BASIS OF BEARINGS:

THE BEARING OF NORTH 84°18'30" WEST ON THE CENTERLINE OF WHITTIER BOUELVARD, AS SHOWN ON PARCEL MAP BOOK 92 / 9, IN THE CITY OF LA HABRA, COUNTY OF ORANGE, STATE OF CALIFORNIA, WAS USED AS THE BASIS OF BEARINGS.

BENCHMARK:

ORANGE COUNTY VERTICAL CONTROL DESIGNATION 2K-15-70

FOUND 3-3/4" OCS ALUMINUM BENCHMARK DISK STAMPED "2K-15-70", SET IN THE SOUTHEASTERLY CORNER OF A 4' X 8' CATCH BASIN. LOCATED IN THE NORTHEASTERLY CORNER OF THE INTERSECTION OF WALNUT STREET AND WHITTIER BOULEVARD, 44' NORTHERLY AND 39.5' EASTLY OF SAID CENTERLINE INTERSECTION.

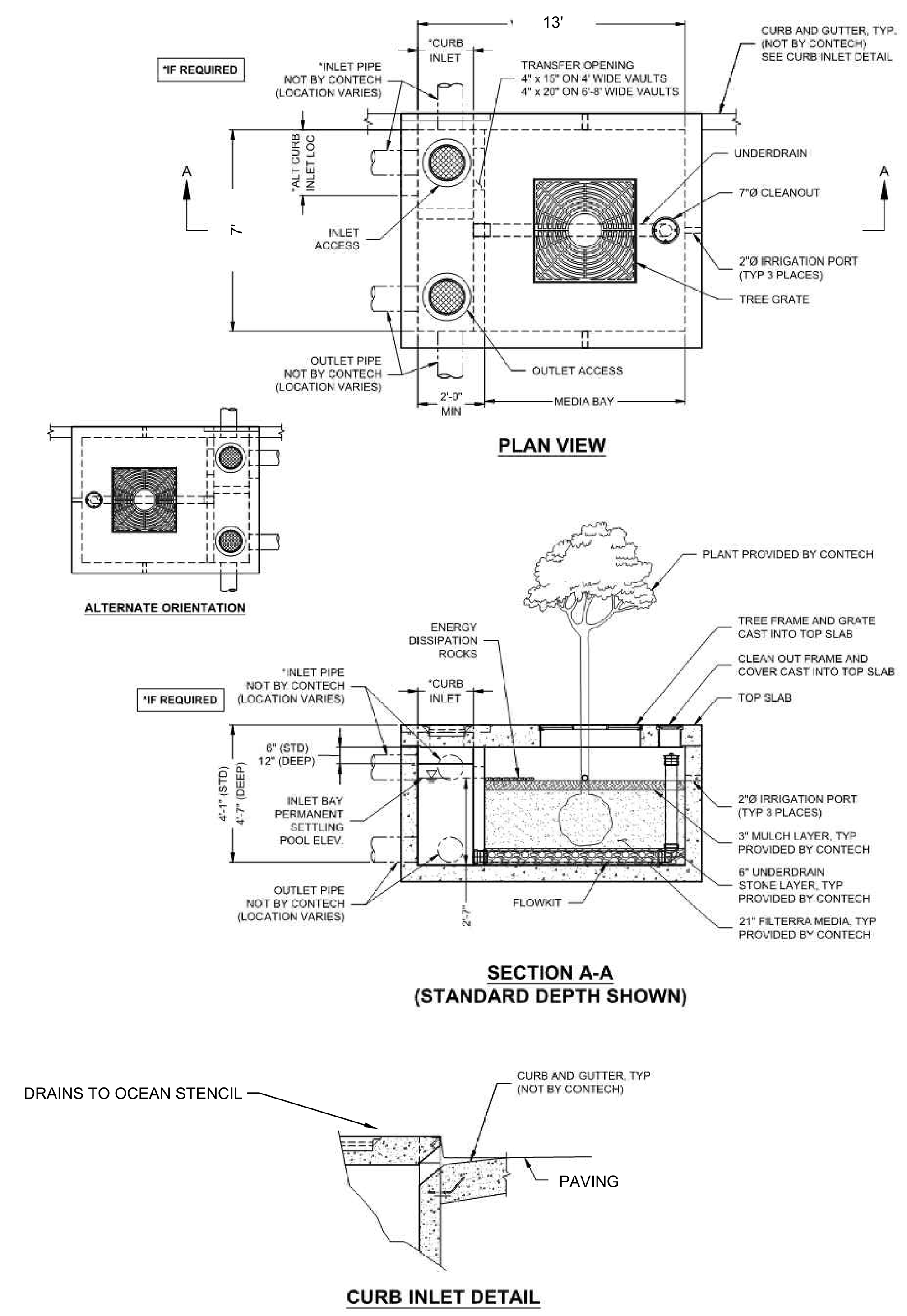
NOTE: THE EXISTENCE OR LOCATION OF ANY UNDERGROUND UTILITIES, PIPES OF STRUCTURES SHOWN ON THIS SURVEY WERE OBTAINED BY A SEARCH OF AVAILABLE RECORDS. THE CONTRACTOR SHALL ASCERTAIN THE TRUE LOCATION AND/OR EXISTENCE OF UNDERGROUND UTILITIES AND SHALL BE RESPONSIBLE FOR ALL DAMAGE TO ANY PUBLIC OR PRIVATE UTILITIES SHOWN OR NOT SHOWN HEREON.

BMP SIZING

Q = (C X DI X DA X 43560/ (12 X 3600)

Q = FLOW
DA = DRAINAGE AREA (ACRES)
DI = DESIGN INTENSITY (IN/HR)
C = RUNOFF COEFFICIENT (UNITLESS)

Q = 0.786 X 0.23 X 0.852 X 43560 / (12 X 3600)
Q = 0.174 CFS
PROPOSED (1) 7X13 FILTERRA UNITS, CAPACITY 0.21 CFS.



PLAN VIEW

ALTERNATE ORIENTATION

SECTION A-A
(STANDARD DEPTH SHOWN)

CURB INLET DETAIL

FILTERRA DETAIL

BMP -STORM DRAIN INLET NO DUMPING STENCIL

SCALE: 1" = 10'

WARE MALCOMB
CIVIL ENGINEERING & SURVEYING

10 edelman
irvine, ca 92618
p 949.660.9128
waremalcomb.com

REGISTERED PROFESSIONAL ENGINEER
LUCAS A. CORSBIE
No. 72588
CIVIL
STATE OF CALIFORNIA

FOR AND ON BEHALF OF WARE MALCOMB

LA HABRA HERITAGE MEDICAL OFFICE BUILDING

PRELIM WQMP EXHIBIT

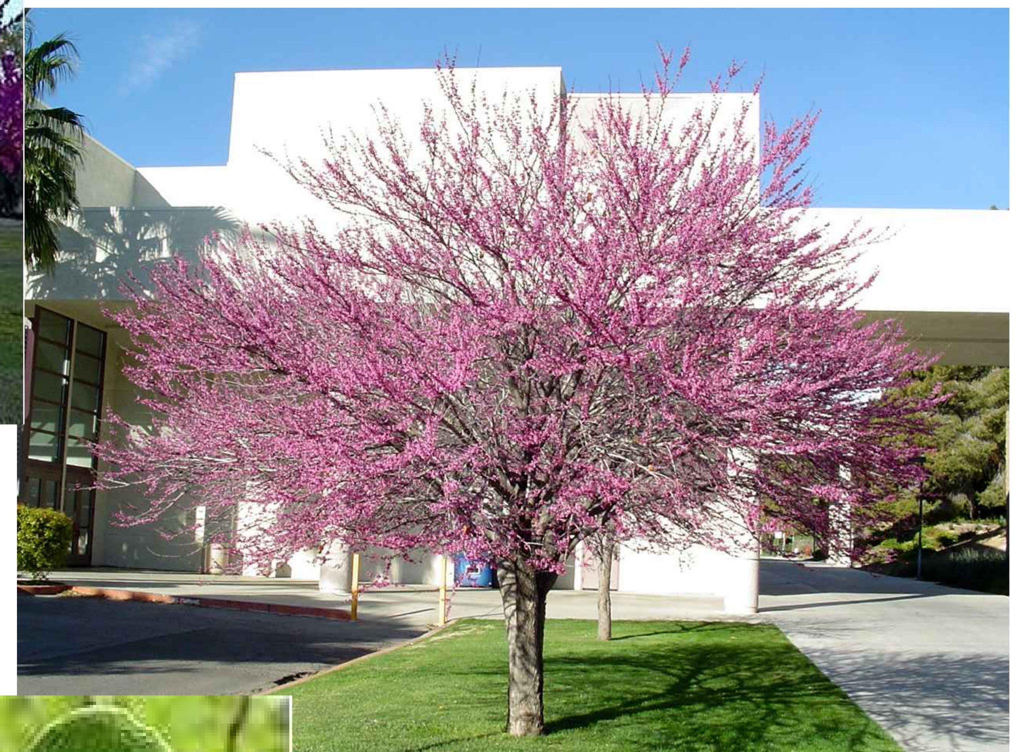
NO.	DATE	REMARKS

JOB NO.:	IRV19-3021
PA / PM:	LC
DRAWN BY:	IH
DATE:	07/13/20
PLOT DATE:	

SHEET
C1.0
Sheet 1 of 1

NOT FOR CONSTRUCTION

CERCIS CANADENSIS /
EASTERN REDBUD

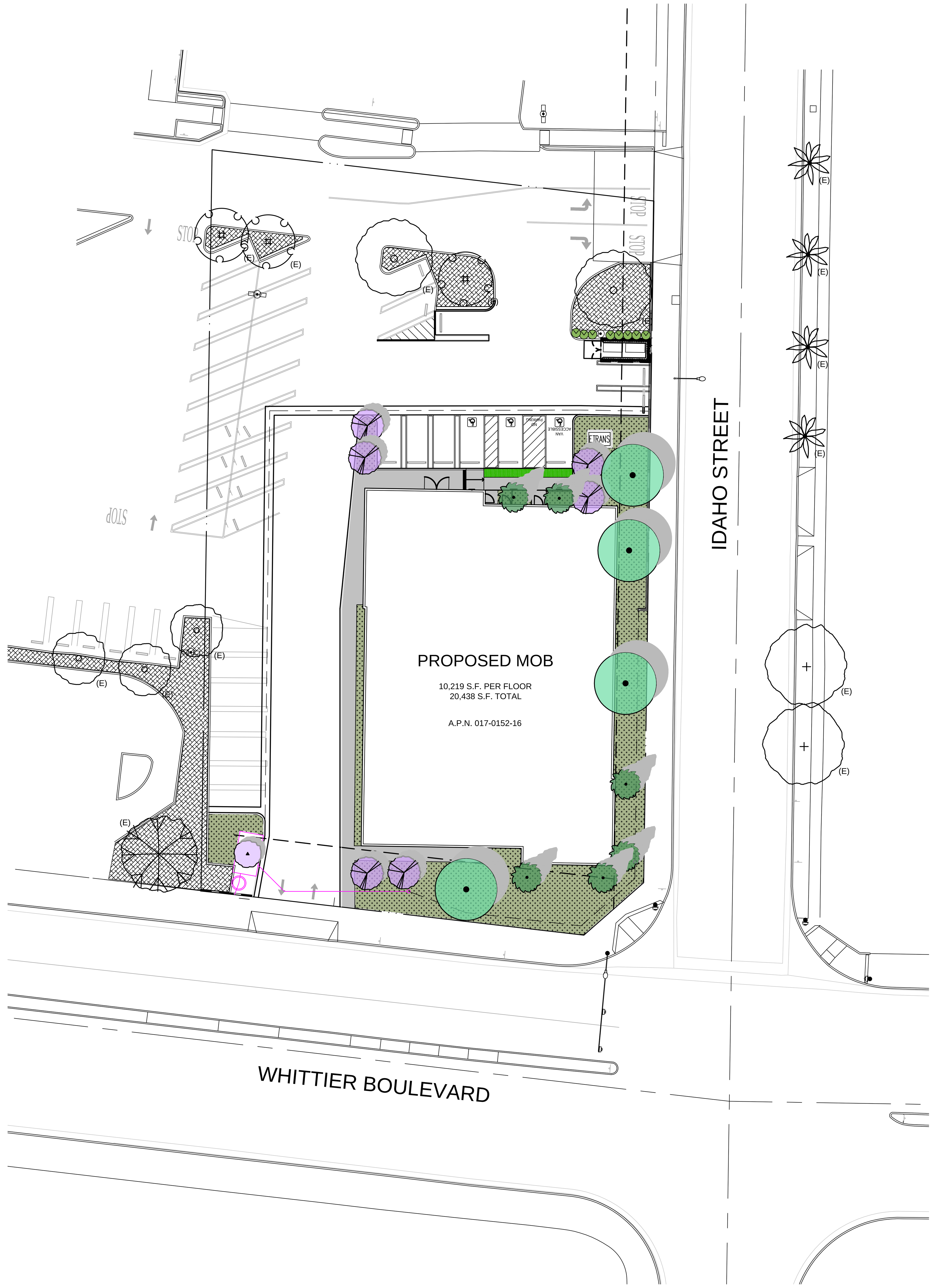


CERCIS OCCIDENTALIS /
WESTERN REDBUD

PODOCARPUS GRACILIOR /
FERN PINE

CUPRESSUS SEMPERVIRENS /
ITALIAN CYPRESS

ARBUTUS MARINA /
NCN (NO COMMON NAME)



PLANTING LEGEND

SYMBOL	BOTANICAL / COMMON NAME	SIZE
PROPOSED TREES		
	ARBUTUS MARINA / NCN	24" BOX MULTI-TRUNK
	CERCIS OCCIDENTALIS / WESTERN REDBUD	24" BOX STANDARD FORM
	CERCIS CANADENSIS / EASTERN REDBUD	15 GALLON STANDARD FORM
	CUPRESSUS SEMPERVIRENS / ITALIAN CYPRESS	24" BOX COLUMN FORM
	OR	
	PODOCARPUS GRACILIOR / FERN PINE	24" BOX COLUMN FORM

SYMBOL	BOTANICAL / COMMON NAME	SIZE
PROPOSED SHRUBS		
	AGAVE DESMETTIANA / DWARF AGAVE	1 GAL / 42" O.C.
	DIETES VEGETA / FORTNIGHT LILY	1 GAL / 36" O.C.
	CALLISTEMON 'LITTLE JOHN' / DWARF BOTTLE BRUSH	1 GAL / 30" O.C.
	HESPERALOE PARVIFLORA / RED YUCCA	1 GAL / 30" O.C.
	NASSELLA TENUISSIMA / MEXICAN FEATHER GRASS	1 GAL / 42" O.C.
	PHORMIUM T. 'SURFER' / SURFER NEW ZEALAND FLAX	1 GAL / 36" O.C.
	ELAEAGNUS P. 'FRUITLANDII' / FRUITLAND SILVERBERRY	5 GAL / 36" O.C.
PROPOSED GROUNDCOVER & VINES		
	SEDUM 'BLUE SPRUCE' / BLUE SPRUCE STONECROP	1 GAL / 18" O.C.
	ROSMARINUS 'HUNTINGTON CARPET' / HUNTINGTON CARPET ROSEMARY	1 GAL / 24" O.C.
	LANTANA 'NEW GOLD' / NEW GOLD LANTANA	1 GAL / 24" O.C.
	TRACHELOSPERMUM JASMINOIDES / STAR JASMINE	1 GAL / 24" O.C.
	FICUS REPENS / CREEPING FIG	5 GAL / PER PLAN

EXISTING TREES

SYMBOL	BOTANICAL / COMMON NAME
EXISTING TREES	
(E)	SCHINUS TEREBINTHIFOLIUS / BRAZILIAN PEPPER TREE
(E)	TRISTANIA CONFERTA / BRISBANE BOX
(E)	EUCALYPTUS SP. / EUCALYPTUS
(E)	PLATANUS ACERIFOLIA / LONDON PLANE TREE

REGISTERED LANDSCAPE ARCHITECT
Colleen M. Nolan
Lic. #5439
(Signature)
07.06.2020
(Date)
STATE OF CALIFORNIA

Colleen M. Nolan
Landscape Architect, #5439
13355 Silverado Court
Corona, CA 92883
714.743.7915 cell cnolan@cox.net

SCALE: 1"=20'-0"

0 20 40 60

NORTH

Preliminary Landscape Plan

Whittier Blvd / Idaho Street
La Habra, CA

WARE MALCOMB

IRV20-0000-00
09.23.2020

SHEET
L1.1

Section VII Educational Materials

Refer to the Orange County Stormwater Program (ocwatersheds.com) for a library of materials available. For the copy submitted to the Permittee, only attach the educational materials specifically applicable to the project. Other materials specific to the project may be included as well and must be attached.

Education Materials			
Residential Material (http://www.ocwatersheds.com)	**Check If Applicable**	**Business Material (http://www.ocwatersheds.com)**	**Check If Applicable**
The Ocean Begins at Your Front Door	☐	Tips for the Automotive Industry	☐
Tips for Car Wash Fund-raisers	☐	Tips for Using Concrete and Mortar	☐
Tips for the Home Mechanic	☐	Tips for the Food Service Industry	☒
Homeowners Guide for Sustainable Water Use	☐	Proper Maintenance Practices for Your Business	☒
Household Tips	☐	**Other Material**	**Check If Attached**
Proper Disposal of Household Hazardous Waste	☐		
Recycle at Your Local Used Oil Collection Center (North County)	☐	Filterra Manufacturer's Manuals	☒
Recycle at Your Local Used Oil Collection Center (Central County)	☐		☐
Recycle at Your Local Used Oil Collection Center (South County)	☐		☐
Tips for Maintaining a Septic Tank System	☐		☐
Responsible Pest Control	☐		☐
Sewer Spill	☐		☐
Tips for the Home Improvement Projects	☐		☐
Tips for Horse Care	☐		☐
Tips for Landscaping and Gardening	☒		☐
Tips for Pet Care	☐		☐
Tips for Pool Maintenance	☐		☐

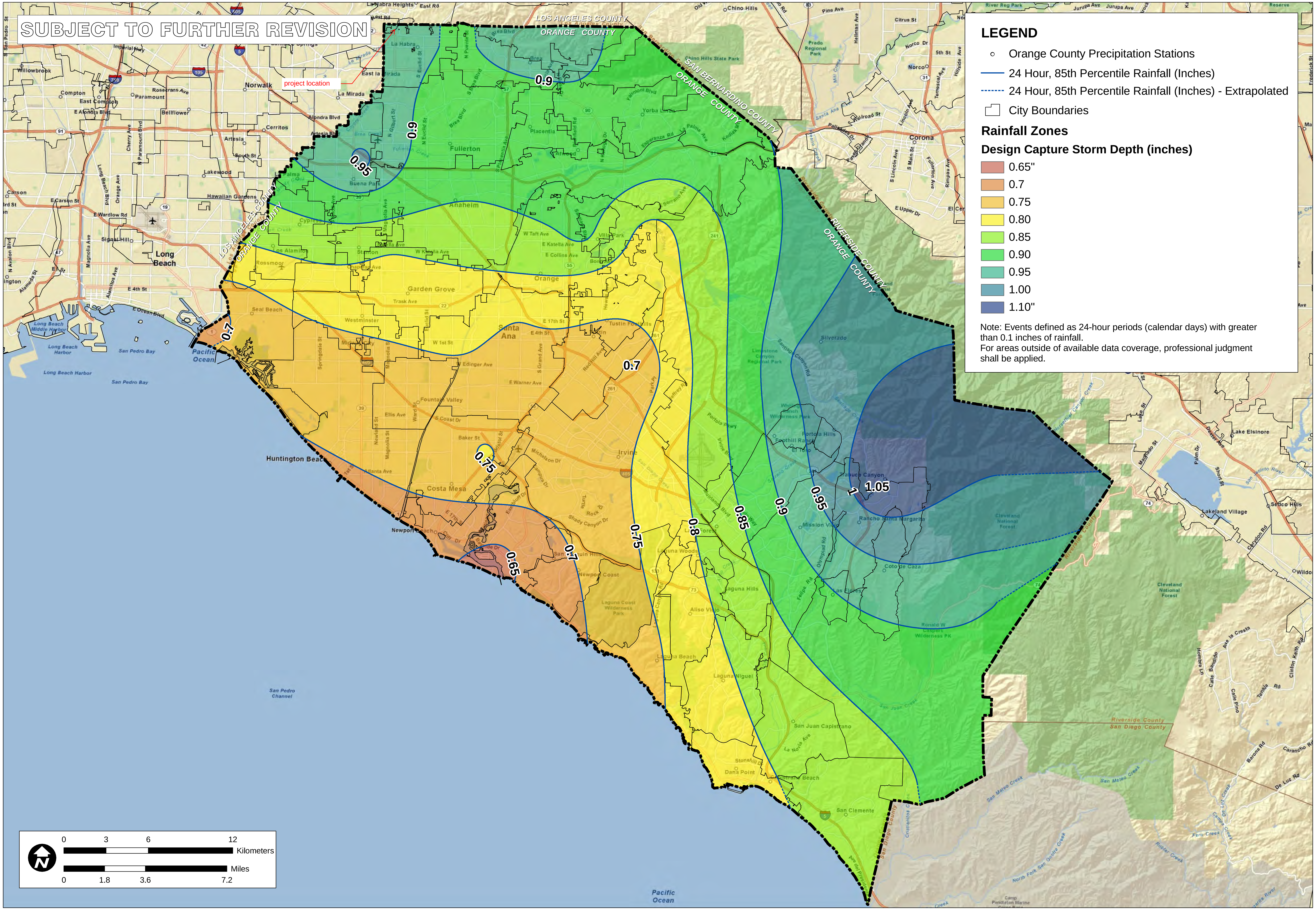
Tips for Residential Pool, Landscape and Hardscape Drains	☐		☐
Tips for Projects Using Paint	☐		☐

Attachment A

Attachment B

Attachment C

Attachment D



TITLE
RAINFALL ZONES

JOB
ORANGE COUNTY TECHNICAL GUIDANCE DOCUMENT
ORANGE CO.
CA

SCALE	1" = 1.8 miles
DESIGNED	TH
DRAWING	TH
CHECKED	BMP
DATE	04/22/10
JOB NO.	9526-E

PACE Advanced Water Engineering

FIGURE
XVI-1

P:\9526E\6-GIS\Mxds\Reports\InfiltrationFeasability_20110215\9526E_FigureXVI-1_RainfallZones_20110215.mxd

NOAA Atlas 14, Volume 6, Version 2
Location name: La Habra, California, USA*
Latitude: 33.9393°, Longitude: -117.9596°
Elevation: 351.24 ft**
* source: ESRI Maps
** source: USGS

POINT PRECIPITATION FREQUENCY ESTIMATES

Sanja Perica, Sarah Dietz, Sarah Heim, Lillian Hiner, Kazungu Maitaria, Deborah Martin, Sandra Pavlovic, Ishani Roy, Carl Trypaluk, Dale Unruh, Fenglin Yan, Michael Yekta, Tan Zhao, Geoffrey Bonnin, Daniel Brewer, Li-Chuan Chen, Tye Parzybok, John Yarchoan

NOAA, National Weather Service, Silver Spring, Maryland

PF_tabular | PF_graphical | Maps_&_aerials

PF tabular

PDS-based point precipitation frequency estimates with 90% confidence intervals (in inches/hour)[1]

Duration	Average recurrence interval (years)									
	1	2	5	10	25	50	100	200	500	1000
5-min	**1.66** (1.39-2.00)	**2.11** (1.76-2.54)	**2.71** (2.26-3.29)	**3.20** (2.65-3.92)	**3.89** (3.10-4.92)	**4.42** (3.44-5.72)	**4.97** (3.77-6.60)	**5.53** (4.08-7.57)	**6.31** (4.46-9.04)	**6.94** (4.72-10.3)
10-min	**1.19** (0.996-1.43)	**1.51** (1.27-1.83)	**1.94** (1.62-2.35)	**2.30** (1.90-2.81)	**2.78** (2.22-3.53)	**3.17** (2.47-4.10)	**3.56** (2.71-4.73)	**3.97** (2.93-5.43)	**4.52** (3.19-6.48)	**4.97** (3.38-7.37)
15-min	**0.956** (0.800-1.16)	**1.22** (1.02-1.47)	**1.57** (1.30-1.90)	**1.85** (1.53-2.26)	**2.25** (1.79-2.84)	**2.55** (1.99-3.31)	**2.87** (2.18-3.81)	**3.20** (2.36-4.38)	**3.65** (2.58-5.22)	**4.00** (2.73-5.94)
30-min	**0.656** (0.548-0.792)	**0.834** (0.698-1.01)	**1.07** (0.894-1.30)	**1.27** (1.05-1.55)	**1.54** (1.23-1.95)	**1.75** (1.36-2.26)	**1.96** (1.49-2.61)	**2.19** (1.61-3.00)	**2.50** (1.76-3.57)	**2.74** (1.87-4.07)
60-min	**0.465** (0.389-0.561)	**0.591** (0.494-0.715)	**0.760** (0.633-0.921)	**0.899** (0.742-1.10)	**1.09** (0.869-1.38)	**1.24** (0.966-1.60)	**1.39** (1.06-1.85)	**1.55** (1.14-2.12)	**1.77** (1.25-2.53)	**1.94** (1.32-2.88)
2-hr	**0.336** (0.280-0.404)	**0.428** (0.358-0.516)	**0.548** (0.457-0.664)	**0.646** (0.534-0.790)	**0.780** (0.622-0.989)	**0.884** (0.689-1.15)	**0.988** (0.752-1.31)	**1.10** (0.809-1.50)	**1.24** (0.878-1.78)	**1.36** (0.924-2.02)
3-hr	**0.279** (0.234-0.337)	**0.356** (0.298-0.431)	**0.457** (0.381-0.553)	**0.538** (0.445-0.658)	**0.648** (0.516-0.821)	**0.732** (0.571-0.948)	**0.817** (0.621-1.09)	**0.904** (0.667-1.24)	**1.02** (0.721-1.46)	**1.11** (0.757-1.65)
6-hr	**0.199** (0.167-0.240)	**0.255** (0.213-0.307)	**0.326** (0.272-0.395)	**0.384** (0.317-0.469)	**0.461** (0.368-0.584)	**0.520** (0.405-0.673)	**0.579** (0.440-0.769)	**0.639** (0.471-0.875)	**0.719** (0.508-1.03)	**0.781** (0.532-1.16)
12-hr	**0.131** (0.110-0.158)	**0.168** (0.140-0.203)	**0.215** (0.179-0.261)	**0.253** (0.209-0.310)	**0.305** (0.243-0.386)	**0.344** (0.268-0.445)	**0.383** (0.291-0.509)	**0.423** (0.312-0.579)	**0.476** (0.336-0.681)	**0.517** (0.352-0.767)
24-hr	**0.091** (0.080-0.105)	**0.116** (0.103-0.135)	**0.150** (0.132-0.174)	**0.177** (0.155-0.207)	**0.214** (0.181-0.258)	**0.242** (0.201-0.298)	**0.270** (0.219-0.341)	**0.299** (0.236-0.388)	**0.339** (0.256-0.457)	**0.369** (0.270-0.515)
2-day	**0.056** (0.049-0.064)	**0.072** (0.064-0.084)	**0.094** (0.083-0.109)	**0.112** (0.098-0.131)	**0.136** (0.116-0.165)	**0.155** (0.129-0.191)	**0.174** (0.141-0.220)	**0.194** (0.153-0.251)	**0.221** (0.167-0.298)	**0.241** (0.177-0.337)
3-day	**0.041** (0.037-0.048)	**0.055** (0.048-0.063)	**0.072** (0.064-0.084)	**0.087** (0.076-0.101)	**0.106** (0.090-0.128)	**0.121** (0.100-0.149)	**0.136** (0.110-0.172)	**0.152** (0.120-0.197)	**0.174** (0.131-0.234)	**0.190** (0.139-0.265)
4-day	**0.033** (0.029-0.038)	**0.044** (0.039-0.051)	**0.059** (0.052-0.068)	**0.071** (0.062-0.083)	**0.088** (0.074-0.106)	**0.100** (0.083-0.123)	**0.113** (0.092-0.143)	**0.127** (0.100-0.164)	**0.145** (0.110-0.196)	**0.159** (0.117-0.222)
7-day	**0.022** (0.019-0.025)	**0.029** (0.025-0.033)	**0.038** (0.034-0.044)	**0.047** (0.041-0.054)	**0.058** (0.049-0.070)	**0.067** (0.055-0.082)	**0.076** (0.061-0.096)	**0.086** (0.067-0.111)	**0.099** (0.075-0.134)	**0.110** (0.080-0.153)
10-day	**0.016** (0.014-0.019)	**0.022** (0.019-0.025)	**0.029** (0.026-0.034)	**0.035** (0.031-0.041)	**0.044** (0.037-0.053)	**0.051** (0.043-0.063)	**0.059** (0.048-0.074)	**0.067** (0.052-0.086)	**0.078** (0.059-0.105)	**0.087** (0.063-0.121)
20-day	**0.010** (0.008-0.011)	**0.013** (0.011-0.015)	**0.017** (0.015-0.020)	**0.021** (0.019-0.025)	**0.027** (0.023-0.032)	**0.031** (0.026-0.038)	**0.036** (0.029-0.045)	**0.041** (0.032-0.053)	**0.049** (0.037-0.066)	**0.055** (0.040-0.076)
30-day	**0.007** (0.007-0.009)	**0.010** (0.009-0.012)	**0.014** (0.012-0.016)	**0.017** (0.015-0.019)	**0.021** (0.018-0.025)	**0.025** (0.020-0.030)	**0.029** (0.023-0.036)	**0.033** (0.026-0.042)	**0.039** (0.029-0.052)	**0.044** (0.032-0.061)
45-day	**0.006** (0.005-0.007)	**0.008** (0.007-0.009)	**0.011** (0.009-0.012)	**0.013** (0.011-0.015)	**0.017** (0.014-0.020)	**0.019** (0.016-0.024)	**0.022** (0.018-0.028)	**0.026** (0.020-0.033)	**0.030** (0.023-0.041)	**0.034** (0.025-0.048)
60-day	**0.005** (0.004-0.006)	**0.007** (0.006-0.008)	**0.009** (0.008-0.011)	**0.011** (0.010-0.013)	**0.014** (0.012-0.017)	**0.017** (0.014-0.020)	**0.019** (0.015-0.024)	**0.022** (0.017-0.028)	**0.026** (0.019-0.035)	**0.029** (0.021-0.040)

[1] Precipitation frequency (PF) estimates in this table are based on frequency analysis of partial duration series (PDS).

Numbers in parenthesis are PF estimates at lower and upper bounds of the 90% confidence interval. The probability that precipitation frequency estimates (for a given duration and average recurrence interval) will be greater than the upper bound (or less than the lower bound) is 5%. Estimates at upper bounds are not checked against probable maximum precipitation (PMP) estimates and may be higher than currently valid PMP values.

Please refer to NOAA Atlas 14 document for more information.

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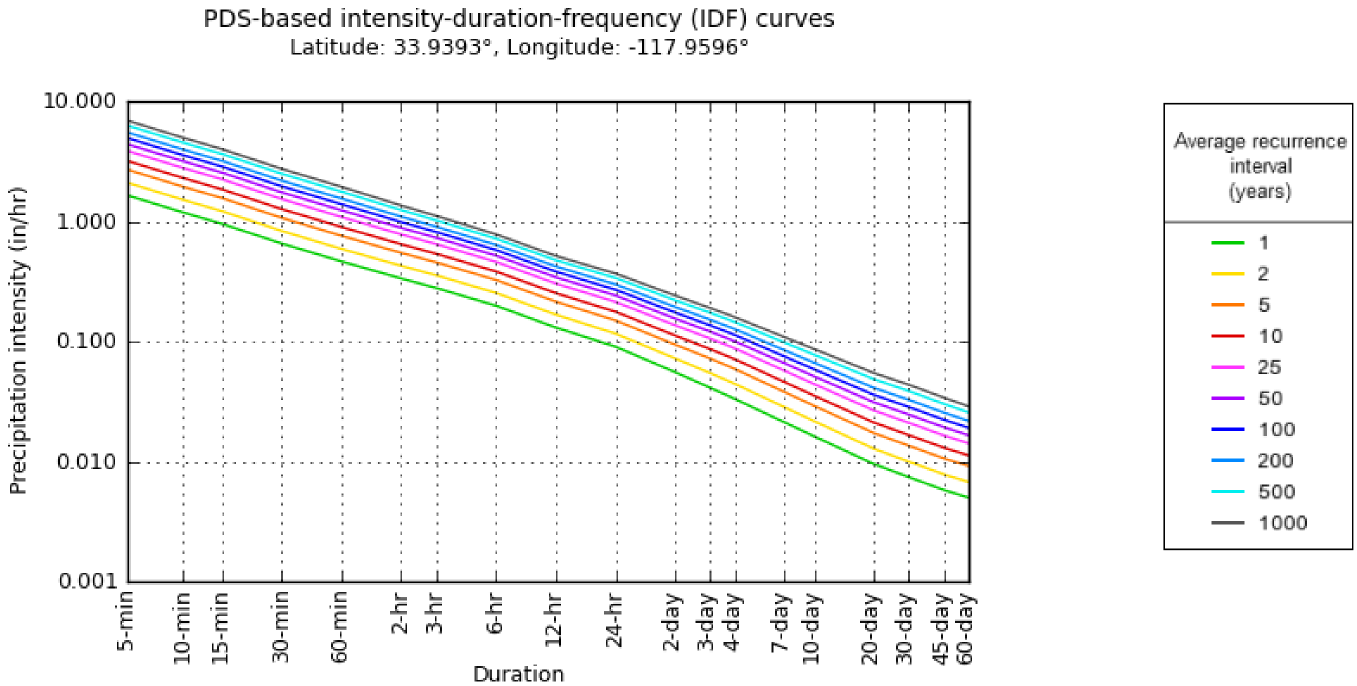
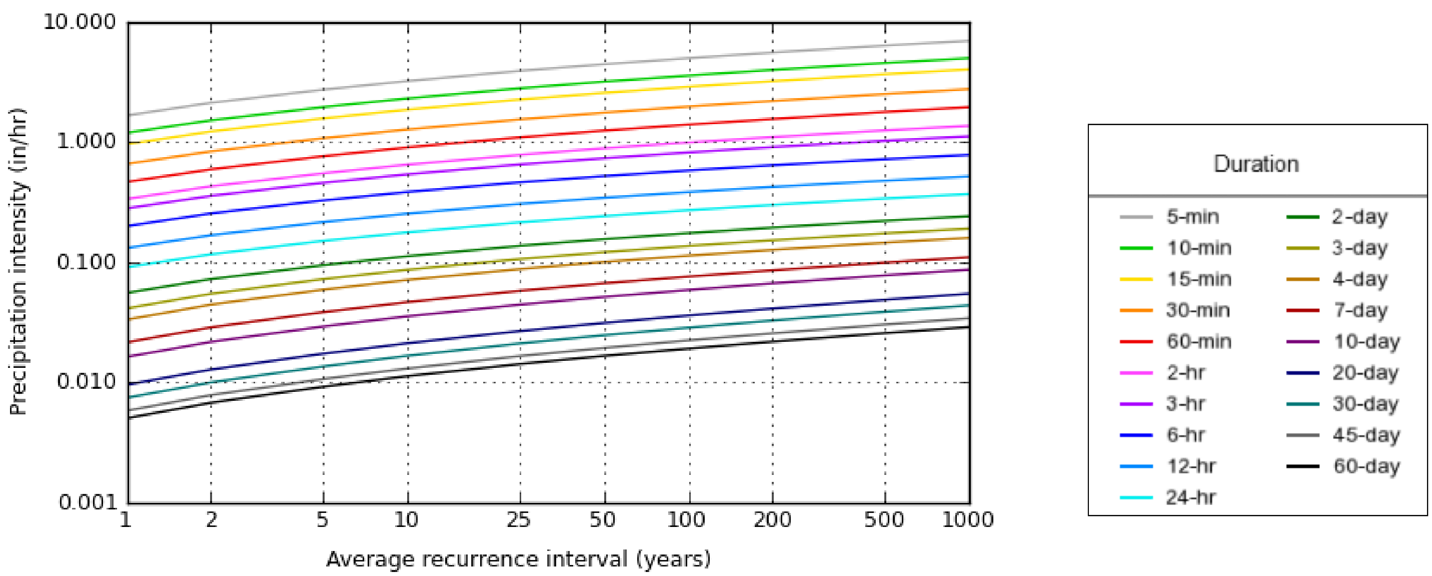
PF graphical





NOAA Atlas 14, Volume 6, Version 2 Created (GMT): Fri Sep 18 22:04:04 2020

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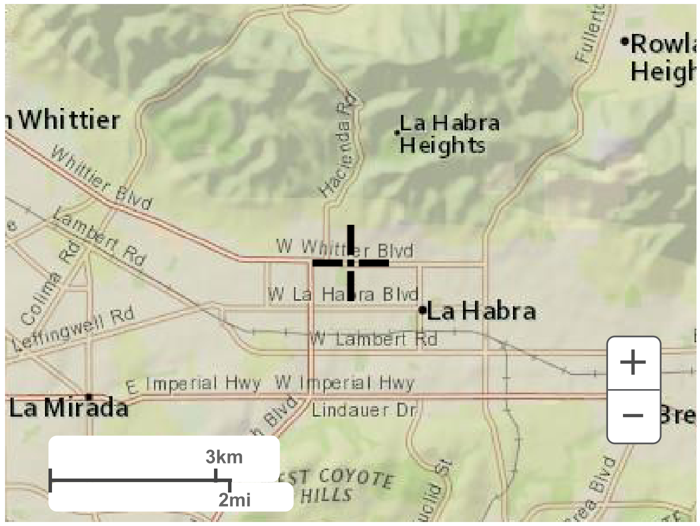
Maps & aerials

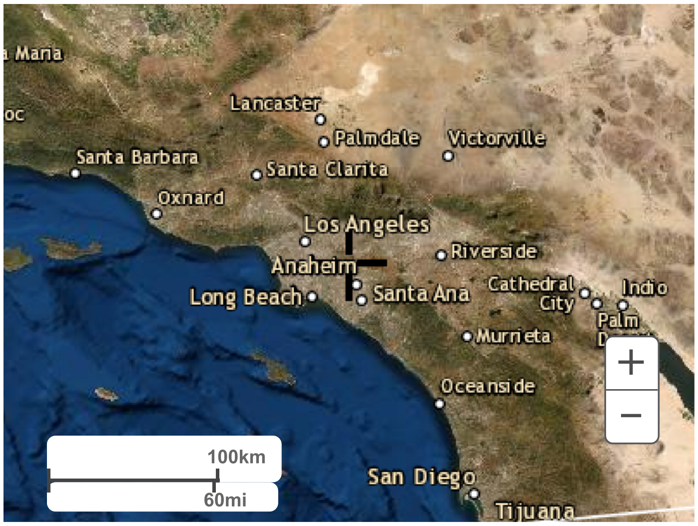
Small scale terrain



Large scale terrain

Large scale map

Large scale aerial



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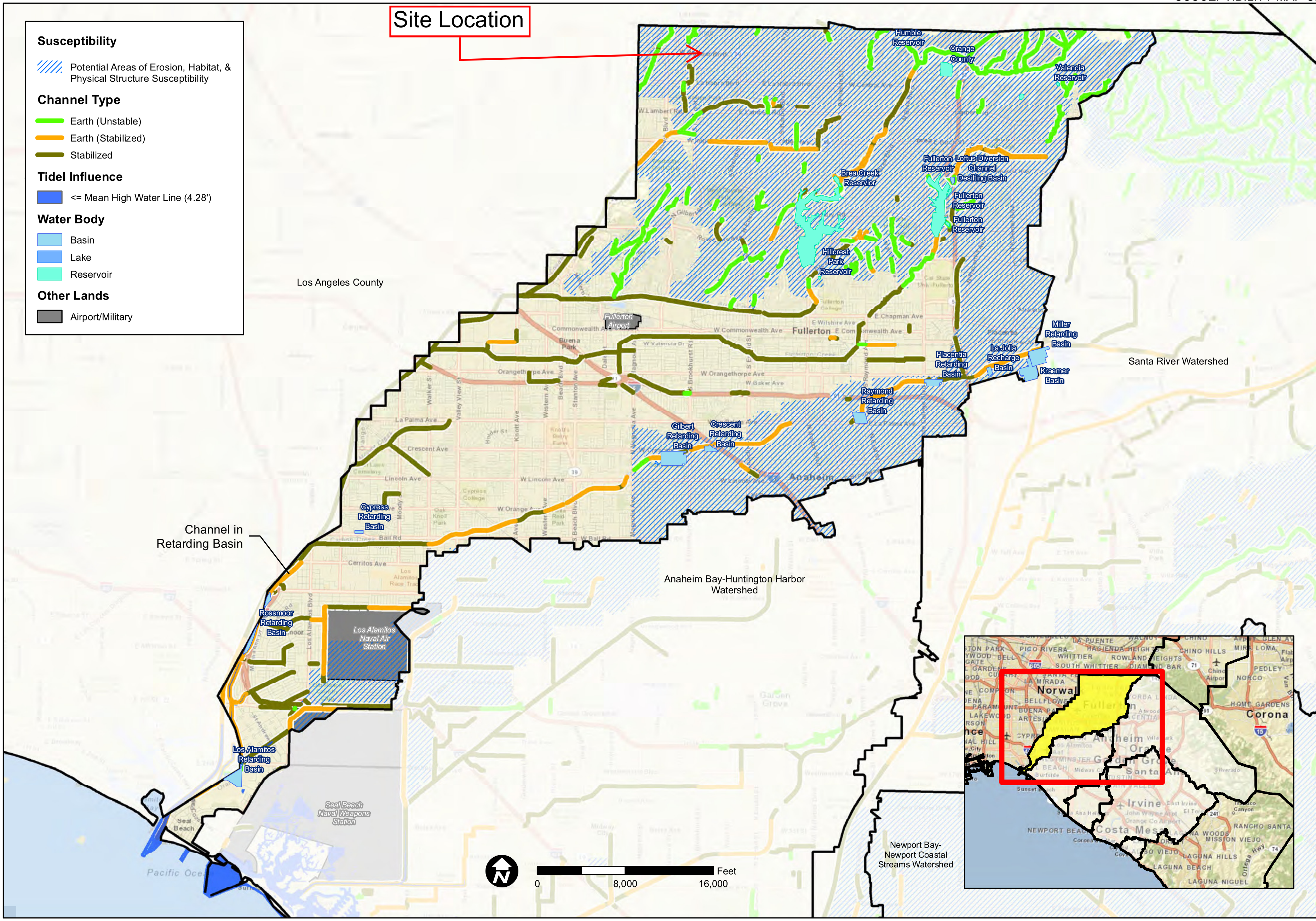
US Department of Commerce
National Oceanic and Atmospheric Administration
National Weather Service
National Water Center
1325 East West Highway
Silver Spring, MD 20910
Questions?: HDSC.Questions@noaa.gov

Disclaimer

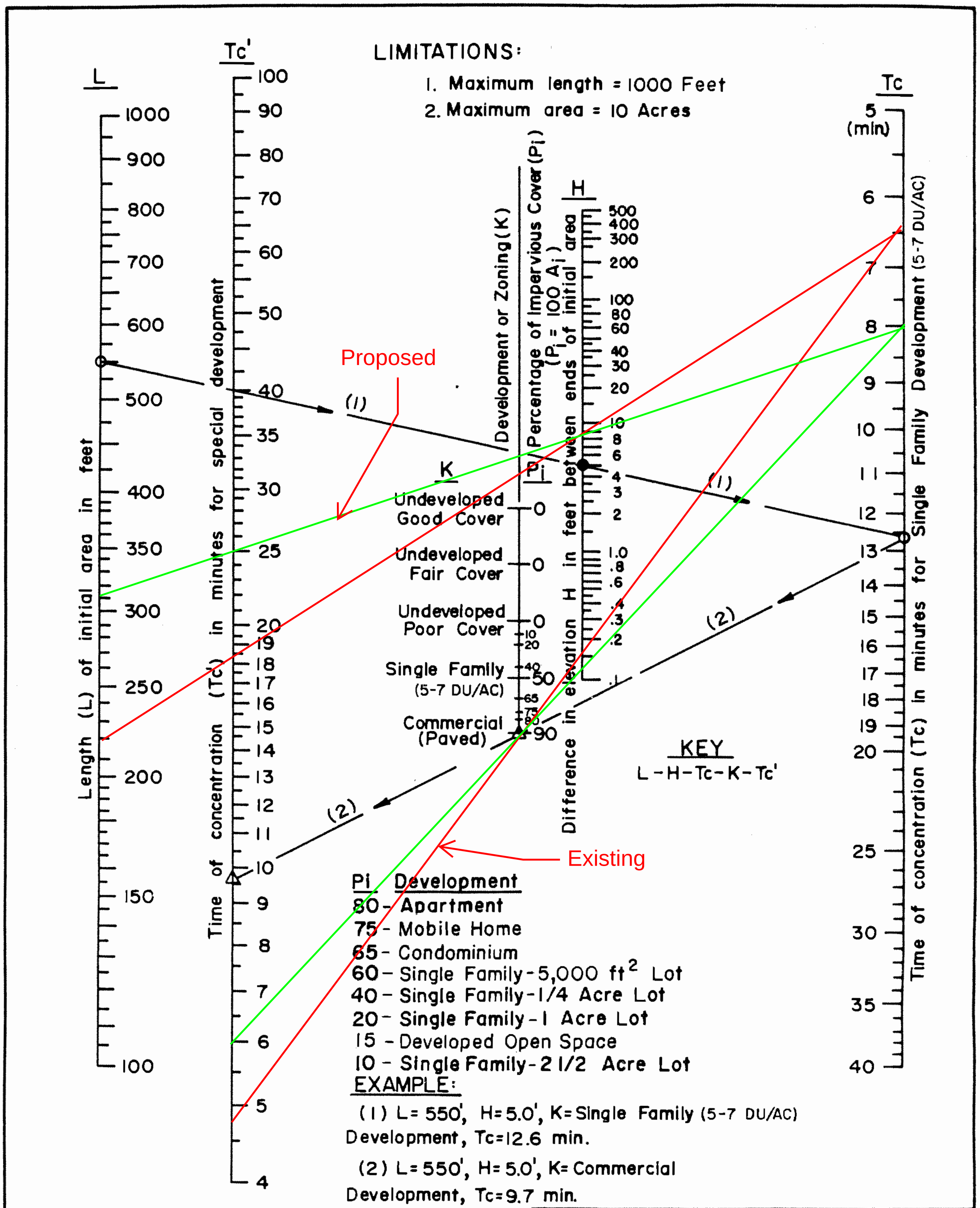


TITLE	SUSCEPTIBILITY ANALYIS SAN GABRIEL-COYOTE CREEK
JOB	ORANGE COUNTY WATERSHED MASTER PLANNING ORANGE CO. CA
SCALE	1" = 8000'
DESIGNED	TH
DRAWING	TH
CHECKED	BMP
DATE	04/30/10
JOB NO.	9526-E

PACE Advanced Water Engineering

1

P:\9526E\6-GIS\Mxds\SuceptabilityMaps_20100505\9526E_SanGabrielCoyoteCreekSusceptibility_20100430.mxd



ORANGE COUNTY

HYDROLOGY MANUAL

TIME OF CONCENTRATION NOMOGRAPH FOR INITIAL SUBAREA

Figure D-1

Worksheet B: Simple Design Capture Volume Sizing Method Existing

Step 1: Determine the design capture storm depth used for calculating volume				
1	Enter design capture storm depth from Figure III.1, d (inches)	d=	0.95	inches
2	Enter the effect of provided HSCs, d_{HSC} (inches) (Worksheet A)	d_{HSC}=	0	inches
3	Calculate the remainder of the design capture storm depth, $d_{remainder}$ (inches) (Line 1 – Line 2)	$d_{remainder}$=	0.95	inches
Step 2: Calculate the DCV				
1	Enter Project area tributary to BMP (s), A (acres)	A=	0.852	acres
2	Enter Project Imperviousness, *imp* (unitless)	imp=	0.85	
3	Calculate runoff coefficient, $C= (0.75 \times imp) + 0.15$	C=	0.786	
4	Calculate runoff volume, $V_{design}= (C \times d_{remainder} \times A \times 43560 \times (1/12))$	V_{design}=	2,542	cu-ft
Step 3: Design BMPs to ensure full retention of the DCV				
Step 3a: Determine design infiltration rate				
1	Enter measured infiltration rate, $K_{observed}$[1] (in/hr) (**Appendix VII**)	$K_{observed}$=		In/hr
2	Enter combined safety factor from Worksheet H, S_{total} (unitless)	S_{total}=		
3	Calculate design infiltration rate, $K_{design} = K_{observed} / S_{total}$	K_{design}=		In/hr
Step 3b: Determine minimum BMP footprint				
4	Enter drawdown time, T (max 48 hours)	T=		Hours
5	Calculate max retention depth that can be drawn down within the drawdown time (feet), $D_{max} = K_{design} \times T \times (1/12)$	D_{max}=		feet
6	Calculate minimum area required for BMP (sq-ft), $A_{min} = V_{design} / d_{max}$	A_{min}=		sq-ft

[1]$K_{observed}$ is the vertical infiltration measured in the field, before applying a factor of safety. If field testing measures a rate that is different than the vertical infiltration rate (for example, three-dimensional borehole percolation rate), then this rate must be adjusted by an acceptable method (for example, Porchet method) to yield the field estimate of vertical infiltration rate, $K_{observed}$. See Appendix VII.

Worksheet B: Simple Design Capture Volume Sizing Method Proposed

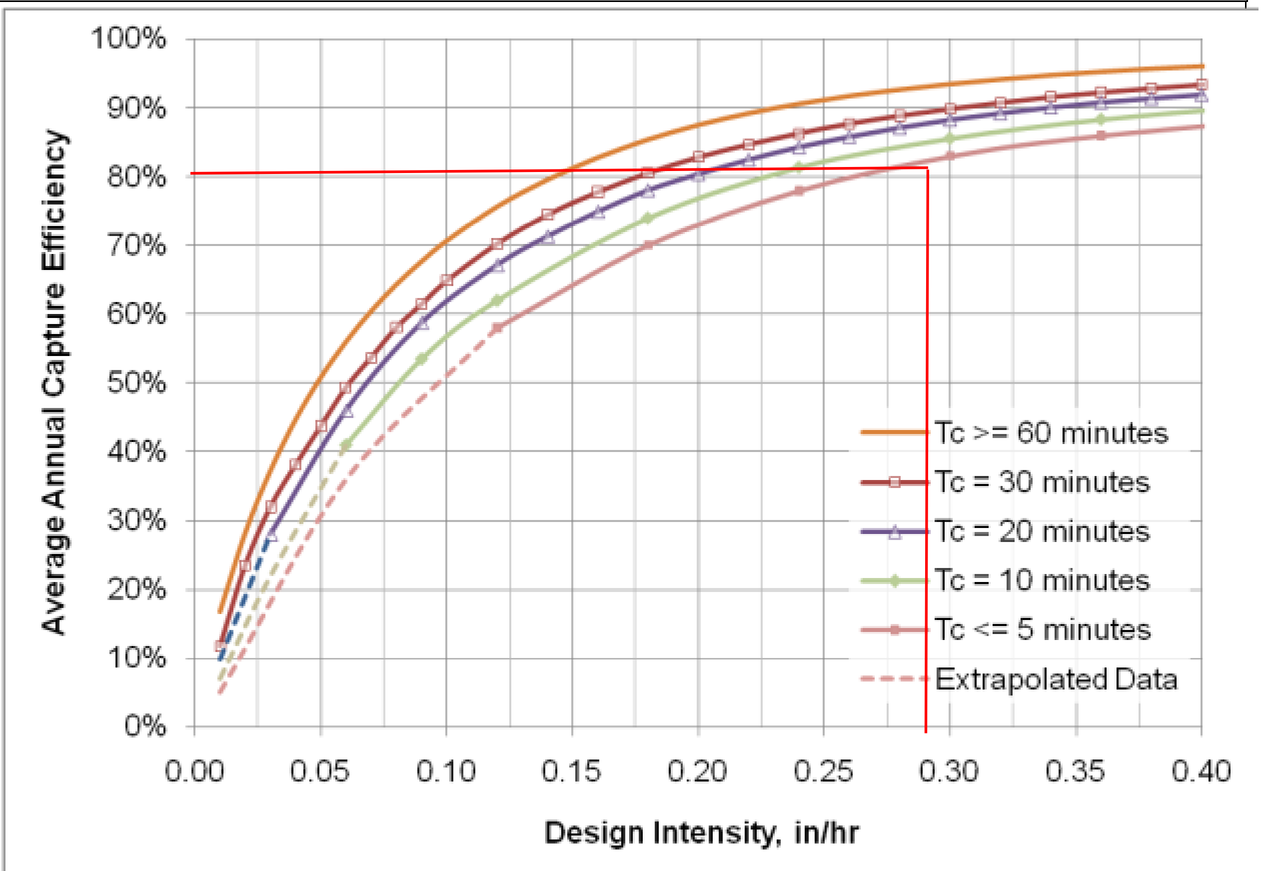
Step 1: Determine the design capture storm depth used for calculating volume				
1	Enter design capture storm depth from Figure III.1, d (inches)	d=	0.95	inches
2	Enter the effect of provided HSCs, d_{HSC} (inches) (Worksheet A)	d_{HSC}=	0	inches
3	Calculate the remainder of the design capture storm depth, $d_{remainder}$ (inches) (Line 1 – Line 2)	$d_{remainder}$=	0.95	inches
Step 2: Calculate the DCV				
1	Enter Project area tributary to BMP (s), A (acres)	A=	0.852	acres
2	Enter Project Imperviousness, *imp* (unitless)	imp=	0.85	
3	Calculate runoff coefficient, $C = (0.75 \times imp) + 0.15$	C=	0.786	
4	Calculate runoff volume, $V_{design} = (C \times d_{remainder} \times A \times 43560 \times (1/12))$	V_{design}=	2,310	cu-ft
Step 3: Design BMPs to ensure full retention of the DCV				
Step 3a: Determine design infiltration rate				
1	Enter measured infiltration rate, $K_{observed}$[1] (in/hr) (Appendix VII)	$K_{observed}$=		In/hr
2	Enter combined safety factor from Worksheet H, S_{total} (unitless)	S_{total}=		
3	Calculate design infiltration rate, $K_{design} = K_{observed} / S_{total}$	K_{design}=		In/hr
Step 3b: Determine minimum BMP footprint				
4	Enter drawdown time, T (max 48 hours)	T=		Hours
5	Calculate max retention depth that can be drawn down within the drawdown time (feet), $D_{max} = K_{design} \times T \times (1/12)$	D_{max}=		feet
6	Calculate minimum area required for BMP (sq-ft), $A_{min} = V_{design} / d_{max}$	A_{min}=		sq-ft

[1] $K_{observed}$ is the vertical infiltration measured in the field, before applying a factor of safety. If field testing measures a rate that is different than the vertical infiltration rate (for example, three-dimensional borehole percolation rate), then this rate must be adjusted by an acceptable method (for example, Porchet method) to yield the field estimate of vertical infiltration rate, $K_{observed}$. See Appendix VII.

Worksheet D: Capture Efficiency Method for Flow-Based BMPs Existing

Step 1: Determine the design capture storm depth used for calculating volume				
1	Enter the time of concentration, T_c (min) (See **Appendix IV.2**)	T_c=	4.75	
2	Using **Figure III.4**, determine the design intensity at which the estimated time of concentration (T_c) achieves 80% capture efficiency, I_1	I_1=	0.29	in/hr
3	Enter the effect depth of provided HSCs upstream, d_{HSC} (inches) (**Worksheet A**)	d_{HSC}=	0	inches
4	Enter capture efficiency corresponding to d_{HSC}, Y_2 (**Worksheet A**)	Y_2=	0	%
5	Using **Figure III.4**, determine the design intensity at which the time of concentration (T_c) achieves the upstream capture efficiency(Y_2), I_2	I_2=	0	
6	Determine the design intensity that must be provided by BMP, $I_{design}= I_1 - I_2$	I_{design}=	0.29	
Step 2: Calculate the design flowrate				
1	Enter Project area tributary to BMP (s), *A* (acres)	A=	0.852	acres
2	Enter Project Imperviousness, *imp* (unitless)	imp=	0.95	
3	Calculate runoff coefficient, $C= (0.75 \times imp) + 0.15$	C=	0.786	
4	Calculate design flowrate, $Q_{design}= (C \times i_{design} \times A)$	Q_{design}=	0.21	cfs
Supporting Calculations				
Describe system:				
Provide time of concentration assumptions:				

Worksheet D: Capture Efficiency Method for Flow-Based BMPs

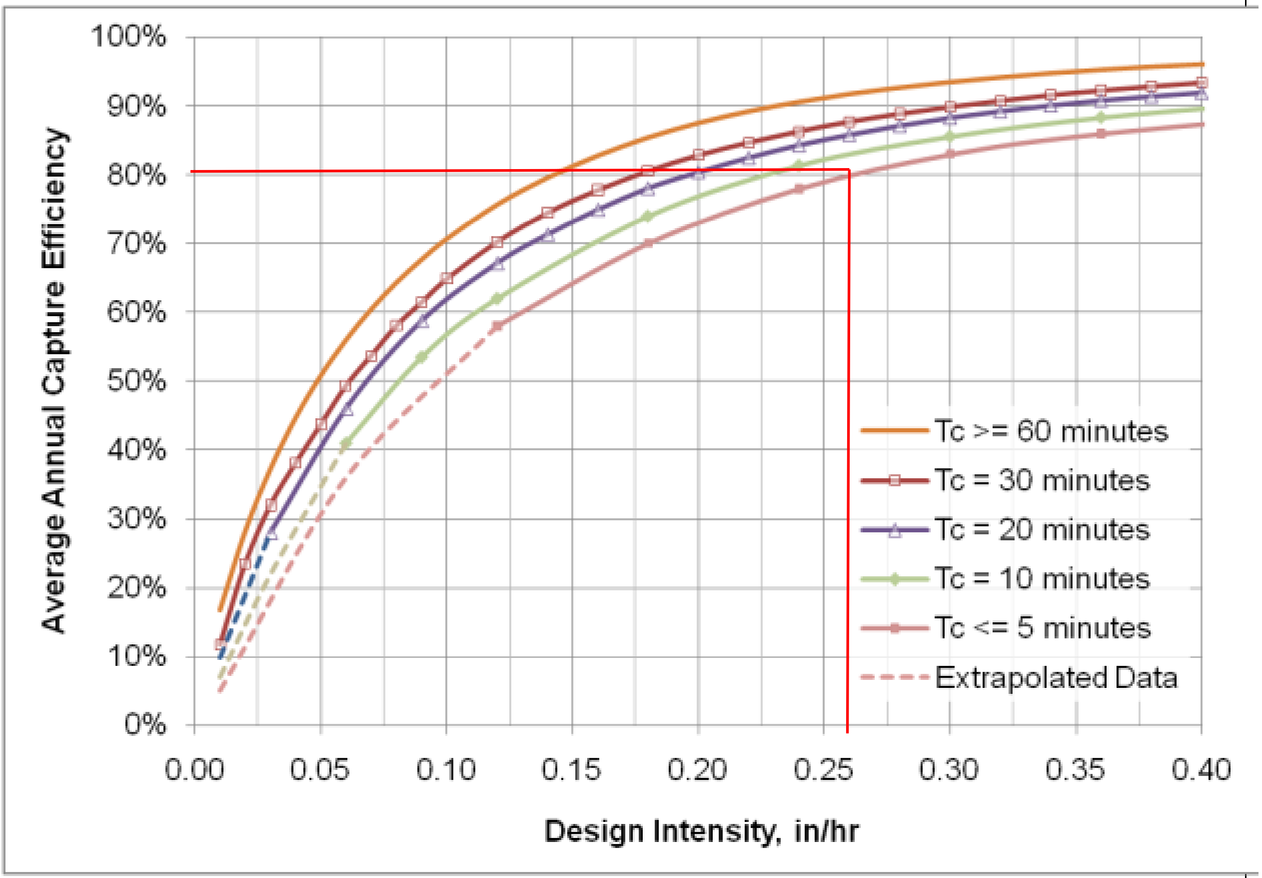
Graphical Operations

Provide supporting graphical operations. See Example III.7.

Worksheet D: Capture Efficiency Method for Flow-Based BMPs Proposed

Step 1: Determine the design capture storm depth used for calculating volume				
1	Enter the time of concentration, T_c (min) (See **Appendix IV.2**)	T_c=	6	
2	Using **Figure III.4**, determine the design intensity at which the estimated time of concentration (T_c) achieves 80% capture efficiency, I_1	I_1=	0.26	in/hr
3	Enter the effect depth of provided HSCs upstream, d_{HSC} (inches) (**Worksheet A**)	d_{HSC}=	0	inches
4	Enter capture efficiency corresponding to d_{HSC}, Y_2 (**Worksheet A**)	Y_2=	0	%
5	Using **Figure III.4**, determine the design intensity at which the time of concentration (T_c) achieves the upstream capture efficiency(Y_2), I_2	I_2=	0	
6	Determine the design intensity that must be provided by BMP, $I_{design}= I_1 - I_2$	I_{design}=	0.26	
Step 2: Calculate the design flowrate				
1	Enter Project area tributary to BMP (s), *A* (acres)	A=	0.852	acres
2	Enter Project Imperviousness, *imp* (unitless)	imp=	0.85	
3	Calculate runoff coefficient, $C = (0.75 \times imp) + 0.15$	C=	0.786	
4	Calculate design flowrate, $Q_{design} = (C \times i_{design} \times A)$	Q_{design}=	0.17	cfs
Supporting Calculations				
Describe system:				
Provide time of concentration assumptions:				

Worksheet D: Capture Efficiency Method for Flow-Based BMPs

Graphical Operations



Provide supporting graphical operations. See Example III.7.

CONTECH ENGINEERED SOLUTIONS

filterra Bioretention Systems

Filterra Sizing Spreadsheet
Uniform Intensity Approach
Storm Intensity = 0.20 in/hr

Filterra Infiltration Rate =	100	(in/hr)
Filterra Flow per Square Foot =	0.0023	(ft3/sec/ft2)

Filterra Flow Rate, Q = 0.0023 ft3/sec x Filterra Surface Area

Rational Method, Q = C x I x A

Site Flowrate, Q = (C x DI x DA x 43560) / (12 x3600)

OR DA = (12 x 3600 x Q) / (C x 43560 x DI)

where

Q =	Flow	(ft3/sec)
DA =	Drainage Area	(acres)
DI =	Design Intensity	(in/hr)
C =	Runoff coefficient	(dimensionless)

			DI 0.2	C 1.00	C 0.85	C 0.50
Available Filterra Box Sizes			**Filterra Flow Rate, Q (ft3/sec)**	100% Imperv. DA (acres)	Commercial max DA (acres)	Residential max DA (acres)
L (ft)	W (ft)	Filterra Surface Area (ft2)				
4	4	16	**0.0370**	0.184	0.216	0.367
6	4	24	**0.0556**	0.275	0.324	0.551
6.5	4	26	**0.0602**	0.298	0.351	0.597
8	4	32	**0.0741**	0.367	0.432	0.735
12	4	48	**0.1111**	0.551	0.648	1.102
6	6	36	**0.0833**	0.413	0.486	0.826
8	6	48	**0.1111**	0.551	0.648	1.102
10	6	60	**0.1389**	0.689	0.810	1.377
12	6	72	**0.1667**	0.826	0.972	1.653
13	7	91	**0.2106**	1.045	1.229	2.089
12	8	96	**0.2222**	1.102	1.296	2.204

Attachment E

Attachment F

Attachment G

Attachment H

GEOTECHNICAL INVESTIGATION
PROPOSED MEDICAL OFFICE BUILDING

1201 West Whittier Boulevard
La Habra, California
for
Northgate Markets

SOUTHERN
CALIFORNIA
GEOTECHNICAL
A California Corporation

July 8, 2020

Northgate Markets
1201 Magnolia Avenue
Anaheim, CA 92801

Attention: Michelle Gutierrez

Project No.: **20G161-1**

Subject: **Geotechnical Investigation**
Proposed Medical Office Building
1201 West Whittier Boulevard
La Habra, California

Ms. Gutierrez:

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

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

Respectfully Submitted,

SOUTHERN CALIFORNIA GEOTECHNICAL, INC.

Daniel W. Nielsen, RCE 77915
Senior Engineer

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

Gregory K. Mitchell, GE 2364
Principal Engineer

REGISTERED PROFESSIONAL ENGINEER
GREGORY K. MITCHELL
No. 2364
Exp. 09/30/20
GEOTECHNICAL
STATE OF CALIFORNIA

Distribution: (2) Addressee

22885 Savi Ranch Parkway ▾ Suite E ▾ Yorba Linda ▾ California ▾ 92887
voice: (714) 685-1115 ▾ fax: (714) 685-1118 ▾ www.socalgeo.com

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

- Most of the borings encountered undocumented fill soils, extending to depths of 5½ to 12± feet. The underlying native alluvium possesses moderate to relatively high strengths. Documentation regarding the placement or compaction of the existing fill soils is not available and therefore these materials are considered to represent undocumented fill. Remedial grading will be necessary in order to provide a subgrade suitable for support of the new structure.
- The undocumented fill soils encountered at Boring No. B-1 extend to a depth of 12± feet and generally consist of very loose to medium dense fine to coarse sands. These soils vary in composition from the majority of the near-surface soils at this site which generally consist of sandy clays and silty clays. The native alluvial soils located immediately beneath the sandy fill soils at Boring No. B-1 possess a strong hydrocarbon odor. Based on these conditions, it is possible that an underground storage tank (UST) was previously removed from the area of Boring No. B-1.
- Most of the near surface soils consist of sandy clays and silty clays. The results of expansion index testing indicate that these soils possess a medium expansion potential (EI= 70).

Site Preparation

- Demolition of the existing building, associated improvements, and the existing pavements will be required in order to facilitate construction of the new building. Debris resultant from demolition should be disposed of offsite. Alternatively, concrete and asphalt debris may be pulverized to a maximum 2-inch particle size, well mixed with the on-site soils, and incorporated into new structural fills or it may be crushed and made into crushed miscellaneous base (CMB). Concrete and asphalt debris may also be crushed to particle sizes between 2 and 4 inches and used for subgrade stabilization.
- Stripping of any existing vegetated areas should remove include all vegetation, including tree root masses and any organic topsoil.
- Remedial grading should be performed within the proposed building area to remove the existing undocumented fill soils any soils disturbed during demolition of the existing building, and a portion of the near surface native alluvial soils. Based on conditions encountered at the boring locations, overexcavation to depths of 5½± to 12± feet will be necessary to remove the existing fill soils. The overexcavation should extend to a depth of at least 5 feet below proposed building pad subgrade elevation. Within the foundation influence zones, the overexcavation should extend to a depth of at least 3 feet below proposed foundation bearing grade.
- Special grading techniques, such as slot cutting, may be required along the east and south property lines in order to perform the recommended remedial grading.
- After the recommended overexcavation has been completed, the resulting subgrade soils should be evaluated by the geotechnical engineer to identify any additional soils that should

be overexcavated. The resulting subgrade should then be scarified to a depth of 10 to 12 inches and thoroughly moisture conditioned (or air dried) to 2 to 4 percent above optimum moisture content. The resulting subgrade should then be recompacted to at least 90 percent of the ASTM D-1557 maximum dry density. The previously excavated soils may then be replaced as compacted structural fill.

- The new parking area subgrade soils are recommended to be scarified to a depth of 12± inches, moisture conditioned to 2 to 4 percent above optimum, and recompacted to at least 90 percent of the ASTM D-1557 maximum dry density.

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 presence of medium expansive soils. Additional reinforcement may be necessary for structural considerations.

Building Floor Slab
- Conventional Slab-on-Grade, 5 inches thick.
- Reinforcement consisting of No. 4 bars at 16-inches on center in both directions, due to presence of medium expansive soils. The actual floor slab reinforcement should be determined by the structural engineer.

Pavements

ASPHALT PAVEMENTS (R = 10)			
Materials	**Thickness (inches)**		
	Auto Parking (TI = 4.0)	Auto Drive Lanes (TI = 5.0)	Light Truck Traffic (TI = 6.0)
Asphalt Concrete	3	3	3½
Aggregate Base	6	9	12
Compacted Subgrade	12	12	12

PORTLAND CEMENT CONCRETE PAVEMENTS (R = 10)		
Materials	**Thickness (inches)**	
	Automobile Parking and Drive Areas	Light Truck Traffic Areas (TI =6.0)
PCC	5	5½
Compacted Subgrade (95% minimum compaction)	12	12

2.0 SCOPE OF SERVICES

The scope of services performed for this project was in accordance with our Proposal No. 20P254, dated June 11, 2020. The scope of services included a visual site reconnaissance, subsurface exploration, field and laboratory testing, and geotechnical engineering analysis to provide criteria for preparing the design of the building foundations, building floor slabs, and parking lot pavements along with site preparation recommendations and construction considerations for the proposed development. The evaluation of the environmental aspects of this site was beyond the scope of services for this geotechnical investigation.

3.0 SITE AND PROJECT DESCRIPTION

3.1 Site Conditions

The subject is located at the northwest corner of West Whittier Boulevard and North Idaho Street in La Habra, California. The site is bounded to the north by asphaltic concrete parking areas for an existing retail development, to the west by a Jack in the Box restaurant, to the south by West Whittier Boulevard, and to the east by North Idaho Street. The general location of the site is illustrated on the Site Location Map, included as Plate 1 of this report.

The site is slightly less than 1 acre in size, and is presently developed with a retail building, 10,305± square feet in size. The building was formerly used as a Petco store, but is now vacant. The building is surrounded by asphaltic concrete pavements and several landscape planters that contain medium bushes and medium to large trees.

Topographic information for the subject site was obtained from a topographic survey prepared by Cal Vada Surveying, Inc. Based on this survey, the parking lot north of the existing building slopes downward to the south at a gradient of 2½± percent. The parking and drive area west of the building slopes downward to the south at a gradient of 4± percent. The existing building possesses a flat floor slab, with a finished floor elevation of 355.68 feet mean sea level (msl). The maximum site elevation is 361.5 feet msl in the northeast corner of the lot. The minimum site elevation is 350.1 feet msl in the southwest corner of the lot.

3.2 Proposed Development

Our office was provided with a conceptual site plan, prepared by Ware Malcomb, for the proposed development. Based on this plan, the site will be developed with one (1) new medical office building (MOB), located in the southeastern region of the site. The MOB will be 2 stories in height, and will have a footprint of 10,000± square feet. The building is expected to be surrounded by asphaltic concrete pavements in the parking and drive areas with some concrete flatwork and landscape planter areas throughout the site.

Based on preliminary structural information provided by the client, we understand that the new building will be a 2-story structure of masonry block and steel frame construction, typically supported on a conventional shallow foundation system with a concrete slab-on-grade floor. Based on the type of construction, maximum column and wall loads are expected to be on the order of 150 kips and 1 to 3 kips per linear foot, respectively.

Grading plans for the proposed development were not available at the time of this report. 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 existing topography, and assuming a relatively balanced site, cuts and fills of less than 3± feet are expected to be necessary to achieve the proposed site grades.

4.0 SUBSURFACE EXPLORATION

4.1 Scope of Exploration/Sampling Methods

The subsurface exploration conducted for this project consisted of four (4) borings advanced to depths of 10 to 25± feet below the existing site grades. All of the borings were logged during drilling by a member of our staff.

Boring Nos. B-1 and B-2 were advanced with hollow-stem augers, by a conventional truck-mounted drilling rig. Boring Nos. B-3 and B-4 were advanced using manually operated hand augering equipment. 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 Plan, included as Plate 2 in Appendix A of this report. The Boring Logs, which illustrate the conditions encountered at the boring locations, as well as the results of some of the laboratory testing, are included in Appendix B.

4.2 Geotechnical Conditions

Pavements

Asphaltic concrete (AC) pavements were encountered at the ground surface at boring Nos. B-1 and B-2. These pavements consist of 3 to 6± inches of asphaltic concrete underlain by 2 to 6± inches of aggregate base.

It should be noted that the location of Boring No. B-2 was offset about 5 feet west of the originally intended location due to the presence of a hard material (possessing the appearance of PCC or slurry) approximately 1-foot below the surface of the pavement. We were not able to identify the composition of this material within this relatively small-diameter boring.

Floor Slab

Boring Nos. B-3 and B-4 were drilled through the floor slab in the interior of the existing building. The floor slab consists of Portland cement concrete (PCC) with thicknesses of 4½ and 6± inches at these boring locations.

Artificial Fill

Artificial fill soils were encountered beneath the pavements at Boring No. B-1 and beneath the floor slab at Boring Nos B-3, and B-4, extending to depths of 5½ to 12± feet below the existing site grades. The artificial fill soils at Boring Nos. B-3 and B-4 generally consist of stiff silty clays and fine sandy clays. The fills soils at Boring No. B-1 generally consist of fine to coarse sands with occasional silty clay nodules. The fill soils possess a disturbed and mottled appearance, resulting in their classification as artificial fill.

Alluvium

Native alluvium was encountered beneath the pavements at Boring No. B-2 and beneath the fill soils at the remaining boring locations. The near-surface native alluvial soils encountered within the upper 5½ to 10± feet generally consist of stiff to very stiff silty clays and fine sandy clays. These soils possess some calcareous veining and nodules. The deeper alluvial soils encountered at Boring Nos. B-1 and B-2 consist of medium dense fine sandy silts and fine to medium sand. These soils possess little calcareous veining and nodules. The alluvial soils encountered at depths of 12 to 22± feet at Boring No. B-1 possess a strong hydrocarbon odor, and the alluvial soils encountered at depths of 8½ to 10± feet at Boring No. B-2 possess a slight hydrocarbon odor. Evaluation of the environmental characteristics of the on-site soils was not a part of our scope of work for this investigation. The native alluvial soils extend at least to the maximum depth explored of 25± feet.

Groundwater

Groundwater was not encountered at any of the borings. Based on the lack of any water within the borings, and the moisture contents of the recovered soil samples, the static groundwater table is considered to have existed at a depth in excess of 25± feet below existing site grades, at the time of the subsurface investigation.

As part of our research, we reviewed historic high groundwater levels reported in the California DMG Open-File Report for the La Habra Quadrangle. Plate 1.2 is a map which displays the historically highest ground water levels using contour lines. Based on the mapped contour lines in the vicinity of the project site, the historic high groundwater level at the subject site is considered to have existed at a depth of 25± feet below existing site grades. We also reviewed readily available groundwater data published on the California State Water Resources Control Board, GeoTracker, website, http://geotracker.waterboards.ca.gov/. The nearest monitoring well with available data in this database is located approximately 3900 feet east of the site. Water level readings within this monitoring well indicate a groundwater level of 48± 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. 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 have been tested to determine their consolidation potential, in accordance with ASTM D-2435. The testing apparatus is designed to accept either natural or remolded samples in a one-inch high ring, approximately 2.416 inches in diameter. Each sample is then loaded incrementally in a geometric progression and the resulting deflection is recorded at selected time intervals. Porous stones are in contact with the top and bottom of the sample to permit the addition or release of pore water. The samples are typically inundated with water at an intermediate load to determine their potential for collapse or heave. The results of the consolidation testing are plotted on Plates C-1 through C-8 in Appendix C of this report.

Maximum Dry Density and Optimum Moisture Content

A representative bulk sample has been tested for its maximum dry density and optimum moisture content. The results have been obtained using the Modified Proctor procedure, per ASTM D-1557 and are presented on Plate C-9 in Appendix C of this report. These tests are generally used to compare the in-situ densities of undisturbed field samples, and for later compaction testing. Additional testing of other soil types or soil mixes may be necessary at a later date.

Soluble Sulfates

A representative sample of the near-surface soils 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 results of the soluble sulfate testing are presented below, and are discussed further in a subsequent section of this report.

Sample Identification	Soluble Sulfates (%)	Sulfate Classification
B-3 @ 0 to 5 feet	0.0118	Not Applicable (S0)

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 and nitrate concentrations of the 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-3 @ 0 to 5 feet	920	7.8	12	76

Expansion Index

The expansion potential of the on-site soils was determined in general accordance with ASTM D-4829. The testing apparatus is designed to accept a 4-inch diameter, 1-in high, remolded sample. The sample is initially remolded to 50± 1 percent saturation and then loaded with a surcharge equivalent to 144 pounds per square foot. The sample is then inundated with water, and allowed to swell against the surcharge. The resultant swell or consolidation is recorded after a 24-hour period. The results of the EI testing are as follows:

Sample Identification	Expansion Index	Expansive Potential
B-4 @ 0 to 5 feet	70	Medium

6.0 CONCLUSIONS AND RECOMMENDATIONS

Based on the results of our review, field exploration, laboratory testing and geotechnical analysis, the proposed development is considered feasible from a geotechnical standpoint. The recommendations contained in this report should be taken into the design, construction, and grading considerations. The recommendations are contingent upon all grading and foundation construction activities being monitored by the geotechnical engineer of record. The Grading Guide Specifications, included as Appendix D, should be considered part of this report, and should be incorporated into the project specifications. The contractor and/or owner of the development should bring to the attention of the geotechnical engineer any conditions that differ from those stated in this report, or which may be detrimental for the development.

6.1 Seismic Design Considerations

The subject site is located in an area which is subject to strong ground motions due to earthquakes. The performance of a site-specific seismic hazards analysis was beyond the scope of this investigation. However, numerous faults capable of producing significant ground motions are located near the subject site. Due to economic considerations, it is not generally considered reasonable to design a structure that is not susceptible to earthquake damage. Therefore, significant damage to structures may be unavoidable during large earthquakes. The proposed 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. Therefore, the possibility of significant fault rupture on the site is considered to be low.

Seismic Design Parameters

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

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

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

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

2019 CBC SEISMIC DESIGN PARAMETERS

Parameter		Value
Mapped Spectral Acceleration at 0.2 sec Period	S_S	1.806
Mapped Spectral Acceleration at 1.0 sec Period	S_1	0.641
Site Class	---	D
Site Modified Spectral Acceleration at 0.2 sec Period	S_{MS}	1.806
Site Modified Spectral Acceleration at 1.0 sec Period	S_{M1}	1.041
Design Spectral Acceleration at 0.2 sec Period	S_{DS}	1.204
Design Spectral Acceleration at 1.0 sec Period	S_{D1}	0.726

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

Liquefaction

Liquefaction is the loss of strength in generally cohesionless, saturated soils when the pore-water pressure induced in the soil by a seismic event becomes equal to or exceeds the overburden pressure. The primary factors which influence the potential for liquefaction include groundwater table elevation, soil type and grain size characteristics, relative density of the soil, initial confining pressure, and intensity and duration of ground shaking. The depth within which the occurrence of liquefaction may impact surface improvements is generally identified as the upper 50 feet below the existing ground surface. Liquefaction potential is greater in saturated, loose, poorly graded fine sands with a mean (d_{50}) grain size in the range of 0.075 to 0.2 mm (Seed and Idriss, 1971). Clayey (cohesive) soils or soils which possess clay particles (d<0.005mm) in excess of 20 percent (Seed and Idriss, 1982) are generally not considered to be susceptible to liquefaction, nor are those soils which are above the historic static groundwater table.

The Seismic Hazards Map for the La Habra Quadrangle, published by the California Geological Survey indicates that the subject site is not located within a designated liquefaction hazard zone. In addition, the subsurface conditions encountered at the boring locations, which consist of relatively high strength cohesive soils, are not considered to be susceptible to liquefaction. Based on these factors, liquefaction is not considered to be a significant design concern for this project.

6.2 Geotechnical Design Considerations

General

Three of the four borings performed within the proposed building area encountered artificial fill soils extending to depths of 5½ to 12± feet. The fill soils at Boring No. B-1 consist of variable strength, fine to coarse sandy soils, possessing very loose to loose relative densities, with the exception of medium dense soils in the upper 1 to 2± feet. These very loose to loose fill soils extend to a depth of approximately 12± feet. The native alluvial soils encountered beneath these fill soils possess a strong hydrocarbon odor. Based on these conditions, it is possible that an underground storage tank (UST) was previously located in the area of Boring No. B-1 area.

The fill soils at the remaining borings consist of silty clay and sandy clay soils with variable strengths and densities. The results of laboratory testing indicate that some of these clayey fill soils are compressible and will consolidate when loaded within the range of the anticipated foundation loads of the proposed structure. No documentation verifying the placement and compaction of the existing fill soils was provided to our office. The existing fill materials are therefore considered to represent undocumented fill. In addition, demolition of the existing building and the surrounding improvements is expected to result in significant disturbance to the near-surface soils. Based on these existing conditions, remedial grading will be necessary within the proposed building area to remove the existing undocumented fill soils and any soils disturbed during demolition, in order to provide a subgrade suitable for support of the new building foundations and floor slabs. The excavated soils may be replaced as compacted structural fill.

Settlement

The recommended remedial grading will remove the existing undocumented fill soils and a portion of the near-surface native alluvium from within the area of the new structure. The native soils that will remain in place beneath the recommended depth of overexcavation will not be subject to significant stress increases from the foundations of the new structure and they possess more favorable consolidation and collapse characteristics than the compressible fill soils. Therefore, following completion of the recommended remedial grading, post-construction settlements are expected to be within tolerable limits.

Expansion

Most of the near surface soils at this site consist of sandy clays and silty clays. Laboratory testing performed on a representative sample of these materials indicates that they possess a medium expansion potential (EI = 70). Based on the presence of expansive soils, special care should be taken to properly moisture condition and maintain adequate moisture content within all subgrade soils as well as newly placed fill soils. The foundation and floor slab design recommendations contained within this report are based on the assumption that the building pad will be underlain by medium expansive soils. It is recommended that additional expansion index testing be conducted at the completion of rough grading to verify the expansion potential of the as-graded building pad.

Soluble Sulfates

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

Corrosion Potential

The results of laboratory testing indicate that the tested sample of the on-site soils possesses a saturated resistivity value of 920 ohm-cm and a pH value of 7.8. 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. Sulfides, and redox potential are factors that are also used in the evaluation procedure. 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 corrosive to ductile iron pipe and other metallic improvements. Therefore, polyethylene encasement, or some other form of protection will be required for iron pipes. However, since SCG does not practice in the area of corrosion engineering, the client may also wish to contact a corrosion engineer to provide a more thorough evaluation.**

A relatively low concentration of chlorides, 12 mg/kg, 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 relatively low chloride concentration 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 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 79 mg/kg. **Based on this test result, the on-site soils are considered to be corrosive to copper pipe. Since SCG does not practice in the area of corrosion engineering, we recommend that the client contact a corrosion engineer to provide recommendations for the protection of copper tubing/pipe in contact with the on-site soils.**

Shrinkage/Subsidence

Removal and recompaction of the near-surface fill and alluvial soils consisting of silty clays, sandy clays, and sandy silts is estimated to result in an average shrinkage of 7 to 12 percent. However, recompaction of the sandy fill soils at Boring No. B-1 is expected to result in an average shrinkage of about 14 to 18 percent. Minor ground subsidence is expected to occur in the soils below the zone of removal, due to settlement and machinery working. The subsidence is estimated to be 0.1± feet. This estimate may be used for grading in areas that are underlain by existing 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 dependant on the type of machinery used, repetitions of use, and dynamic effects, all of which are difficult to assess precisely.

Grading and Foundation Plan Review

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

6.3 Site Grading Recommendations

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

Site Stripping and Demolition

Any existing improvements that will not remain in place for use with the new development should be removed in their entirety. This should include all foundations, floor slabs, utilities, and any other subsurface improvements associated with the existing building. All debris resultant from demolition should be disposed of offsite in accordance with local regulations. Concrete and asphalt debris may be pulverized to a maximum 2-inch particle size and incorporated into new structural fills.

Stripping of vegetation, including turf grass, shrubs, and possibly large trees will be required in the area of any existing landscape planters that are demolished. Removal of the trees should include any associated root masses. All organic materials should be disposed of offsite. The actual extent of site stripping should be determined by the geotechnical engineer at the time of grading, based on the organic content and the stability of the encountered materials.

<u>Treatment of Existing Soils: Building Pad</u>

It is recommended that remedial grading be performed within the new building area to remove the existing undocumented fill soils, any soils disturbed during demolition, and a portion of the near surface alluvium. Based on conditions encountered at the boring locations, overexcavation to a depths of 5½ to 12± feet will be required in the proposed building area to remove the existing fill soils. If deeper areas of existing fill soils are encountered, greater depths of overexcavation may be required. The overexcavation should also extend to a depth of at least 5 feet below the proposed building pad grade.

Within the influence zones of the new foundations, the overexcavation should extend to a depth of at least 3 feet below proposed foundation bearing grade. The overexcavation should extend at least 5 feet beyond the building perimeters, and to an extent equal to the depth of new fill below the foundation bearing grade. If the proposed structure incorporates any exterior columns (such as for a building canopy or overhang) the overexcavation should also encompass these areas.

The remedial grading activities within the south and east portions of the proposed building area will require excavation in close proximity to the south and east property lines. Specialized grading techniques such as slot cutting may be required in order to complete the new overexcavation. If slot cutting is required, the geotechnical engineer should be contacted for supplementary recommendations.

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, very moist soils will be encountered at or near the base of the recommended overexcavation. Stabilization of the exposed overexcavation subgrade soils may be necessary. 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 or geotextile, could be necessary. In this event, 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 at least 2 to 4 percent above optimum moisture content, and recompacted to at least 90 percent of the ASTM D-1557 maximum dry density. The previously excavated soils may then be replaced as compacted structural fill.

Treatment of Existing Soils: Parking and Drive Areas

Based on economic considerations, overexcavation of the existing fill soils in the new parking and drive areas is not considered warranted, with the exception of areas where lower strength, or unstable, soils are identified by the geotechnical engineer during grading.

Subgrade preparation in the new parking and drive areas should initially consist of removal of all soils disturbed during stripping and demolition operations. The geotechnical engineer should then evaluate the subgrade to identify any areas of additional unsuitable soils. The subgrade soils should then be scarified to a depth of 12± inches, moisture conditioned to 2 to 4 percent above 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 parking and drive areas assume that the owner and/or developer can tolerate minor amounts of movement within the proposed parking areas. The grading recommendations presented above do not completely mitigate the extent of existing undocumented fill soils and medium expansive soils in the parking areas. As such, settlement and associated pavement distress could occur. Typically, repair of such distressed areas involves significantly lower costs than completely mitigating these soils at the time of construction. If the owner cannot tolerate the risk of such movements, the parking and drive areas should be overexcavated to provide for a new layer of compacted structural fill, extending to a depth of at least 2 feet below proposed pavement subgrade elevation.

Treatment of Existing Soils: Flatwork

The proposed development may include some areas of Portland cement concrete flatwork. Based on conditions encountered at the boring locations, it is expected that these areas of flatwork will be underlain by moist to very moist medium expansive soils. The presence of these soils possesses a minor risk of heave and damage to new flatwork, which will be relatively lightly loaded. Based on economic considerations, flatwork is typically constructed immediately over medium expansive soils. However, if the owner desires to obtain greater protection against heaving of flatwork, a layer of very low expansive select structural fill could be placed below the flatwork areas. Typically, this layer of select fill is 1 to 2 feet in thickness.

Subgrade preparation in the new flatwork areas should initially consist of removal of all soils disturbed during stripping and demolition operations. The geotechnical engineer should then evaluate the subgrade to identify any areas of additional unsuitable soils. The subgrade soils should then be scarified to a depth of 12± inches, moisture conditioned to 3 to 5 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 fill 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.

Treatment of Existing Soils: Retaining Walls and Site Walls

The existing soils within the areas of any proposed retaining walls and non-retaining site walls should be overexcavated to a depth of 3 feet below foundation bearing grade and replaced as compacted structural fill, as discussed above for the proposed building pad. Any undocumented fill soils within any of these foundation areas should be removed in their entirety. The overexcavation areas should extend at least 5 feet beyond the foundation perimeters, and to an extent equal to the depth of fill below the new foundations. Any erection pads used to construct the walls are considered to be part of the foundation system with respect to these overexcavation recommendations. The overexcavation subgrade soils should be evaluated by the geotechnical engineer prior to scarifying, moisture conditioning, and recompacting the upper 12 inches of exposed subgrade soils. The previously excavated soils may then be replaced as compacted structural fill.

Fill Placement

- Fill soils should be placed in thin (6± inches), near-horizontal lifts, moisture conditioned to 2 to 4 percent above the optimum moisture content, and compacted.
- On-site soils may be used for fill provided they are cleaned of any debris to the satisfaction of the geotechnical engineer. All fill should conform with the recommendations presented in the Grading Guide Specifications, included as Appendix D. On-site soils may be used for fill provided they are cleaned of any debris to the satisfaction of the geotechnical engineer. **It should be noted that most of the soils at this site currently possess moisture contents above the anticipated moisture content. Therefore, some drying of these materials will be required in order to achieve a moisture content suitable for recompaction.**
- 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 La Habra.
- 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). It is recommended that materials in excess of 3 inches in size not be used for utility trench backfill. Compacted trench backfill should conform to the requirements of the local grading code, and more restrictive requirements may be indicated by the City of La Habra. All utility trench backfills should be witnessed by the geotechnical engineer. The trench backfill soils should be compaction tested where possible; probed and visually evaluated elsewhere.

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

6.4 Construction Considerations

Excavation Considerations

The near-surface soils at this site generally consist of sandy clays and silty clays. These materials are expected to be relatively stable within shallow excavations. However, some of the existing fill soils consist of very loose to loose sandy soils. These sandy soils may be subject to caving. If caving occurs within shallow excavations, flattened excavation slopes may be sufficient to provide excavation stability. Deeper excavations may require some form of external stabilization such as shoring or bracing. Maintaining adequate moisture content within the near-surface soils will improve excavation stability. Temporary excavation slopes should be no steeper than 1.5h:1v in clayey soils and 2h:1v in sandy soils. All excavation activities on this site should be conducted in accordance with Cal-OSHA regulations.

Remedial grading for the proposed structure will require excavation in close proximity to the south and east property lines. The contractor should take all necessary provisions to protect the existing improvements, including sidewalks. As discussed previously, slot cutting procedures may be necessary during remedial grading operations in these areas. The geotechnical engineer should observe the conditions and determine the appropriate slot cutting procedures at the time of site grading.

If slot cutting is required, typically A-B-C slots on 6 to 8-foot centers are suitable to maintain excavation stability. Initially, no soil should be removed from the zone defined by a 1h:1v outward projection beginning at a point 1 foot outside the edge of the existing sidewalk. The geotechnical engineer should be present to observe the soil conditions and determine the appropriate slot cutting procedures at the time of site grading.

Expansive Soils

The near surface on-site soils are expected to possess a medium expansion potential. Therefore, care should be given to proper moisture conditioning of all building pad subgrade soils to a moisture content of 2 to 4 percent above the Modified Proctor optimum during site grading. All imported fill soils should have very low expansive (EI < 20) characteristics. **In addition to adequately moisture conditioning the subgrade soils and fill soils during grading, special care must be taken to maintain moisture content of these soils at 2 to 4 percent above the Modified Proctor optimum. This will require the contractor to frequently moisture condition these soils throughout the grading process, unless grading occurs during a period of relatively wet weather**.

Due to the presence of expansive soils at this site, provisions should be made to limit the potential for surface water to penetrate the soils immediately adjacent to the structure. These provisions should include directing surface runoff into rain gutters and area drains, reducing the extent of landscaped areas around the structure, and sloping the ground surface away from the building. Where possible, it is recommended that landscaped planters not be located immediately adjacent to the building. If landscaped planters around the buildings are necessary, it is recommended that drought tolerant plants or a drip irrigation system be utilized, to minimize the potential for deep moisture penetration around the 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 offsite.
- Enclosed planters adjoining, or in close proximity to proposed structure, should be sealed at the bottom and provided with subsurface collection systems and outlet pipes.
- Depressed planters should be raised with soil to promote runoff (minimum drainage gradient two percent or five percent, see above), and/or equipped with area drains to eliminate ponding.
- Drainage outfall locations should be selected to avoid erosion of slopes and/or properly armored to prevent erosion of graded surfaces. No drainage should be directed over or towards adjoining slopes.
- All drainage devices should be maintained on a regular basis, including frequent observations during the rainy season to keep the drains free of leaves, soil and other debris.

- Landscape irrigation should conform to the recommendations of the landscape architect and should be performed judiciously to preclude either soaking or excessive drying of the foundation soils. This should entail regular watering during the drier portions of the year and little or no irrigation during the rainy season. Automatic sprinkler systems should, therefore, be switched to manual operation during the rainy season. Good irrigation practice typically requires frequent application of limited quantities of water that are sufficient to sustain plant growth, but do not excessively wet the soils. Ponding and/or run-off of irrigation water are indications of excessive watering.

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

<u>Groundwater</u>

The static groundwater table at this site is considered to exist at a depth in excess of 25± feet. Therefore, groundwater is not expected to impact grading or foundation construction activities.

6.5 Foundation Design and Construction

Based on the preceding grading recommendations, it is assumed that the new building pad will be underlain by new structural fill soils used to replace the existing undocumented fill soils and a portion of the near-surface alluvial soils. These structural fill soils are expected to extend to a depth of at least 3 feet below proposed foundation bearing grade. Based on this subsurface profile, the proposed structure may be supported on conventional shallow foundations.

<u>Foundation Design Parameters</u>

New square and rectangular footings may be designed as follows:

- Maximum, net allowable soil bearing pressure: 2,500 lbs/ft^2.
- Minimum wall/column footing width: 14 inches/24 inches.
- Minimum longitudinal steel reinforcement within strip footings: Six (6) No. 5 rebars (3 top and 3 bottom), due to the presence of medium expansive 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 or seismic loads. The minimum steel reinforcement recommended above is based on geotechnical considerations; additional reinforcement may be necessary for structural considerations. The actual design of the foundations should be determined by the structural engineer.

Foundation Construction

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

The foundation subgrade soils should also be properly moisture conditioned to 2 to 4 percent above the Modified Proctor optimum, to a depth of at least 12 inches below bearing grade. **Since it is typically not feasible to increase the moisture content of the floor slab and foundation subgrade soils once rough grading has been completed, care should be taken to maintain the moisture content of the building pad subgrade soils throughout the construction process.**

Estimated Foundation Settlements

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

Lateral Load Resistance

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

- Passive Earth Pressure: 250 lbs/ft^3
- Friction Coefficient: 0.25

These are allowable values, and include a factor of safety. When combining friction and passive resistance, the passive pressure component should be reduced by one-third. These values assume that footings will be poured directly against compacted structural fill. The maximum allowable passive pressure is 3000 lbs/ft^2.

6.6 Floor Slab Design and Construction

Subgrades which will support new floor slabs should be prepared in accordance with the recommendations contained in the ***Site Grading Recommendations*** section of this report. Based on the anticipated grading which will occur at this site, the floor of the new building may be constructed as a conventional slab-on-grade supported on existing or newly placed structural fill soils, extending to a depth of at least 5 feet below existing grade. Based on geotechnical considerations, the floor slab may be designed as follows:

- Minimum slab thickness: 5 inches.

- Minimum slab reinforcement: No. 4 bars at 16 inches on-center, in both directions, due to presence of medium expansive soils at this site. The actual floor slab reinforcement should be determined by the structural engineer, based upon the imposed loading.

- Slab underlayment: If moisture sensitive floor coverings will be used or if vapor transmission into the area above the building slab is problematic, then minimum slab underlayment should consist of a moisture vapor barrier constructed below the entire area of the proposed slab. 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 15-mil 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 patios and sidewalks should be prepared in accordance with the recommendations contained in Section 6.3 of this report. Based on these recommendations, the exterior flatwork will be supported on existing fill soils that have been scarified and moisture conditioned to a depth of 12 inches and recompacted to 90 percent of the ASTM D-1557 maximum dry density. Based on geotechnical considerations, exterior slabs on grade which are not subjected to any vehicular traffic may be designed as follows:

- Minimum slab thickness: 4 inches

- Minimum slab reinforcement: No. 4 bars at 18 inches on center, in both directions.

- Moisture condition the flatwork subgrade soils to 2 to 4 percent of the optimum moisture content, to a depth of at least 12 inches.

- 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.

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

- Where the flatwork is adjacent to a landscape planter or another area with exposed soil, it should incorporate a turned down edge. This turned down edge should be at least 12 inches in depth and 6 inches in width. The turned down edge should incorporate longitudinal steel reinforcement consisting of at least one No. 4 bar.

- Flatwork which is constructed immediately adjacent to the new structure should be dowelled into the perimeter foundations in a manner determined by the structural engineer.

- Some cracking of exterior flatwork at this site should be expected, due to the presence of expansive soils.

6.8 Retaining Wall Design and Construction

Although not indicated on the site plan, some small retaining walls (less than 6 feet in height) 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 near-surface soils consist of medium expansive silty clays and fine sandy clays, which are not considered suitable for retaining wall backfill.** We have provided retaining wall design parameters assuming the use of imported sandy materials for retaining wall backfill. The imported sandy soils should possess an internal angle of friction of at least 30 degrees when compacted to 90 percent of the ASTM D-1557 maximum dry density.

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

RETAINING WALL DESIGN PARAMETERS

Design Parameter		Soil Type: Imported Sandy Soils
Internal Friction Angle (ϕ)		30°
Unit Weight		130 lbs/ft^3
Equivalent Fluid Pressure:	Active Condition (level backfill)	43 lbs/ft^3
	Active Condition (2h:1v backfill)	70 lbs/ft^3
	At-Rest Condition (level backfill)	65 lbs/ft^3

The walls should be designed using a soil-footing coefficient of friction of 0.25 and an equivalent passive pressure of 250 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. Based on the site plan provided to our office, it is not expected that any walls in excess of 6 feet in height will be required for this project. In the event that such walls are required, our office should be contacted for supplementary design parameters.

Retaining Wall Foundation Design

The retaining wall foundations should be supported within newly placed compacted structural fill, extending to a depth of at least 3 feet below proposed foundation bearing grade. Foundations to support new retaining walls should be designed in accordance with the general Foundation Design Parameters presented in a previous section of this report.

Backfill Material

Retaining walls should be backfilled with imported sandy soils. Onsite soils are not recommended for use as retaining wall backfill. 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 supported on the existing fill and/or native soils that have been scarified, moisture conditioned, and recompacted. These materials generally consist of silty clays and sandy clays. These materials are expected to exhibit fair to poor pavement support characteristics, with estimated R-values of 10 to 20. Since R-value testing was not included in the scope of services for the current project, the subsequent pavement designs are based upon a conservatively assumed R-value of 10. 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 may be desirable to perform R-value testing after the completion of rough grading to verify the R-value of the as-graded parking subgrade.

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 seven operational traffic days per week.

Traffic Index	No. of Heavy Trucks per Day
4.0	0
5.0	1
6.0	3

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 = 10)			
Materials	**Thickness (inches)**		
	Auto Parking (TI = 4.0)	Auto Drive Lanes (TI = 5.0)	Light Truck Traffic (TI = 6.0)
Asphalt Concrete	3	3	3½
Aggregate Base	6	9	12
Compacted Subgrade	12	12	12

The aggregate base course should be compacted to at least 95 percent of the ASTM D-1557 maximum dry density. The asphaltic concrete should be compacted to at least 95 percent of the Marshall maximum density, as determined by ASTM D-2726. The aggregate base course may consist of crushed aggregate base (CAB) or crushed miscellaneous base (CMB), which is a recycled gravel, asphalt and concrete material. The gradation, R-Value, Sand Equivalent, and Percentage Wear of the CAB or CMB should comply with appropriate specifications contained in the current edition of the "Greenbook" Standard Specifications for Public Works Construction.

Portland Cement Concrete

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

PORTLAND CEMENT CONCRETE PAVEMENTS (R = 10)		
Materials	**Thickness (inches)**	
	Automobile Parking and Drive Areas	Light Truck Traffic Areas (TI =6.0)
PCC	5	5½
Compacted Subgrade (95% minimum compaction)	12	12

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

7.0 GENERAL COMMENTS

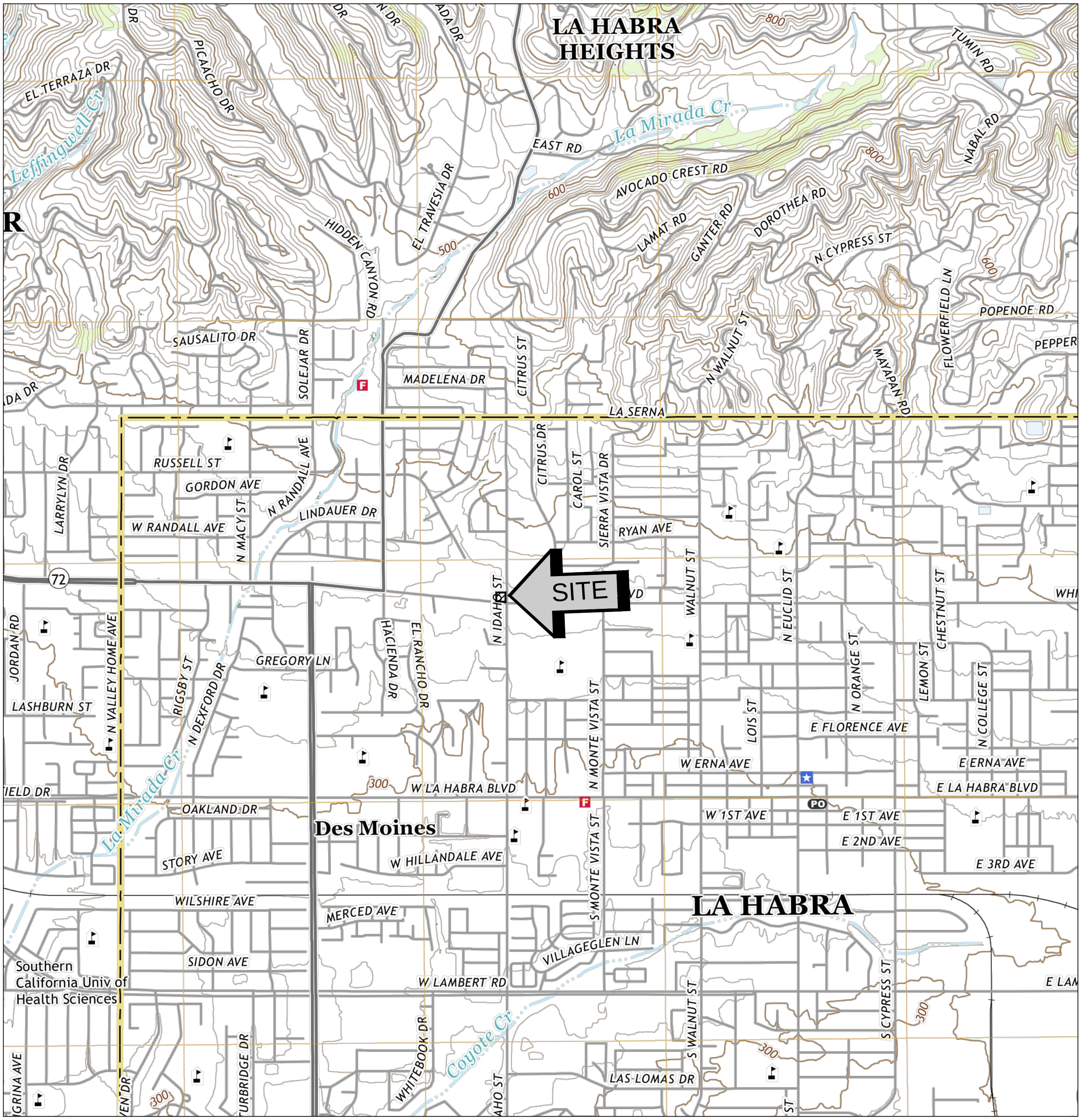
This report has been prepared as an instrument of service for use by the client, in order to aid in the evaluation of this property and to assist the architects and engineers in the design and preparation of the project plans and specifications. This report may be provided to the contractor(s) and other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

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

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

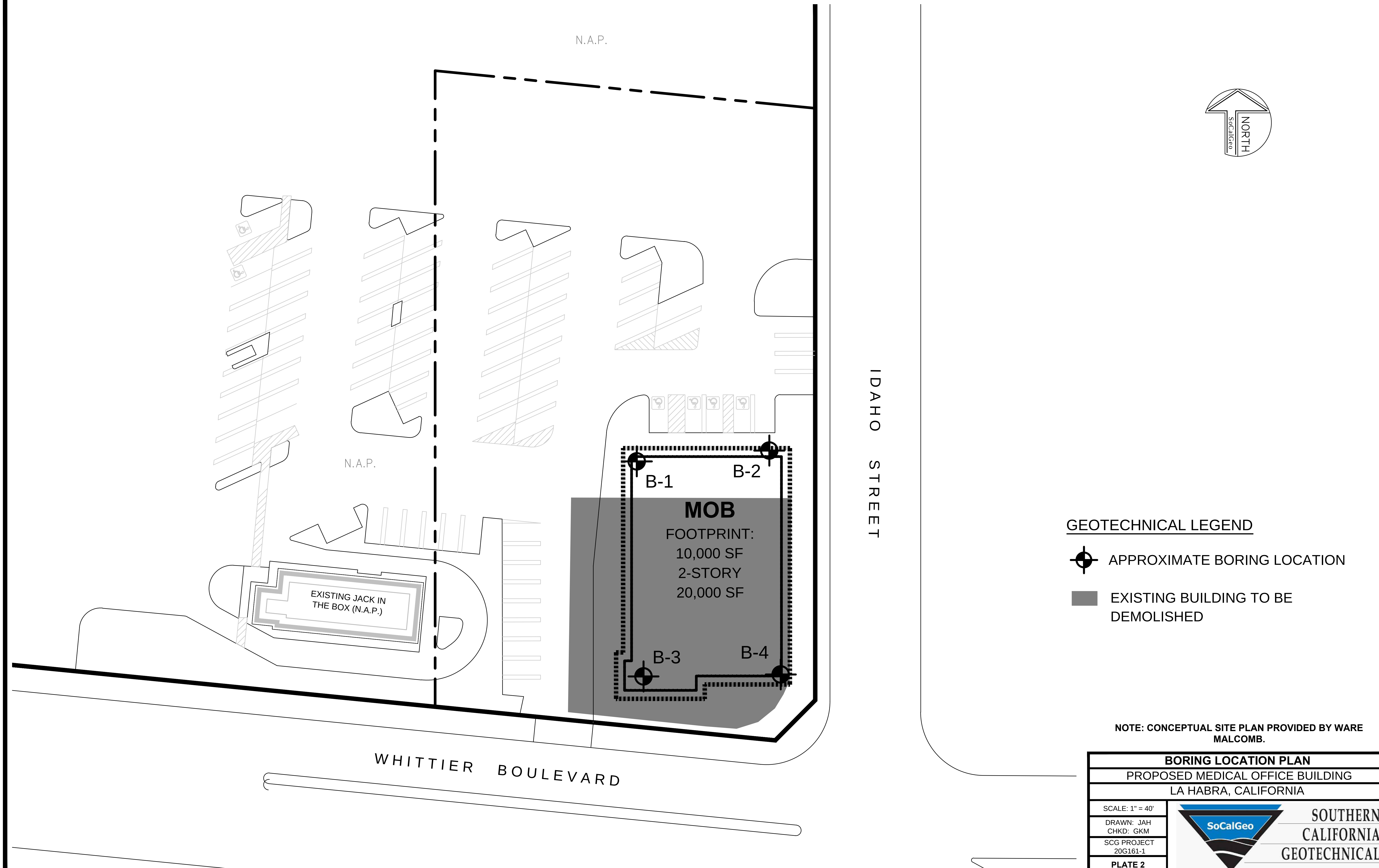
The analysis, conclusions, and recommendations contained within this report have been promulgated in accordance with generally accepted professional geotechnical engineering practice. No other warranty is implied or expressed.

APPENDIX A



SOURCE: USGS TOPOGRAPHIC MAP OF THE LA HABRA QUADRANGLE, ORANGE COUNTY, CALIFORNIA, 2018

SITE LOCATION MAP	
PROPOSED MEDICAL OFFICE BUILDING	
LA HABRA, CALIFORNIA	
SCALE: 1" = 2000'	SOUTHERN CALIFORNIA GEOTECHNICAL
DRAWN: JAH CHKD: GKM	
SCG PROJECT 20G161-1	
PLATE 1	



NOTE: CONCEPTUAL SITE PLAN PROVIDED BY WARE MALCOMB.

BORING LOCATION PLAN

PROPOSED MEDICAL OFFICE BUILDING

LA HABRA, CALIFORNIA

SCALE: 1" = 40'

DRAWN: JAH
CHKD: GKM

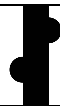
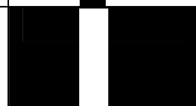
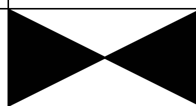
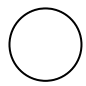
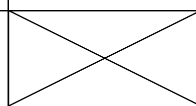
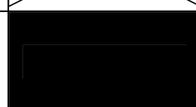
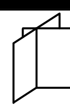
SCG PROJECT 20G161-1

PLATE 2

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL

APPENDIX B

BORING LOG LEGEND

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

COLUMN DESCRIPTIONS

DEPTH: Distance in feet below the ground surface.

SAMPLE: Sample Type as depicted above.

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

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

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

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

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

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

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

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

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

SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS		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.: 20G161-1
PROJECT: Proposed Medical Office Building
LOCATION: La Habra, California

DRILLING DATE: 6/22/20
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jamie Hayward

WATER DEPTH: Dry
CAVE DEPTH: 15 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
					6± inches Asphaltic Concrete, 2± inches Aggregate Base							
	X	20			FILL: Gray Brown fine to coarse Sand, trace fine Gravel, occasoinal Red Brown Silty Clay nodules, mottled, loose to medium dense-moist to very moist	113	11					
	X	9				98	14					
5	X	5				102	18					
	X	4			FILL: Gray fine to coarse Sand, very loose-damp	105	4					
10	X	7			FILL: Gray Brown fine to coarse Sand, occasional Red Brown Silty Clay nodules, loose-damp to moist	100	6					
15	X	11			ALLUVIUM: Green Gray fine Sandy Silt, little Calcareous veining, strong hydrocarbon odor, slightly porous, medium dense-moist to very moist @ 13½ to 15 feet, trace to little Clay		20					
20	X	18					11					
25	X	22			Gray fine to medium Sand, trace coarse Sand, trace fine Gravel, medium dense-damp		4					
					Boring Terminated at 25 feet							

TBL 20G161-1.GPJ SOCALGEO.GDT 7/9/20

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-2

JOB NO.: 20G161-1
PROJECT: Proposed Medical Office Building
LOCATION: La Habra, California

DRILLING DATE: 6/22/20
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jamie Hayward

WATER DEPTH: Dry
CAVE DEPTH: 16 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
					3± inches Asphaltic Concrete, 6± inches Aggregate Base							
	X	14	4.5		ALLUVIUM: Red Brown fine Sandy Clay, trace medium to coarse Sand, stiff to very stiff-very moist		19					
	X	13	3.0				22					
5												
	X	19			Red Brown to Brown fine Sandy Silt with occasional thinly interbedded fine Sandy Clay and Clayey fine Sand lenses, little Calcareous veining/nodules, trace medium to coarse Sand, medium dense-very moist		26					
	X	40			@ 8½ to 10 feet, slight hydrocarbon odor, dense		16					
10												
	X	15					18					
15												
	X	18					18					
20												
	X	24					16					
25												
					Boring Terminated at 25 feet							

TBL 20G161-1.GPJ SOCALGEO.GDT 7/9/20

TEST BORING LOG

PLATE B-2

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-3

JOB NO.: 20G161-1
PROJECT: Proposed Medical Office Building
LOCATION: La Habra, California

DRILLING DATE: 6/22/20
DRILLING METHOD: Hand Auger
LOGGED BY: Ryan Bremer

WATER DEPTH: Dry
CAVE DEPTH: 10 feet
READING TAKEN: At Completion

SURFACE ELEVATION: --- MSL

DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					4¼± inches Portland Cement Concrete							
	X		4.5		FILL: Dark Brown Silty Clay, trace fine Sand, very stiff-moist	109	15					
	X		4.5		FILL: Red Brown to Brown fine Sandy Clay, trace Silt, trace fine Gravel, mottled, very stiff-moist to very moist	84	21					
5	X		3.0		@ 5 to 6 feet, trace Brick fragments	99	13					
	X		3.0		ALLUVIUM: Light Brown to Brown fine Sandy Clay, some Calcareous veining, little Silt, trace medium to coarse Sand, very stiff-very moist	88	20					
	X		3.0			94	20					
10					Boring Terminated at 10 feet							

TBL 20G161-1.GPJ SOCALGEO.GDT 7/9/20

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-4

JOB NO.: 20G161-1
PROJECT: Proposed Medical Office Building
LOCATION: La Habra, California

DRILLING DATE: 6/22/20
DRILLING METHOD: Hand Auger
LOGGED BY: Ryan Bremer

WATER DEPTH: Dry
CAVE DEPTH: 10 feet
READING TAKEN: At Completion

SURFACE ELEVATION: --- MSL

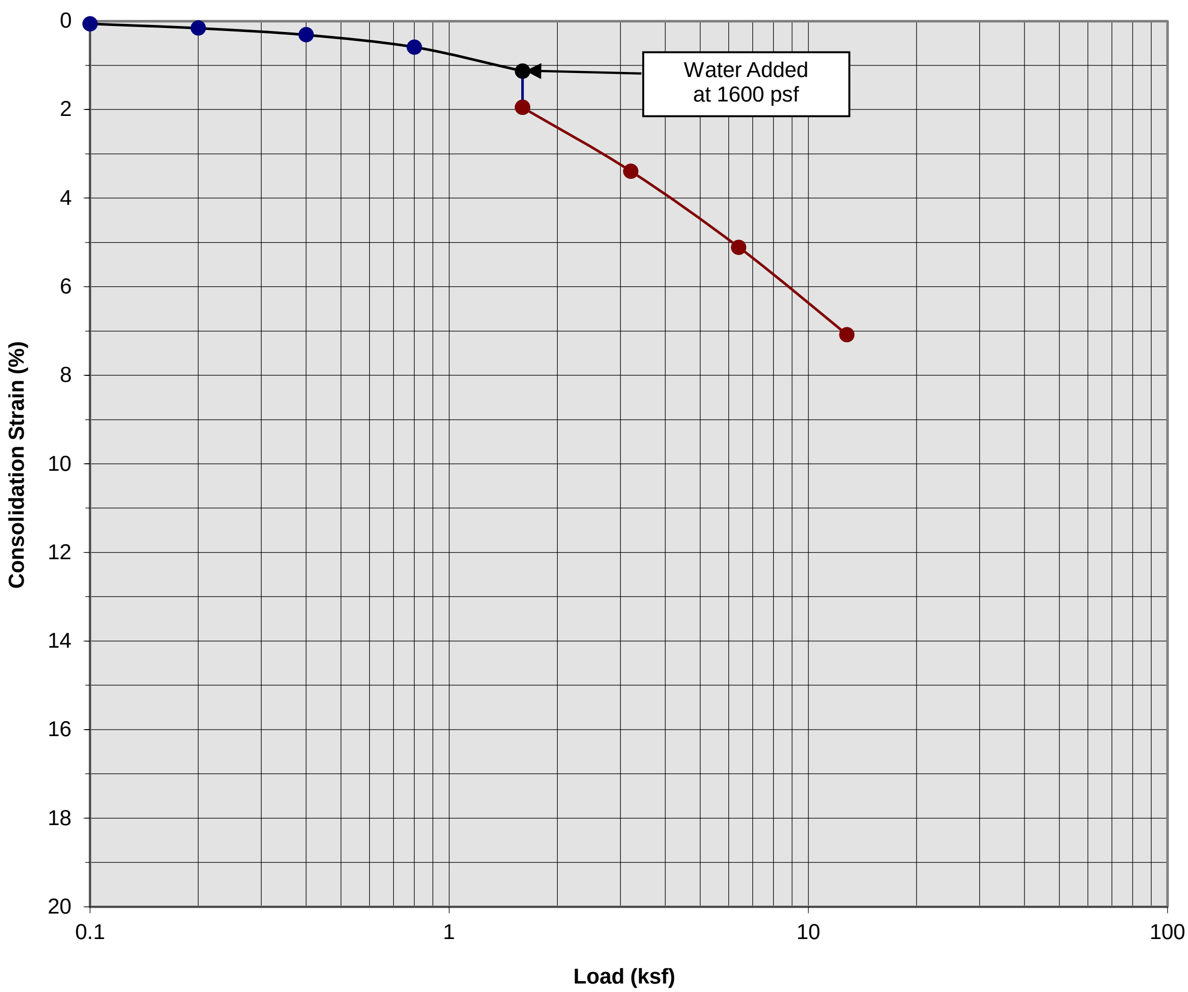
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					6± inches Portland Cement Concrete							
	X		3.0		FILL: Dark Brown fine Sandy Clay to Silty Clay, mottled, very stiff-very moist	82	18					EI = 70 @ 0 to 5 feet
	X		3.0		FILL: Dark Brown Silty Clay, trace fine Sand, very stiff-very moist	89	18					
5	X		3.0		FILL: Red Brown to Dark Brown fine Sandy Clay, trace to little Silt, mottled, very stiff-moist	83	19					
	X		2.5		ALLUVIUM: Red Brown Silty Clay, stiff-moist to very moist	97	20					
	X		2.5		@ 9 feet, some Calcareous veining	97	23					
10												

Boring Terminated at 10 feet

TBL 20G161-1.GPJ SOCALGEO.GDT 7/9/20

APPENDIX C

Consolidation/Collapse Test Results

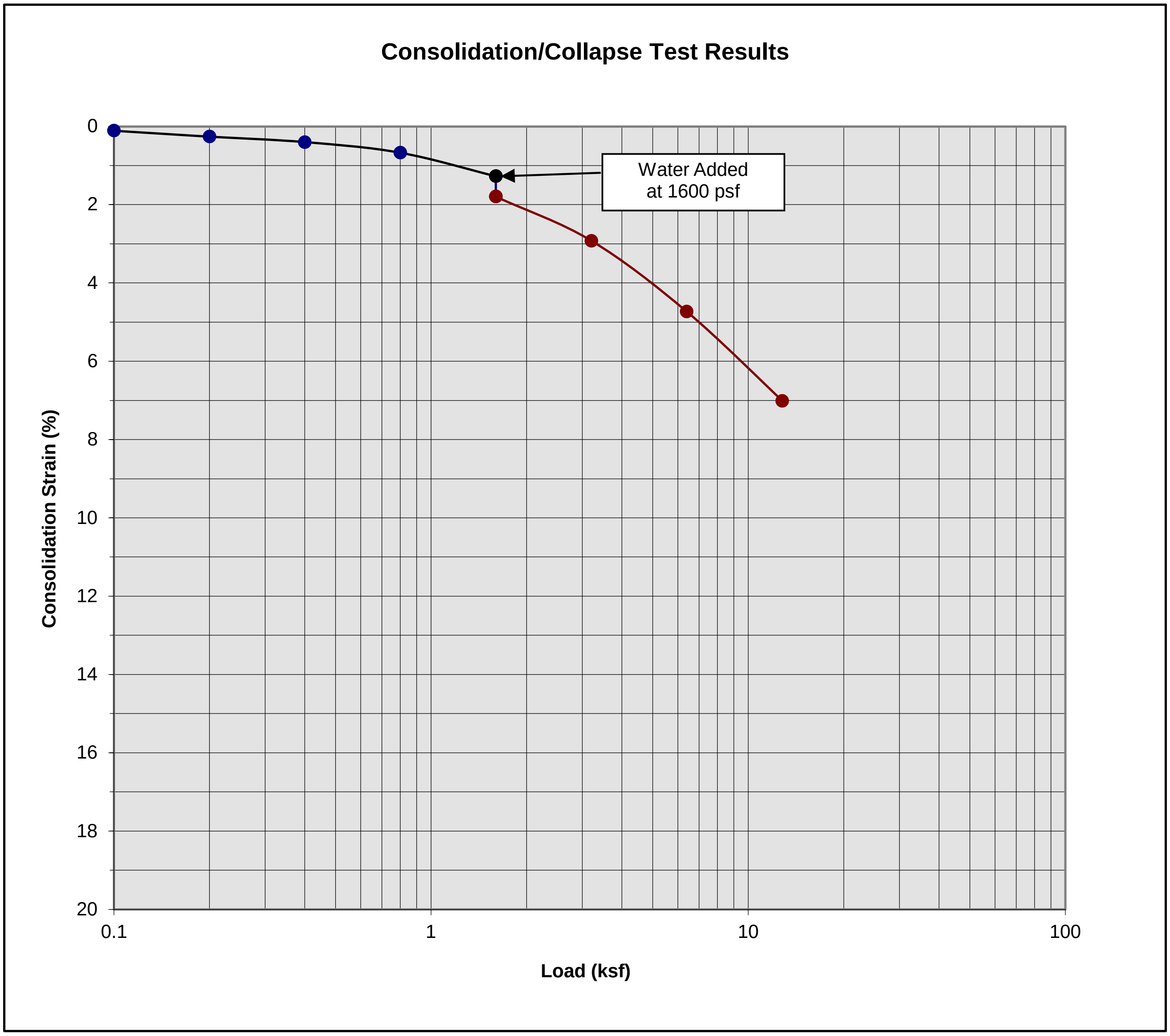


Classification: FILL: Gray Brown fine to coarse Sand, occasional Silty Clay nodules

Boring Number:	B-1	Initial Moisture Content (%)	15
Sample Number:	---	Final Moisture Content (%)	16
Depth (ft)	3 to 4	Initial Dry Density (pcf)	99.1
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	106.3
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.82

Proposed Medical Office Building
La Habra, California
Project No. 20G161-1
PLATE C- 1

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

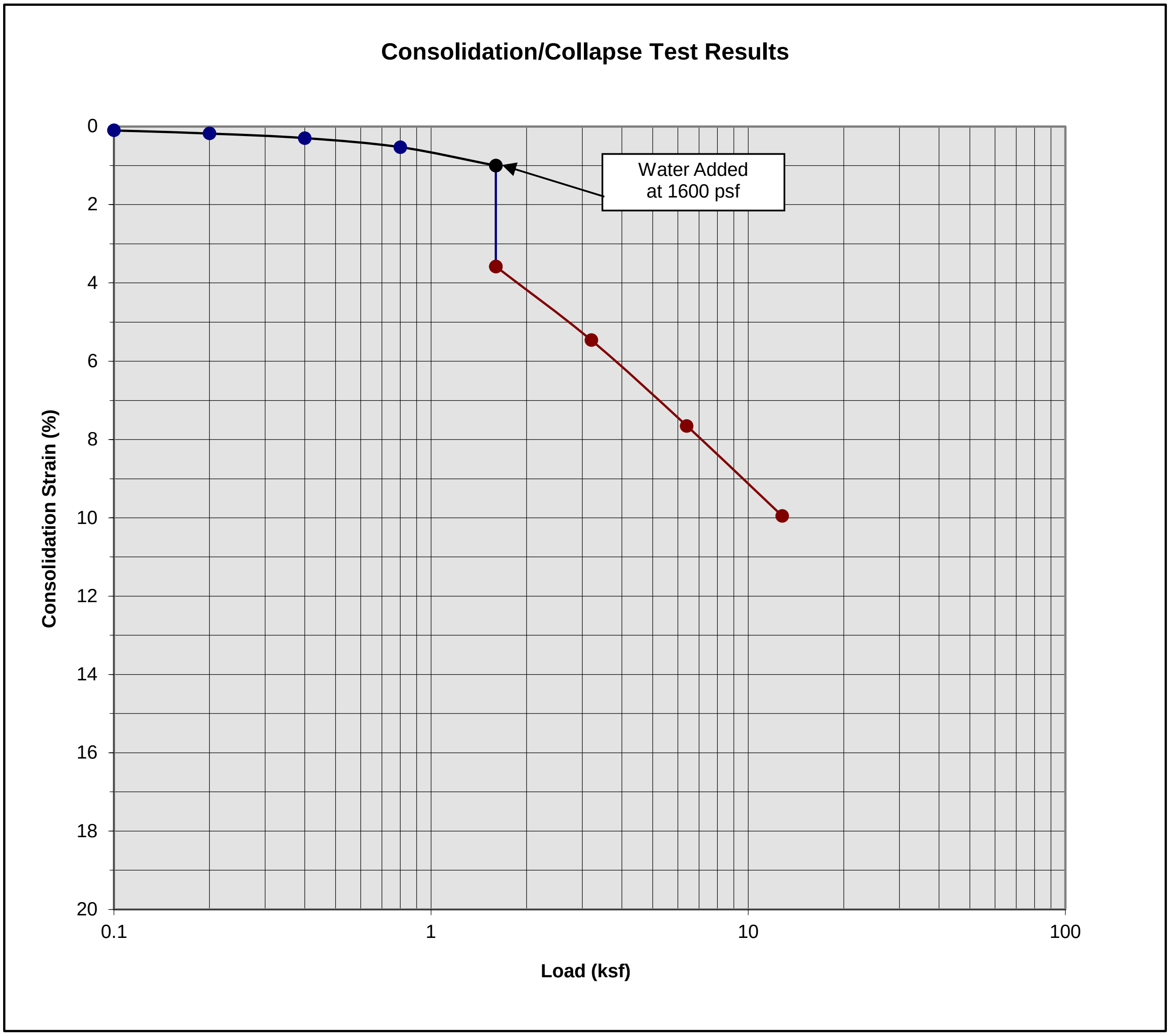


Classification: FILL: Gray Brown fine to coarse Sand, occasional Silty Clay nodules

Boring Number:	B-1	Initial Moisture Content (%)	18
Sample Number:	---	Final Moisture Content (%)	20
Depth (ft)	5 to 6	Initial Dry Density (pcf)	101.6
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	109.7
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.52

Proposed Medical Office Building
La Habra, California
Project No. 20G161-1
PLATE C- 2

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

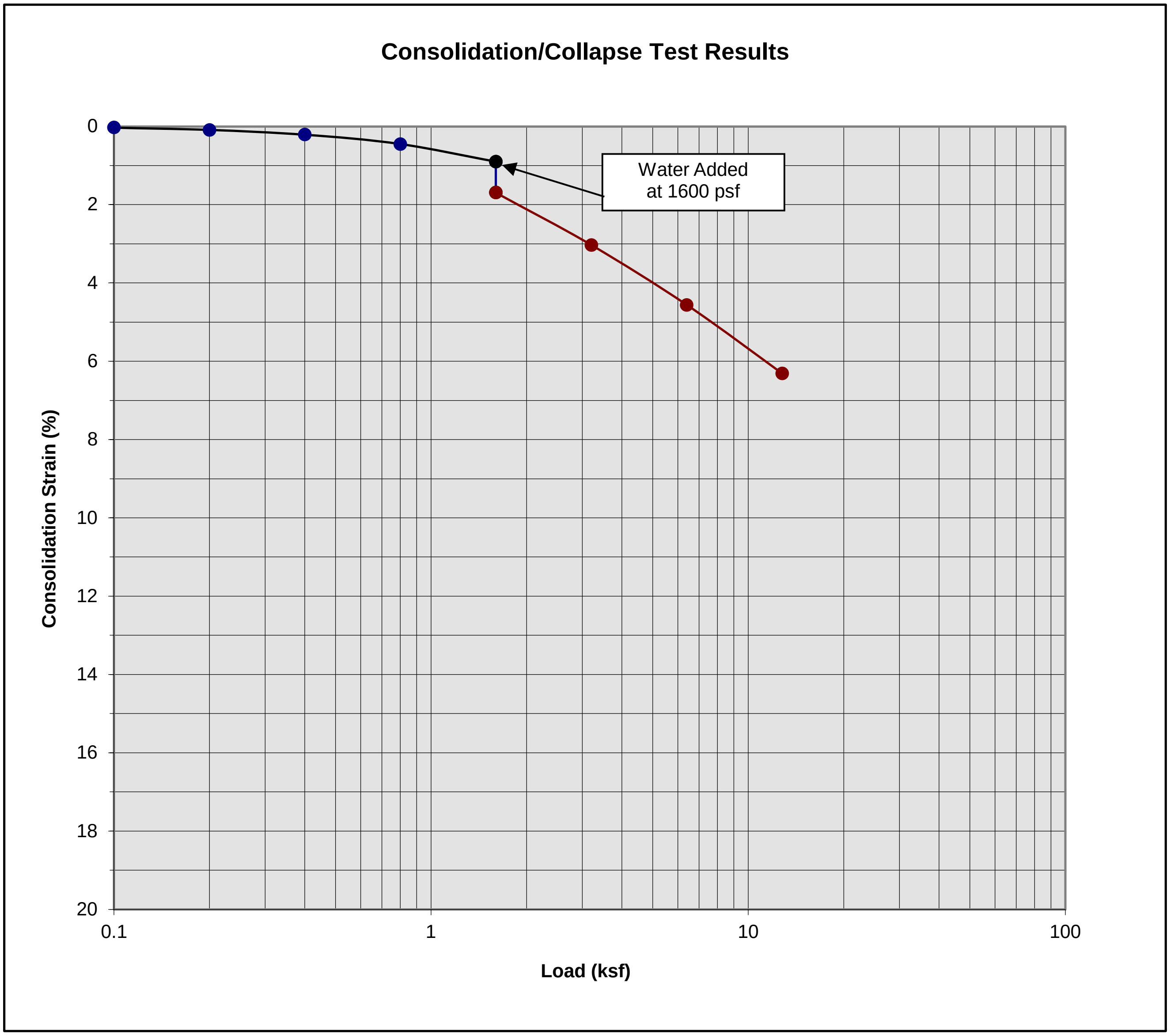


Classification: FILL: Gray fine to coarse Sand

Boring Number:	B-1	Initial Moisture Content (%)	4
Sample Number:	---	Final Moisture Content (%)	13
Depth (ft)	7 to 8	Initial Dry Density (pcf)	105.7
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	117.0
Specimen Thickness (in)	1.0	Percent Collapse (%)	2.58

Proposed Medical Office Building
La Habra, California
Project No. 20G161-1
PLATE C- 3

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

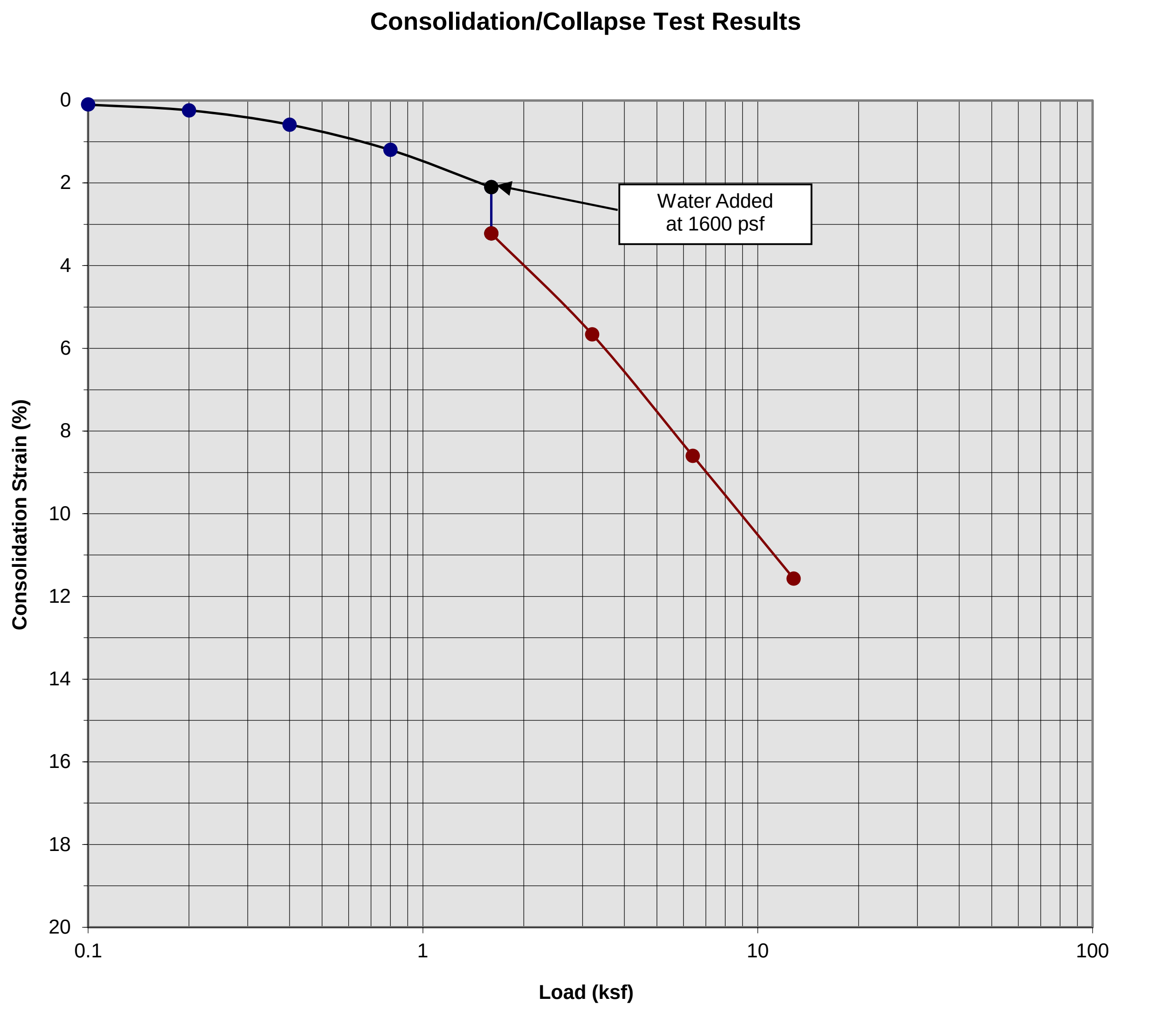


Classification: FILL: Gray Brown fine to coarse Sand, occasional Silty Clay nodules

Boring Number:	B-1	Initial Moisture Content (%)	6
Sample Number:	---	Final Moisture Content (%)	11
Depth (ft)	9 to 10	Initial Dry Density (pcf)	100.6
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	107.4
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.79

Proposed Medical Office Building
La Habra, California
Project No. 20G161-1
PLATE C- 4

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

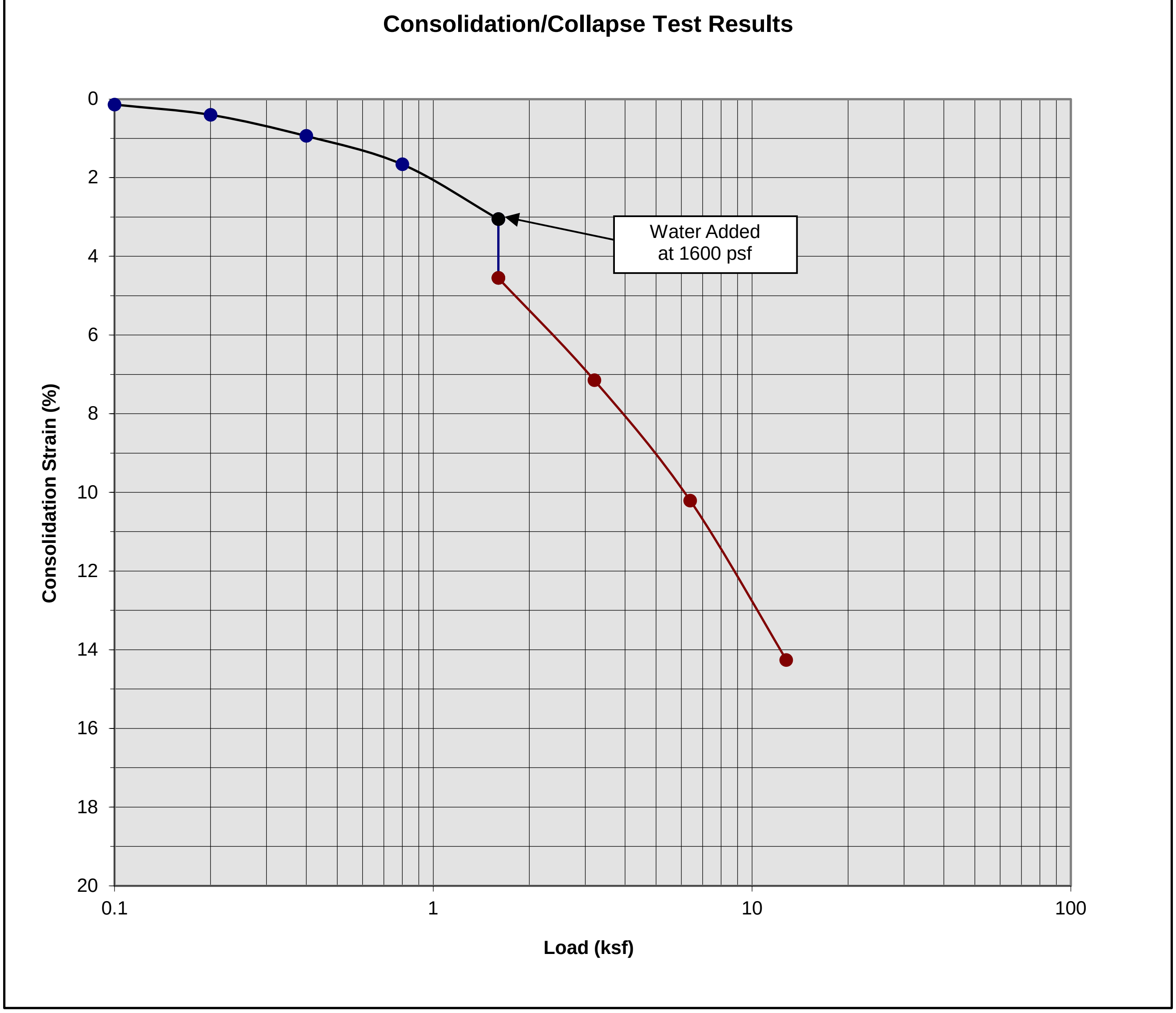


Classification: FILL: Dark Brown Silty Clay, trace fine Sand

Boring Number:	B-4	Initial Moisture Content (%)	18
Sample Number:	---	Final Moisture Content (%)	22
Depth (ft)	3 to 4	Initial Dry Density (pcf)	89.8
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	101.0
Specimen Thickness (in)	1.0	Percent Collapse (%)	1.12

Proposed Medical Office Building
La Habra, California
Project No. 20G161-1
PLATE C- 5

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

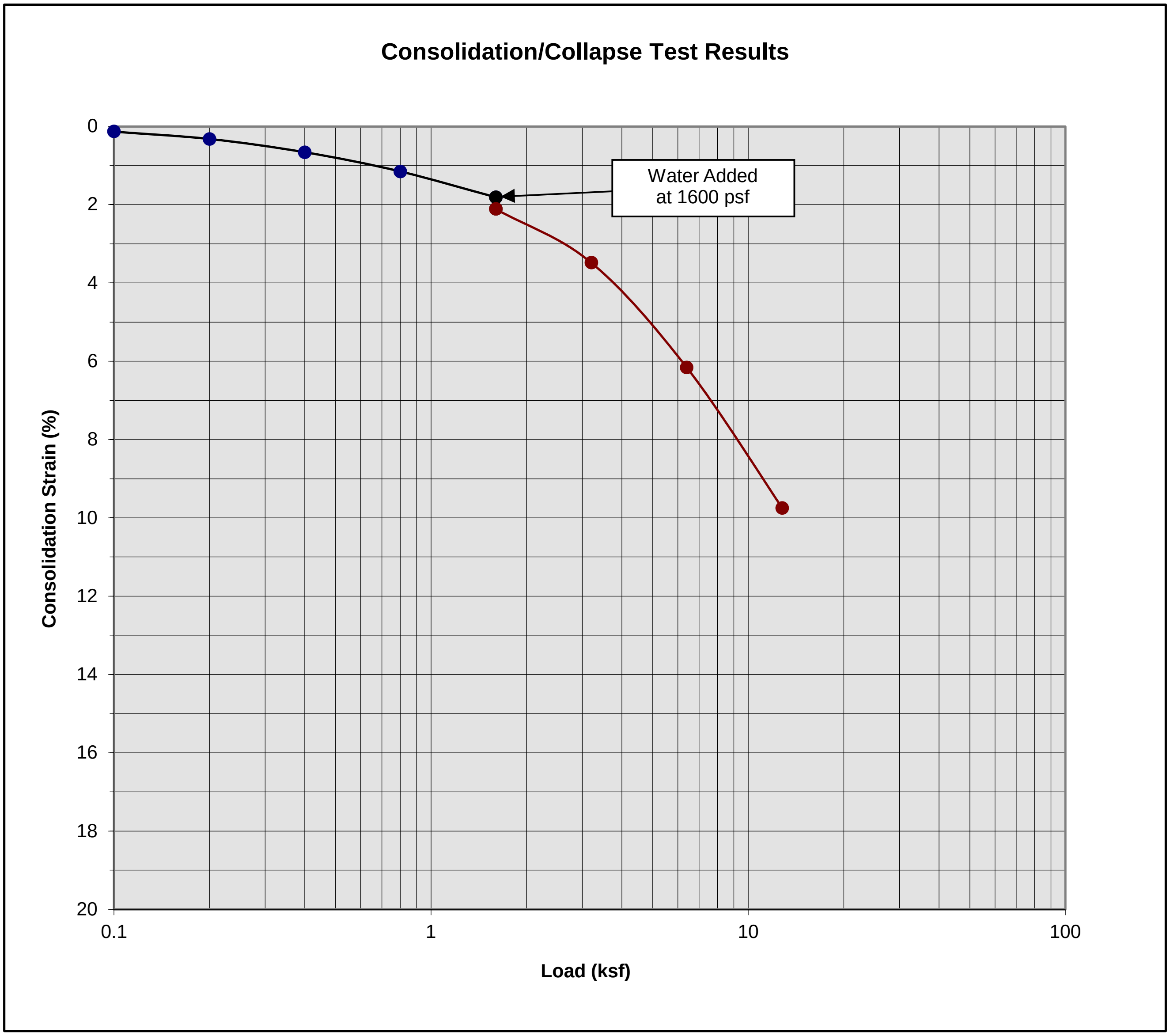


Classification: FILL: Red Brown to Dark Brown fine Sandy Clay, trace to little Silt

Boring Number:	B-4	Initial Moisture Content (%)	19
Sample Number:	---	Final Moisture Content (%)	27
Depth (ft)	5 to 6	Initial Dry Density (pcf)	83.2
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	96.9
Specimen Thickness (in)	1.0	Percent Collapse (%)	1.50

Proposed Medical Office Building
La Habra, California
Project No. 20G161-1
PLATE C- 6

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation



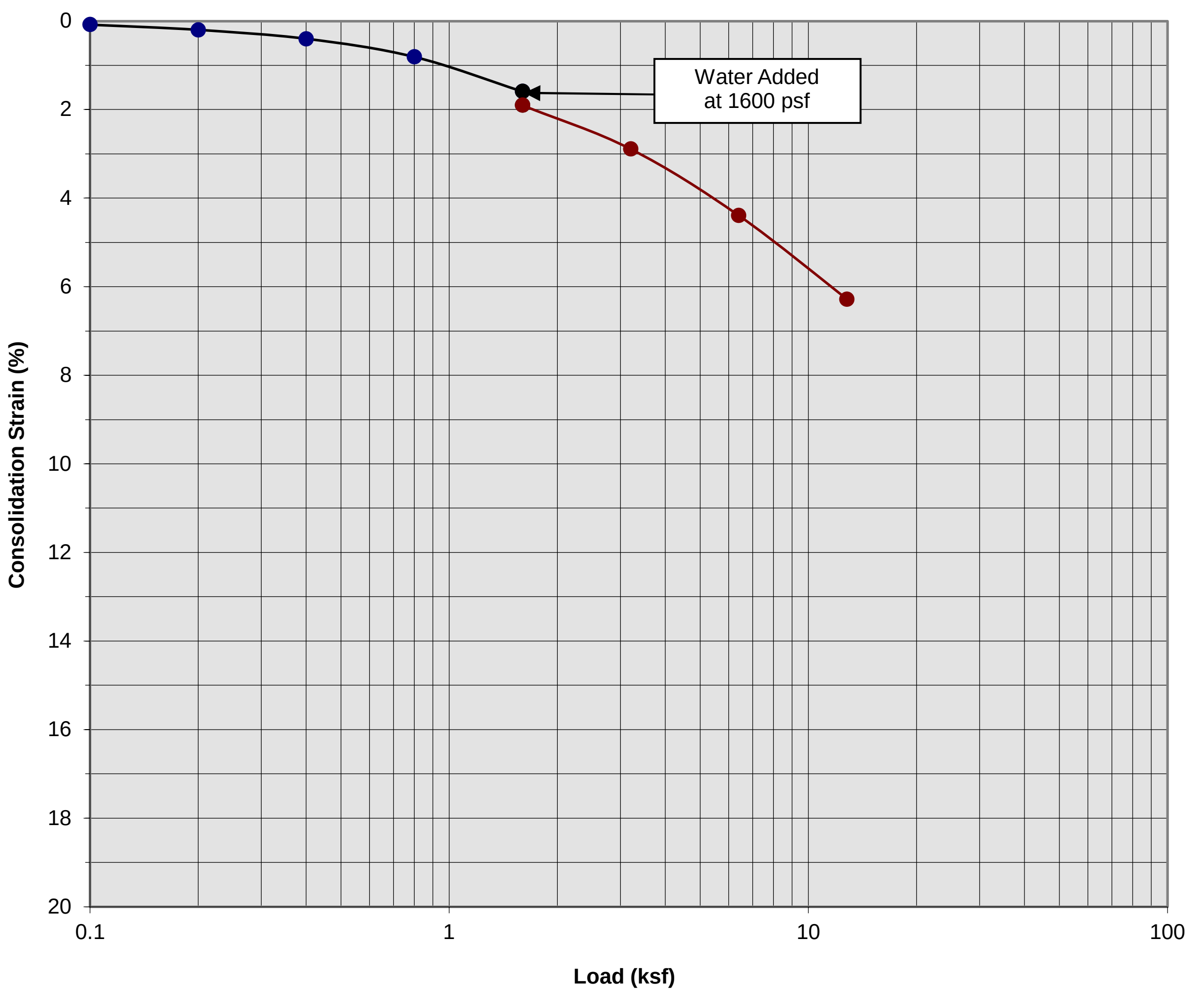
Classification: Red Brown Silty Clay

Boring Number:	B-4	Initial Moisture Content (%)	20
Sample Number:	---	Final Moisture Content (%)	22
Depth (ft)	7 to 8	Initial Dry Density (pcf)	96.8
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	107.1
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.30

Proposed Medical Office Building
La Habra, California
Project No. 20G161-1
PLATE C- 7

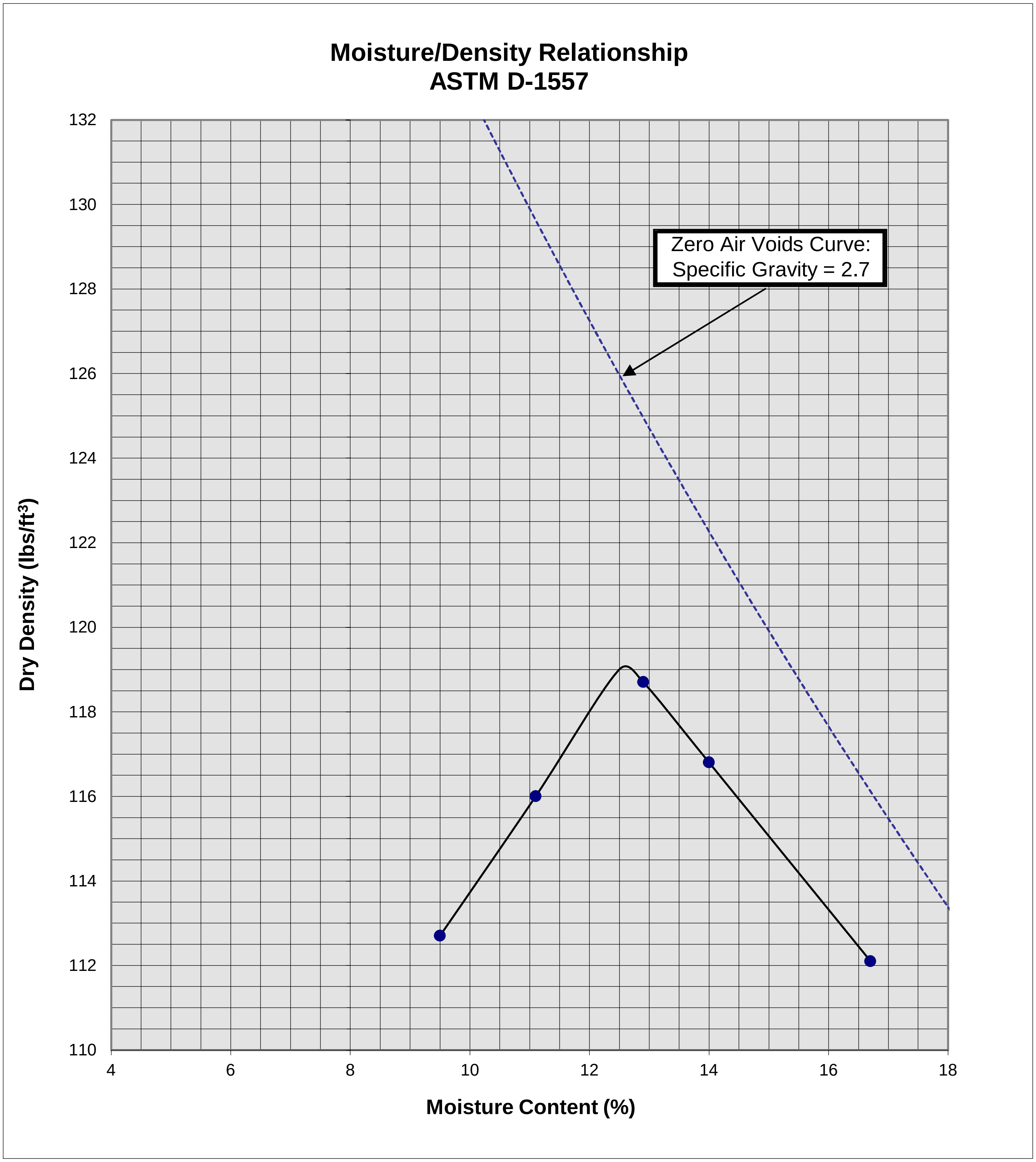
SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

Consolidation/Collapse Test Results



Classification: Red Brown Silty Clay

Boring Number:	B-4	Initial Moisture Content (%)	22
Sample Number:	---	Final Moisture Content (%)	26
Depth (ft)	9 to 10	Initial Dry Density (pcf)	97.3
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	103.9
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.31



Soil ID Number	B-4 @ 0-5'
Optimum Moisture (%)	12.5
Maximum Dry Density (pcf)	119
Soil Classification	Dark Brown fine Sandy Clay to Silty Clay

Proposed Medical Office Building
La Habra, California
Project No. 20G161-1
PLATE C-9

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

APPENDIX D

GRADING GUIDE SPECIFICATIONS

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

General

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

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

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

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

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

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

Site Preparation

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

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

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

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

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

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

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

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

<u>Compacted Fills</u>

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

Foundations

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

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

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

Fill Slopes

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

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

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

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

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

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

Cut Slopes

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

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

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

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

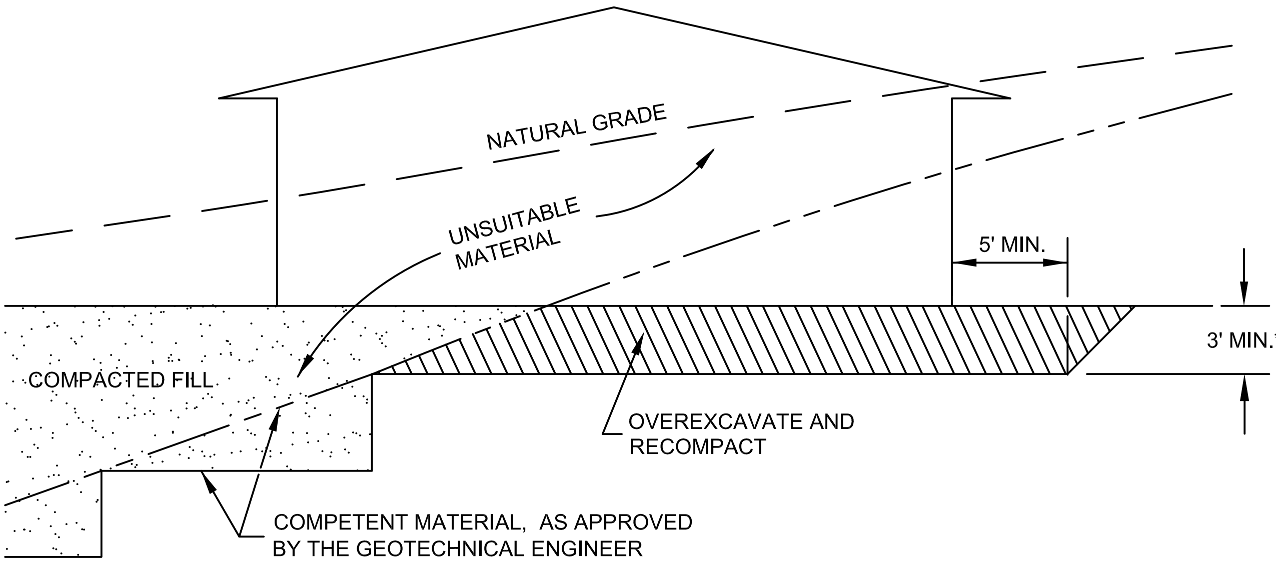
Subdrains

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

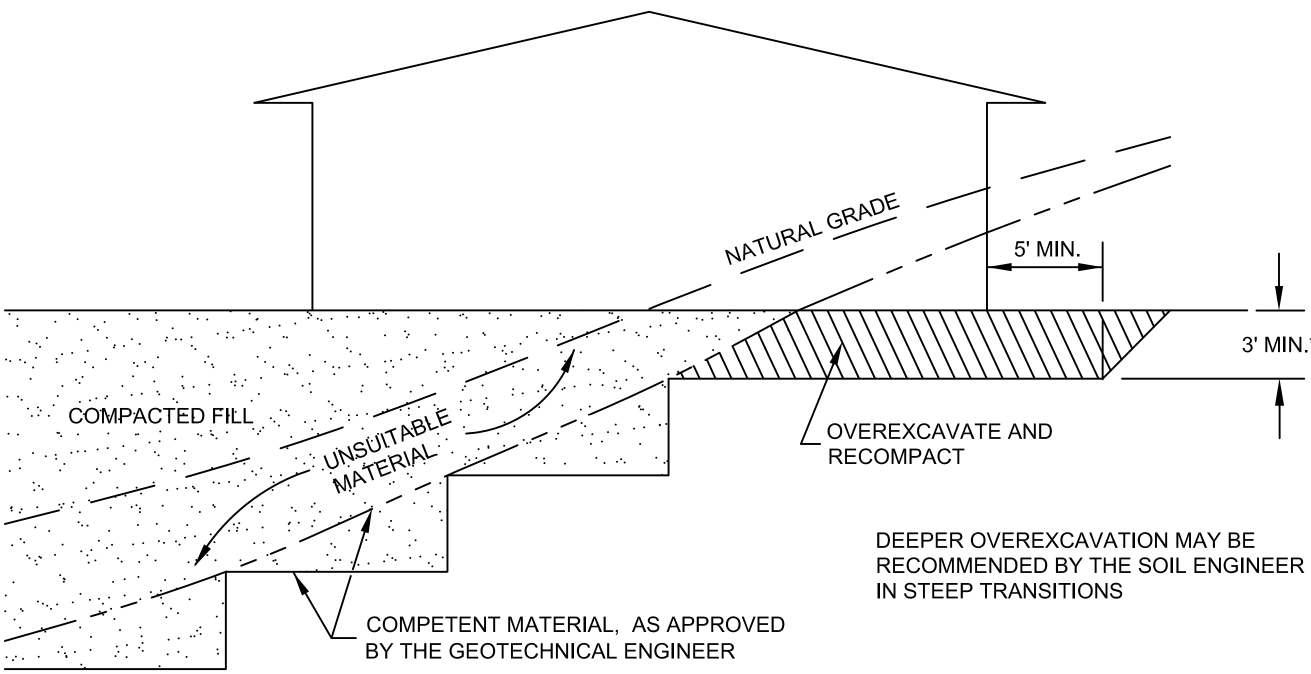
- Plastic pipe may be used for subdrains provided it is Schedule 40 or SDR 35 or equivalent. Pipe should be protected against breakage, typically by placement in a square-cut (backhoe) trench or as recommended by the manufacturer.

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

CUT LOT



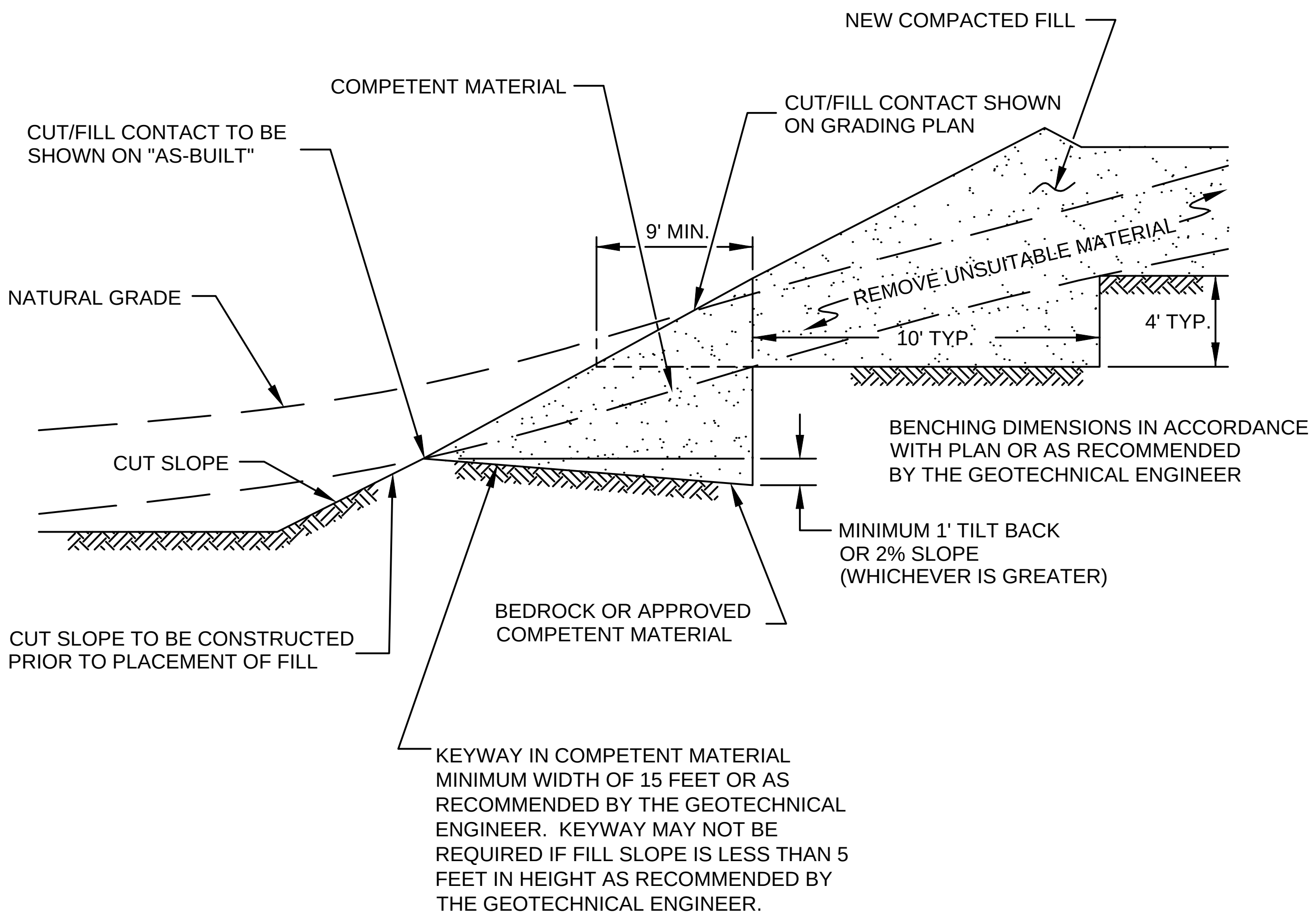
CUT/FILL LOT (TRANSITION)



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

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

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



FILL ABOVE CUT SLOPE DETAIL

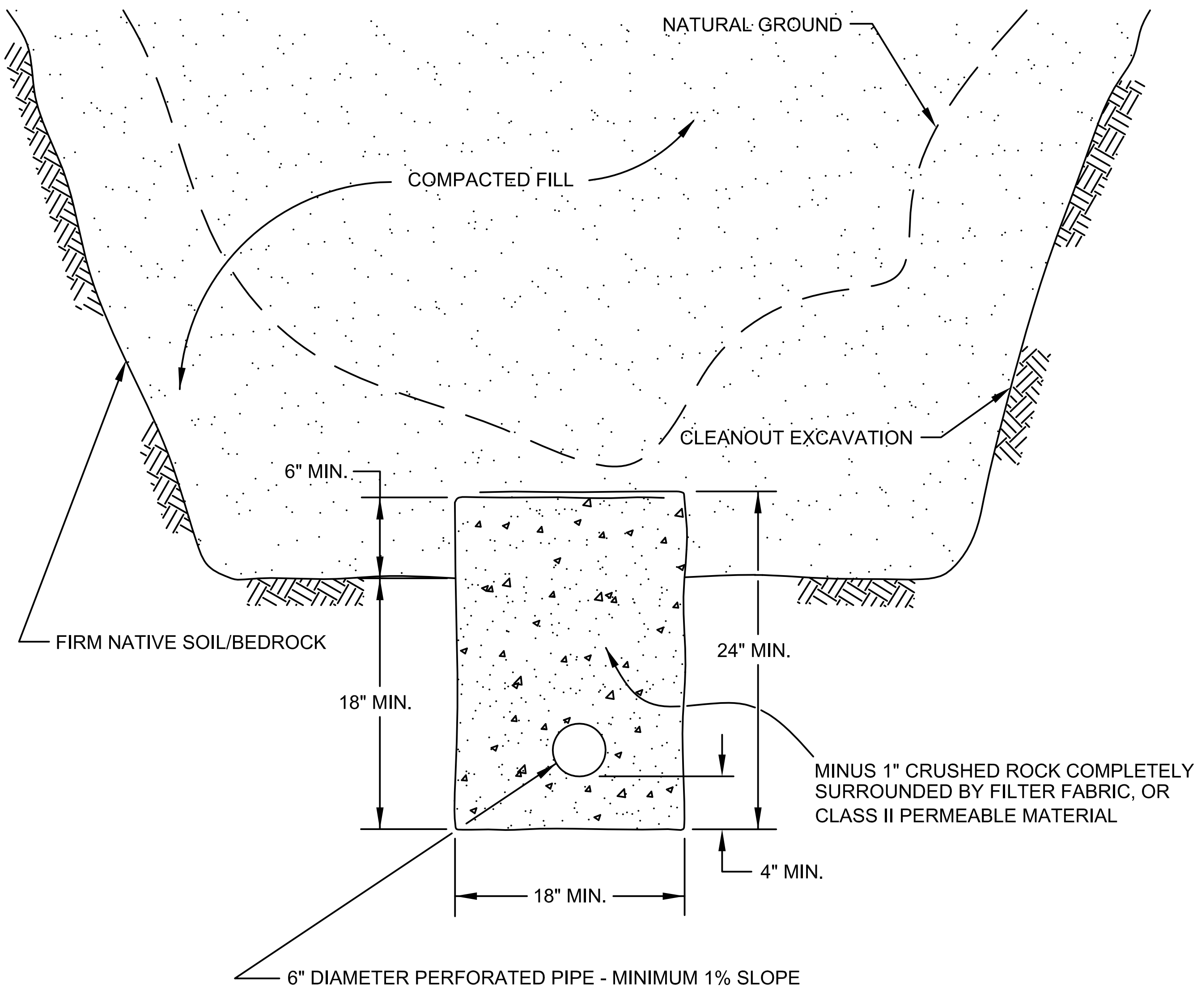
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-2

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL



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

SCHEMATIC ONLY
NOT TO SCALE

CANYON SUBDRAIN DETAIL

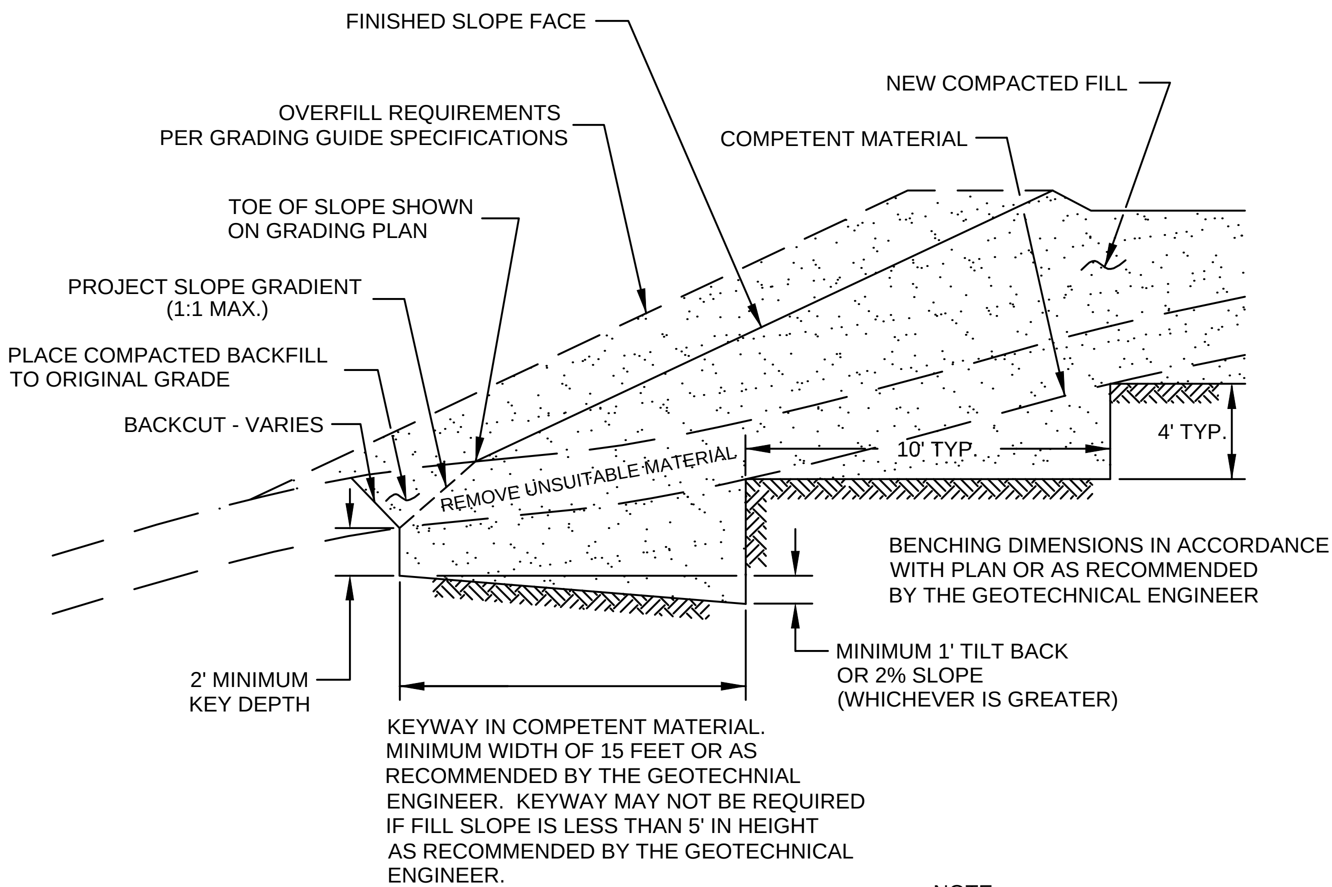
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-3

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL



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

FILL ABOVE NATURAL SLOPE DETAIL

GRADING GUIDE SPECIFICATIONS

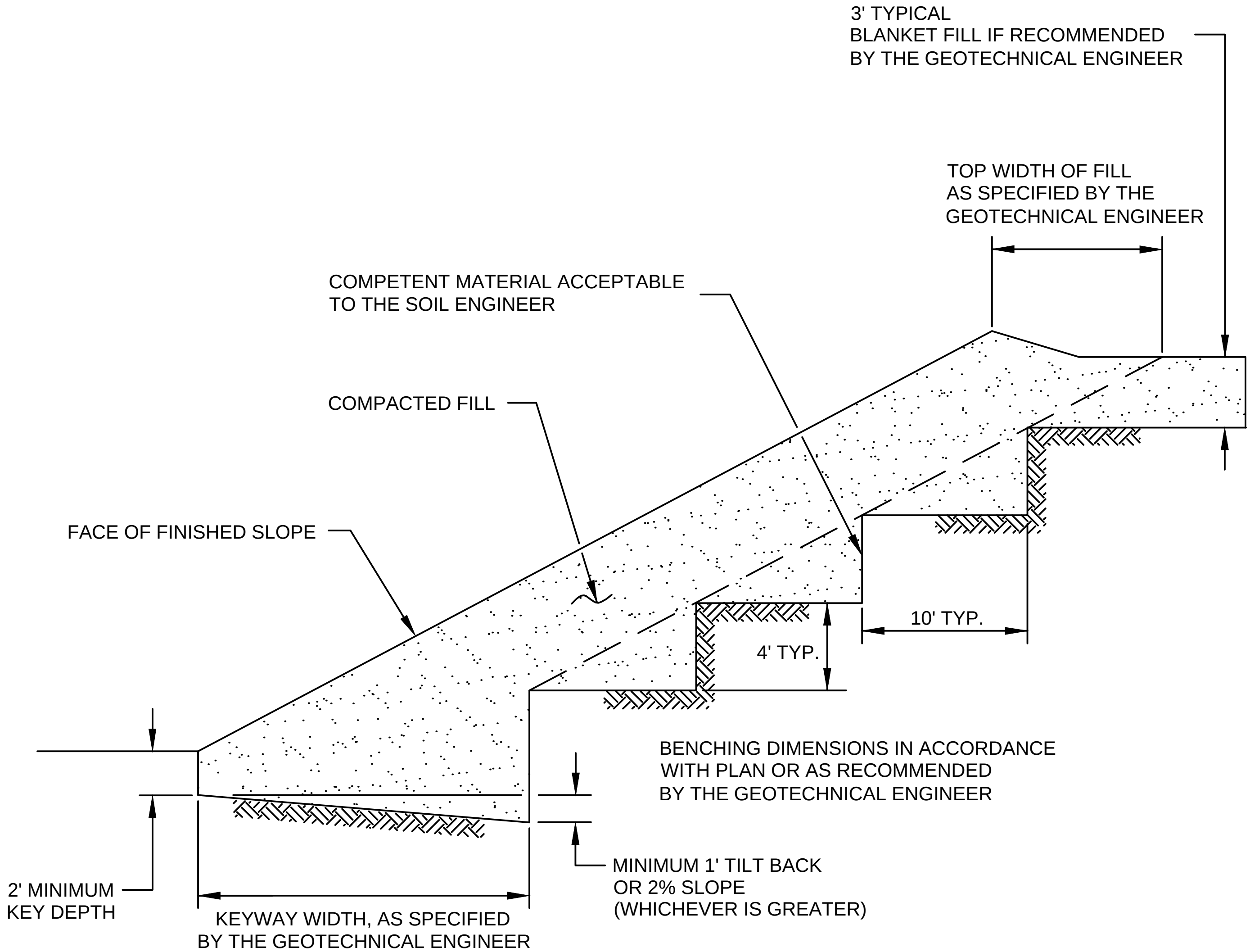
NOT TO SCALE

DRAWN: JAS
CHKD: GKM

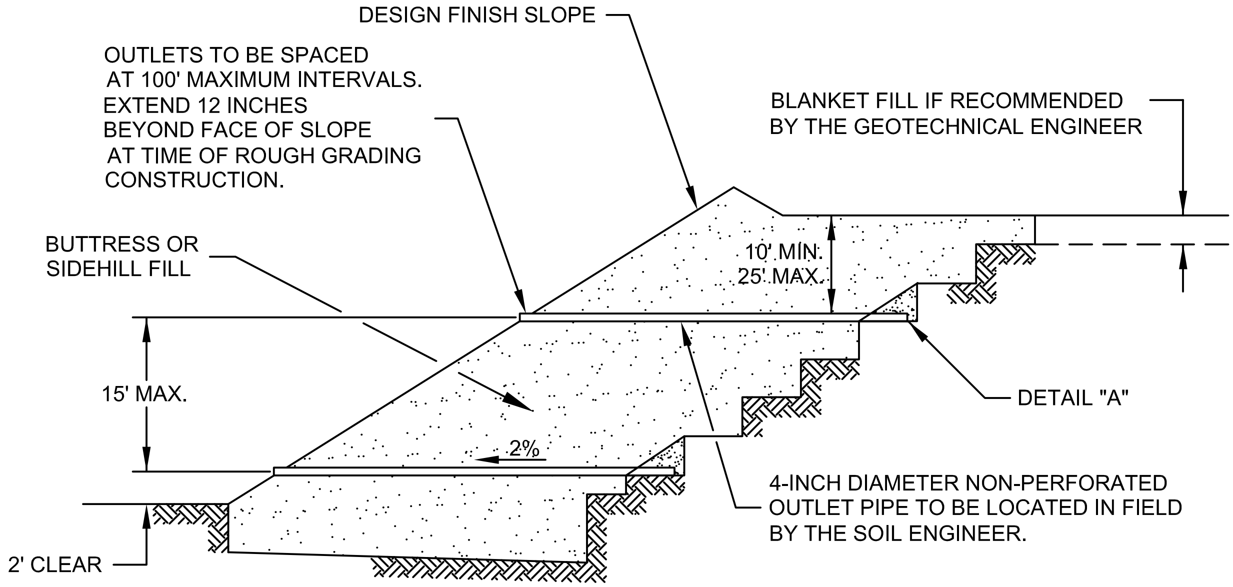
PLATE D-4

SoCalGeo

SOUTHERN
CALIFORNIA
GEOTECHNICAL



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



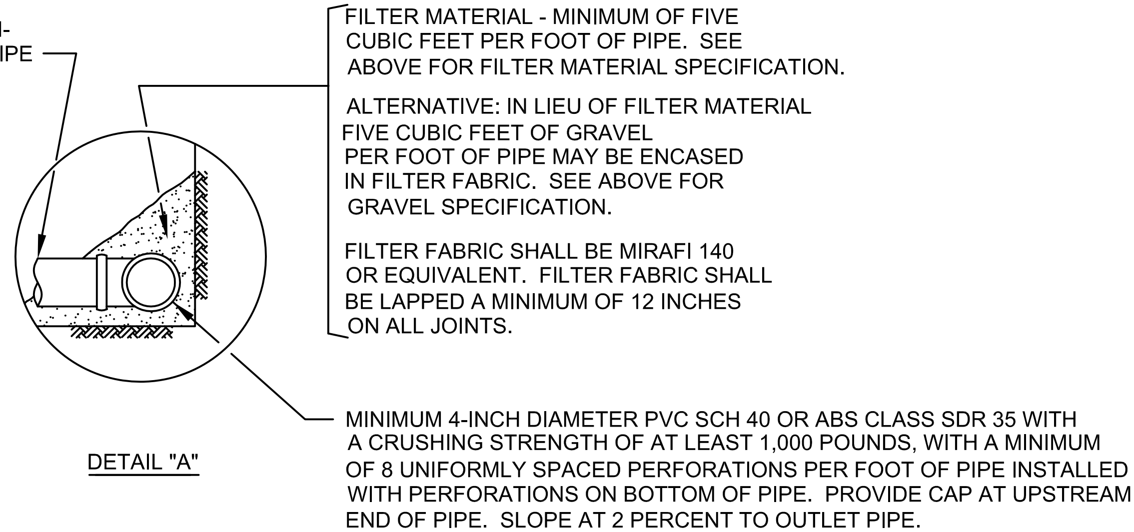
"FILTER MATERIAL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT: (CONFORMS TO EMA STD. PLAN 323)

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

"GRAVEL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT:

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

SAND EQUIVALENT = MINIMUM OF 50



DETAIL "A"

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

SLOPE FILL SUBDRAINS

GRADING GUIDE SPECIFICATIONS

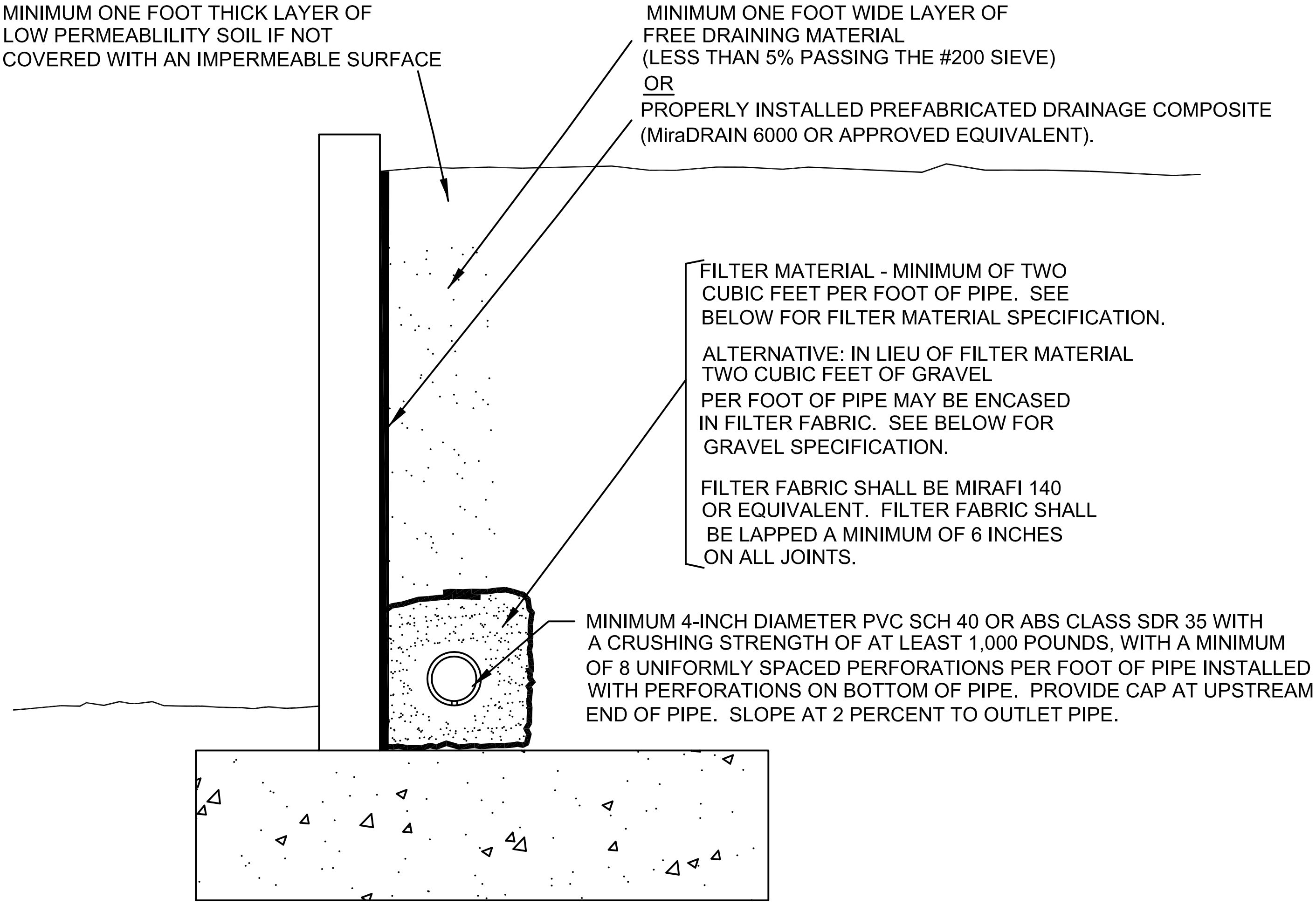
NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-6

SoCalGeo

SOUTHERN CALIFORNIA GEOTECHNICAL



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

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

"GRAVEL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT:

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

SAND EQUIVALENT = MINIMUM OF 50

RETAINING WALL BACKDRAINS

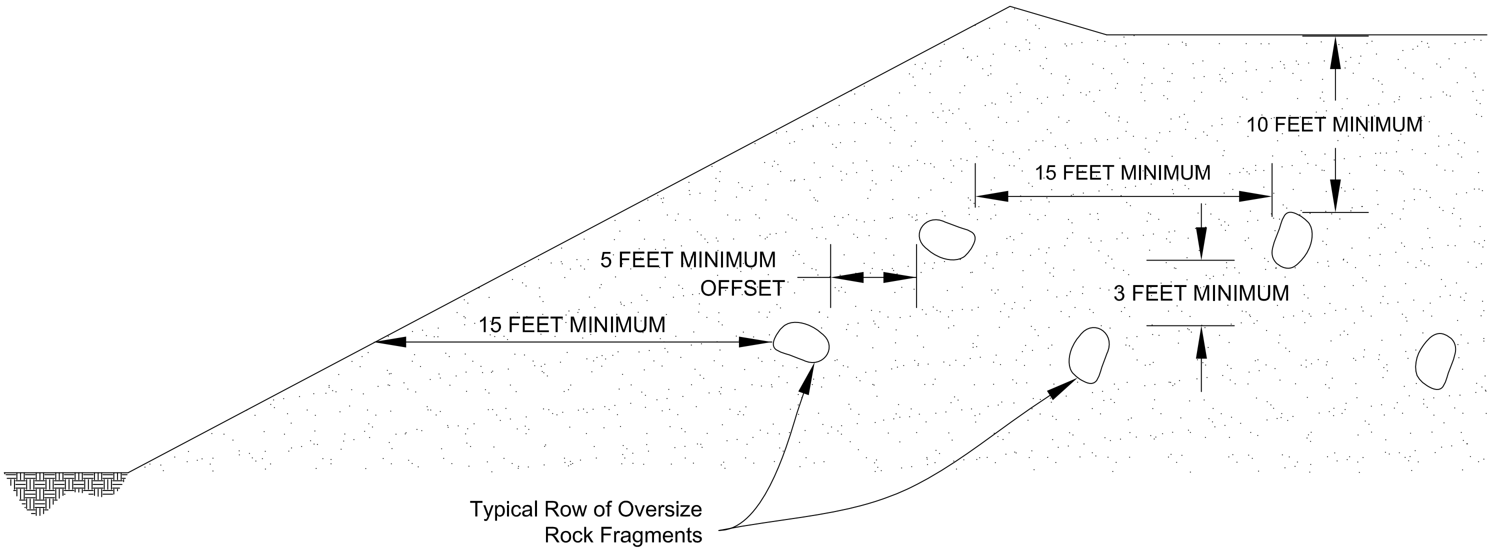
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

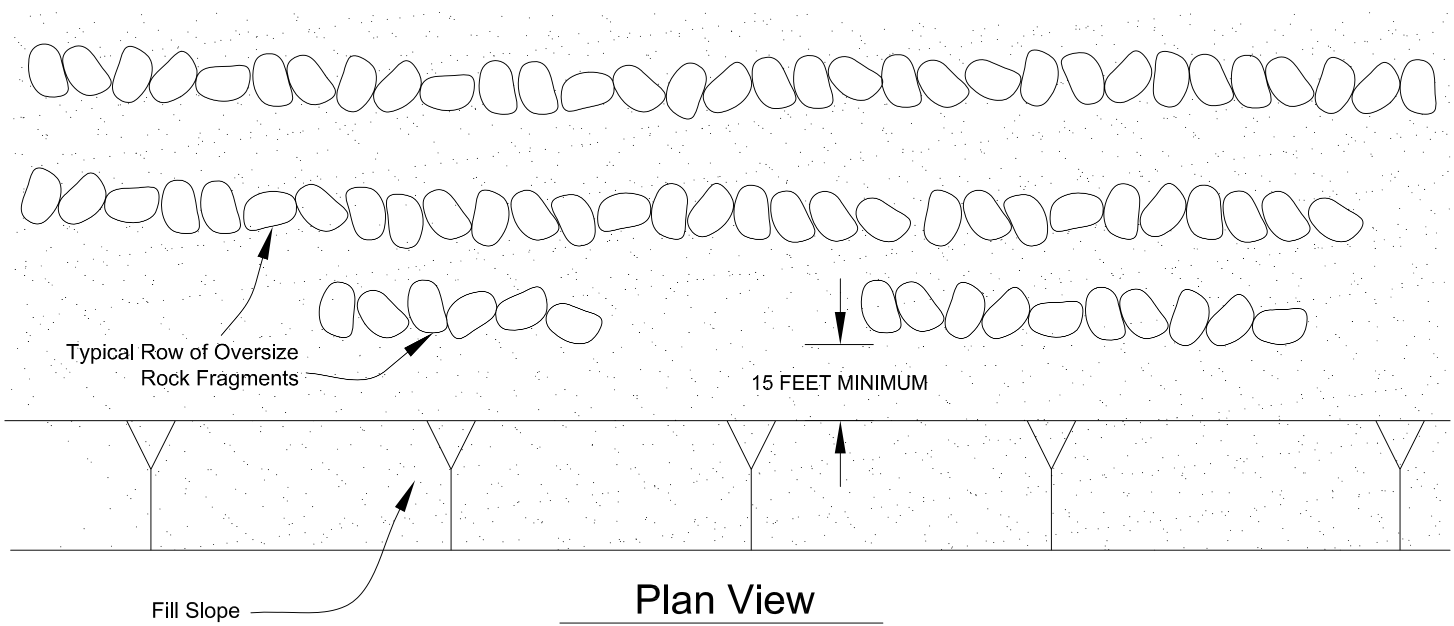
DRAWN: JAS
CHKD: GKM

PLATE D-7

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL



Section View



Plan View

PLACEMENT OF OVERSIZED MATERIAL

GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: PM
CHKD: GKM

PLATE D-8

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL

APPENDIX E

OSHPD

Latitude, Longitude: 33.939430, -117.959497

Date	6/23/2020, 11:45:01 AM
Design Code Reference Document	ASCE7-16
Risk Category	III
Site Class	D - Stiff Soil

Type	Value	Description
S_S	1.806	MCE_R ground motion. (for 0.2 second period)
S_1	0.641	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.806	Site-modified spectral acceleration value
S_{M1}	null -See Section 11.4.8	Site-modified spectral acceleration value
S_{DS}	1.204	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	0.785	MCE_G peak ground acceleration
F_{PGA}	1.1	Site amplification factor at PGA
PGA_M	0.864	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
SsRT	1.806	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	2.01	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	2.348	Factored deterministic acceleration value. (0.2 second)
S1RT	0.641	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.711	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.881	Factored deterministic acceleration value. (1.0 second)
PGAd	0.944	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.898	Mapped value of the risk coefficient at short periods
C_{R1}	0.901	Mapped value of the risk coefficient at a period of 1 s

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

SEISMIC DESIGN PARAMETERS - 2019 CBC

PROPOSED MEDICAL OFFICE BUILDING

LA HABRA, CALIFORNIA

DRAWN: JAH
CHKD: GKM

SCG PROJECT
20G161-1

PLATE E-1

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL

Attachment I

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

July 9, 2020

Northgate Markets
1201 Magnolia Avenue
Anaheim, CA 92801

Attention: Michelle Gutierrez

Project No.: **20G161-2**

Subject: **Results of Infiltration Testing**
Proposed Medical Office Building
1201 West Whittier Boulevard
La Habra, California

Reference: Geotechnical Investigation, Proposed Medical Office Building, 1201 West Whittier Boulevard, La Habra, California, prepared by Southern California Geotechnical, Inc. (SCG) for Northgate Markets, SCG Project No. 20G161-1, dated July 9, 2020.

Ms. Gutierrez:

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. 19P254, dated June 11, 2020. The scope of services included visual site reconnaissance, subsurface exploration, field and laboratory testing, and engineering analysis to determine the infiltration rates of the onsite soils. The infiltration testing was performed in general accordance with the guidelines published by Orange County: Technical Guidance Document for the Preparation of Conceptual/Preliminary and/or Project Water Quality Management Plans (WQMPs), Appendix VII. These guidelines are dated December 20, 2013.

Site and Project Description

The subject is located at the northwest corner of West Whittier Boulevard and North Idaho Street in La Habra, California. The site is bounded to the north by asphaltic concrete parking areas for an existing retail development, to the west by a Jack in the Box restaurant, to the south by West Whittier Boulevard, and to the east by North Idaho Street. The street address of the site is 1201 West Whittier Boulevard. The general location of the site is illustrated on the Site Location Map, included as Plate 1 of this report.

The site is slightly less than 1 acre in size, and is presently developed with a retail building, 10,305± square feet in size. The building was formerly used as a Petco store, but is now unoccupied. The building is surrounded by asphaltic concrete pavements and several landscape planters that contain medium bushes and medium to large trees.

Topographic information for the subject site was obtained from a topographic survey prepared by Cal Vada Surveying, Inc. Based on this survey, the parking lot north of the existing building slopes downward to the south at a gradient of 2½± percent. The parking and drive area west of the building slopes downward to the south at a gradient of 4± percent. The existing building possesses a flat floor slab, with a finished floor elevation of 355.68 feet mean sea level (msl). The maximum site elevation is 361.5 feet msl in the northeast corner of the lot. The minimum site elevation is 350.1 feet msl in the southwest corner of the lot.

Proposed Development

Our office was provided with a conceptual site plan, prepared by Ware Malcomb, for the proposed development. Based on this plan, the site will be developed with one (1) new medical office building (MOB), located in the southeastern region of the site. The MOB will be 2 stories in height, and will have a footprint of 10,000± square feet. The building is expected to be surrounded by asphaltic concrete pavements in the parking and drive areas with some concrete flatwork and landscape planter areas throughout the site.

The proposed development will use on-site storm water infiltration. Based on up information provided to our office, the infiltration area will be located in the southwestern region of the site. It is expected that a relatively shallow bioswale or below grade chamber system, extending to a maximum depth of 5 to 8± feet will be utilized. We have provided an option to perform infiltration testing within the area of the proposed infiltration system.

Concurrent Study

Southern California Geotechnical, Inc. (SCG) concurrently conducted a geotechnical investigation at the subject site. As part of this study, four (4) borings were advanced to depths of 10 to 25± feet below existing site grades. Asphaltic concrete (AC) pavements were encountered the ground surface at Boring Nos. B-1 and B-2. These pavements consist of 3 to 6± inches of asphaltic concrete underlain by 2 to 6± inches of aggregate base. Boring Nos. B-3 and B-4 were drilled through the Portland cement concrete (PCC) slab. The slab is 4½ to 6± inches thick at these locations. Artificial fill soils were encountered beneath the pavements at three of the boring locations, extending to depths of 5½ to 12± feet below the existing site grades. The artificial fill soils generally consist of silty clays and fine sandy clays with trace to little silt, and clay with trace fine sand and silt. Native alluvium was encountered beneath the artificial fill or pavements at all of the boring locations, extending to at least the maximum depth explored of 25± feet. The near surface native alluvial soils within the upper 5½ to 12± feet generally consist of stiff to very stiff silty clays and fine sandy clays. These soils possess some calcareous veining and nodules. Deeper alluvial soils, extending to at least 25± feet, generally consist of medium dense fine sandy silts and fine to medium sands.

Subsurface Exploration

Scope of Exploration

The subsurface exploration for the infiltration testing consisted of two (2) infiltration borings, advanced to depths of 8± feet below existing site grades. The borings were logged during drilling by a member of our staff. The approximate locations of the infiltration borings (identified as I-1 and I-2) are indicated on the Infiltration Test Location Plan, enclosed as Plate 2 of this report.

Geotechnical Conditions

Pavements

Asphaltic concrete (AC) pavements were encountered at both boring locations. These pavements consisted of 2 to 4½± inches of AC underlain by 3 to 4± inches of aggregate base.

Alluvium

Native alluvial soils were encountered beneath the pavements at both of the boring locations. The alluvium consists of stiff silty clays and fine sandy clays, extending to the maximum explored depth of 8± feet below existing site grades.

Groundwater

Groundwater was not encountered at any of the borings for this study nor for the concurrent study. Based on the lack of any water within the borings, and the moisture contents of the recovered soil samples, the static groundwater table is considered to have existed at a depth in excess of 25± feet below the existing site grades at the time of the subsurface investigation

As part of our research, we reviewed historic high groundwater levels reported in the CA DMG Open-File Report for the La Habra Quadrangle. Plate 1.2 is a map which displays the historically highest ground water levels using contour lines. Based on the mapped contour lines in the vicinity of the project site, the historic high groundwater level at the subject site is considered to have existed at a depth of 25± feet below existing site grades. We also reviewed readily available groundwater data published on the California State Water Resources Control Board, GeoTracker, website, http://geotracker.waterboards.ca.gov/. The nearest monitoring well with available data in this database is located approximately 3900 feet east of the site. Water level readings within this monitoring well indicate a groundwater level of 48± feet below the ground surface in March 2020.

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 the Orange County guidelines: Technical Guidance Document for the Preparation of Conceptual/Preliminary and/or Project Water Quality Management Plans (WQMPs), Appendix VII.

Pre-soaking

In accordance with the county infiltration standards, all of the infiltration test borings were pre-soaked prior to the infiltration testing. The pre-soaking process consisted of filling the test borings by inverting a full 5-gallon bottle of clear water supported over each hole so that the water level reaches a level of at least 5 times the hole's radius above the gravel at the bottom of each hole. The pre-soaking was completed after all of the water had percolated through each test hole or after 15 hours since initiating the pre-soak. Based on the results of the pre-soaking process, different infiltration procedures were used during the infiltration testing at the infiltration boring locations.

Infiltration Testing

Following the pre-soaking process of the infiltration test borings, SCG performed the infiltration testing. Each test hole was filled with water to a depth of at least 5 times the hole's radius above the gravel at the bottom of each test hole, and less than or equal to the water level used during the pre-soaking process. In accordance with the Orange County guidelines, since "non-sandy soils" were encountered at the bottom of both Infiltration Borings, readings were taken at 30-minute intervals for a total of 6 hours. After each reading, the borings were refilled to the correct water level above the gravel at the bottom of each test hole. The water level readings are presented on the spreadsheets enclosed with this report. The infiltration rates for each of the timed intervals are also tabulated on the spreadsheets.

The infiltration rates from the test are tabulated in inches per hour. In accordance with typically accepted practice, it is recommended that the most conservative reading from the latter part of the infiltration tests be used as the design infiltration rate. The rates are summarized below:

Infiltration Test No.	**Depth (feet)**	**Soil Description**	**Infiltration Rate (inches/hour)**
I-1	8	Dark Brown to Brown fine Sandy Clay, little medium Sand	0.5
I-2	8	Brown Silty Clay, trace fine Sand	0.3

Moisture Content

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

Grain Size Analysis

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

Design Recommendations

Two (2) infiltration tests were performed at the subject site. The infiltration rates at these locations range from 0.3 to 0.5 inches per hour. **Based on the results from Infiltration Test Nos. I-1 and I-2 we recommend that an infiltration rate of 0.3 inches per hour be used for the proposed below-grade chamber system near the southwestern side**

of the existing building. These rates do not incorporate a factor of safety. The design rate should incorporate an appropriate factor of safety.

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

Construction Considerations

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

Infiltration versus Permeability

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

Location of Infiltration Systems

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

General Comments

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

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

Closure

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

Respectfully Submitted,

SOUTHERN CALIFORNIA GEOTECHNICAL, INC.

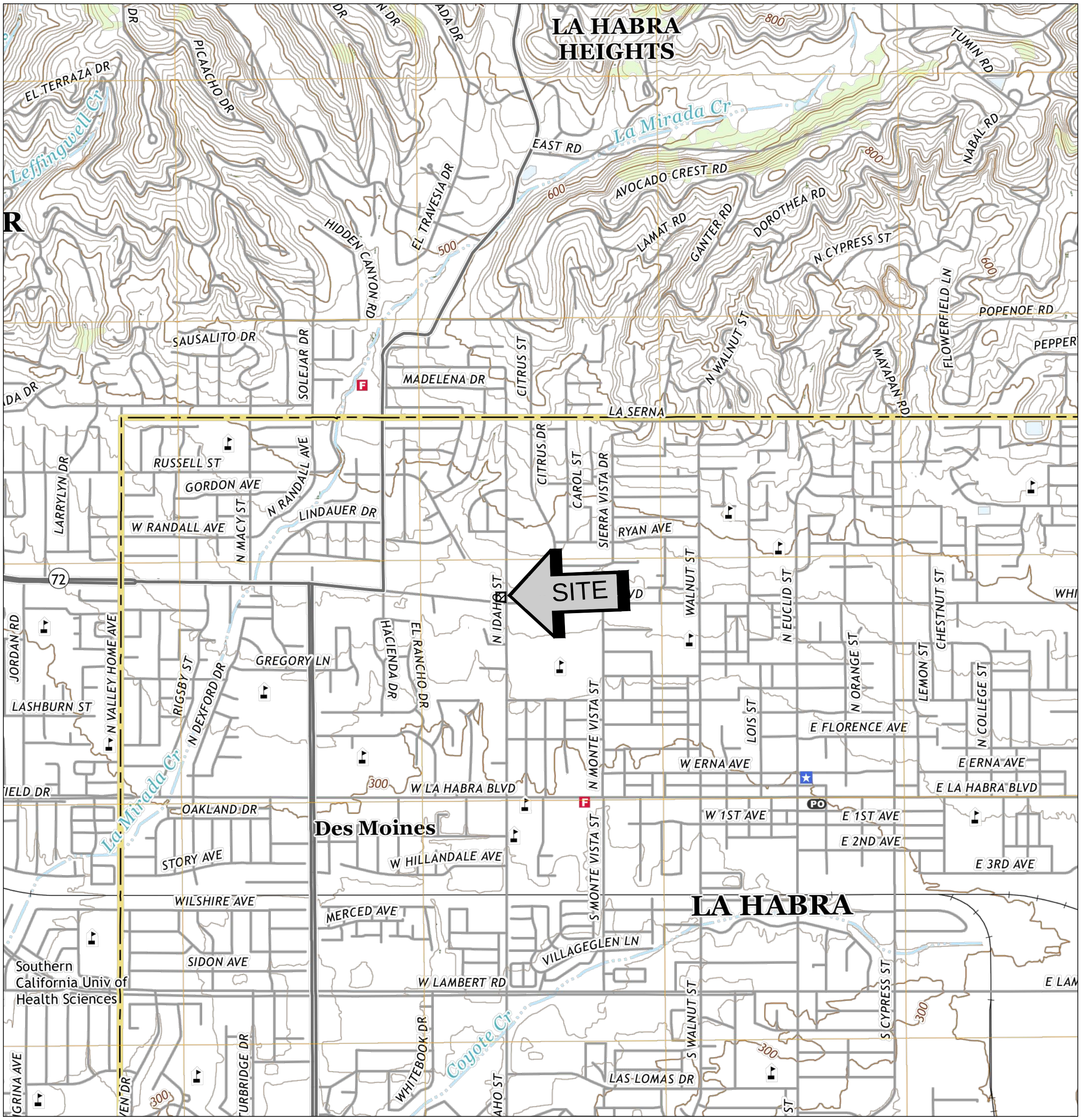
Ryan Bremer
Staff Geologist

REGISTERED PROFESSIONAL ENGINEER
GREGORY K. MITCHELL
No. 2364
Exp. 09/30/20
GEOTECHNICAL
STATE OF CALIFORNIA

Gregory K. Mitchell, GE 2364
Principal Engineer

Distribution: (1) Addressee

Enclosures: Plate 1 - Site Location Map
Plate 2 - Infiltration Test Location Plan
Boring Log Legend and Logs (4 pages)
Infiltration Test Results Spreadsheets (2 pages)
Grain Size Distribution Graphs (2 pages)



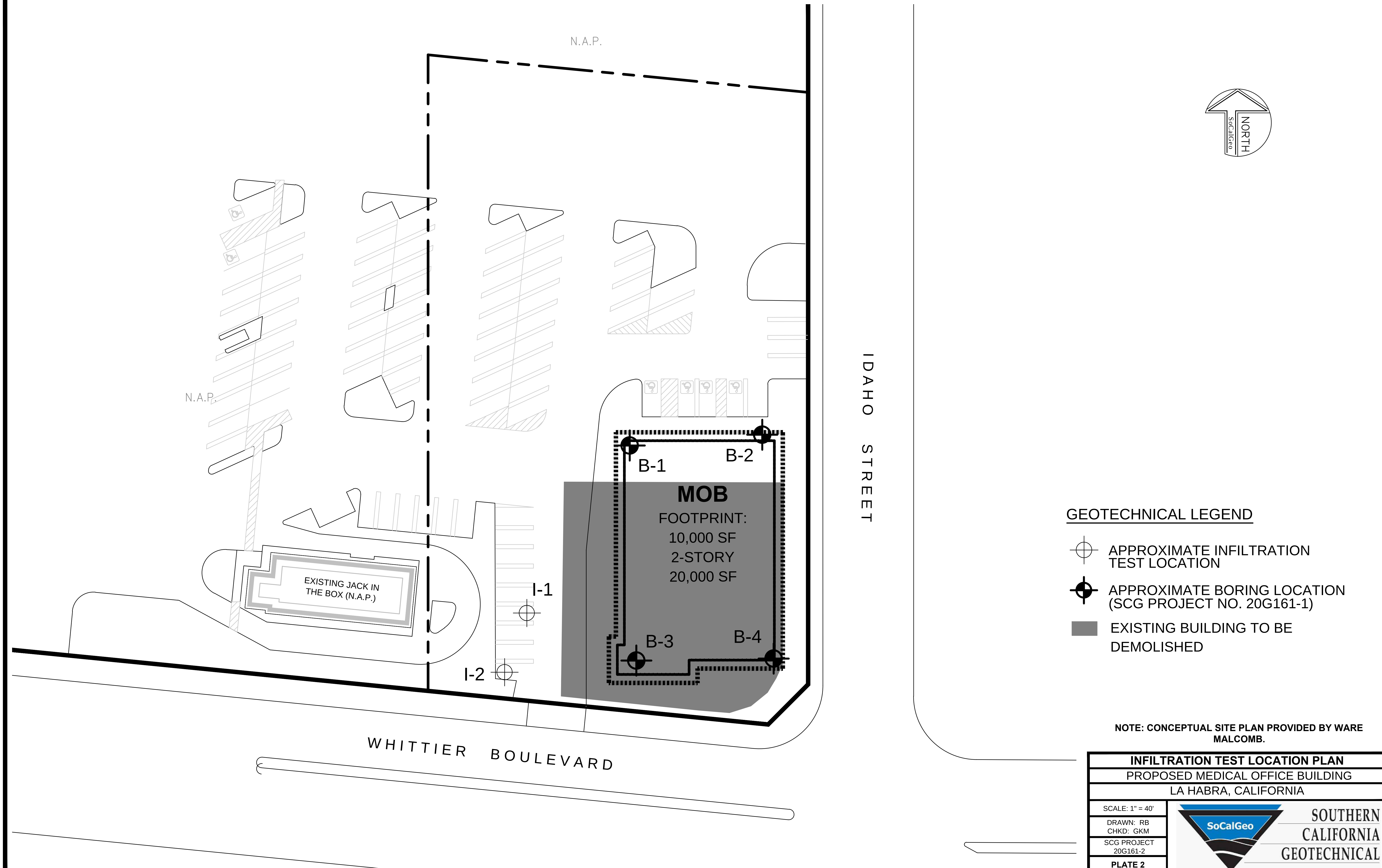
SOURCE: USGS TOPOGRAPHIC MAP OF THE LA HABRA QUADRANGLE, ORANGE COUNTY, CALIFORNIA, 2018

SITE LOCATION MAP

PROPOSED MEDICAL OFFICE BUILDING

LA HABRA, CALIFORNIA

SCALE: 1" = 2000'	SOUTHERN CALIFORNIA GEOTECHNICAL
DRAWN: JAH CHKD: GKM	
SCG PROJECT 20G161-2	
PLATE 1	



GEOTECHNICAL LEGEND

APPROXIMATE INFILTRATION TEST LOCATION

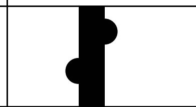
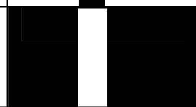
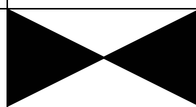
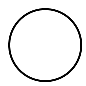
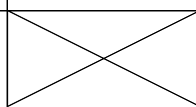
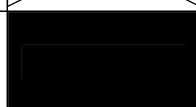
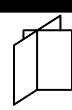
APPROXIMATE BORING LOCATION (SCG PROJECT NO. 20G161-1)

EXISTING BUILDING TO BE DEMOLISHED

NOTE: CONCEPTUAL SITE PLAN PROVIDED BY WARE MALCOMB.

INFILTRATION TEST LOCATION PLAN	
PROPOSED MEDICAL OFFICE BUILDING	
LA HABRA, CALIFORNIA	
SCALE: 1" = 40'	SOUTHERN CALIFORNIA GEOTECHNICAL
DRAWN: RB CHKD: GKM	
SCG PROJECT 20G161-2	
PLATE 2	

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. I-1

JOB NO.: 20G161-2
PROJECT: Proposed Medical Office Building
LOCATION: La Habra, California

DRILLING DATE: 6/22/20
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jamie Hayward

WATER DEPTH: Dry
CAVE DEPTH: N/A
READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: 351.3' MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					2± inches Asphaltic Concrete, 4± inches Aggregate Base							
					ALLUVIUM Dark Brown to Brown fine Sandy Clay, little medium Sand, stiff-very moist							
	X	17	2.0				19					
	X	15	2.5				18					
5												
	X	11	2.5				17			71		
					Boring Terminated at 8 Feet							

TBL 20G161-2.GPJ SOCALGEO.GDT 7/9/20

TEST BORING LOG

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. I-2

JOB NO.: 20G161-2
PROJECT: Proposed Medical Office Building
LOCATION: La Habra, California

DRILLING DATE: 6/22/20
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jamie Hayward

WATER DEPTH: Dry
CAVE DEPTH: N/A
READING TAKEN: At Completion

SURFACE ELEVATION: 350.8' 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
					4½± inches Asphaltic Concrete, 3± inches Aggregate Base							
	X	14	2.0		ALLUVIUM Dark Brown to Red Brown fine Sandy Clay, trace medium Sand, stiff-very moist		19					
	X	15	3.0				22					
5												
	X	8	3.0		Brown Silty Clay, trace fine Sand, stiff-very moist		27			95		

Boring Terminated at 8 Feet

TBL 20G161-2.GPJ SOCALGEO.GDT 7/9/20

TEST BORING LOG

PLATE B-2

INFILTRATION CALCULATIONS

Project Name	Proposed Medical Office Building
Project Location	La Habra, California
Project Number	20G161-2
Engineer	Ryan Bremer

Test Hole Diameter	8.00	(in)
Test Depth	8.00	(ft)

Infiltration Test Hole	I-1

Interval Number		Time	Time Interval (min)	Water Depth (ft)	Change in Water Level (ft)	Preadjusted Percolation Rate Q (min/in)	Preadjusted Percolation Rate Q (in/hr)	Reduction Factor	Infiltration Rate Q (in/hr)	
PS1	Initial	8:00 AM	25.0	5.90	0.28	7.4	8.1	6.9	1.2	PRE-SOAK
	Final	8:25 AM		6.18						
PS2	Initial	8:25 AM	25.0	6.18	0.14	14.9	4.0	6.3	0.6	
	Final	8:50 AM		6.32						
1	Initial	8:50 AM	30.0	6.32	0.18	13.9	4.3	5.8	0.7	INFILTRATION TESTING
	Final	9:20 AM		6.50						
2	Initial	9:20 AM	30.0	6.15	0.17	14.7	4.1	6.3	0.6	
	Final	9:50 AM		6.32						
3	Initial	9:50 AM	30.0	6.32	0.17	14.7	4.1	5.8	0.7	
	Final	10:20 AM		6.49						
4	Initial	10:20 AM	30.0	6.00	0.27	9.3	6.5	6.6	1.0	
	Final	10:50 AM		6.27						
5	Initial	10:50 AM	30.0	6.15	0.17	14.7	4.1	6.3	0.6	
	Final	11:20 AM		6.32						
6	Initial	11:20 AM	30.0	6.32	0.17	14.7	4.1	5.8	0.7	
	Final	11:50 AM		6.49						
7	Initial	11:50 AM	30.0	6.11	0.18	13.9	4.3	6.4	0.7	
	Final	12:20 PM		6.29						
8	Initial	12:20 PM	30.0	6.29	0.17	14.7	4.1	5.9	0.7	
	Final	12:50 PM		6.46						
9	Initial	12:50 PM	30.0	6.00	0.14	17.9	3.4	6.8	0.5	
	Final	1:20 PM		6.14						
10	Initial	1:20 PM	30.0	6.14	0.14	17.9	3.4	6.4	0.5	
	Final	1:50 PM		6.28						
11	Initial	1:50 PM	30.0	6.28	0.12	20.8	2.9	6.0	0.5	
	Final	2:20 PM		6.40						
12	Initial	2:20 PM	30.0	6.12	0.13	19.2	3.1	6.4	0.5	
	Final	2:50 PM		6.25						

Infiltration Rate = (Preadjusted Percolation Rate)/(Reduction Factor)
Reduction Factor (Rf) = ((2d-Δd)/(DIA))+1

Where:
d = Initial Water Depth
Δd = Average/Final Water Level Drop
DIA = Diameter of the boring (in.)

INFILTRATION CALCULATIONS

Project Name	Proposed Medical Office Building
Project Location	La Habra, California
Project Number	20G161-2
Engineer	Ryan Bremer

Test Hole Diameter	8.00	(in)
Test Depth	8.00	(ft)

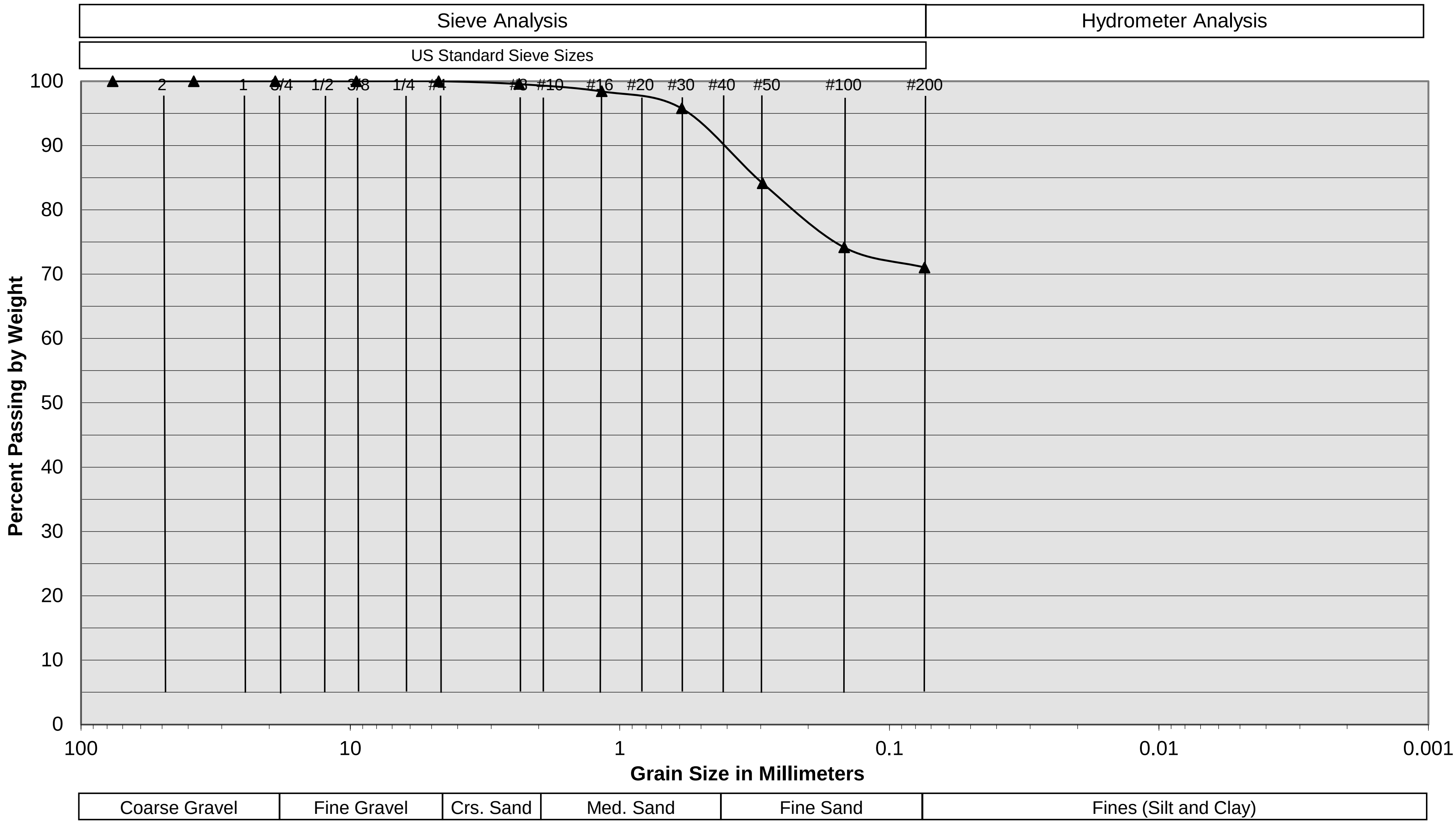
Infiltration Test Hole	I-2

Interval Number		Time	Time Interval (min)	Water Depth (ft)	Change in Water Level (ft)	Preadjusted Percolation Rate Q (min/in)	Preadjusted Percolation Rate Q (in/hr)	Reduction Factor	Infiltration Rate Q (in/hr)	
PS1	Initial	8:05 AM	25.0	5.73	0.19	11.0	5.5	7.5	0.7	PRE-SOAK
	Final	8:30 AM		5.92						
PS2	Initial	8:30 AM	25.0	5.92	0.13	16.0	3.7	7.0	0.5	
	Final	8:55 AM		6.05						
1	Initial	8:55 AM	30.0	6.05	0.14	17.9	3.4	6.6	0.5	INFILTRATION TESTING
	Final	9:25 AM		6.19						
2	Initial	9:25 AM	30.0	6.19	0.10	25.0	2.4	6.3	0.4	
	Final	9:55 AM		6.29						
3	Initial	9:55 AM	30.0	6.26	0.10	25.0	2.4	6.1	0.4	
	Final	10:25 AM		6.36						
4	Initial	10:25 AM	30.0	5.90	0.13	19.2	3.1	7.1	0.4	
	Final	10:55 AM		6.03						
5	Initial	10:55 AM	30.0	6.03	0.17	14.7	4.1	6.7	0.6	
	Final	11:25 AM		6.20						
6	Initial	11:25 AM	30.0	6.20	0.09	27.8	2.2	6.3	0.3	
	Final	11:55 AM		6.29						
7	Initial	11:55 AM	30.0	6.29	0.07	35.7	1.7	6.0	0.3	
	Final	12:25 PM		6.36						
8	Initial	12:25 PM	30.0	6.00	0.10	25.0	2.4	6.9	0.4	
	Final	12:55 PM		6.10						
9	Initial	12:55 PM	30.0	6.10	0.12	20.8	2.9	6.5	0.4	
	Final	1:25 PM		6.22						
10	Initial	1:25 PM	30.0	6.22	0.10	25.0	2.4	6.2	0.4	
	Final	1:55 PM		6.32						
11	Initial	1:55 PM	30.0	6.00	0.09	27.8	2.2	6.9	0.3	
	Final	2:25 PM		6.09						
12	Initial	2:25 PM	30.0	6.09	0.09	27.8	2.2	6.6	0.3	
	Final	2:55 PM		6.18						

Infiltration Rate = (Preadjusted Percolation Rate)/(Reduction Factor)
Reduction Factor (Rf) = ((2d-Δd)/(DIA))+1

Where:
d = Initial Water Depth
Δd = Average/Final Water Level Drop
DIA = Diameter of the boring (in.)

Grain Size Distribution



Sample Description	I-1 @ 6½ to 8 feet
Soil Classification	Dark Brown to Brown fine Sandy Clay, little medium Sand

Proposed Medical Office Building
La Habra, California
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PLATE C-1

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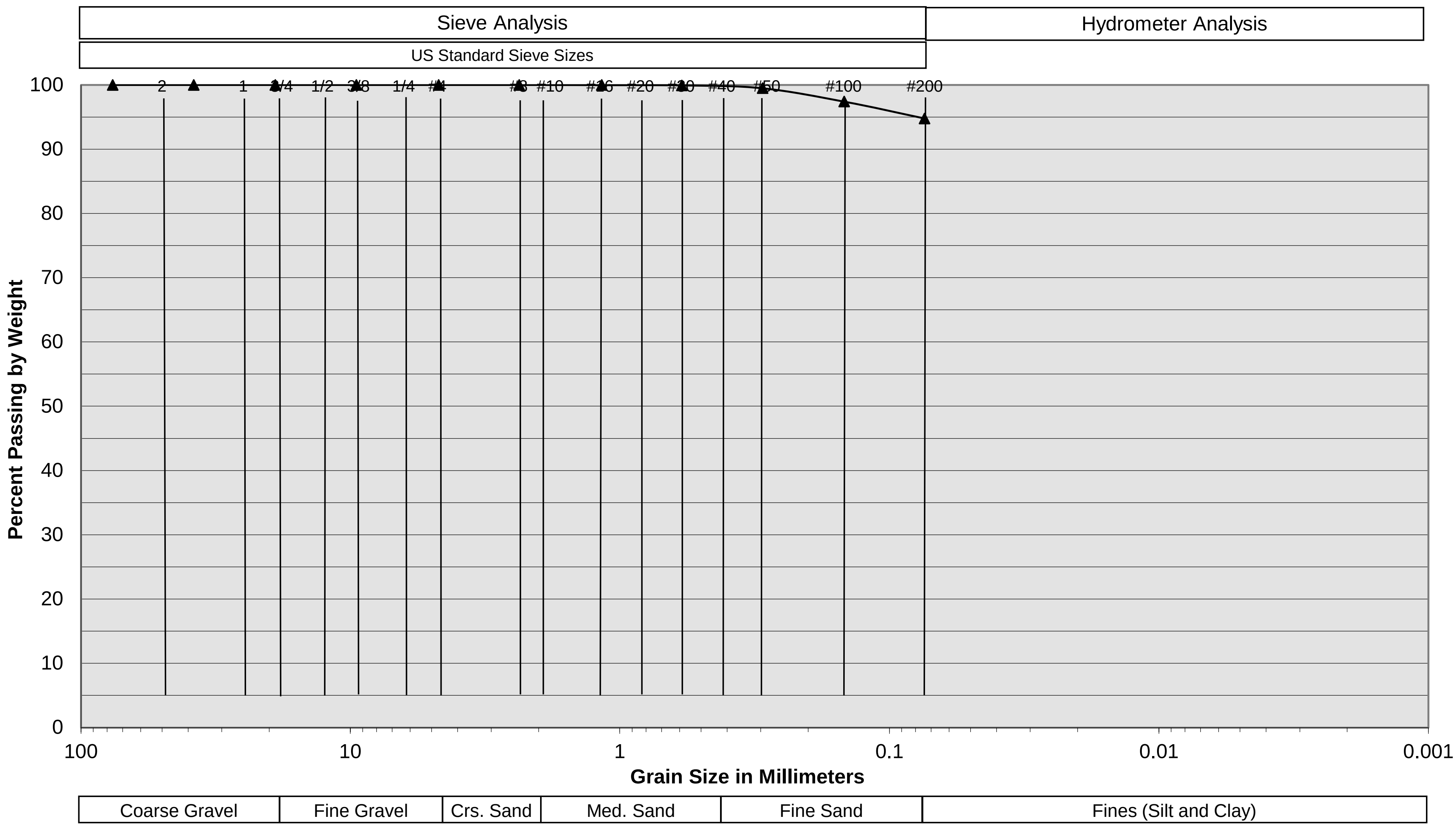
Grain Size Distribution



Sample Description	I-2 @ 6½ to 8 feet
Soil Classification	Brown Silty Clay, little to some Silt, trace fine Sand

Proposed Medical Office Building
La Habra, California
Project No. 20G161-2
PLATE C-2

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Attachment J