APPENDIX C

Water Quality Management Plan

WATER QUALITY MANAGEMENT PLAN (WQMP)

Huntington Beach Vans Skate Park

REC JN 16469
October 13, 2011
Revised October 26, 2011

RICK ENGINEERING COMPANY

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& Building

Water Quality Management Plan (WQMP)

Project Name:

Huntington Beach Vans Skate Park

Prepared for:

Site Design Group, Inc.

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REGISTERED PROFESSIONAL ENGINEER
MARTEN L. ANDERSON
NO. 51313
Exp. 06-30-12
CIVIL
STATE OF CALIFORNIA

Project Owner's Certification			
Permit/Application No.		Grading Permit No.	
Tract/Parcel Map No.		Building Permit No.	
CUP, SUP, and/or APN (Specify Lot Numbers if Portions of Tract)			APN 142-073-03

This Water Quality Management Plan (WQMP) has been prepared for Site Design Group, Inc. by Rick Engineering Company. 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:			
Title			
Company			
Address			
Email			
Telephone #			
Signature		Date	

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Appendix C ..... **Notice of Transfer of Responsibility**

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.		Tract/Parcel Map No.	APN 142-073-03
Additional Information/ Comments:	The project is bound by McFadden Avenue to the north, railroad to the east, Center Avenue to the south, and Gothard Street to the west, within the City of Huntington Beach.		
Water Quality Conditions			
Water Quality Conditions (list verbatim)	New development projects that create 10,000 square feet or more of impervious surface Parking Lots 5,000 square feet or more		
Watershed-Based Plan Conditions			
Provide applicable conditions from watershed - based plans including WIHMPs and TMDLS.	According to the Technical Guidance Document, Watershed Infiltration and Hydromodification Management Plan (WIHMP) will be prepared for the Anaheim Bay-Huntington Harbour watershed by May 2012.		

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):	New development projects that create 10,000 square feet or more of impervious surface Parking Lots 5,000 square feet or more			
Project Area (ft²): 118,380	Number of Dwelling Units: none		SIC Code: Industry Code 7999: Miscellaneous Amusement and Recreation	
Narrative Project Description:	The project site is located on APN 142-073-03. The Zoning Map for the City of Huntington Beach shows that the site is designated as SP-14-Specific Plan Zone. The site measures 2.72 acres, is vacant, and consists of dirt and minimal vegetation. The post-project site will consist of a skate park, permeable parking lots, retail building, and landscaped areas. The post-project site will be graded to drain into the permeable parking lots, which will filter the stormwater to remove pollutants and store it underground until it is discharged into the street after the peak of the storm.			
Project Area	Pervious		Impervious	
	Area (acres or sq ft)	Percentage	Area (acres or sq ft)	Percentage
Pre-Project Conditions	2.72 acres	100%	0 acres	0%
Post-Project Conditions	1.62 acres	60%	1.10 acres	40%
Drainage Patterns/Connections	The site is located in the Anaheim Bay-Huntington Harbour Watershed and discharges to the East Garden Grove Wintersburg Channel, which drains to Huntington Harbour, Anaheim Bay, and Bolsa Chica State Beach. Anaheim Bay is listed on the 2006 CWA Section 303(d) list for Pesticides, Metals, Organic Compounds, and Sediment (Dieldrin, Nickel, Polychlorinated Biphenyls (PCBs), and Sediment Toxicity). Huntington Harbour is listed on the 2006 CWA Section 303(d) list for Pesticides, Metals, Pathogens, Organic Compounds, and Sediment (Chlordane, Copper, Lead, Nickel, Pathogens, PCBs, and Sediment Toxicity). Bolsa Chica State Beach is listed on the 2006 CWA Section 303(d) list for Metals (Copper and Nickel).			

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.*

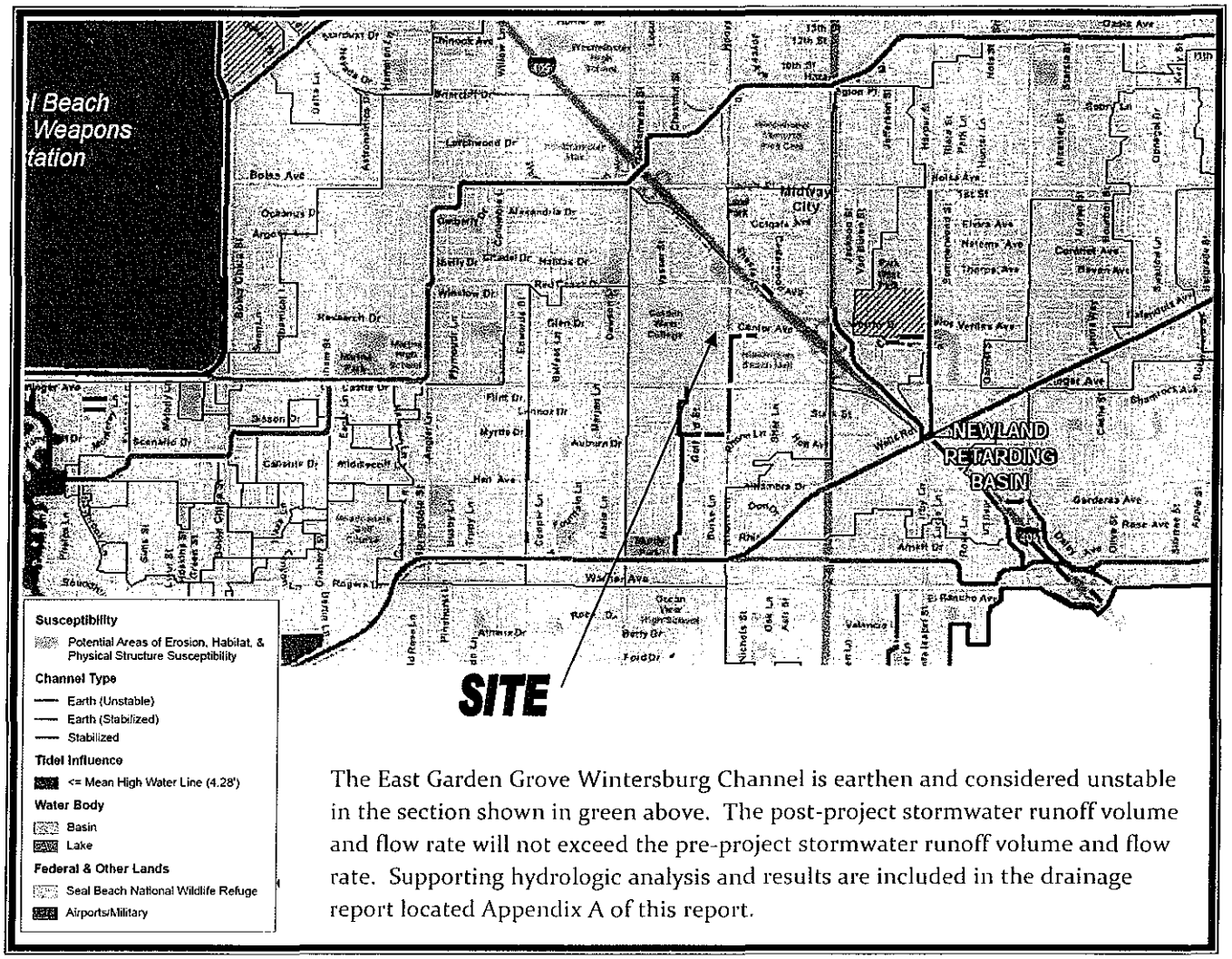
Pollutants of Concern			
Pollutant	Circle One: E=Expected to be of concern	N=Not Expected to be of concern	Additional Information and Comments
Suspended-Solid/ Sediment	E	N	Pollutant of concern because Anaheim Bay and Huntington Harbour is listed on the 2006 CWA Section 303(d) list for Sediment.
Nutrients	E	N	
Heavy Metals	E	N	Pollutant of concern because Anaheim Bay, Huntington Harbour, and Bolsa Chica State Beach is listed on the 2006 CWA Section 303(d) list for Metals.
Pathogens (Bacteria/Virus)	E	N	Pollutant of concern because Huntington Harbour is listed on the 2006 CWA Section 303(d) list for Pathogens.
Pesticides	E	N	Pollutant of concern because Anaheim Bay and Huntington Harbour is listed on the 2006 CWA Section 303(d) list for Pesticides.
Oil and Grease	E	N	
Toxic Organic Compounds	E	N	Pollutant of concern because Anaheim Bay and Huntington Harbour is listed on the 2006 CWA Section 303(d) list for Organic Compounds.
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.

☐ No – Show map

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

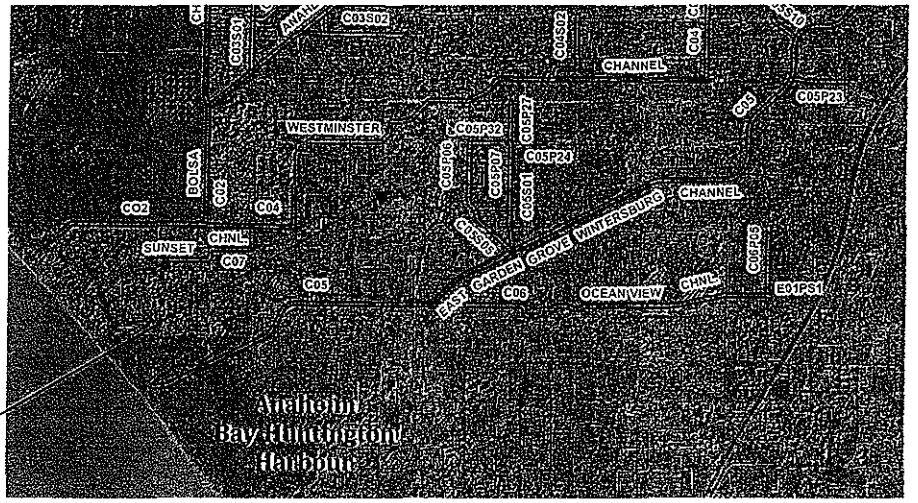


The East Garden Grove Wintersburg Channel is earthen and considered unstable in the section shown in green above. The post-project stormwater runoff volume and flow rate will not exceed the pre-project stormwater runoff volume and flow rate. Supporting hydrologic analysis and results are included in the drainage report located Appendix A of this report.

II.4 Post Development Drainage Characteristics

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

The site is located in the Anaheim Bay-Huntington Harbour Watershed and discharges to the East Garden Grove Wintersburg Channel, which drains to Huntington Harbour, Anaheim Bay, and Bolsa Chica State Beach. The post-project site will be graded with a high point near the center and will drain north and south into the permeable parking lots at the ends of the site. The HCOC is being addressed by the proposed parking lot sub-base storage areas that will detain the incremental increase and ensure that the post development runoff volume does not exceed that of the predevelopment and time of concentration is not less than that for the predevelopment condition. The parking lots have been sized to store the incremental increase of stormwater runoff volume produced by the development of the site for the 100-year design storm, which is more than the HCOC required 2-year design storm. The parking lot pavers and sub-base will filter the stormwater to remove pollutants and store it underground. A perforated pipe system will be constructed within the sub-base section to allow removal of any stormwater that does not infiltrate into the soil beneath. After the peak of the storm, the stored water will be pumped out of the perforated pipes and discharged into Center Avenue and McFadden Avenue thru curb outlets. The pump system shall be designed so that the discharge will not to exceed the pre-project flow rates at any time. As the stormwater flows down Center Avenue and McFadden Avenue it will be directed thru the gutter and down Gothard Avenue to the intersection of Gothard Avenue and Edinger Avenue. The water will enter the City of Huntington Beach storm drain system thru an existing catch basin that leads to a 24" storm drain line in Gothard Avenue which discharges to the East Garden Grove Wintersburg Channel.



SITE

II.5 Property Ownership/Management

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

There will not be a homeowners or property owners association formed for this project. The entire site will be owned and managed by the City of Huntington Beach. Site features include skate park, permeable parking lots, retail building, and landscaped areas.

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	M-sp-d
Location/Address	APN 142-073-03
	The project is bound by McFadden Avenue to the north, railroad to the east, Center Avenue to the south, and Gothard Street to the west, within the City of Huntington Beach.
Land Use	Proposed Skate Park
Zoning	SP-14-Specific Plan Zone
Acreage	2.72 acres
Predominant Soil Type	C

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	*0.7 inch Design Capture Storm Depth (per TCD Figure XVI-1)*
Topography	*The existing site is relatively flat and gently slopes in a southerly direction. The topography drops approximately four feet from the northerly boundary along McFadden Avenue to the southerly boundary along Center Avenue.*
Drainage Patterns/Connections	*The existing site drainage sheet flows in a southwesterly direction towards Center Avenue. The proposed project will create a high point in the center of the parcel and direct stormwater to the permeable parking lots located on the north and south ends of the site.*
Soil Type, Geology, and Infiltration Properties	*The project site's soil is Type C (per TCD Figure XVI-2a). Type C soils typically have low infiltration rates when thoroughly wetted.*

Site Characteristics (continued)	
Hydrogeologic (Groundwater) Conditions	*The geotechnical report prepared by Southern California Geotechnical noted that the depth to historic high groundwater is ±7 feet. The borings from the geotechnical report encountered groundwater between 12 and 14 feet.*
Geotechnical Conditions (relevant to infiltration)	*The geotechnical report noted that artificial fill and debris was encountered in most of the borings. Additionally, high groundwater may make the site substandard for infiltration.*
Off-Site Drainage	*There is no concern for off-site run-on to the project. The project is bound by McFadden Avenue to the north and Center Avenue to the south (which are both improved roadways with curb and gutter). The project is bound by a railroad to the east and a vacant parcel to the west which drains in a southwesterly direction.*
Utility and Infrastructure Information	*There is no record or indication that any utilities exist on-site.*

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	The site is located in the Anaheim Bay-Huntington Harbour Watershed and discharges to the East Garden Grove Wintersburg Channel, which drains to Huntington Harbour, Anaheim Bay, and Bolsa Chica State Beach.
303(d) Listed Impairments	Anaheim Bay is listed on the 2006 CWA Section 303(d) list for Pesticides, Metals, Organic Compounds, and Sediment (Dieldrin, Nickel, Polychlorinated Biphenyls (PCBs), and Sediment Toxicity). Huntington Harbour is listed on the 2006 CWA Section 303(d) list for Pesticides, Metals, Pathogens, Organic Compounds, and Sediment (Chlordane, Copper, Lead, Nickel, Pathogens, PCBs, and Sediment Toxicity). Bolsa Chica State Beach is listed on the 2006 CWA Section 303(d) list for Metals (Copper and Nickel).
Applicable TMDLs	n/a
Pollutants of Concern for the Project	The projects expected pollutants are suspended solids/sediments, nutrients, heavy metals, pathogens (bacteria/virus), pesticides, oil and grease, toxic organic compounds, and trash and debris. The projects expected pollutant of concern (expected site pollutant and listed on 303(d) list) is Metals.
Environmentally Sensitive and Special Biological Significant Areas	n/a

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.*

(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?	YES ☐	NO ☒
If yes, describe WIHMP feasibility criteria or regional/sub-regional LID opportunities.	n/a	

Project Performance Criteria (continued)	
If HCOC exists, list applicable hydromodification control performance criteria (Section 7.II-2.4.2.2 in MWQMP)	Post development runoff volume for the two-year storm does not exceed that of the predevelopment condition by more than five percent and time of concentration of the post-development storm event is not less than that for the predevelopment condition by more than five percent.
List applicable LID performance criteria (Section 7.II-2.4.3 from MWQMP)	Priority projects must infiltrate, harvest and use, evapotranspire, or biotreat/biofilter, the 85th percentile, 24-hour storm event (Design Capture Volume). A properly designed biotreatment system may only be considered if infiltration, harvest and use, and evapotranspiration (ET) cannot be feasibly implemented for the full design capture volume.
List applicable treatment control BMP performance criteria (Section 7.II-3.2.2 from MWQMP)	Satisfaction of LID performance criteria also fully satisfies treatment control performance criteria. If it is not feasible to meet LID performance criteria through retention and/or biotreatment provided on-site or at a sub-regional scale, then treatment control BMPs shall be provided on-site or off-site prior to discharge to waters of the US.
Calculate LID design storm capture volume for Project.	DCV = C(ft^3) x d(in) x A(ac) x 43,560(ft^2/ac) x 1/12 (in/ft) where C = runoff coefficient = 0.75 x *imp* + 0.15 = (0.75)(0.4) + 0.15 = 0.45 *imp* = impervious fraction of drainage area (ranges from 0 to 1) d = storm depth = 0.7 inch Design Capture Storm Depth (per TCD Figure XVI-1) A = tributary area = 2.72 acre project site DCV = (0.45)(0.7)(2.72)(43,560)(1/12) DCV = 3,110 ft^3

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 site will consist of a skate park, permeable parking lots, retail building, and landscaped areas. The WQMP plot plan shows the site features and configuration. The post-project site will be graded to drain into the permeable parking lots, which will filter the stormwater to remove pollutants and store it underground until it is discharged into the street after the peak of the storm. The proposed parking lot infiltration areas are designed to remove pollutants from stormwater via filtration thru media and soil. Additionally, activity restrictions should be enforced to prevent the pollutants of concern from coming in contact with stormwater. The project is anticipated to have hydrologic conditions of concern due to the fact that portions of the East Garden Grove Wintersburg Channel are earthen and considered unstable. To address the HCOC, the proposed parking lot sub-base storage areas will detain the incremental increase and ensure that the post development runoff volume does not exceed that of the predevelopment and time of concentration is not less than that for the predevelopment condition. The post-project site is not anticipated to have any significant impacts downstream. The total Design Capture Volume for the project is 3,110 ft^3.

Approximate coordinates for the permeable parking lots are as follows:

North Parking Lot: Latitude 33.736662, Longitude -117.998486

South Parking Lot: Latitude 33.734324, Longitude -117.998421

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:	☐

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.

Area of pavers required=$DCV/n_r d_r$ = 3,110/(.35 x 1)= 8,885 ft^2

North Parking Lot (11,277 ft^2)+ South Parking Lot (18,215 ft^2)= 29,492 ft^2

→ more than adequate paver area in both lots

The stormwater is not expected to infiltrate into the ground because the depth to historic groundwater is ±7 feet. The Infiltration BMP feasibility Worksheet indicates that "infiltration of any volume is not feasible." The stormwater will be filtered thru the permeable pavement and media below to remove pollutants and it will be stored below ground before the perforated underdrains collect the stormwater and discharge it to the neighboring streets after the peak of the storm. The planter areas within the project will be porous and stormwater should infiltrate or evaporate within them.

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.

The stormwater is not expected to infiltrate into the ground because the depth to historic groundwater is ±7 feet. The Infiltration BMP feasibility Worksheet indicates that "infiltration of any volume is not feasible." The stormwater will be filtered thru the permeable pavement and media below to remove pollutants and it will be stored below ground before the perforated underdrains collect the stormwater and discharge it to the neighboring streets after the peak of the storm. The storage media beneath the paver will act as an underground detention area for the stormwater. The planter areas within the project will be porous and stormwater should infiltrate or evaporate within them. Stormwater from sidewalks and hardscape areas will be directed to the planter areas to promote infiltration and evapotranspiration.

As noted in the Technical Guidance Document, specific fact sheets have not been developed for evapotranspiration-based BMPs because evapotranspiration is a significant reduction process in all hydrologic source control BMPs, surface-based infiltration BMPs, and biotreatment BMPs. In order to design the maximum feasible evapotranspiration areas amended soils consistent with the biotreatment BMP fact sheets will be used. The long and narrow planter areas located on-site will be designed to function as a bio-swales to promote as much evapotranspiration as possible.

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 Strom 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.

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.

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

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.

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)	☐	☒	No hazardous wastes are expected to be generated.
N6	Local Industrial Permit Compliance	☐	☒	The City of Huntington Beach does not issue water quality permits.
N7	Spill Contingency Plan	☐	☒	No spills are expected to be generated as part of this project.
N8	Underground Storage Tank Compliance	☐	☒	No underground storage tanks are proposed.
N9	Hazardous Materials Disclosure Compliance	☐	☒	No hazardous materials are expected to be generated as part of this project.
N10	Uniform Fire Code Implementation	☒	☐	
N11	Common Area Litter Control	☒	☐	
N12	Employee Training	☒	☐	
N13	Housekeeping of Loading Docks	☐	☒	No loading docks are proposed.
N14	Common Area Catch Basin Inspection	☒	☐	
N15	Street Sweeping Private Streets and Parking Lots	☒	☐	
N16	Retail Gasoline Outlets	☐	☒	No retail gas outlets are 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	☐	☒	None proposed.
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	☒	☐	
	Incorporate requirements applicable to individual priority project categories (from SDRWQCB NPDES Permit)	☒	☐	
S6	Dock areas	☐	☒	None proposed.
S7	Maintenance bays	☐	☒	None proposed.
S8	Vehicle wash areas	☐	☒	None proposed.
S9	Outdoor processing areas	☐	☒	None proposed.
S10	Equipment wash areas	☐	☒	None proposed.
S11	Fueling areas	☐	☒	None proposed.
S12	Hillside landscaping	☐	☒	None proposed.
S13	Wash water control for food preparation areas	☐	☒	None proposed.
S14	Community car wash racks	☐	☒	None proposed.

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.*

Description of Proposed Project				
Project Types that Qualify for Water Quality Credits (Select all that apply):				
☐Redevelopment projects that reduce the overall impervious footprint of the project site.	☐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.		☐ 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).	
☐ 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).		☐ 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		☐ 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).
☐Developments with dedication of undeveloped portions to parks, preservation areas and other pervious uses.	☐ Developments in a city center area.	☐ Developments in historic districts or historic preservation areas.	☐ 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.	☐In-fill projects, the conversion of empty lots and other underused spaces into more beneficially used spaces, such as residential or commercial areas.
Calculation of Water Quality Credits (if 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.*

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**
Education for Property Owners, Tenants and Occupants	City of Huntington Beach	Educational materials are included in this WQMP. The owner shall distribute additional copies of handouts.	The educational material provided should be reviewed yearly as well as when there is a change in ownership.
Activity Restrictions	City of Huntington Beach	Any activity that may affect surrounding areas or the downstream receiving waters (such as car washes or leaving trash bin lids open) is strictly prohibited.	Trash areas shall be checked before and after a major storm event. As well as on a monthly basis to reduce debris.
Common Area Landscape Management	City of Huntington Beach	Inspect landscaping and irrigation systems and repair/replace if needed.	The landscaping and irrigation systems shall be monitored monthly.
BMP Maintenance	City of Huntington Beach	Inspect systems; remove trash and debris and repair/replace if needed.	The BMPs shall be monitored before and after a major storm event and monthly.

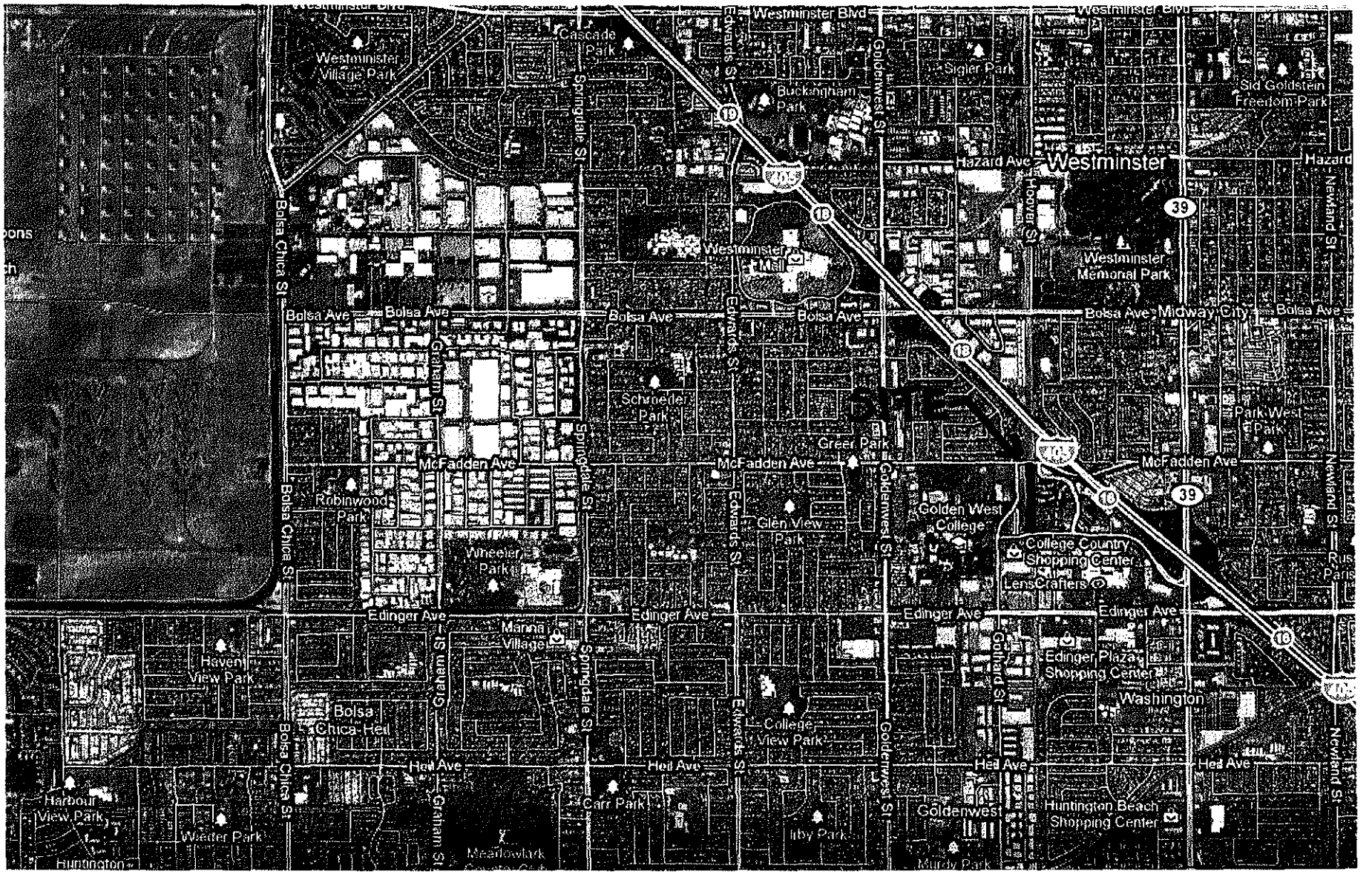
BMP Inspection/Maintenance			
BMP	**Reponsible Party(s)**	**Inspection/ Maintenance Activities Required**	**Minimum Frequency of Activities**
Common Area Litter Control	City of Huntington Beach	Wastes (debris, vegetation, etc) shall be properly disposed. No stormwater runoff shall pass through trash storage areas.	Trash areas shall be checked before and after a major storm event. As well as on a monthly basis to reduce debris.
Employee Training	City of Huntington Beach	Educational materials are included in this WQMP. The owner shall distribute additional copies of handouts.	The educational material provided should be reviewed yearly as well as when there is a new employee.
Common Area Catch Basin Inspection	City of Huntington Beach	Inspect systems; remove trash and debris and repair/replace if needed.	The catch basins shall be monitored before and after a major storm event and monthly. Cleaning required annually.
Street Sweeping Private Streets and Parking Lots	City of Huntington Beach	A street sweeper shall clean the privately maintained streets and parking areas to reduce debris.	A street sweeper shall clean the privately maintained streets and parking areas monthly.
Storm Drain Stenciling and Signage	City of Huntington Beach	Inspect stenciling and signage and repair/replace if needed.	The stenciling and signage shall be monitored annually.
Trash and Waste Storage Areas	City of Huntington Beach	Wastes (debris, vegetation, etc) shall be properly disposed. No stormwater runoff shall pass through trash storage areas.	Trash storage areas shall be checked before and after a major storm event. As well as on a monthly basis to reduce debris.

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



RICK ENGINEERING COMPANY
Riverside

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San Diego - Orange - San Luis Obispo - Bakersfield - Sacramento - Phoenix - Tucson

HUNTINGTON BEACH VANS SKATE PARK LOCATION MAP

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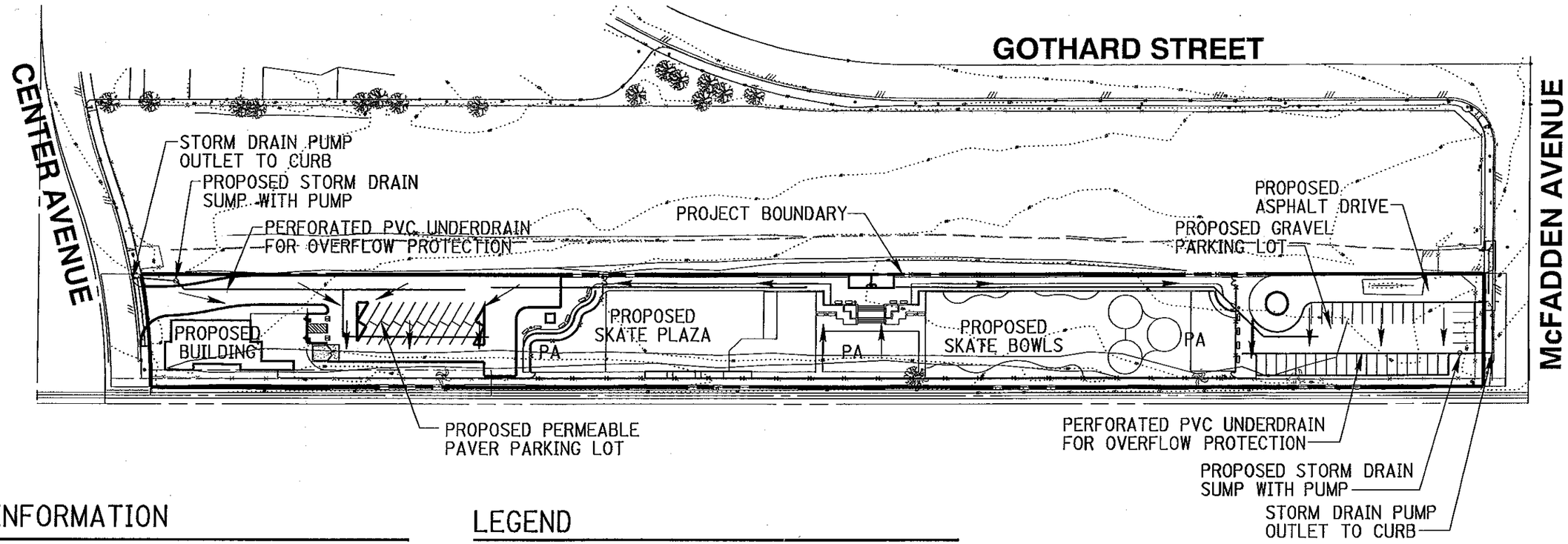

SITE INFORMATION

1. APN: 142-073-03
2. EXISTING ZONING: SP-14
3. AREA: 2.72 ACRES

LEGEND

— — —	PROJECT BOUNDARY
——►	DIRECTION OF FLOW
———	PROPOSED BUILDING FOOTPRINT
PA	LANDSCAPE/PLANTED AREA

SCALE: 1" = 100'

100 0 100 200

WQMP EXHIBIT:

THIS PLAN PRESENTS THE RECOMMENDATIONS OF RICK ENGINEERING COMPANY FOR THE TITLE PROJECT BASED ON COMPLETED PHASES OF CONSTRUCTION. STORM WATER POLLUTION PREVENTION IS THE RESPONSIBILITY OF THE PROPERTY OWNER OR THEIR DELEGATED RESPONSIBLE PARTY. THE RECOMMENDATIONS PRESENTED ON THIS PLAN SHOULD BE CONSTANTLY REVIEWED AND AMENDED AS APPROPRIATE BY THE PROPERTY OWNER OR THEIR DELEGATED RESPONSIBLE PARTY AS SITE CONDITIONS CHANGE.

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HUNTINGTON BEACH VANS SKATE PARK SITE PLAN

NOT FOR CONSTRUCTION - EXHIBIT FOR WQMP ONLY

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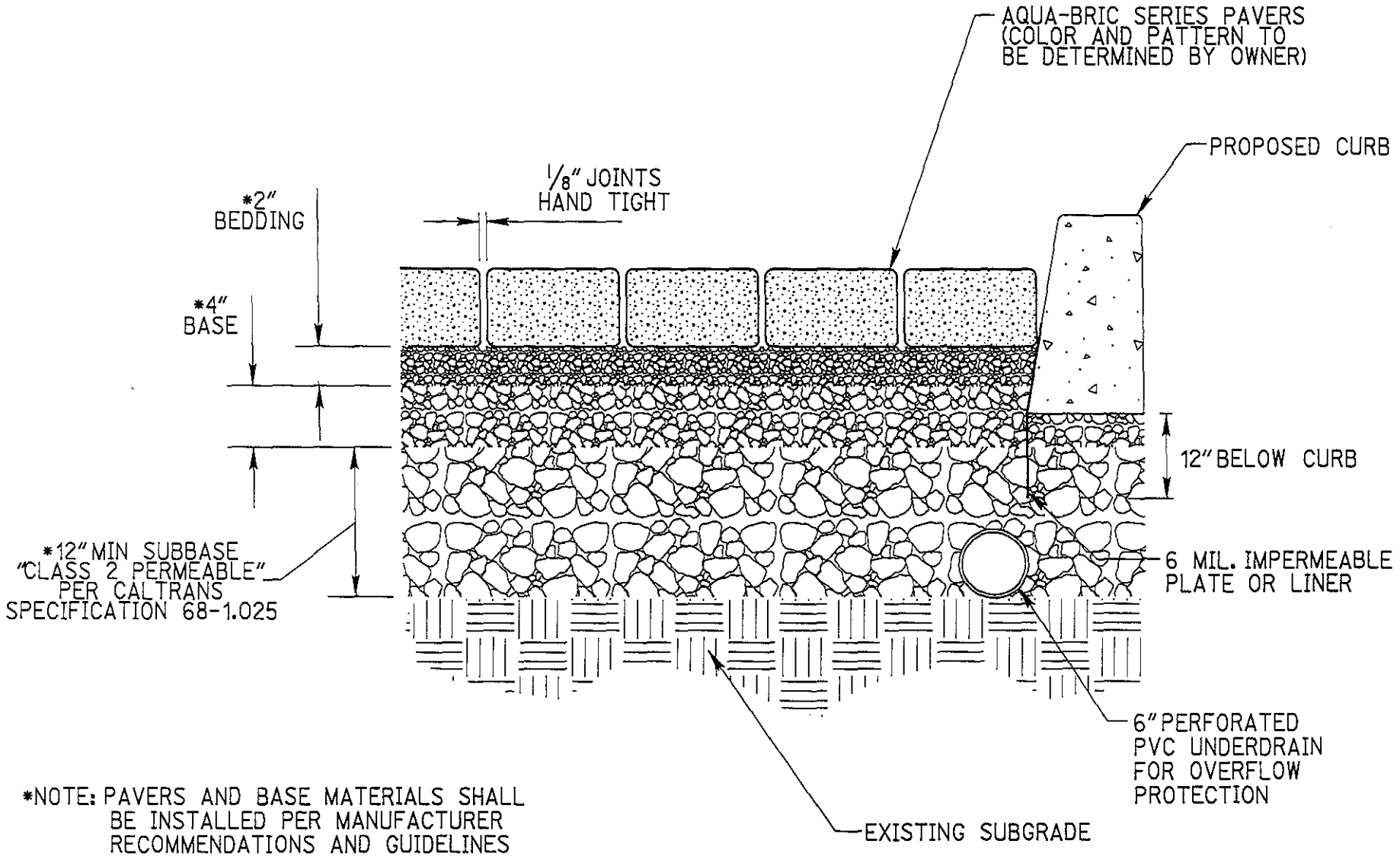

*NOTE: PAVERS AND BASE MATERIALS SHALL BE INSTALLED PER MANUFACTURER RECOMMENDATIONS AND GUIDELINES

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HUNTINGTON BEACH VANS SKATE PARK PERMEABLE PAVERS DETAIL

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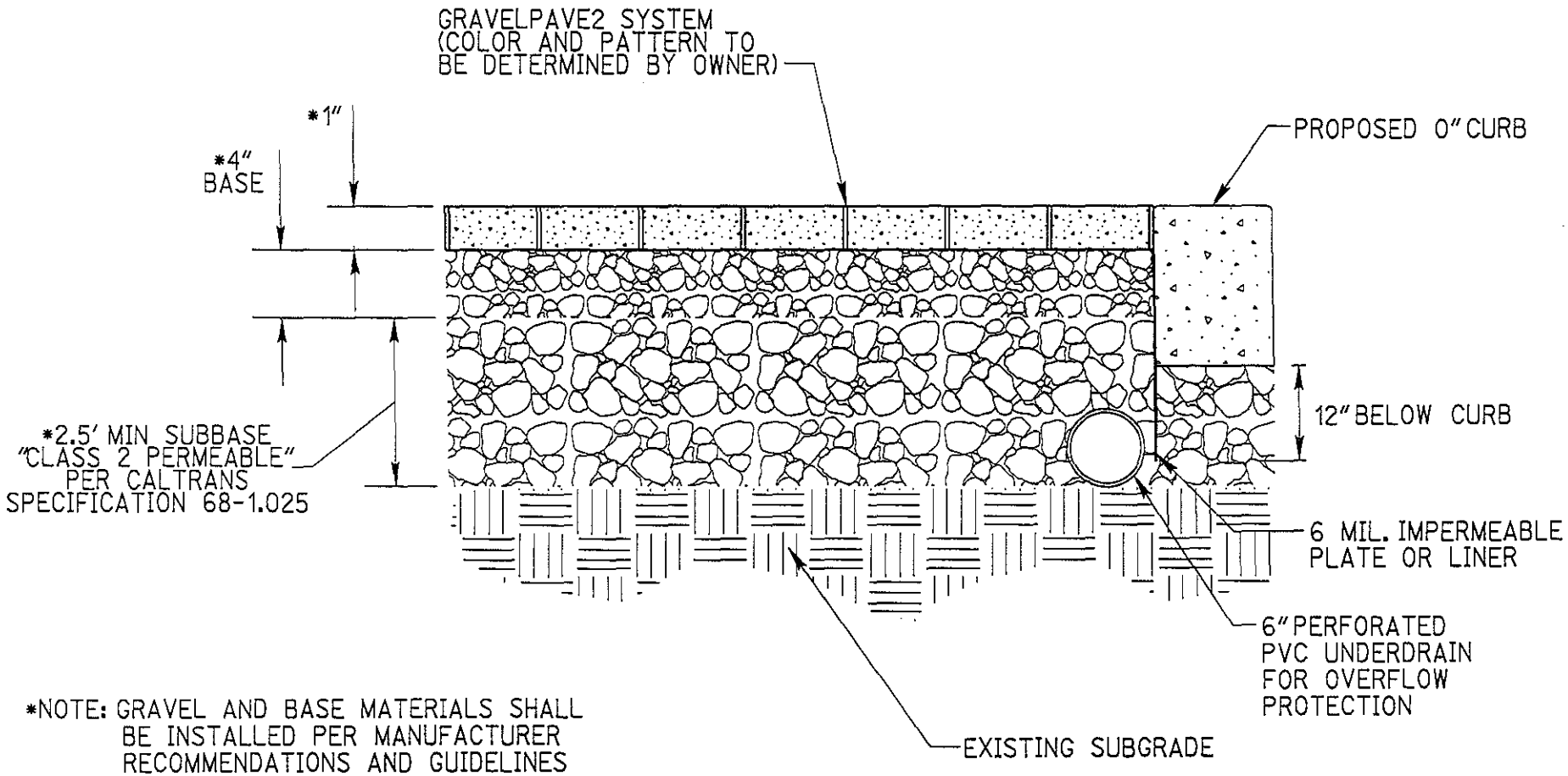

*NOTE: GRAVEL AND BASE MATERIALS SHALL BE INSTALLED PER MANUFACTURER RECOMMENDATIONS AND GUIDELINES

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HUNTINGTON BEACH VANS SKATE PARK GRAVEL PARKING LOT DETAIL

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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)	☒	Permeable Paver Information	☒
Recycle at Your Local Used Oil Collection Center (Central County)	☐	GravelPave Information	☒
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	☒		☐

Rational Method Hydrology Results (cfs)

	2-Year Storm		100-Year Storm	
	Pre-Project	Post-Project	Pre-Project	Post-Project
Southerly Basin	0.98	1.79	3.30	4.96
Northerly Basin	0.68	1.36	2.21	3.77
Total	1.66	3.15	5.51	8.73

Unit Hydrograph Results (cubic feet)

	2-Year Storm		100-Year Storm	
	Pre-Project	Post-Project	Pre-Project	Post-Project
Southerly Basin	7,173	7,836	27,011	25,409
Northerly Basin	2,967	6,213	11,176	20,146
Total	10,140	14,049	38,187	45,555

SUBBASE DESIGN

* SUBBASE IS APPROXIMATELY 35% VOID SPACE

SOUTHERLY BASIN – 18,215 ft^2 OF PAVERS

100-YEAR POST-PROJECT VOLUME – 25,409 ft^3

100-YEAR PRE-PROJECT VOLUME – 27,011 ft^3

2-YEAR POST-PROJECT VOLUME – 7,836 ft^3

2-YEAR PRE-PROJECT VOLUME – 7,173 ft^3

663 ft^3 DETENTION VOLUME REQUIRED.

(18,215 ft^2 PAVERS)(12" SUBBASE)(35% VOID SPACE) = 6,375 ft^3 VOID SPACE

∴ 12" SUBBASE OK

NORTHERLY BASIN – 11,277 ft^2 GRAVEL

100-YEAR POST-PROJECT VOLUME – 20,146 ft^3

100-YEAR PRE-PROJECT VOLUME – 11,176 ft^3

8,970 ft^3 DETENTION VOLUME REQUIRED

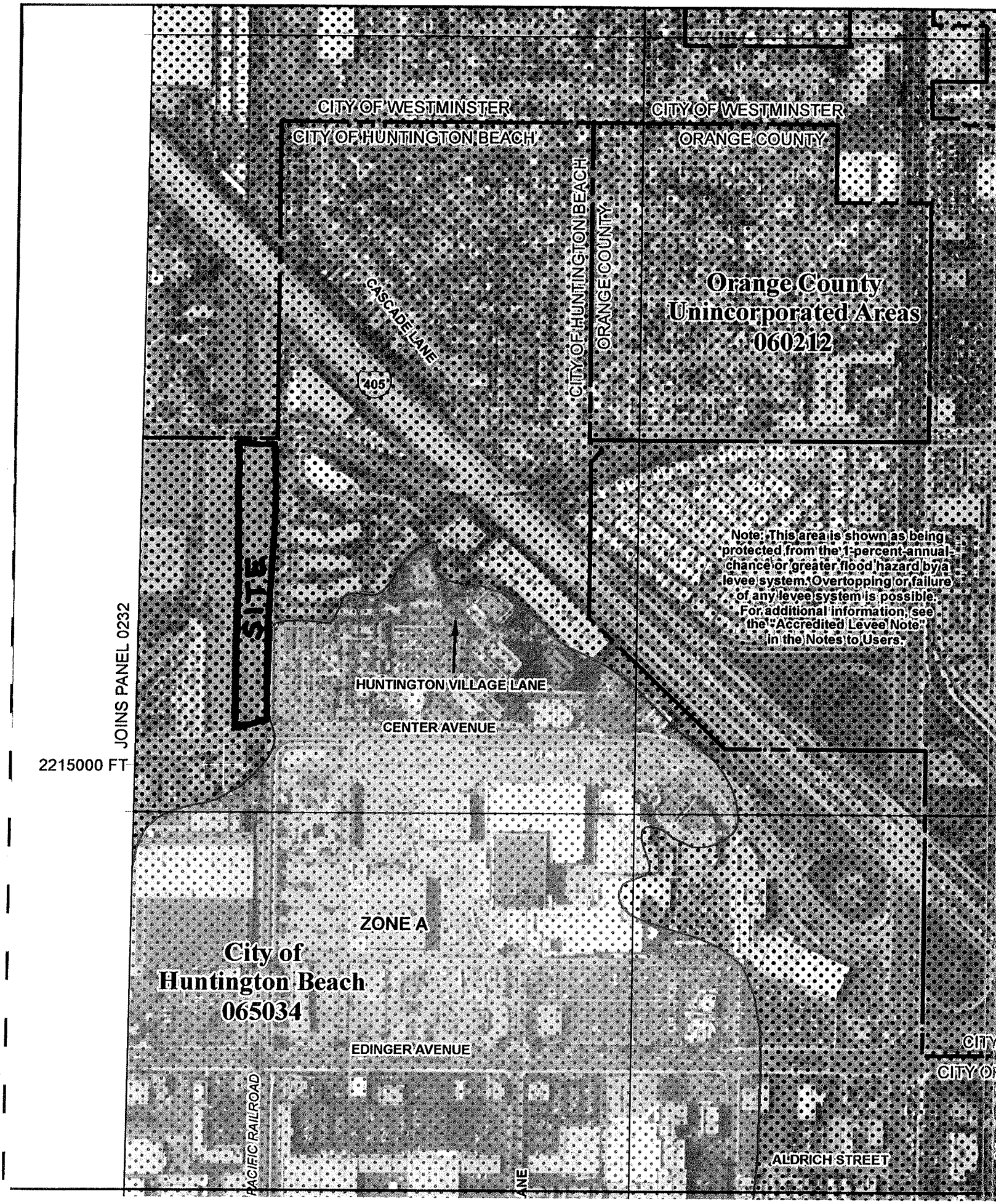
2-YEAR POST-PROJECT VOLUME – 6,213 ft^3

2-YEAR PRE-PROJECT VOLUME – 2,967 ft^3

3,246 ft^3

(11,277 ft^2)(2.5' SUBBASE)(35% VOID SPACE) = 9,867 ft^3 VOID SPACE

∴ 2.5' SUBBASE OK


CITY OF WESTMINSTER
CITY OF HUNTINGTON BEACH
CITY OF WESTMINSTER
ORANGE COUNTY
CITY OF HUNTINGTON BEACH
ORANGE COUNTY
Orange County
Unincorporated Areas
060212
CASCADE LANE
405
SITE
Note: This area is shown as being protected from the 1-percent-annual-chance or greater flood hazard by a levee system. Overtopping or failure of any levee system is possible. For additional information, see the "Accredited Levee Note" in the Notes to Users.
JOINS PANEL 0232
2215000 FT
HUNTINGTON VILLAGE LANE
CENTER AVENUE
ZONE A
City of
Huntington Beach
065034
EDINGER AVENUE
PACIFIC RAILROAD
ALDRICH STREET

LEGEND

SPECIAL FLOOD HAZARD AREAS SUBJECT TO INUNDATION BY THE 1% ANNUAL CHANCE FLOOD

The 1% annual flood (100-year flood), also known as the base flood, is the flood that has a 1% chance of being equaled or exceeded in any given year. The Special Flood Hazard Area is the area subject to flooding by the 1% annual chance flood. Areas of Special Flood Hazard include Zones A, AE, AH, AO, AR, A99, V, and VE. The Base Flood Elevation is the water-surface elevation of the 1% annual chance flood.

ZONE A	No Base Flood Elevations determined.
ZONE AE	Base Flood Elevations determined.
ZONE AH	Flood depths of 1 to 3 feet (usually areas of ponding); Base Flood Elevations determined.
ZONE AO	Flood depths of 1 to 3 feet (usually sheet flow on sloping terrain); average depths determined. For areas of alluvial fan flooding, velocities also determined.
ZONE AR	Special Flood Hazard Area formerly protected from the 1% annual chance flood by a flood control system that was subsequently decertified. Zone AR indicates that the former flood control system is being restored to provide protection from the 1% annual chance or greater flood.
ZONE A99	Area to be protected from 1% annual chance flood by a Federal flood protection system under construction; no Base Flood Elevations determined.
ZONE V	Coastal flood zone with velocity hazard (wave action); no Base Flood Elevations determined.
ZONE VE	Coastal flood zone with velocity hazard (wave action); Base Flood Elevations determined.

FLOODWAY AREAS IN ZONE AE

The floodway is the channel of a stream plus any adjacent floodplain areas that must be kept free of encroachment so that the 1% annual chance flood can be carried without substantial increases in flood heights.

OTHER FLOOD AREAS

ZONE X	Areas of 0.2% annual chance flood; areas of 1% annual chance flood with average depths of less than 1 foot or with drainage areas less than 1 square mile; and areas protected by levees from 1% annual chance flood.

OTHER AREAS

ZONE X	Areas determined to be outside the 0.2% annual chance floodplain.
ZONE D	Areas in which flood hazards are undetermined, but possible.

ional Flood Insurance Program at 1-800-638-6620.

MAP SCALE 1" = 500'

50 0 500 1000 FEET

METERS

NFIP

NATIONAL FLOOD INSURANCE PROGRAM

PANEL 0251J

FIRM

FLOOD INSURANCE RATE MAP

ORANGE COUNTY, CALIFORNIA
AND INCORPORATED AREAS

PANEL 251 OF 539

(SEE MAP INDEX FOR FIRM PANEL LAYOUT)

CONTAINS:

COMMUNITY	NUMBER	PANEL	SUFFIX
FOUNTAIN VALLEY, CITY OF	060218	0251	J
HUNTINGTON BEACH, CITY OF	065034	0251	J
ORANGE COUNTY	060212	0251	J
WESTMINSTER, CITY OF	060237	0251	J

Notice to User: The **Map Number** shown below should be used when placing map orders; the **Community Number** shown above should be used on insurance applications for the subject community.

MAP NUMBER
06059C0251J

MAP REVISED
DECEMBER 3, 2009

Federal Emergency Management Agency

This is an official copy of a portion of the above referenced flood map. It was extracted using F-MIT On-Line. This map does not reflect changes or amendments which may have been made subsequent to the date on the title block. For the latest product information about National Flood Insurance Program flood maps check the FEMA Flood Map Store at www.msc.fema.gov

Appendix B

Geotechnical Report

GEOTECHNICAL INVESTIGATION AND LIQUEFACTION EVALUATION PROPOSED SKATE PARK AND RETAIL BUILDING

SEC McFadden Avenue and Gothard Street
Huntington Beach, California
for
SITE Design Group, Inc.

April 12, 2011

SITE Design Group, Inc.
915 Camino Del Mar, Suite 150
Del Mar, California 92014

Attention: Ms. Cristal Garcia

Project No.: **11G113-1**

Subject: **Geotechnical Investigation and Liquefaction Evaluation**
Proposed Skate Park and Retail Building
SEC McFadden Avenue and Gothard Street
Huntington Beach, California

Dear Ms. Garcia:

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

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

Respectfully Submitted,

SOUTHERN CALIFORNIA GEOTECHNICAL, INC.

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

Daniel W. Nielsen, RCE 77915
Project Engineer

PROFESSIONAL GEOLOGIST
JOHN A. SEMINARA
No. 2125
CERTIFIED ENGINEERING GEOLOGIST
STATE OF CALIFORNIA

John A. Seminara, CEG 2125
Principal Geologist

Distribution: (2) Addressee

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A Plate 1: Site Location Map
Plate 2: Boring Location Plan
B Boring Logs
C Laboratory Test Results
D Grading Guide Specifications
E Seismic Design Parameters
F Liquefaction Evaluation Spreadsheets

1.0 EXECUTIVE SUMMARY

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

Site Preparation

- Initial site preparation should include stripping of grass and weed growth and any near surface organic material from the proposed building and improvement areas. Any organic materials should be disposed of off-site.
- Portions of the subject site are underlain by a surficial layer of fill soils, extending to depths of 1½ to 3± feet. The underlying native alluvium consists of compressible sandy silts, silty sands, silty clays and clayey silts. Several samples of soils recovered from depths of 1 to 15± feet contained minor amounts of organic peat.
- It is recommended that remedial grading be performed within the proposed building area to remove a portion of the existing unsuitable, compressible alluvial soils in order to control foundation settlements to acceptable levels. The overexcavation is recommended to extend to a depth of 5 feet below existing site grades, or 5 feet below the proposed building pad grade, whichever is greater. The overexcavation should also extend to a depth of at least 4 feet below all new footings.
- The presence of potentially compressible organic alluvial soils indicates the potential for long term settlements due to secondary consolidation. The magnitude of these settlements is expected to be within the range of 0.1 to 0.2 inches over the next 20 years, but cannot be precisely determined. To limit the possibility of secondary consolidation settlement, any soils identified during overexcavation as organic material or peat deposits should be removed in their entirety and disposed of off site.
- After overexcavation has been completed, the resulting subgrade soils should be evaluated by the geotechnical engineer to identify any additional soils that should be overexcavated. The resulting subgrade should then be scarified to a depth of 12 inches, and moisture conditioned or air dried to a moisture content of 2 to 4 percent above optimum. The previously excavated soils may then be replaced as compacted structural fill. All fill soils should be compacted to at least 90 percent of the ASTM D-1557 maximum dry density.
- The new parking area subgrade soils are recommended to be scarified to a depth of 12± inches, thoroughly moisture conditioned and recompacted to at least 90 percent of the ASTM D-1557 maximum dry density.

Building Foundations

- Conventional shallow foundations, supported in newly placed compacted fill.
- 1,500 lbs/ft^2 maximum allowable soil bearing pressure.
- Reinforcement consisting of at least six (6) No. 5 rebars (3 top and 3 bottom) in strip footings due to the liquefaction potential and expansion potential of the onsite soils. Additional reinforcement may be necessary for structural considerations.

Flatwork and Above Grade Skate Park Features

- Minimum slab thickness: 4 inches

- Minimum slab reinforcement: No. 4 bars at 18 inches on center, in both directions.
- Consideration should be given to the use of thickened edges, based on the presence of medium expansive soils.
- The existing soils in the proposed flatwork areas are recommended to be overexcavated to a depth of 2 feet below proposed subgrade elevation. The subgrade overexcavation soils within the flatwork and above grade skate park features should be evaluated by the geotechnical engineer to verify their suitability to serve as a structural fill subgrade.
- 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 to achieve a moisture content of 2 to 4 percent above the optimum moisture content. The subgrade soils should then be recompacted to at least 90 percent of the ASTM D-1557 maximum dry density. The flatwork areas should then be raised to grade with very low to non-expansive (EI < 20) structural fill.

Building Floor Slab

- Conventional Slab-on-Grade, at least 5 inches thick.
- Reinforcement consisting of at least No. 4 bars at 18-inches on-center, in both directions. The actual thickness and reinforcement of the floor slabs 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	3½	4
Aggregate Base	6	9	11
Compacted Subgrade	12	12	12

2.0 SCOPE OF SERVICES

The scope of services performed for this project was in accordance with our Proposal No. 11P130, dated February 8, 2011. 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. Based on the location of the subject site, this investigation also included a site specific liquefaction evaluation. The evaluation of the environmental aspects of this site was beyond the scope of services for this geotechnical investigation.

3.0 SITE AND PROJECT DESCRIPTION

3.1 Site Conditions

The subject site is located at the southeast corner of McFadden Avenue and Gothard Street in Huntington Beach, California. The site is bordered to the north by Mc Fadden Avenue, to the west by Gothard Street and a transit center, to the south by Center Avenue, and to the east by railroad tracks. The general location of the site is illustrated on the Site Location Map, included as Plate 1 in Appendix A of this report.

The subject site is a nearly rectangular shaped parcel, approximately 6.65 acres in size. The site is currently vacant and undeveloped. Ground surface cover generally consists of moderate to dense grass and weed growth with some areas of exposed soil except near the northeast corner of the site where ground surface cover appears to consist of gravel and broken pieces of asphalt. An asphalt driveway also traverses the site in the north-south direction.

Detailed topographic information was not available at the time of this report. Visually, site grades descend to the south at an estimated gradient of approximately 1 percent. There was estimated to be up to 10± feet of elevation differential across the site.

3.2 Proposed Development

The conceptual site plan indicates that the subject site will be developed into a skate park with a 3,000 ft^2 retail building located in the southern portion of the site. The site plan indicates that proposed development will also include a 12,000 ft^2 skate plaza with improvements constructed at or above the finished grade, a 15,000 ft^2 rectangular area for bowl features, several areas for temporary event seating, a vendor/event staging area, and parking lots. The new improvements are expected to be surrounded by Portland cement concrete flatwork and may include some limited areas of landscaping. We expect that the parking areas will be paved with asphaltic concrete.

Detailed structural information was not provided with the preliminary project information. We assume that the retail building will be of wood frame or masonry block construction, supported on conventional shallow foundations. Based on the assumed construction, maximum column and wall loads are expected to be on the order of 40 kips and 1 to 2 kips per linear foot, respectively. We assume that the new skate park features will be constructed of shotcrete/Portland cement concrete. It is also assumed that the bottom of some of the bowl features will be approximately 5 to 10± feet below the existing grade.

Detailed grading plans for the proposed development are not yet available. Based on an assumed relatively flat site, minor cuts and fills of up to 1 to 2± feet are assumed to be necessary to achieve the proposed site grades. In the areas of the proposed bowl features, cuts of 5 to 10± feet are may be necessary.

4.0 SUBSURFACE EXPLORATION

4.1 Scope of Exploration/Sampling Methods

The subsurface exploration conducted for this project consisted of six (6) borings, advanced to depths of 5 to 50± feet below currently existing site grades. The 50± foot boring was performed as a part of the liquefaction evaluation. All of the borings were logged during drilling by a member of our staff. It should be noted that at the time of this report, a new site plan has been provided since the date that we drilled the borings. The locations of some of the skate park features and the retail building has been altered since the subsurface exploration was performed.

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

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

4.2 Geotechnical Conditions

Artificial Fill

Artificial fill soils were encountered at all of the boring locations except B-1. These soils generally consist of very loose to medium dense silty sands and sandy silts and medium stiff silty clays and sandy clays extending to depths of 1½ to 3± feet. In general, the soils classified as artificial fill possess a disturbed appearance and occasional fine gravel. Additionally, Boring Nos. B-4 and B-6 encountered artificial debris in the fill materials including concrete and wood fragments.

Alluvium

Native alluvial soils were encountered at the ground surface at Boring No. B-1 and beneath the fill soils at all of the other boring locations. A thin layer of gray brown peat material was observed at Boring No. B-1 at depths of 1 to 2± feet. This thin peat material was measured to be approximately ¾ to 1 inch in thickness. In addition, several layers of soils with varying amounts of organic material (peat) were observed at Boring Nos. B-1, B-2, B-3 and B-4 between depths of 3½ to 12± feet.

The native alluvial soils extending to the maximum depth explored of 50± feet generally consist of interbedded soft to medium stiff silty clays and clayey silts with varying amounts of fine sand, and loose to medium dense silty fine sands and fine sandy silts with varying clay content.

Groundwater

Free water was encountered at a depth of 14± feet during the drilling of Boring No. B-1, 12± feet during the drilling of Boring No. B-3, and 13± feet during the drilling of Boring No. B-4. No free water was encountered within Boring Nos. B-2, B-5 and B-6. Delayed readings were not possible at the boring locations due to the fact that the borings caved after the augers were withdrawn from the boreholes. The static groundwater table is considered to have existed at a depth of 12 to 14± feet at the time of subsurface exploration. As noted in a subsequent section of this report, the California Geological Survey (CGS) has indicated that the depth to the historic high groundwater table in this area is 7± feet.

4.3 Peat Deposits

Research indicates that the site is within an area of possible peat deposits. The primary available reference applicable to the subject site is the map titled Distribution of Peat Deposits in Orange County, California, by James R. Evans, 1976. This map is part of the Open-File Report 79-8 LA, Environmental Geology of Orange County, California, published by the California Division of Mines and Geology.

The location of the site is mapped as an "area where peat deposits are a strong probability, but distribution and thickness is unknown." It should be noted that the site is relatively close to an area of known peat deposits of small thickness (0.5 to 5.0 feet) that is approximately 8 feet below ground surface to the top of a peat layer.

5.0 LABORATORY TESTING

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

Classification

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

In-situ Density and Moisture Content

The density has been determined for selected relatively undisturbed ring samples. These densities were determined in general accordance with the method presented in ASTM D-2937. The results are recorded as dry unit weight in pounds per cubic foot. The moisture contents are determined in accordance with ASTM D-2216, and are expressed as a percentage of the dry weight. These test results are presented on the Boring Logs.

Consolidation

Selected soil samples have been tested to determine their consolidation potential, in accordance with ASTM D-2435. The testing apparatus is designed to accept either natural or remolded samples in a one-inch high ring, approximately 2.416 inches in diameter. Each sample is then loaded incrementally in a geometric progression and the resulting deflection is recorded at selected time intervals. Porous stones are in contact with the top and bottom of the sample to permit the addition or release of pore water. The samples are typically inundated with water at an intermediate load to determine their potential for collapse or heave. The results of the consolidation testing are plotted on Plates C-1 through C-4 in Appendix C of this report.

Expansion Index

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

Sample Identification	Expansion Index	Expansive Potential
B-1 @ 0 to 5 feet	65	Medium

Grain Size Analysis

Limited grain size analyses have been performed on several selected samples, in accordance with ASTM D-1140. These samples were washed over a #200 sieve to determine the percentage of fine-grained material in each sample, which is defined as the material which passes the #200 sieve. The weight of the portion of the sample retained on each screen is recorded and the percentage finer or coarser of the total weight is calculated. The results of these tests are presented on the boring logs.

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 result of the soluble sulfate testing is presented below, and is discussed further in a subsequent section of this report.

Sample Identification	Soluble Sulfates (%)	Sulfate Classification
B-1 @ 0 to 5 feet	0.0067	Negligible

Resistivity and pH Testing

Representative bulk samples of soil collected from the building area were submitted to a subcontracted analytical laboratory for determination of electrical resistivity and pH. The resistivity of the soils is a measure of their potential to attack buried metal improvements such as utility lines. The results of the resistivity and pH testing are presented below, and are discussed further in a subsequent section of this report.

Sample Identification	Resistivity (ohm-cm)	pH
B-1 @ 0 to 5 feet	2700	7.6

Organic Content

Representative near-surface soil samples were submitted to our laboratory for determination of organic content in accordance with ASTM Test Method 2974. The results of the testing are as follows:

Sample Identification	Organic Content (%)
B-1 @ 3 to 4 feet	1.4
B-1 @ 5 to 6 feet	3.0
B-1 @ 7 to 8 feet	4.9
B-1 @ 9 to 10 feet	1.7

Atterberg Limits

Atterberg Limits testing (ASTM D-4318) was performed on several of the samples recovered at Boring No. B-1. This test is used to determine the Liquid Limit and Plastic Limit of the soil. The Plasticity Index is the difference between the two limits. Plasticity Index is a general indicator of the expansive potential of the soil, with higher numbers indicating higher expansive potential. Soils with a PI greater than 25 are considered to have a high plasticity, and a high expansion potential. The results of the Atterberg Limits testing are presented on the boring logs.

6.0 CONCLUSIONS AND RECOMMENDATIONS

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

6.1 Seismic Design Considerations

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

Faulting and Seismicity

Research of available maps indicates that the subject site is not located within an Alquist-Priolo Earthquake Fault Zone. Therefore, the possibility of significant fault rupture on the site is considered to be low.

Seismic Design Parameters

Based on standards in place at the time of this report, the proposed development must be designed in accordance with the requirements of the 2010 edition of the California Building Code (CBC).

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

The 2010 CBC Seismic Design Parameters have been generated using Earthquake Ground Motion Parameters, a software application developed by the United States Geological Survey. This software application, available at the USGS web site calculates seismic design parameters in accordance with the 2010 CBC, utilizing a database of deterministic site accelerations at 0.01 degree intervals. The table below is a compilation of the data provided by the USGS

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

2010 CBC SEISMIC DESIGN PARAMETERS

Parameter		Value
Mapped Spectral Acceleration at 0.2 sec Period	S_S	1.534
Mapped Spectral Acceleration at 1.0 sec Period	S_1	0.552
Site Class	---	E*
Short-Period Site Coefficient at 0.2 sec Period	F_a	0.9
Long-Period Site Coefficient at 1.0 sec Period	F_v	2.4
Site Modified Spectral Acceleration at 0.2 sec Period	S_{MS}	1.381
Site Modified Spectral Acceleration at 1.0 sec Period	S_{M1}	1.325
Design Spectral Acceleration at 0.2 sec Period	S_{DS}	0.920
Design Spectral Acceleration at 1.0 sec Period	S_{D1}	0.883

*The 2010 CBC requires that Site Class F be assigned to any profile containing soils vulnerable to potential failure or collapse under seismic loading, such as liquefiable soils. For Site Class F, the site coefficients are to be determined in accordance with Section 11.4.7 of ASCE 7.05. However, Section 20.3.1 of ASCE 7.05 indicates that for sites with structures having a fundamental period of vibration equal to or less than 0.5 seconds, the site class is determined using the standard procedures. **If the proposed structure has a fundamental period greater than 0.5 seconds, SCG should be contacted to revise these seismic design parameters.**

Liquefaction

Research of the Newport Beach Quadrangle, 7.5 minute Seismic Hazard Zone Map, published by the California Geological Survey (CGS) indicates that the site is located in a designated liquefaction hazard zone. Therefore, the scope of this investigation included a detailed liquefaction evaluation in order to determine the site-specific liquefaction potential.

The liquefaction was performed using the reported historic groundwater depth of 7 feet. The primary reference used to determine the historic groundwater depths in this area is CGS Open File Report 97-08, the Seismic Hazard Evaluation of the Newport Beach Quadrangle.

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 liquefaction analysis was conducted in accordance with the requirements of Special Publication 117A (CDMG, 2008), and currently accepted practice (SCEC, 1997). The liquefaction potential of the subject site was evaluated using the empirical method originally developed by Seed, et al. (Seed and Idriss 1971). This method predicts the earthquake-induced liquefaction potential of the site based on a given design earthquake magnitude and peak ground acceleration at the subject site. This procedure essentially compares the cyclic resistance ratio (CRR) [the cyclic stress ratio required to induce liquefaction for a cohesionless soil stratum at a given depth] with the earthquake-induced cyclic stress ratio (CSR) at that depth from a specified design earthquake (defined by a peak ground surface acceleration and an associated earthquake moment magnitude). The current version of a generally accepted baseline chart (Youd and Idriss, 1997) is used to determine CRR as a function of the corrected SPT N-value $(N_1)_{60}$. The factor of safety against liquefaction is defined as CRR/CSR. Based on Special Publication 117A, a factor of safety of at least 1.3 is required in order to demonstrate that a given soil stratum is non-liquefiable.

The liquefaction analysis procedure is tabulated on the spreadsheet forms included in Appendix F of this report. The liquefaction analysis was performed for Boring B-1, which was advanced to a depth of 50± feet. The liquefaction potential of the site was analyzed utilizing a design peak ground acceleration (PGA) of 0.37g for a magnitude 6.8 seismic event. The design PGA was obtained in accordance with the 2010 CBC and ASCE 7-05 and is equal to S_{DS} divided by 2.5. The analysis was performed using groundwater at 7 feet, which is expected to be representative of a conservative, historic groundwater elevation at the subject site.

If liquefiable soils are identified, the potential settlements that could occur as a result of liquefaction are determined using the procedure developed by Tokimatsu and Seed (1987). This procedure uses the induced cyclic stress ratio, the corrected N-value and the earthquake magnitude to determine the expected volumetric strain of saturated sands subjected to earthquake shaking. This analysis is also documented on the spreadsheets included in Appendix F.

Conclusions and Recommendations

The results of the liquefaction analysis have identified potentially liquefiable soils at the site. The potentially liquefiable soils are generally located between depths of 8 and 11± feet, 17 to 19± feet, 23 feet to 31± feet, and 48½ to 50± feet. Soils which are located above the historic groundwater table (7 feet), or possess factors of safety in excess of 1.3 are considered non-liquefiable. The zones of silty clays, clayey silts, and sandy clays are considered non-liquefiable due to their fine grained, cohesive, characteristics and the results of the Atterberg limits testing with respect to the requirements of Special publication 117A. Settlement analyses were conducted for each of the potentially liquefiable strata.

Based on the settlement analysis (also tabulated on the spreadsheet in Appendix F) total dynamic (liquefaction induced) settlements of 3.9± inches could be expected at the boring

location. The associated differential settlement would therefore be on the order of 2.6± inches. The estimated differential settlement could be assumed to occur across a distance of 100 feet, indicating a maximum angular distortion of about 0.0022 inches per inch. This settlement is considered to be within the structural tolerances of a typical building supported on a shallow foundation system. However, it should be noted that minor to moderate repairs, including repair of damaged drywall and stucco, etc., could be required after the occurrence of liquefaction-induced settlements.

A shallow foundation system can be designed to resist the effects of the anticipated differential settlements, to the extent that the structure would not catastrophically fail. Designing the proposed structure to remain completely undamaged during a seismic event that could occur once every 2,500± years (the code specified return period used in the liquefaction analysis) is not considered to be economically feasible. Based on this understanding, the use of a shallow foundation system is considered to be the most economical means of supporting the proposed structure.

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

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

6.2 Geotechnical Design Considerations

General

Boring No. B-1 which was drilled within the proposed building area generally encountered low density, low to moderate strength native alluvial soils within the depths of the foundation influence zone of the proposed building. Additionally, these soils contain traces of organic materials at various depths. These soils are considered unsuitable for the support of the proposed structure. Therefore, remedial grading is recommended to replace a portion of the unsuitable native soils with compacted structural fill soils. Additionally, all of the other borings encountered a surficial layer of artificial fill. No documentation has been provided for these fill materials. The fill soils also possess variable strengths and are considered unsuitable for the support of the proposed building and skate park improvements. Additional remedial grading is also recommended for the skate park improvement areas.

Several samples of native soils recovered from depths of 1 to 15± feet contain trace amounts of peat. As discussed in Section 4.3 of this report, mapping performed by Evans does not specifically identify peat deposits at the subject site, but indicates that the site is located in an area where peat deposits are possible. Nearby peat deposits have been mapped, beginning at a depth of 8± feet, with a thickness of 0.5 to 5.0 feet. The borings drilled at the subject site generally encountered trace amounts of peat interbedded within clayey or silty soils. The peat deposits at the sample depths were generally less than ¼ inch in thickness except at Boring No. B-1 at 1 to 2 feet, where a peat deposit was measured to be approximately ¾ to 1 inch in thickness. With the current scope of field exploration, it was not practical to determine the extent and thickness of the peat deposits in the area of the proposed building and proposed improvements with any significant accuracy. The presence of these deposits could result in additional long term total and differential settlements. SCG can provide no assurance that excessive future settlements will not occur. However, this report provides recommendations for remedial grading which will identify and remove peat deposits from the building pad and improvement areas to at least the depth of overexcavation. Any such materials identified in any new foundation areas should be removed and replaced with non-organic compacted fill.

As discussed in a previous section of this report, potentially liquefiable soils were identified at this site. This report provides recommendations to reduce the potential for excessive differential movements between the proposed improvements. However, the owner and/or developer should be aware of the potential for settlements to occur in the event of liquefaction, as discussed in Section 6.1 of this report.

Settlement

The results of laboratory testing indicate that the soils at the subject site generally possess poor to moderate consolidation characteristics. This report provides recommendations for remedial grading to remove and replace compressible soils with non-organic structural fill. Where new building footings, designed for a relatively low allowable soil bearing pressure, are founded in newly placed structural fill soils, the settlements are expected to be within tolerable limits. Within areas of new above-grade skate park improvements, the recommended remedial grading will remove the artificial fill soils and the upper portion of the compressible native alluvium, and replace these materials as compacted structural fill. Therefore, following completion of the recommended grading, post-construction settlements of the new improvements are expected to be within tolerable limits.

The presence of the peat deposits, identified at several of the boring locations, may result in long term settlements associated with secondary consolidation. The amount of secondary consolidation will depend on the thickness and moisture content of the peat deposit in any given location. These parameters appear to vary significantly between the boring locations, and are also expected to vary widely across the building and improvement areas. Further considerations related to settlement associated with secondary consolidation are presented in Section 6.5 of this report.

Expansion

Laboratory testing performed on representative samples of the near surface soils indicates that these materials possess a medium expansion potential (EI = 65). Based on the presence of potentially expansive soils at this site, 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 made in consideration of the expansion index test results. 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.

Corrosion Potential

The results of the electrical resistivity and pH testing indicate that samples of the on-site soils have a resistivity of 2700 ohm-cm, a pH value of 7.6 and relatively high in-situ moisture contents. These test results have been evaluated in accordance with guidelines published by the Ductile Iron Pipe Research Association (DIPRA). The DIPRA guidelines consist of a point system by which characteristics of the soils are used to quantify the corrosivity characteristics of the site. Resistivity and pH are two of the five factors that enter into the evaluation procedure. Relative soil moisture content as well as redox potential and sulfides are also included. Although redox potential and sulfide testing were not part of the scope of services for this project, we have evaluated the corrosivity characteristics of the on-site soils using resistivity, pH and moisture content. **Based on these factors, and utilizing the DIPRA procedure, the on-site soils are considered to be slightly corrosive to ductile iron pipe. Therefore, it is expected that polyethylene protection will be required for cast iron or ductile iron pipes.** If a more detailed evaluation is desired, redox potential and sulfide content should be determined for the on-site soils. The client may also wish to contact a corrosion engineer to provide a more thorough evaluation.

Shrinkage/Subsidence

Removal and recompaction of the near surface native soils is estimated to result in an average shrinkage of 12 to 17 percent. Minor ground subsidence is expected to occur in the soils below the zone of removal, due to settlement and machinery working. The subsidence is estimated to be 0.2± feet. This estimate may be used for grading in areas that are underlain by native alluvial soils.

These estimates are based on previous experience and the subsurface conditions encountered at the boring locations. The actual amount of subsidence is expected to be variable and will be 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 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.

Soluble Sulfates

The results of the soluble sulfate testing indicate that the selected samples of the on-site soils contain negligible concentrations of soluble sulfates with respect to the American Concrete Institute (ACI) Publication 318-05 Building Code Requirements for Structural Concrete and Commentary, Section 4.3. Therefore, specialized concrete mix designs are not considered to be necessary, with regard to sulfate protection purposes. It is, however, recommended that additional soluble sulfate testing be conducted at the completion of rough grading to verify the soluble sulfate concentrations of the soils which are present at the proposed improvement grades.

6.3 Site Grading Recommendations

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

Site Stripping

Initial site preparation should include stripping of any vegetation from the proposed building and improvement areas. Based on conditions observed at the time of the subsurface exploration, this will include areas of grass and weed growth throughout the site. Any organic soils encountered during site stripping should be removed and disposed of off-site. The actual extent of stripping should be determined in the field by a representative of the geotechnical engineer, based on the organic content and the stability of the encountered materials.

Treatment of Existing Soils: Building Pad

Remedial grading should be performed within the building pad area in order to remove the unsuitable, compressible, upper portion of the native alluvial and fill soils. In order to provide a relatively uniform support condition for the new structure and to limit settlement, it is recommended that the existing soils within the proposed building area be overexcavated to a depth of 5 feet below existing grade and to a depth of 5 feet below proposed building pad subgrade elevation. The depth of overexcavation should also be sufficient to provide at least 4 feet of newly placed compacted structural fill below the bearing grade of all foundations.

Following completion of the overexcavation, the subgrade soils within the building areas should be evaluated by the geotechnical engineer to verify their suitability to serve as the structural fill subgrade, as well as to support the foundation loads of the new structures. At a minimum, the soils exposed at the base of the overexcavation should possess an in-situ density equal to at least 85 percent of the ASTM D-1557 maximum dry density. This evaluation should include probing and proofrolling to identify any soft, loose or otherwise unstable soils that must be removed. The character and composition of the possible fill soils encountered at several of the boring locations should also be determined at this time. If these materials are determined to represent organic soils or peat deposits, they should also be removed in their entirety and

disposed of off-site. **Some localized areas of deeper excavation may be required if additional fill materials or, loose, porous, low density, organic or otherwise unsuitable materials are encountered at the base of the overexcavation.**

Based on conditions encountered at the exploratory boring locations, some zones of moist/very moist soils will be encountered at or near the base of the recommended overexcavation. Some subgrade stabilization may be required. 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, may 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 to achieve a moisture content of 2 to 4 percent above optimum moisture content. The subgrade soils should then be recompacted to at least 90 percent of the ASTM D-1557 maximum dry density. The previously excavated non-organic soils may then be replaced as compacted structural fill.

Treatment of Existing Soils: Retaining Walls and Site Walls

The existing soils within the areas of proposed retaining walls should be overexcavated to a depth of 2 feet below foundation bearing grade and replaced as compacted structural fill, as discussed above for the proposed building area. Any undocumented fill soils should also be removed from the retaining wall areas. Subgrade soils in areas of non-retaining site walls should be overexcavated to a depth of 1 foot below proposed bearing grade. In both cases, the overexcavation subgrade soils should be evaluated by the geotechnical engineer prior to scarifying, moisture conditioning and recompacting the upper 12 inches of exposed subgrade soils. The previously excavated soils may then be replaced as compacted structural fill.

In the areas of the proposed below grade bowl features where overexcavation is not practical, the side walls and bottom of the excavations should be compacted at the exposed surface until the soil is firm and unyielding.

Treatment of Existing Soils: Flatwork and Above Grade Skate Park Features

As discussed above, the subject site is generally underlain by soils with medium expansive potential. Concrete flatwork is especially susceptible to movement when the subgrade soils are allowed to increase or decrease in moisture content. This is primarily due to the fact that flatwork is relatively lightly loaded and is typically exposed to moisture infiltration from landscape planters or other surface water runoff. It is therefore recommended that the proposed concrete flatwork areas be overexcavated to provide for a new layer of very low to non-expansive fill.

The existing soils in the proposed flatwork areas are recommended to be overexcavated to a depth of 2 feet below proposed subgrade elevation. The subgrade overexcavation soils within the flatwork and above grade skate park features should be evaluated by the geotechnical engineer to verify their suitability to serve as a structural fill subgrade. This evaluation should

include probing to identify any soft, loose or otherwise unstable soils that should be removed. Some localized areas of deeper excavation may be required if unsuitable fill materials or loose, porous, or low density native soils are encountered at the existing soil subgrade level.

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 to achieve a moisture content of 2 to 4 percent above the optimum moisture content. The subgrade soils should then be recompacted to at least 90 percent of the ASTM D-1557 maximum dry density. **The flatwork areas should then be raised to grade with very low to non-expansive (EI < 20) structural fill.**

If the owner is willing to tolerate increased amounts of movement within the perimeter flatwork, the recommended layer of very low to non-expansive fill could be eliminated. However, the owner must be willing to accept the risk of increased movement which could result in cracking or other distress within the concrete flatwork.

Treatment of Existing Soils: Parking and Drive Areas

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

Subgrade preparation in the new parking areas should initially consist of removal of all soils disturbed during stripping and demolition operations. The geotechnical engineer should then evaluate the subgrade to identify any areas of additional unsuitable soils. The subgrade soils should then be scarified to a depth of 12± inches, moisture conditioned to 2 to 4 percent above optimum, and recompacted to at least 90 percent of the ASTM D-1557 maximum dry density. Based on the presence of surficial undocumented 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.

The grading recommendations presented above for the proposed parking and drive areas assume that the owner and/or developer can tolerate minor amounts of settlement within the proposed parking areas. The grading recommendations presented above do not mitigate the extent of potentially compressible soils and undocumented fill soils in the parking areas. As such, settlement and associated pavement distress could occur. Typically, repair of such distressed areas involves significantly lower costs than completely mitigating these soils at the time of construction. If the owner cannot tolerate the risk of such settlements, the parking and drive areas should be graded in a manner similar to that described for the building area.

Fill Placement

- Fill soils should be placed in thin (6± inches), near-horizontal lifts, moisture conditioned to 2 to 4 percent above the optimum moisture content, and compacted.
- On-site soils may be used for fill provided they are cleaned of any debris and organic materials to the satisfaction of the geotechnical engineer. Select imported soils or aggregate base may be utilized as structural fill.

- All grading and fill placement activities should be completed in accordance with the requirements of the 2010 CBC and the grading code of the city of Huntington Beach.
- All fill soils should be compacted to at least 90 percent of the ASTM D-1557 maximum dry density. Fill soils should be well mixed.
- Compaction tests should be performed periodically by the geotechnical engineer as random verification of compaction and moisture content. These tests are intended to aid the contractor. Since the tests are taken at discrete locations and depths, they may not be indicative of the entire fill and therefore should not relieve the contractor of his responsibility to meet the job specifications.

Imported Structural Fill

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

Utility Trench Backfill

In general, all utility trench backfill soils should be compacted to at least 90 percent of the ASTM D-1557 maximum dry density. As an alternative, a clean sand (minimum Sand Equivalent of 30) may be placed within trenches and compacted in place (jetting or flooding is not recommended). Compacted trench backfill should conform to the requirements of the local grading code, and more restrictive requirements may be indicated by the city of Huntington Beach. 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 95 percent of the ASTM D-1557 standard. Pea gravel backfill should not be used for these trenches.

6.4 Construction Considerations

Excavation Considerations

The near-surface soils generally consist of silty sands, sandy silts, clayey silts and silty clays. These materials are generally loose or medium stiff and may be subject to significant caving. Where caving does occur, flattened excavation slopes may be sufficient to provide excavation stability. On a preliminary basis, the inclination of temporary slopes should not exceed 1.5h:1v. Deeper excavations may require some form of external stabilization such as shoring or bracing. Maintaining adequate moisture content within the near-surface soils will improve excavation stability. All excavation activities on this site should be conducted in accordance with Cal-OSHA regulations.

Moisture Sensitive Subgrade Soils

Some of the near-surface soils possess appreciable silt and clay content and will become unstable if exposed to significant moisture infiltration or disturbance by construction traffic. In addition, based on their granular content, some of the on-site soils will be susceptible to erosion. Therefore, the site should be graded to prevent ponding of surface water and to prevent water from running into excavations.

If the construction schedule dictates that site grading will occur during a period of wet weather, allowances should be made for costs and delays associated with drying the on-site soils or import of a less moisture sensitive fill material. Grading during wet or cool weather may also increase the depth of overexcavation in the pad areas as well as the need for and/or the thickness of the crushed stone stabilization layer, discussed in Section 6.3 of this report.

Expansive Soils

The near-surface soils at this site generally consist of low to medium expansive clayey silts and silty clays. 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 low expansive characteristics. **In addition to adequately moisture conditioning the subgrade soils and fill soils during grading, special care must be taken to maintain moisture content of these soils at 2 to 4 percent above the Modified Proctor optimum. This will require the contractor to frequently moisture condition these soils throughout the grading process, unless grading occurs during a period of relatively wet weather.**

Groundwater

The static groundwater table at this site is considered to exist at a depth of 12 to 14± feet. Based on the interbedding of the soils encountered at the site, it is possible that perched water may be encountered above the relatively impermeable clay layers encountered at various depths. Additionally, very moist soils were encountered at depths greater than 5± feet. Therefore, it is possible that some seepage will occur in excavations deeper than 5± feet. It should be noted that the California Geological Survey (CGS) has identified the depth to the historic high groundwater table at this site at 7± feet. **Provisions to incorporate de-watering measures during the construction of the proposed building and skate park improvements may be necessary to ensure that seepage of perched groundwater may be removed from the excavations.**

Dry Weather and Wet Weather Construction

The on-site soils generally consist of moist to very moist sandy silts, silty sands, silty clays, and clayey silts. These materials are considered to be susceptible to subgrade instability, even in the event of dry weather. The previously presented grading recommendations are considered applicable for dry weather conditions or conditions of minor rainfall. In the event of heavy rainfall, provisions should be made to utilize imported select structural fill in all areas where fill is required, including the foundation and floor slab overexcavation areas and pavement subgrades. The on-site soils are not expected to be suitable for use as structural fill during a

period of wet weather. During wet weather construction, the use of a crushed stone stabilization layer within all overexcavation areas should also be assumed. Local practice dictates that wet weather conditions be assumed during the period of October 15 to April 15.

6.5 Foundation Design and Construction

Based on the preceding grading recommendations, new building foundations will be underlain by newly placed structural fill soils extending to a depth of at least 4 feet below the proposed foundation bearing grade. Based on this subsurface profile, and based on the design considerations presented in Section 6.1 of this report, the proposed structure may be supported on shallow foundations.

Foundation Design Parameters

New square and rectangular footings may be designed as follows:

- Maximum, net allowable soil bearing pressure: 1,500 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 liquefaction potential and expansion potential of the on-site 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 standard geotechnical practice, given the magnitude of predicted liquefaction-induced settlements, the expansion potential of the on-site soils and the structure type proposed for this site. Additional rigidity may be necessary for structural considerations, or to resist the effects of the liquefaction-induced differential settlements discussed in Section 6.1. The actual design of the foundations should be determined by the structural engineer.

Foundation Construction

The foundation subgrade soils should be evaluated at the time of overexcavation, as discussed in Section 6.3 of this report. It is further recommended that the foundation subgrade soils be evaluated by the geotechnical engineer immediately prior to steel or concrete placement. Soils suitable for direct foundation support should consist of existing compacted structural fill, or newly placed compacted structural fill materials that have been 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 static settlements of shallow foundations designed and constructed in accordance with the previously presented recommendations are estimated to be less than 1.0 and 0.5 inches, respectively. Differential movements are expected to occur over a 40-foot span, thereby resulting in an angular distortion of less than 0.002 inches per inch. These settlements are in addition to the liquefaction-induced settlements previously discussed in Section 6.1 of this report.

As discussed previously, the area of the new structure may be underlain by potentially compressible peat deposits. Therefore, it is not considered reasonable to reduce the potential settlements of any new foundation elements to levels that are significantly below the estimated long term settlements of the overall structure. The estimated long term settlements associated with the existing peat deposits that were encountered at the boring locations are estimated to be in the range of 0.1 to 0.2 inches over a 20-year period.

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 2,000 lbs/ft^2.

6.6 Floor Slab Design and Construction

Subgrades which will support new floor slabs should be prepared in accordance with the recommendations contained in the ***Site Grading Recommendations*** section of this report. Based on the anticipated grading which will occur at this site, and based on the design

considerations presented in Section 6.1 of this report, the floor of the proposed structure may be constructed as a conventional slab-on-grade supported on existing or newly placed structural fill, extending to a depth of at least 5 feet below finished pad grade. Based on geotechnical considerations, the floor slabs may be designed as follows:

- Minimum slab thickness: 5 inches.

- Minimum slab reinforcement: No. 4 bars at 18-inches on-center, in both directions. The actual floor slab reinforcement should be determined by the structural engineer, based upon the imposed loading.

- Modulus of subgrade reaction: 100 lbs/in^3

- Slab underlayment: If moisture sensitive floor coverings are to be used, a moisture vapor barrier should be constructed below the entire slab area of the proposed garage. The moisture vapor barrier should meet or exceed the Class A rating as defined by ASTM E 1745-97 and have a permeance rating less than 0.01 perms as described in ASTM E 96-95 and ASTM E 154-88. The moisture vapor barrier should be properly constructed in accordance with all applicable manufacturer specifications. Given that a rock free subgrade is anticipated and that a capillary break is not required, sand below the barrier is not required. The need for sand and/or the amount of sand above the moisture vapor barrier should be specified by the structural engineer or concrete contractor. The selection of sand above the barrier is not a geotechnical engineering issue and hence outside our purview. Where moisture sensitive floor coverings are not anticipated, the vapor barrier may be 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. The steel reinforcement recommendations presented above are based on standard geotechnical practice, given the magnitude of predicted liquefaction-induced settlements, and the structure type proposed for this site. Additional rigidity may be necessary for structural considerations, or to resist the effects of the liquefaction-induced differential settlements discussed in Section 6.1

6.7 Exterior Flatwork Design and Construction

Subgrades which will support new exterior slabs-on-grade for patios, above grade skate park features, sidewalks and driveways should be prepared in accordance with the recommendations contained in the ***Grading Recommendations*** section of this report. These recommendations

assume that the flatwork will be underlain by a 2 foot thick layer of very low to non-expansive (EI < 20) structural fill. In the event that the owner elects to place the flatwork directly upon the existing subgrade soils, the geotechnical engineer should be contacted for supplementary recommendations. Based on geotechnical considerations, exterior slabs on grade 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 a moisture content of 2 to 4 percent above 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. **Minor cracking and/or movement of exterior concrete slabs on grade should be expected.**
- Expansion or felt joints should be used at the interface of exterior slabs on grade to permit relative movement.

Thickened Edges

Where the outer edges of concrete flatwork are to be bordered by landscaping, thickened edges should be used to prevent excessive infiltration and accumulation of water under the slabs. Thickened edges should be 6 to 8 inches wide, extend 12 inches below the tops of the finish slab surfaces, and be reinforced with a minimum of two No. 4 bars, one top and one bottom.

6.8 Retaining Wall Design and Construction

Skate park features consisting of below grade bowls and ramps will act as retaining walls and should be designed to withstand lateral earth pressures. These features are expected to extend to depths of 5 to 10 feet. It is expected that the retained soils will consist of structural fill soils or approved excavated native slopes. Additionally, although not indicated on the site plan, some small retaining walls (less than 4 to 5± 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. We have provided parameters for two different types of wall backfill: on-site soils and imported select aggregate base material. The near-surface on-site soils generally consist of loose silty sands and sandy silts and medium stiff silty clays and clayey silts. 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.

RETAINING WALL DESIGN PARAMETERS

Design Parameter		Soil Type	
		Imported Aggregate Base	Onsite soils
Internal Friction Angle (ϕ)		38°	26°
Unit Weight		130 lbs/ft^3	120 lbs/ft^3
Equivalent Fluid Pressure:	Active Condition (level backfill)	30 lbs/ft^3	47 lbs/ft^3
	Active Condition (2h:1v backfill)	44 lbs/ft^3	81 lbs/ft^3
	At-Rest Condition (level backfill)	50 lbs/ft^3	68 lbs/ft^3

Regardless of the backfill type, 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 addition to the lateral earth pressures presented above, the 2010 CBC requires that for structures assigned to Seismic Design Categories D through F, retaining walls should be designed for lateral earth pressures due to earthquake motion. However, since the bowl features and ramps are not associated with the proposed building, it may not be necessary to design these features for seismic lateral earth pressures. However, the recommended seismic pressure distribution has been provided. The recommended seismic pressure distribution is triangular in shape, with a maximum magnitude of 20H lbs/ft^2, where H is the overall height of the wall. The maximum pressure should be assumed to occur at the top of the wall, decreasing to 0 at the base of the wall. The seismic pressure distribution is based on the Mononobe-Okabe equation, utilizing a peak ground acceleration of 0.37g. This peak site acceleration is equal to $S_{DS}/2.5$, in accordance with the 2010 CBC.

Retaining Wall Foundation Design

The foundations for conventional retaining walls should be supported within newly placed compacted structural fill, extending to a depth of at least 2 feet below the proposed bearing grade. Foundations to support new retaining walls should be designed in accordance with the general Foundation Design Parameters presented in a previous section of this report. Since overexcavation is not practical in the areas of the proposed bowl features, these soils should be compacted at the exposed surface until they are firm and unyielding.

Backfill Material

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

It is recommended that a 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. For conventional retaining walls 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.

For the proposed bowl features, we expect that a synthetic drain panel will be the most practical drainage system. We recommend that a pressure relief valve be installed at the base of all the basins to help reduce the possibility of the bowl structure uplifting due to hydrostatic pressures from temporary build up of perched water beneath the skate park.

6.9 Pavement Design Parameters

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

Pavement Subgrades

It is anticipated that the new pavements will be primarily supported on a layer of compacted structural fill, consisting of scarified, thoroughly moisture conditioned and recompacted existing soils. The on-site soils generally consist of sandy silts and silty sands with areas of silty clay to clayey silt. Based on these subsurface conditions, an assumed R-value of 10 has been used for the subsequent pavement designs. Any fill material imported to the site should have support characteristics equal to or greater than that of the on-site soils and be placed and compacted under engineering controlled conditions. It is recommended that R-value testing be performed after completion of rough grading. Depending upon the results of the R-value testing, it may be feasible to use thinner pavement sections in some areas of the site.

Asphaltic Concrete

Presented below are the recommended thicknesses for new flexible pavement structures consisting of asphaltic concrete over a granular base. An alternate pavement section has been provided for use in parking stall areas due to the anticipated lower traffic intensity in these areas. However, truck traffic must be excluded from areas where the thinner pavement section is used; otherwise premature pavement distress may occur. 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.

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	3½	4
Aggregate Base	6	9	11
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.

7.0 GENERAL COMMENTS

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

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

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

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

8.0 REFERENCES

Blake, Thomas F., FRISKSP, A Computer Program for the Probabilistic Estimation of Peak Acceleration and Uniform Hazard Spectra Using 3-D Faults as Earthquake Sources, Version 4.00, 2000.

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

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

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

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

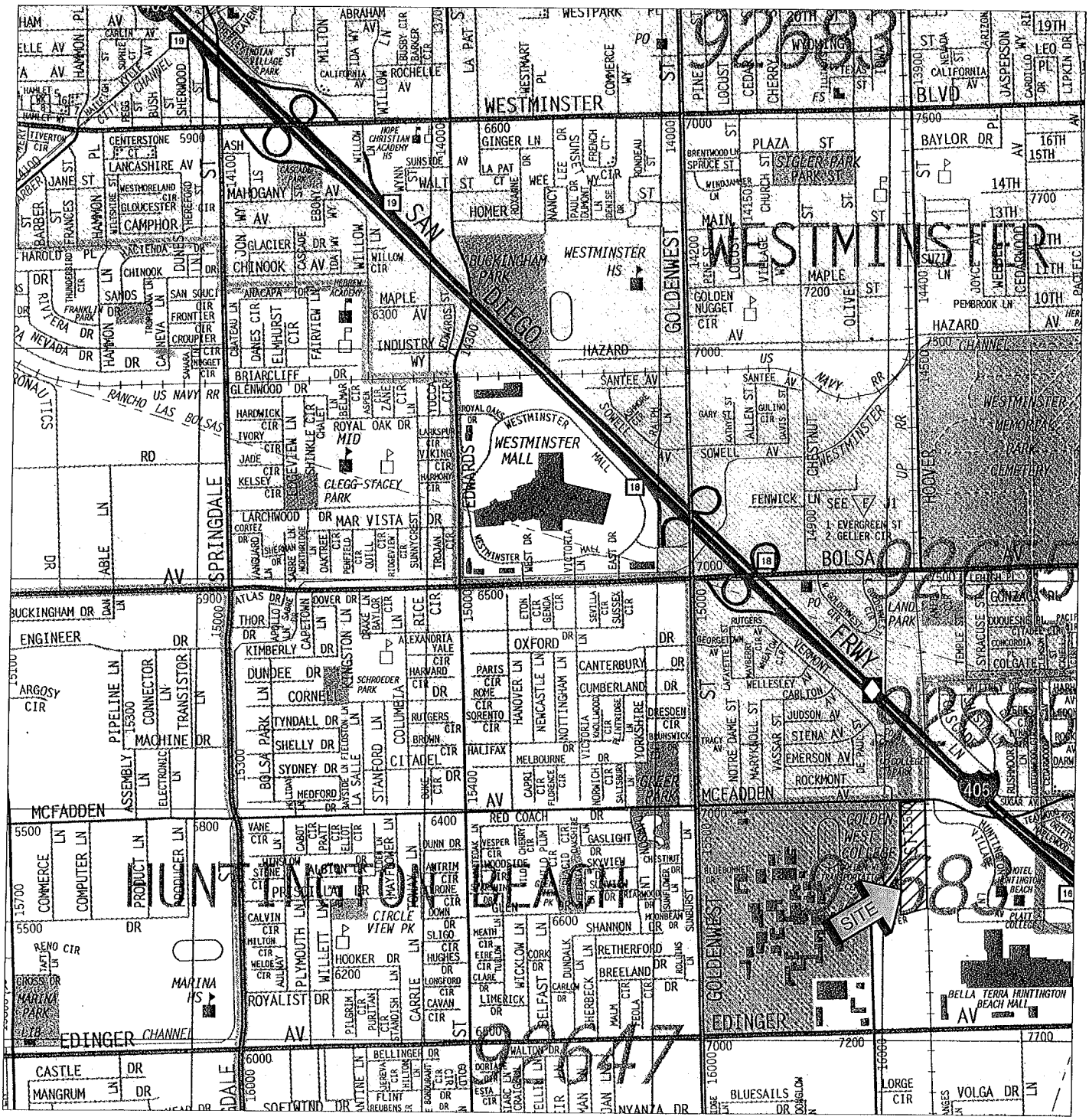
Sadigh, K., Chang, C. –Y., Egan, J. A., Makdisi. F., Youngs, R. R., "*Attenuation Relationships for Shallow Crustal Earthquakes Based on California Strong Motion Data*", Seismological Research Letters, Seismological Society of America, Volume 68, Number 1, January/ February 1997, pp. 180-189.

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

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

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

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



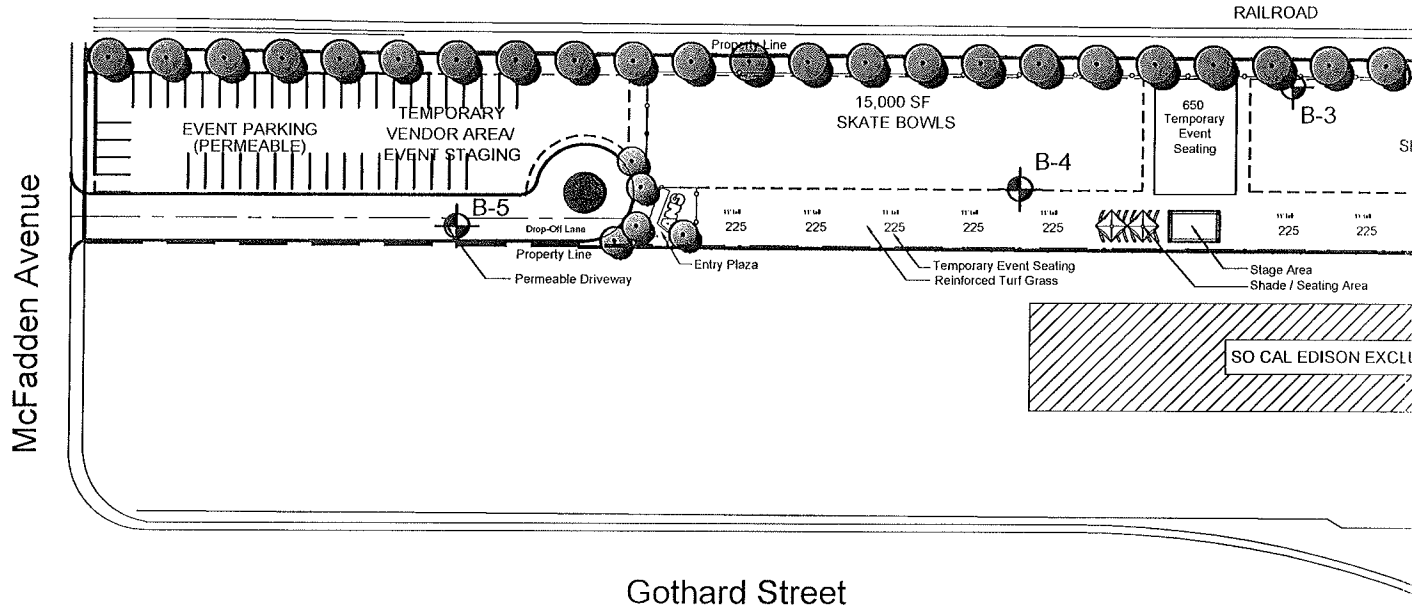
SOURCE: ORANGE COUNTY
THOMAS GUIDE, 2008

SITE LOCATION MAP
PROPOSED SKATE PARK AND RETAIL BUILDING
HUNTINGTON BEACH, CALIFORNIA
SCALE: 1" = 2400'
DRAWN: DWN CHKD: JAS
SCG PROJECT 11G113-1
PLATE 1

SoCalGeo

SOUTHERN CALIFORNIA GEOTECHNICAL

APPENDIX B



GEOTECHNICAL LEGEND

APPROXIMATE BORING LOCATION

NOTE: BASE MAP PREPAR

BORING LOG LEGEND

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

COLUMN DESCRIPTIONS

DEPTH: Distance in feet below the ground surface.

SAMPLE: Sample Type as depicted above.

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

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

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

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

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

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

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

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

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

SOIL CLASSIFICATION CHART

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

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-1

JOB NO.: 11G113
PROJECT: Skate Park and Retail Buildings
LOCATION: Huntington Beach, California

DRILLING DATE: 3/23/11
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Daniel Nielsen

WATER DEPTH: Dry
CAVE DEPTH: 7 feet
READING TAKEN: At Completion

SURFACE ELEVATION: --- MSL

FIELD RESULTS					DESCRIPTION	LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG		DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
	X	14			ALLUVIUM: Dark Gray Brown fine Sandy Silt, little Clay, some Organics and Peat deposits, Organic odor, loose-damp to moist	104	16					EI = 65 @ 0 to 5'
	X	9			Gray Brown Silty fine Sand, little Iron oxide staining, loose-damp to moist	88	15					
5	X	8	2.0			80	33					
	X	10	1.5		Dark Gray Brown to Black Silty Clay, trace Iron oxide staining, trace to little Organics medium stiff to stiff-damp	78	42					
10	X	12			Gray Brown Silty fine Sand to fine Sandy Silt, loose-moist to very moist	88	33					
					Gray Brown fine Sandy Silt, trace Iron oxide staining, loose-wet							
	X	6					36			81		
15			0.75		@ 14 feet, Groundwater encountered during drilling Dark Gray Brown Silty Clay, medium stiff-very moist		60			92		
					Dark Gray Brown fine Sandy Silt, medium dense-very moist							
	X	9					31			66		
20			0.75		Dark Gray Brown to Black Silty Clay, slight Organic odor, stiff-very moist		45	61	28	89		
	X	9			Light Gray Brown Silty fine Sand, loose to medium dense-wet		29			33		
25												
	X	10					25			23		
30												
					Gray Brown Clayey Silt, medium stiff-very moist to wet							
	X	8	1.75				45	46	27	94		

TBL 11G113.GPJ SOCALGEO.GDT 4/12/11

TEST BORING LOG

PLATE B-1a

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-1

JOB NO.: 11G113
PROJECT: Skate Park and Retail Buildings
LOCATION: Huntington Beach, California

DRILLING DATE: 3/23/11
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Daniel Nielsen

WATER DEPTH: Dry
CAVE DEPTH: 7 feet
READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION (Continued)	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					Gray Brown Clayey Silt, medium stiff-very moist to wet							
					Gray Brown fine Sandy Silt, trace to little Clay, loose to medium dense-wet							
	X	9					29	44	28	67		
40												
	X	12					35			70		
45			2.0		Gray Brown Silty Clay, medium stiff-very moist		42	51	27	97		
	X	18			Gray Brown Silty fine Sand to fine Sandy Silt, medium dense-wet		34			48		
50												
					Boring Terminated at 50'							

TBL 11G113.GPJ SOCALGEO.GDT 4/12/11

TEST BORING LOG

PLATE B-1b

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-2

JOB NO.: 11G113
PROJECT: Skate Park and Retail Buildings
LOCATION: Huntington Beach, California

DRILLING DATE: 3/23/11
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Daniel Nielsen

WATER DEPTH: Dry
CAVE DEPTH: 8 feet
READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
	X	13	4.0		FILL: Brown Clayey fine Sand and fine Sandy Clay, little fine Gravel, mottled, loose to medium stiff-moist	102	16					
	X	8	4.0		ALLUVIUM: Brown Clayey fine Sand to fine Sandy Clay, trace coarse Sand, trace Organics, loose to medium stiff-moist	99	20					
5	X	9			Light Gray Brown Silty fine Sand to fine Sandy Silt, loose-damp	70	12					
	X	11	2.75		Dark Gray Brown to Black Silty Clay, little Iron oxide staining, trace Organics, medium stiff-very moist	77	40					
10	X	12	1.75			86	36					
					Boring Terminated at 10'							

TBL 11G113.GPJ SOCALGEO.GDT 4/12/11

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-3

JOB NO.: 11G113
PROJECT: Skate Park and Retail Buildings
LOCATION: Huntington Beach, California

DRILLING DATE: 3/23/11
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Daniel Nielsen

WATER DEPTH: Dry
CAVE DEPTH: 8 feet
READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					FILL: Brown fine Sandy Silt, some Clay, little fine Gravel, loose-moist							
	X	9	4.5+		ALLUVIUM: Dark Brown fine Sandy Clay, loose-moist		18 16					
	X	7			Light Gray Brown Silty fine Sand, trace calcareous veining, little Iron oxide staining, loose-damp to moist		7					
5												
	X	5			Gray Brown to Brown Silty fine Sand, loose-very moist to wet		36					
			2.5		Brown Silty Clay, trace to little Peat deposits, loose-very moist		40					
	X	6	2.25		Dark Gray Brown fine Sandy Silt, loose-very moist		36 22					
10												
					@ 12 feet, Groundwater encountered during drilling Gray Brown fine Sand, loose-wet							
	X	5			Interbedded Gray Brown Clayey Silt and Silty fine Sand, loose-very moist		35 48					
15												
					Dark Gray Brown Silty Clay, trace Organics, medium stiff-wet							
	X	7	2.25				40					
20												
					Boring Terminated at 20'							

TBL 11G113.GPJ SOCALGEO.GDT 4/12/11

TEST BORING LOG

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-4

JOB NO.: 11G113
PROJECT: Skate Park and Retail Buildings
LOCATION: Huntington Beach, California

DRILLING DATE: 3/23/11
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Daniel Nielsen

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

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
	X	10			FILL: Brown to Dark Brown Silty fine Sand, trace Wood fragments, trace Portland cement concrete fragments, loose to medium dense-moist		9					
	X	9			ALLUVIUM: Light Gray Brown fine Sand, loose-dry to damp		4					
5												
	X	7	3.75		Brown Silty Clay to fine Sandy Clay, trace calcareous veining, trace Peat deposits, some Iron oxide staining, medium stiff-very moist		35					
	X	6	2.0				34					
10												
	X	8			Gray Brown to Brown Silty fine Sand to fine Sandy Silt, loose-wet @ 13 feet, Groundwater encountered during drilling		32					
15												
					Boring Terminated at 15'							

TBL 11G113.GPJ SOCALGEO.GDT 4/12/11

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-5

JOB NO.: 11G113
PROJECT: Skate Park and Retail Buildings
LOCATION: Huntington Beach, California

DRILLING DATE: 3/23/11
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Daniel Nielsen

WATER DEPTH: Dry
CAVE DEPTH: 3.5 feet
READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
	X	4			FILL: Brown Silty fine Sand to fine Sandy Silt, little Clay, soft to very loose-damp to moist		11					
	X	9			ALLUVIUM: Light Gray to Dark Gray Brown Silty fine Sand to fine Sandy Silt, trace Clay, loose-damp		13					
5					Boring Terminated at 5'							

TBL 11G113.GPJ SOCALGEO.GDT 4/12/11

TEST BORING LOG

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-6

JOB NO.: 11G113
PROJECT: Skate Park and Retail Buildings
LOCATION: Huntington Beach, California

DRILLING DATE: 3/23/11
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Daniel Nielsen

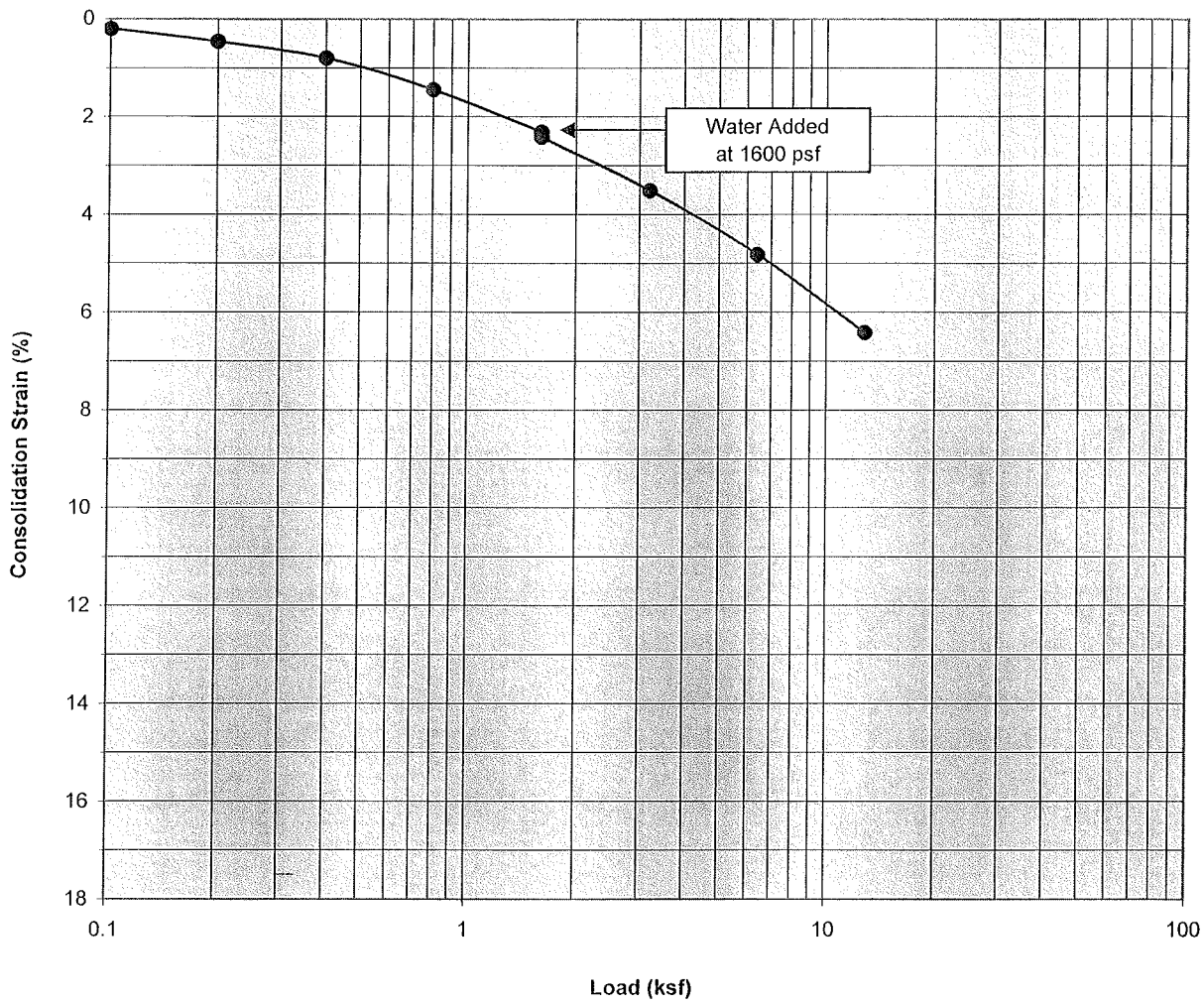
WATER DEPTH: Dry
CAVE DEPTH: 3.5 feet
READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
	X	10			FILL: Dark Gray Brown to Dark Brown Silty fine Sand to fine Sandy Silt, some trace fine root fibers, trace Wood fragments, loose to medium dense-moist		18					
	X	12			ALLUVIUM: Light Gray Brown fine Sandy Silt, trace Iron oxide staining, medium dense-damp		10					
5					Boring Terminated at 5'							

TBL 11G113.GPJ SOCALGEO.GDT 4/12/11

APPENDIX C

Consolidation/Collapse Test Results



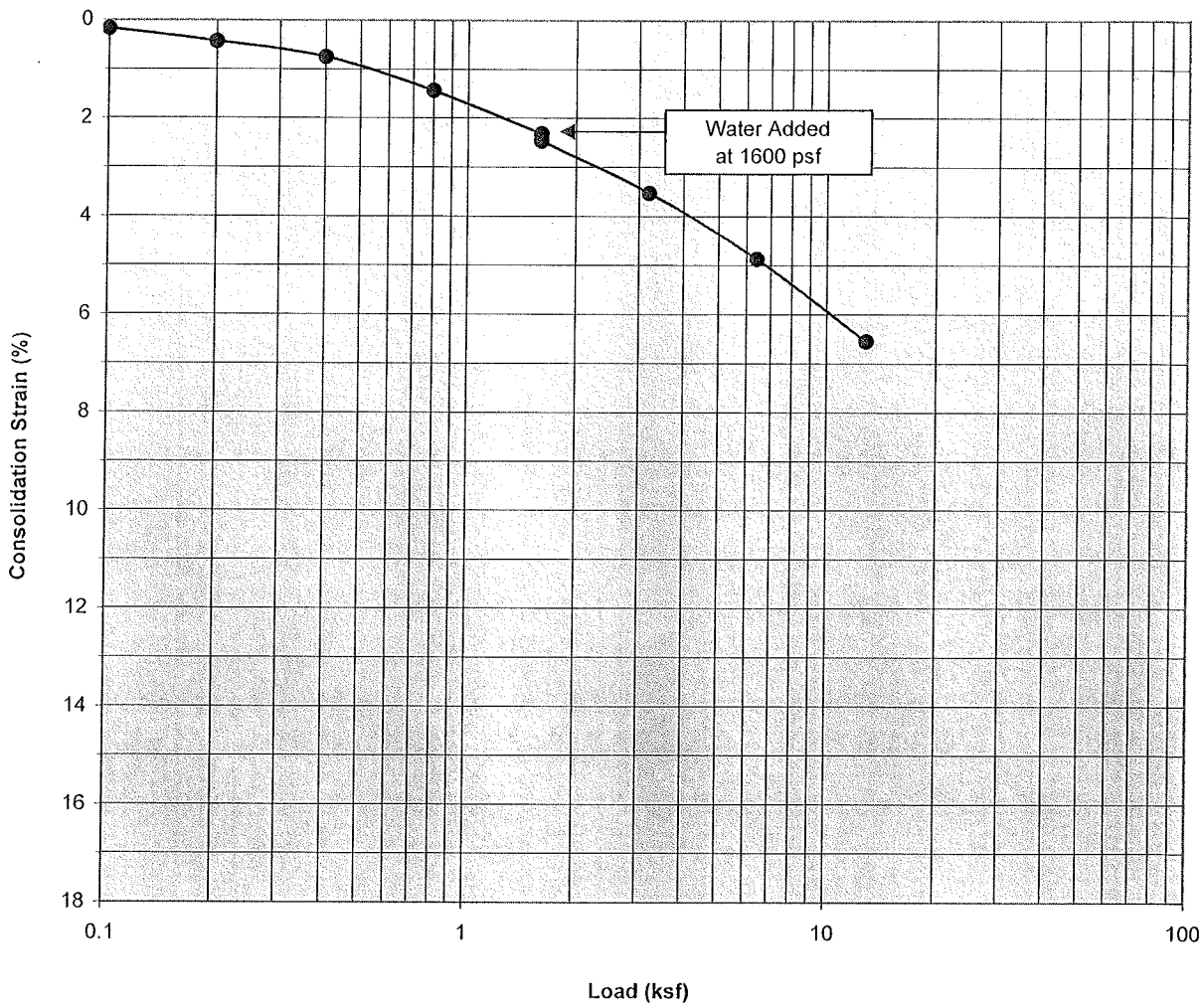
Classification: Dark Gray Brown Silty fine Sand to fine Sandy Silt

Boring Number:	B-1	Initial Moisture Content (%)	15
Sample Number:	---	Final Moisture Content (%)	19
Depth (ft)	1 to 2	Initial Dry Density (pcf)	103.7
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	109.1
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.11

Skate Park and Retail Buildings
Huntington Beach, California
Project No. 11G113
PLATE C- 1

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL

Consolidation/Collapse Test Results



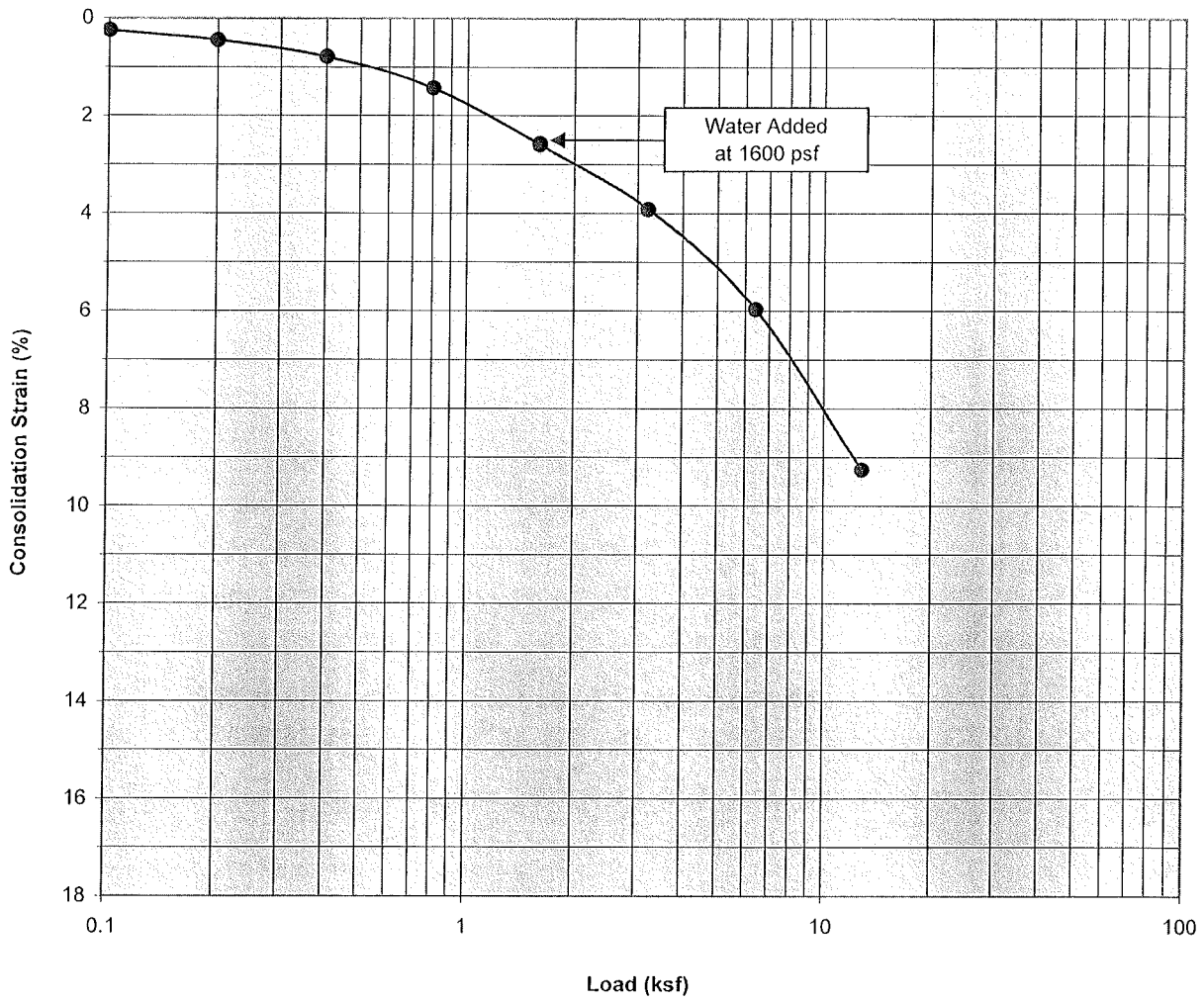
Classification: Gray Brown Silty fine Sand

Boring Number:	B-1	Initial Moisture Content (%)	17
Sample Number:	---	Final Moisture Content (%)	29
Depth (ft)	3 to 4	Initial Dry Density (pcf)	89.6
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	96.4
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.15

Skate Park and Retail Buildings
Huntington Beach, California
Project No. 11G113
PLATE C- 2

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL

Consolidation/Collapse Test Results



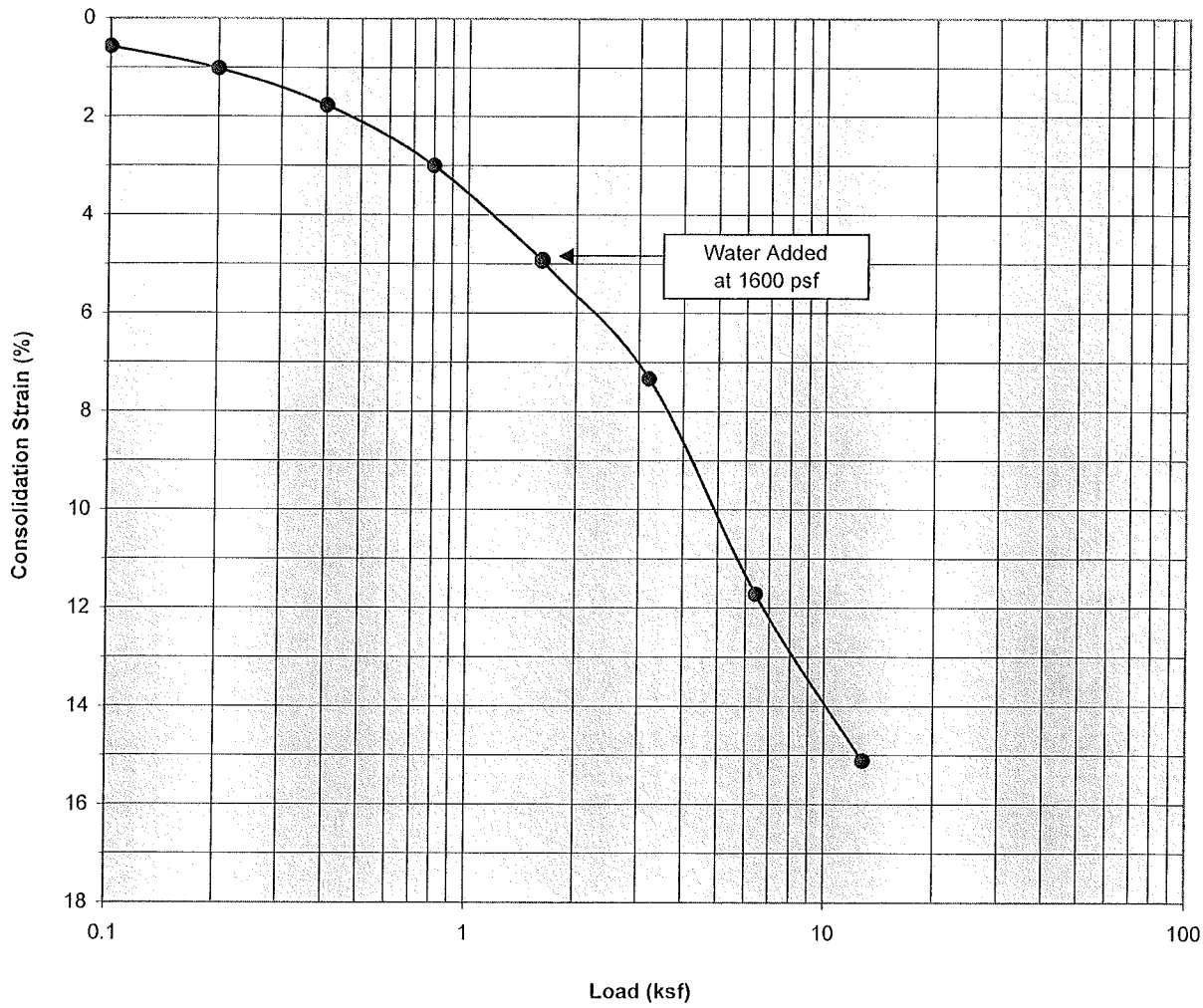
Classification: Gray Brown Silty fine Sand

Boring Number:	B-1	Initial Moisture Content (%)	32
Sample Number:	---	Final Moisture Content (%)	37
Depth (ft)	5 to 6	Initial Dry Density (pcf)	78.7
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	85.5
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.02

Skate Park and Retail Buildings
Huntington Beach, California
Project No. 11G113
PLATE C- 3

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL

Consolidation/Collapse Test Results



Classification: Gray Brown Silty fine Sand

Boring Number:	B-1	Initial Moisture Content (%)	43
Sample Number:	---	Final Moisture Content (%)	38
Depth (ft)	7 to 8	Initial Dry Density (pcf)	78.1
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	92.5
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.05

Skate Park and Retail Buildings
Huntington Beach, California
Project No. 11G113
PLATE C- 4

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL

APPENDIX D

GRADING GUIDE SPECIFICATIONS

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

General

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

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

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

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

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

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

Site Preparation

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

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

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

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

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

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

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

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

Compacted Fills

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

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

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

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

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

  - Fill materials consisting of soil meeting the minimum moisture content requirements and free of oversize material should be placed between and over the rows of rock or

concrete. Ample water and compactive effort should be applied to the fill materials as they are placed in order that all of the voids between each of the fragments are filled and compacted to the specified density.

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

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

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

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

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

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

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

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

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

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

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

Foundations

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

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

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

Fill Slopes

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

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

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

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

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

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

Cut Slopes

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

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

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

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

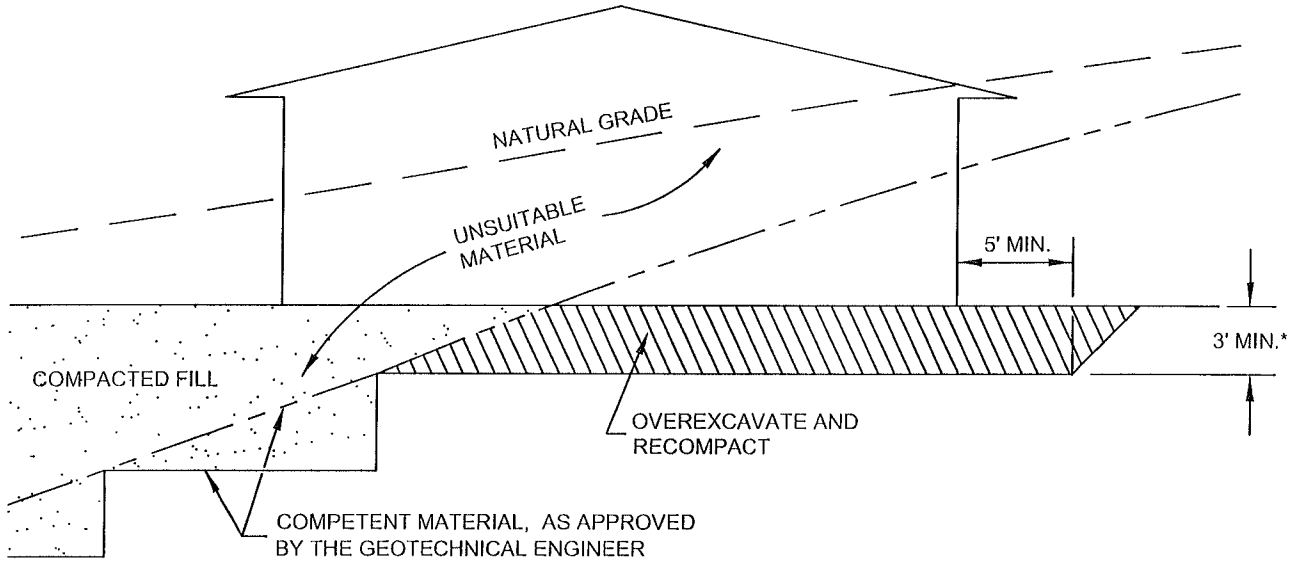
Subdrains

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

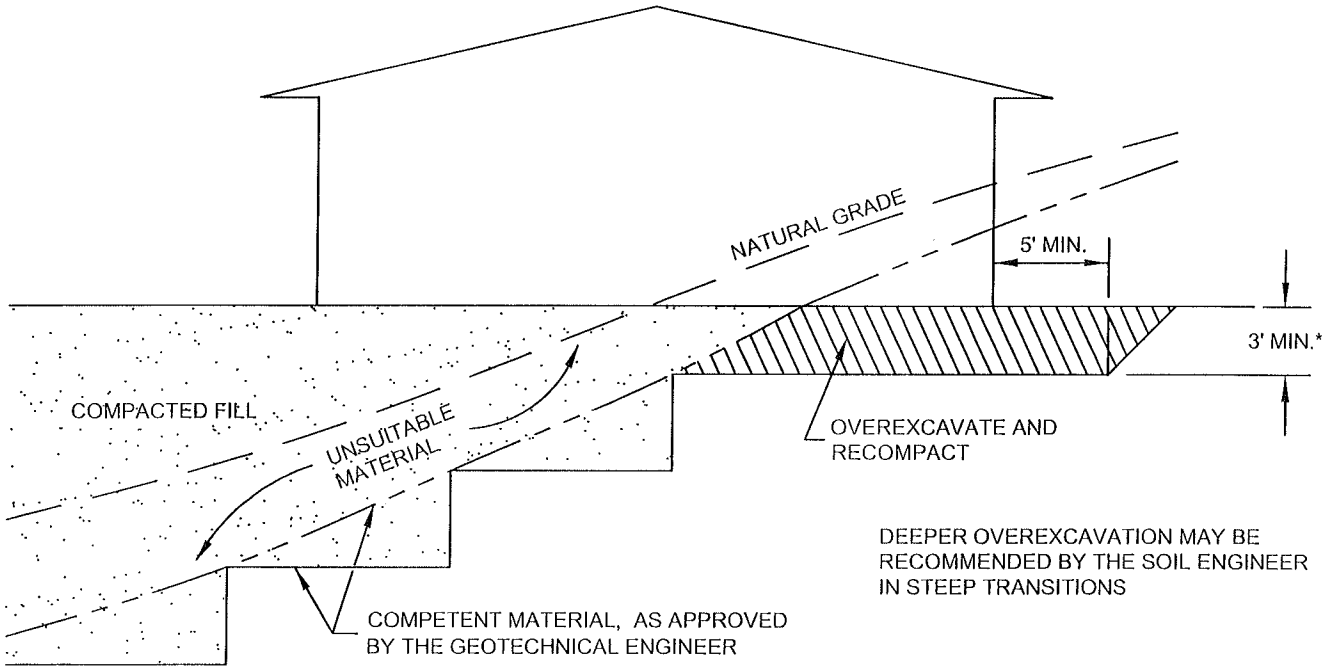
- Plastic pipe may be used for subdrains provided it is Schedule 40 or SDR 35 or equivalent. Pipe should be protected against breakage, typically by placement in a square-cut (backhoe) trench or as recommended by the manufacturer.

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

CUT LOT



CUT/FILL LOT (TRANSITION)



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

TRANSITION LOT DETAIL

GRADING GUIDE SPECIFICATIONS

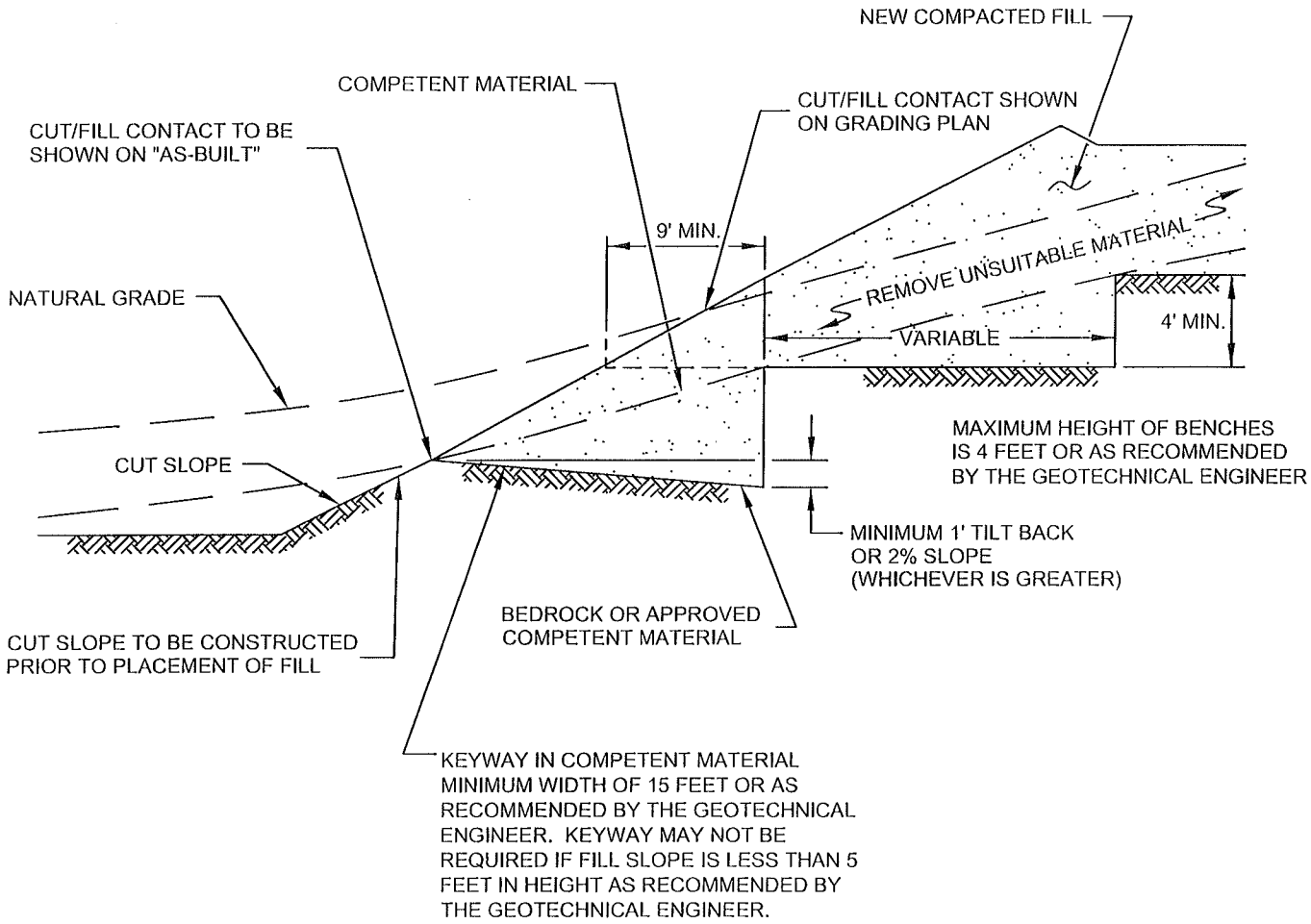
NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-1

SoCalGeo

SOUTHERN CALIFORNIA GEOTECHNICAL



FILL ABOVE CUT SLOPE DETAIL

GRADING GUIDE SPECIFICATIONS

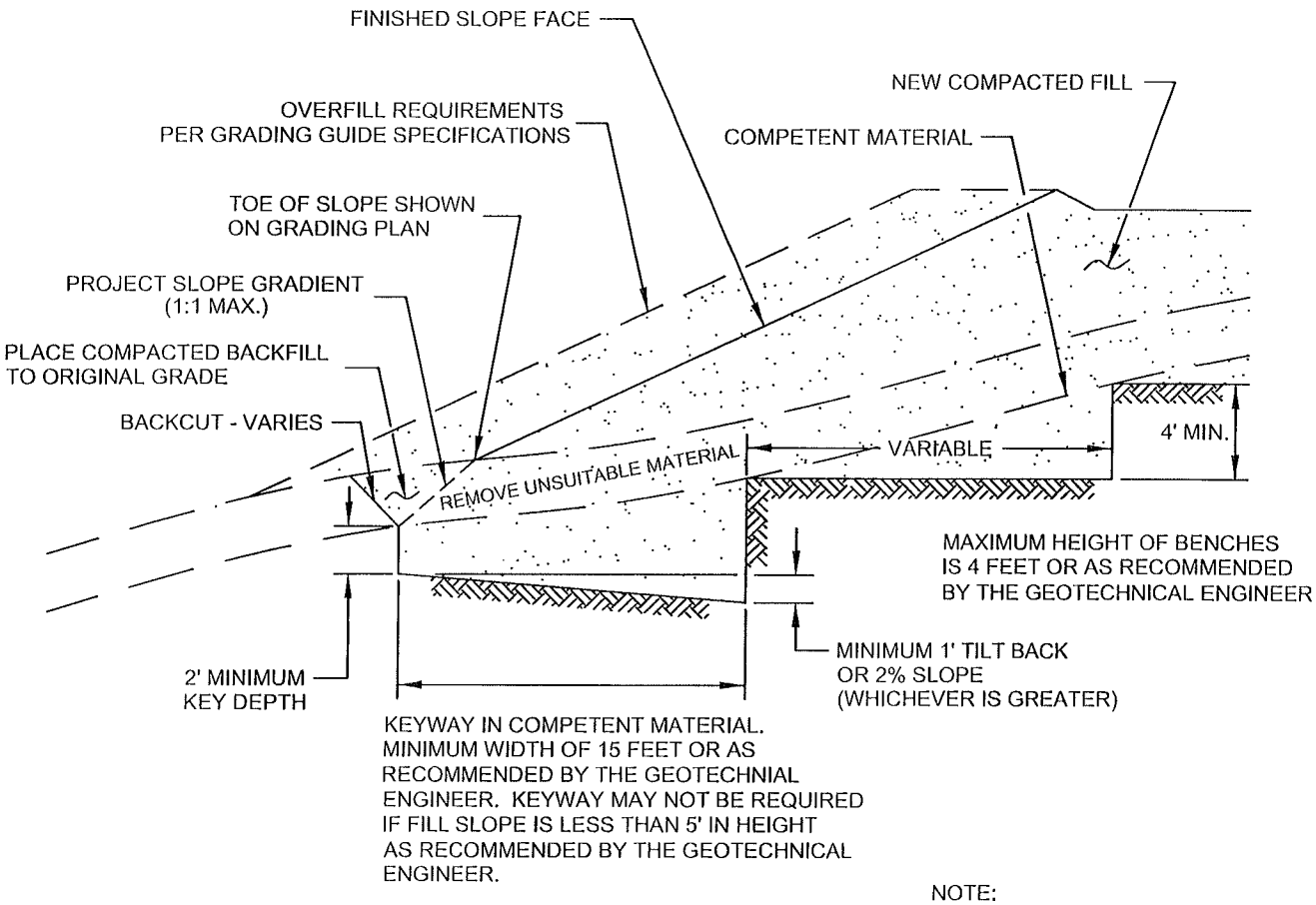
NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-2

SoCalGeo

SOUTHERN CALIFORNIA GEOTECHNICAL



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

FILL ABOVE NATURAL SLOPE DETAIL

GRADING GUIDE SPECIFICATIONS

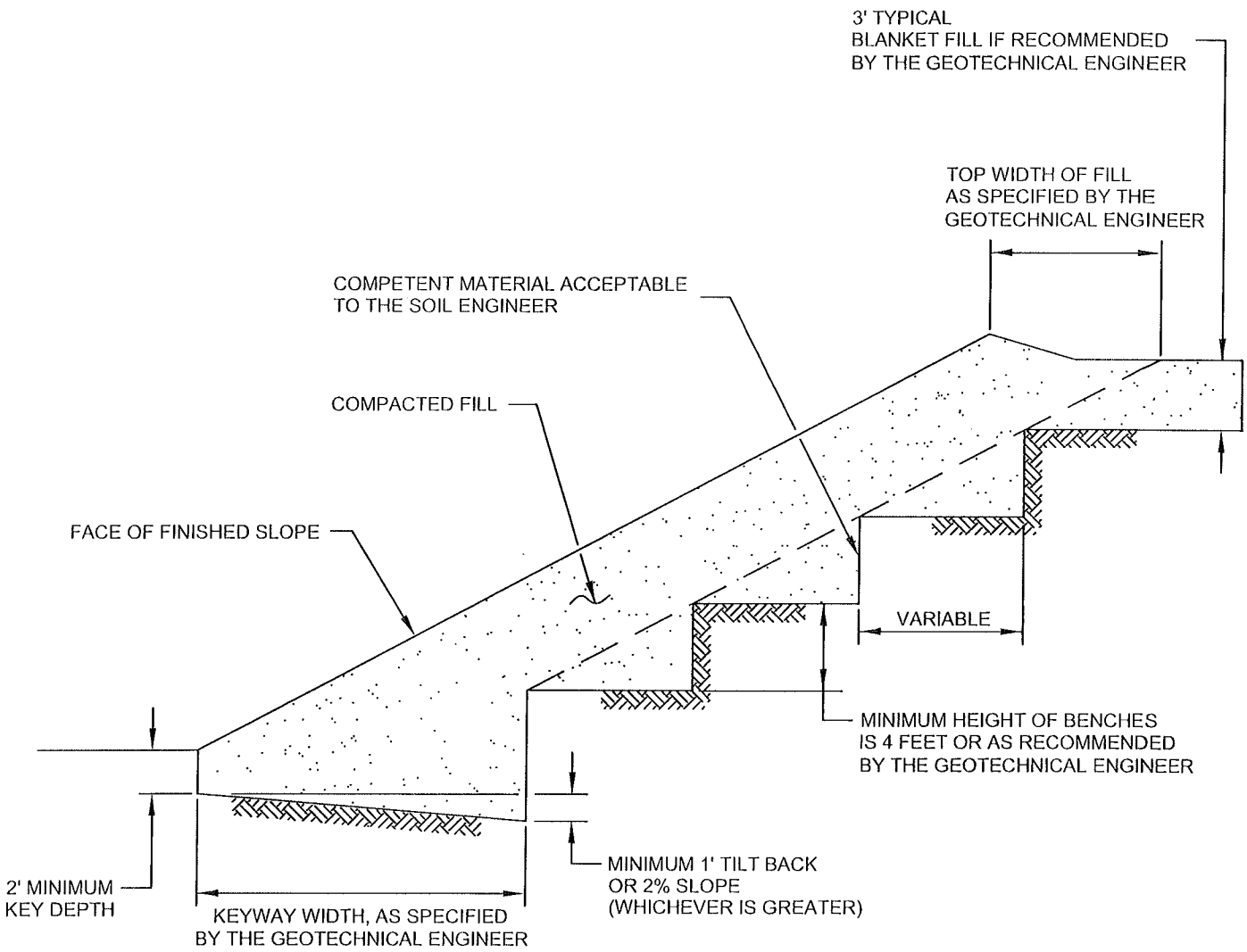
NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-4

SoCalGeo

SOUTHERN CALIFORNIA GEOTECHNICAL



STABILIZATION FILL DETAIL

GRADING GUIDE SPECIFICATIONS

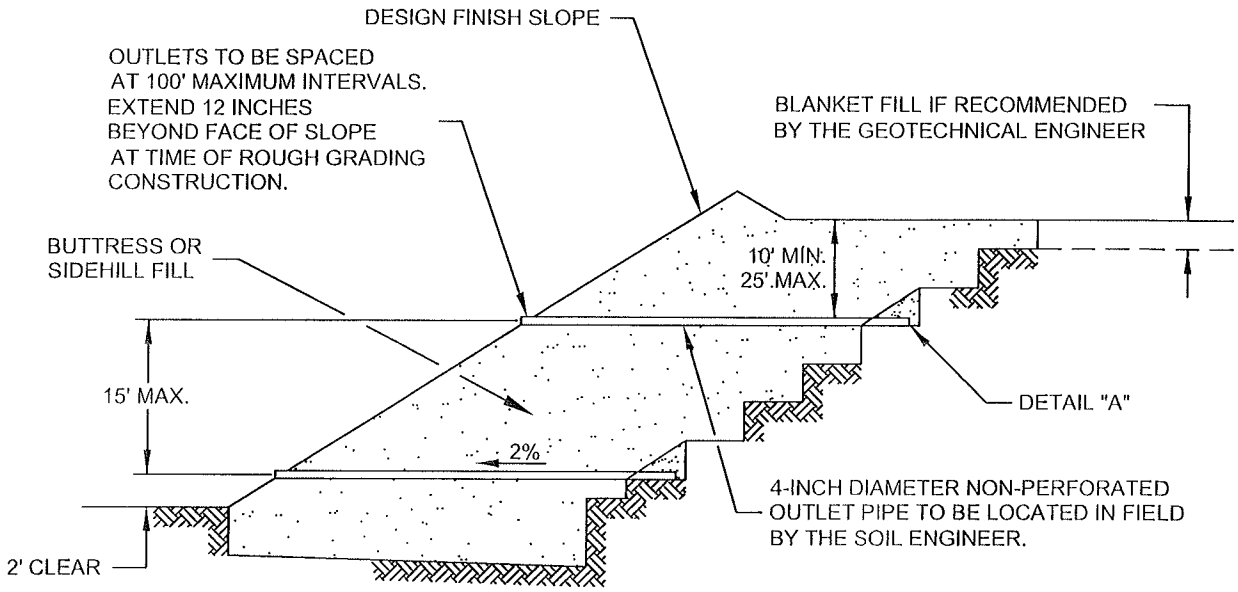
NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-5

SoCalGeo

SOUTHERN
CALIFORNIA
GEOTECHNICAL



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

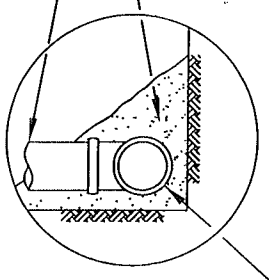
SIEVE SIZE	PERCENTAGE PASSING
1"	100
3/4"	90-100
3/8"	40-100
NO. 4	25-40
NO. 8	18-33
NO. 30	5-15
NO. 50	0-7
NO. 200	0-3

"GRAVEL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT:

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

SAND EQUIVALENT = MINIMUM OF 50

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



DETAIL "A"

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

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

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

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

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

SLOPE FILL SUBDRAINS

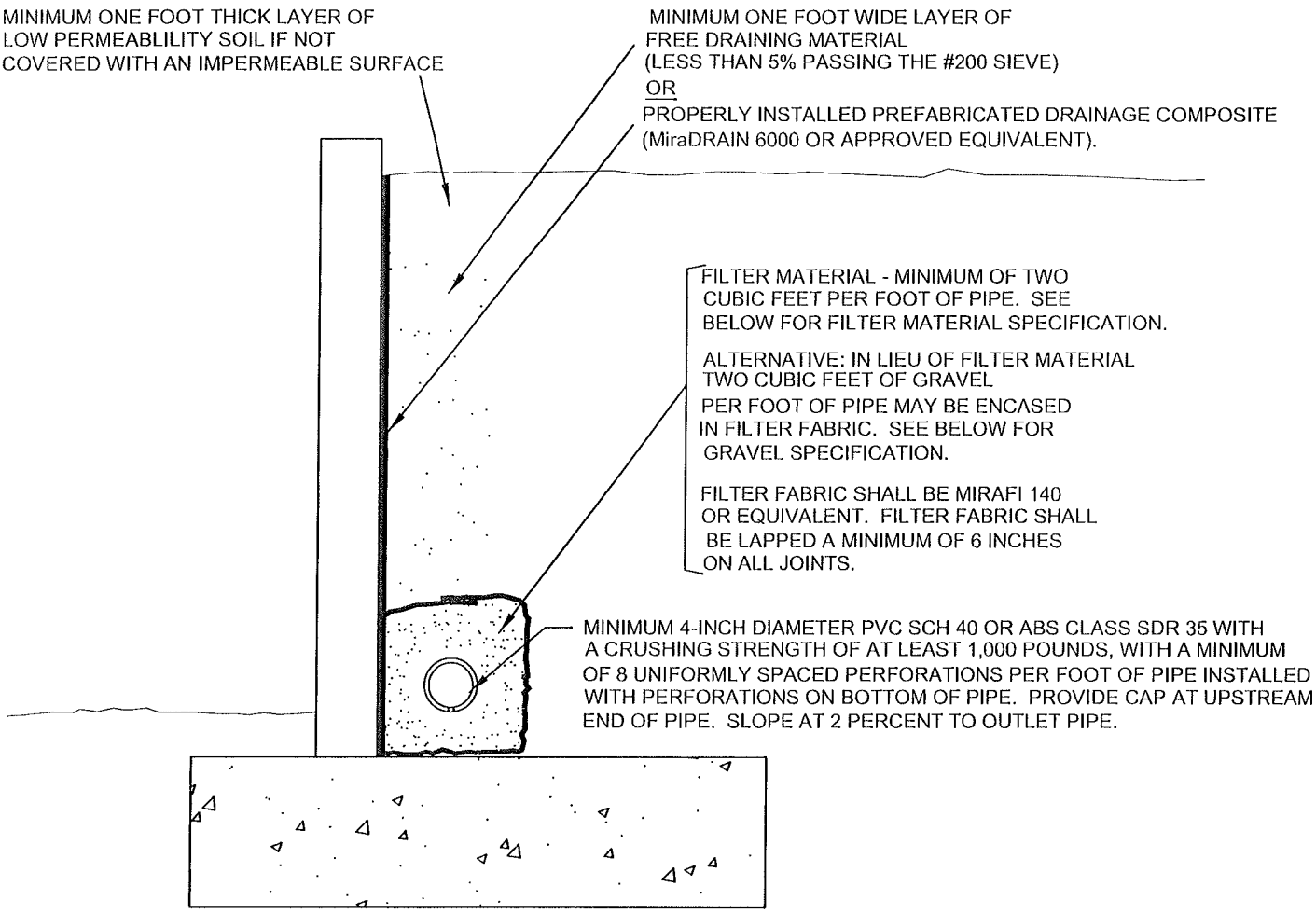
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-6

SOUTHERN CALIFORNIA GEOTECHNICAL



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

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

"GRAVEL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT:

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

SAND EQUIVALENT = MINIMUM OF 50

RETAINING WALL BACKDRAINS

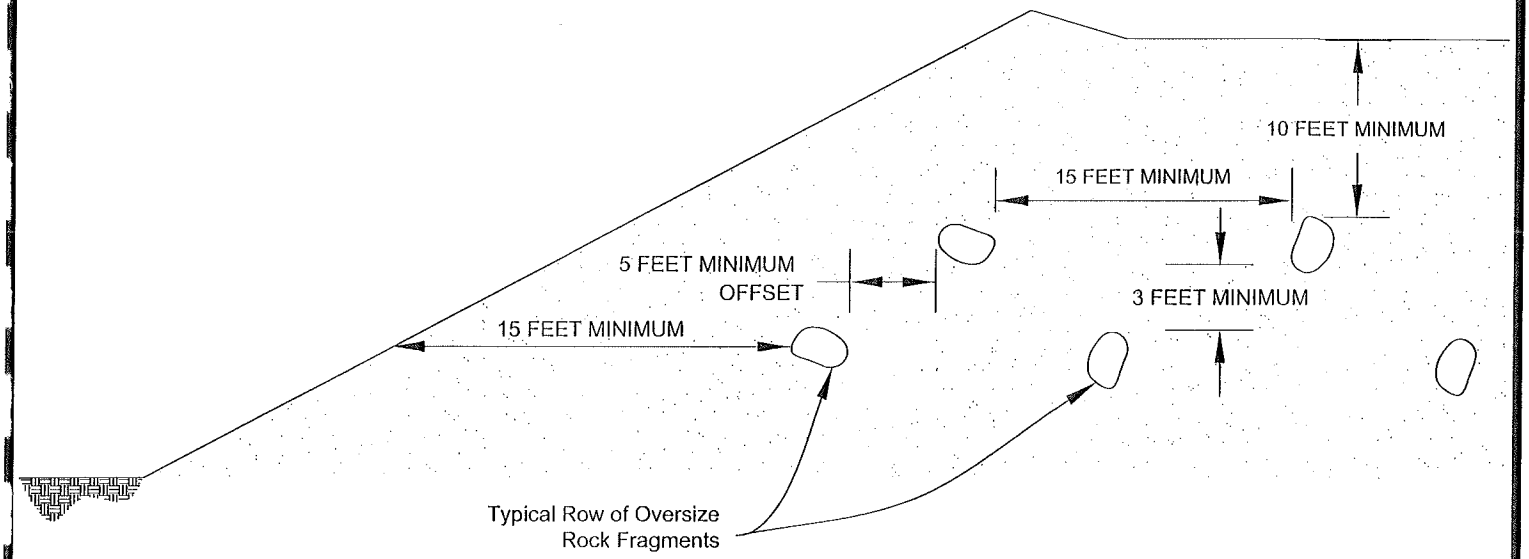
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

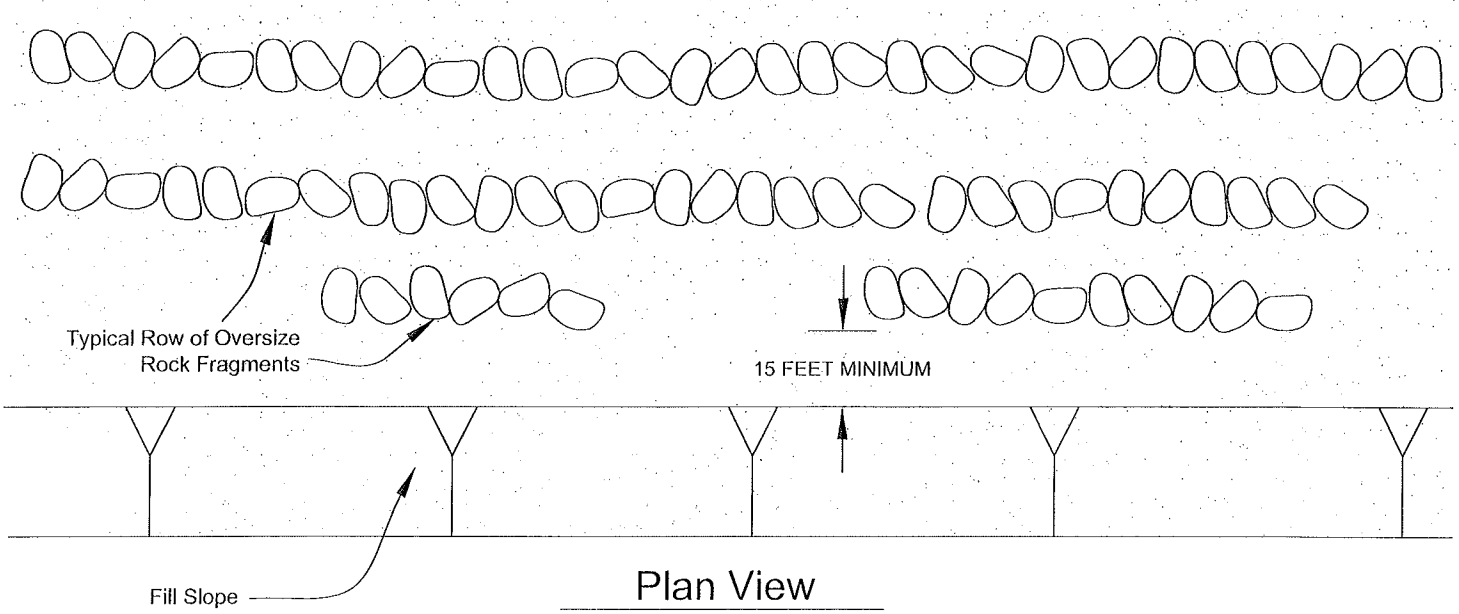
DRAWN: JAS
CHKD: GKM

PLATE D-7

SOUTHERN CALIFORNIA GEOTECHNICAL



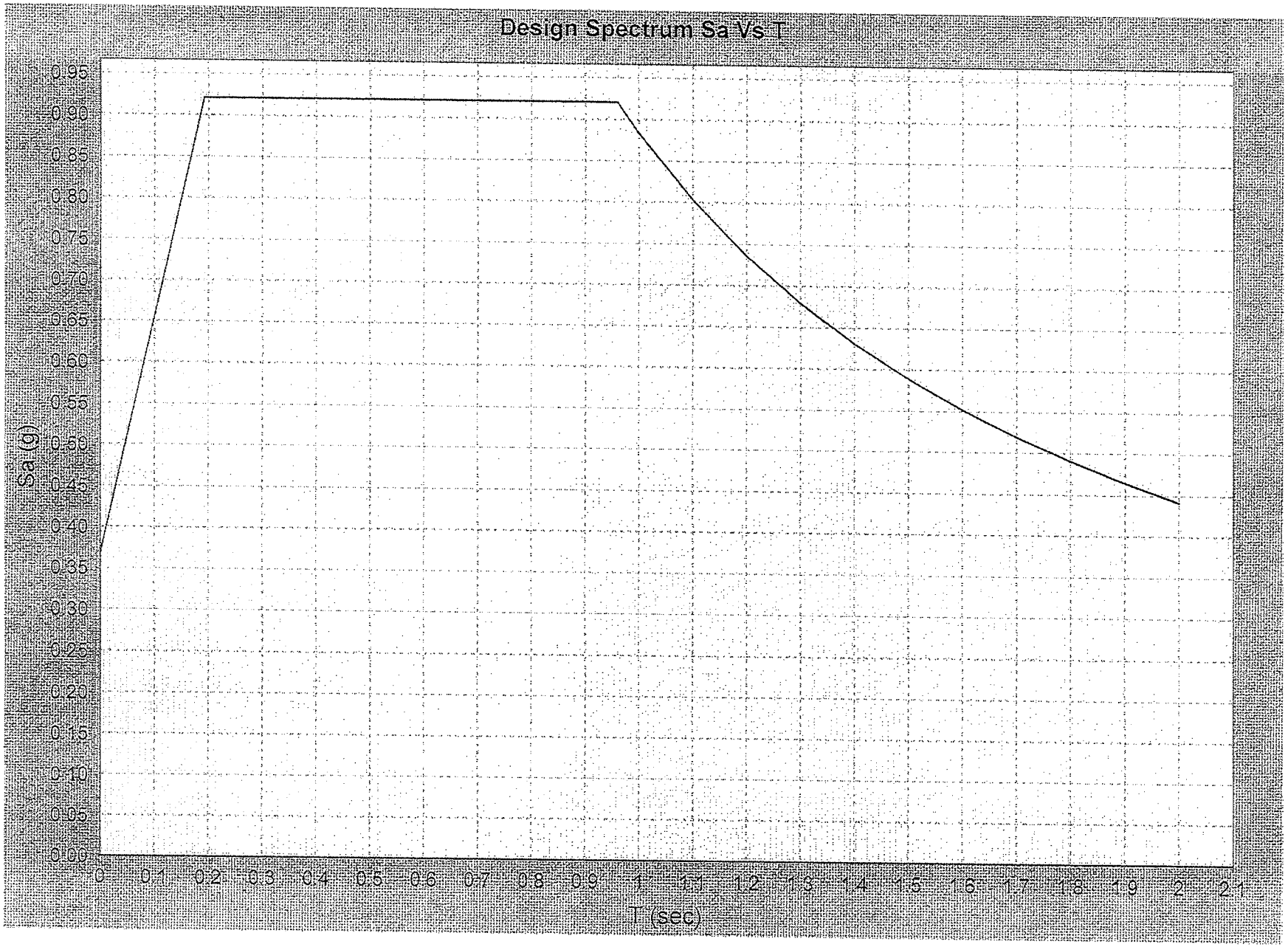
Section View



Plan View

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

APPENDIX E


Design Spectrum Sa Vs T
Sa (g)
0.95
0.90
0.85
0.80
0.75
0.70
0.65
0.60
0.55
0.50
0.45
0.40
0.35
0.30
0.25
0.20
0.15
0.10
0.05
0.00
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1
1.1
1.2
1.3
1.4
1.5
1.6
1.7
1.8
1.9
2
2.1
T (sec)

Conterminous 48 States
2009 International Building Code
Latitude = 33.734234
Longitude = -117.99840500000002
Spectral Response Accelerations Ss and S1
Ss and S1 = Mapped Spectral Acceleration Values
Site Class B - Fa = 1.0 ,Fv = 1.0
Data are based on a 0.01 deg grid spacing

Period (sec)	Sa (g)	
0.2	1.534	(Ss, Site Class B)
1.0	0.552	(S1, Site Class B)

Conterminous 48 States
2009 International Building Code
Latitude = 33.734234
Longitude = -117.99840500000002
Spectral Response Accelerations SMs and SM1
SMs = Fa x Ss and SM1 = Fv x S1
Site Class E - Fa = 0.9 ,Fv = 2.4

Period (sec)	Sa (g)	
0.2	1.381	(SMs, Site Class E)
1.0	1.325	(SM1, Site Class E)

Conterminous 48 States
2009 International Building Code
Latitude = 33.734234
Longitude = -117.99840500000002
Design Spectral Response Accelerations SDs and SD1
SDs = 2/3 x SMs and SD1 = 2/3 x SM1
Site Class E - Fa = 0.9 ,Fv = 2.4

Period (sec)	Sa (g)	
0.2	0.920	(SDs, Site Class E)
1.0	0.883	(SD1, Site Class E)

LIQUEFACTION EVALUATION

Project Name	Skate Park and Retail Bldg
Project Location	Huntington Beach
Project Number	11G113
Engineer	DWN

Design Acceleration	0.37	(g)
Design Magnitude	6.8	
Historic High Depth to Groundwater	7	(ft)
Current Depth to Groundwater	70	(ft)

Boring No. B-1

Sample Depth (ft)	Depth to Top of Layer (ft)	Depth to Bottom of Layer (ft)	Depth to Midpoint (ft)	Uncorrected N-Value	Unit Weight of Soil (pcf)	Fines Content (%)	Energy Correction	C_N	Rod Length Correction	$(N_1)_{60}$	$(N_1)_{60CS}$	Overburden Stress (σ_o) (psf)	Eff. Overburden Stress (Hist. Water) (σ_o') (psf)	Eff. Overburden Stress (Curr. Water) (σ_o') (psf)	Stress Reduction Coefficient (r_d)	Cyclic Stress Ratio to Cause Liquefaction (M=7.5)	Cyclic Stress Ratio to Cause Liquefaction (M=6.8)	Cyclic Stress Ratio Induced by Design Earthquake	Factor of Safety	Comments
							(1)	(2)	(3)	(4)	(5)				(6)	(7)	(8)	(9)		
6	0	7	3.5		120		1.3	2.18	0.75	0.0	0.0	420	420	420	0.99	0.05	0.06	0.24	N/A	Above Water Table
8.5	8	11	9.5	9	120		1.3	1.32	0.75	11.6	11.6	1140	984	1140	0.98	0.13	0.16	0.27	0.59	**Liquefiable**
14	11	14.5	12.75	6	120	40	1.3	1.14	0.85	7.6	14.1	1530	1171	1530	0.97	0.15	0.20	0.30	N/A	Non-liquefiable: PI>12
14.5	14.5	17	15.75	6	120	92	1.3	1.03	0.85	6.8	13.2	1890	1344	1890	0.96	0.14	0.18	0.33	N/A	Non-liquefiable: PI>12
19	17	19	18	9	120	66	1.3	0.96	0.95	10.7	17.8	2160	1474	2160	0.96	0.19	0.25	0.34	0.73	**Liquefiable**
19.5	19	23	21	9	120	89	1.3	0.89	0.95	9.9	16.9	2520	1646	2520	0.95	0.18	0.23	0.35	N/A	Non-liquefiable: PI>12
24.5	23	27	25	9	120	33	1.3	0.82	0.95	9.1	15.5	3000	1877	3000	0.94	0.17	0.22	0.36	0.60	**Liquefiable**
29.5	27	31	29	10	120	23	1.3	0.76	0.95	9.4	14.3	3480	2107	3480	0.93	0.15	0.20	0.37	0.54	**Liquefiable**
34.5	31	37	34	8	120	94	1.3	0.70	1	7.3	13.7	4080	2395	4080	0.90	0.15	0.19	0.37	N/A	Non-liquefiable: PI>12
39.5	37	42	39.5	9	120	67	1.3	0.65	1	7.6	14.1	4740	2712	4740	0.85	0.15	0.20	0.36	N/A	Non-liquefiable: PI>12
44	42	44	43	12	120	70	1.3	0.62	1	9.7	16.7	5160	2914	5160	0.82	0.18	0.23	0.35	N/A	Non-liquefiable: PI>12
44.5	44	48	46	12	120	97	1.3	0.60	1	9.4	16.3	5520	3086	5520	0.80	0.18	0.23	0.34	N/A	Non-liquefiable: PI>12
49.5	48	50	49	18	120	48	1.3	0.58	1	13.6	21.4	5880	3259	5880	0.78	0.23	0.30	0.34	0.89	**Liquefiable**

Notes:

* Assumed

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

(2) Overburden Correction, Lao and Whitman, 1986, $C_N = (2.0 \text{ ksf} / p'_o)^{1/2}$

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

(4) N-value corrected for energy, rod length, and overburden

(5) N-value corrected for fines content per Eq. 5 (Youd and Idriss, 1997). Allows use of base curve, Fig 2 (Youd and Idriss, 1997)

(6) Calculated by Eq. 2 (Youd and Idriss, 1997), gives same results as Fig 40 of Seed and Idriss, ASCE, September 1971

(7) Per Figure 2, base curve (Youd and Idriss, 1997) using $(N_1)_{60CS}$. Curve also presented as Fig 7.1 (SCEC, 1997). INDET indicates that the $(N_1)_{60}$ plots to the right of the vertical portion of the base curve, and the Cyclic Stress Ratio required to induce liquefaction is indeterminant. The layer is non-liquefiable.

(8) Corrected for Magnitude Weighting using revised Idriss factors (Fig 12, Youd and Idriss (1997) and Fig 7.2, SCEC (1997))

τ 0.65(σ) a

APPENDIX F

(9) Per Seed and Idriss, ASCE, September 1971 $\frac{\tau_{av}}{\sigma_o'} = \frac{0.65(\sigma_o)}{\sigma_o'} * \frac{a_{max}}{g} * r_d$

(10) Per SCEC (1997), thefollowing guidelines apply to the factor of safety against liquefaction:

Consequence of Liquefaction	$(N_1)_{60}$ (clean sand)	Factor of Safety
Settlement	<= 15	1.1
	>= 30	1.0
Surface Manifestation	<= 15	1.2
	>= 30	1.0
Lateral Spread	<= 15	1.3
	>= 30	1.0

LIQUEFACTION INDUCED SETTLEMENTS

Project Name	Skate Park and Retail Bldg
Project Location	Huntington Beach
Project Number	11G113
Engineer	DWN

Boring No. B-1

Sample Depth (ft)	Depth to Top of Layer (ft)	Depth to Bottom of Layer (ft)	Depth to Midpoint (ft)	$(N_1)_{60}$	N_{CORR}	$(N_1)_{60\text{-}CORR}$	Liquefaction Factor of Safety	Cylclic Stress Ratio Induced by Design Earthquake (M=6.8)	Magnitude Weighting Factor	Cyclic Stress Ratio Induced by Design Earthquake (M=7.5)			Volmetric Strain (%)	Total Deformation of Layer (in)	Comments
				(1)	(2)	(3)	(4)	(5)	(6)	(7)			(8)		
6	0	7	3.5	0.0	0	0.0	N/A	0.24	1.28	0.19			0.0	0.00	Above Water Table
8.5	8	11	9.5	11.6	0	11.6	0.59	0.27	1.28	0.21			2.3	0.81	**Liquefiable**
14	11	14.5	12.75	7.6	3	10.6	N/A	0.30	1.28	0.24			0.0	0.00	Non-liquefiable: PI>12
14.5	14.5	17	15.75	6.8	5	11.8	N/A	0.33	1.28	0.25			0.0	0.00	Non-liquefiable: PI>12
19	17	19	18	10.7	5	15.7	0.73	0.34	1.28	0.26			1.9	0.44	**Liquefiable**
19.5	19	23	21	9.9	5	14.9	N/A	0.35	1.28	0.27			0.0	0.00	Non-liquefiable: PI>12
24.5	23	27	25	9.1	3	12.1	0.60	0.36	1.28	0.28			2.3	1.08	**Liquefiable**
29.5	27	31	29	9.4	2	11.4	0.54	0.37	1.28	0.29			2.4	1.15	**Liquefiable**
34.5	31	37	34	7.3	5	12.3	N/A	0.37	1.28	0.29			0.0	0.00	Non-liquefiable: PI>12
39.5	37	42	39.5	7.6	5	12.6	N/A	0.36	1.28	0.28			0.0	0.00	Non-liquefiable: PI>12
44	42	44	43	9.7	5	14.7	N/A	0.35	1.28	0.27			0.0	0.00	Non-liquefiable: PI>12
44.5	44	48	46	9.4	5	14.4	N/A	0.34	1.28	0.27			0.0	0.00	Non-liquefiable: PI>12
49.5	48	50	49	13.6	4	17.6	0.89	0.34	1.28	0.26			1.6	0.39	**Liquefiable**
										Total Deformation (in)				3.88	

Notes:

* Assumed

(1) N_{60} calculated previously for the individual layer

(2) Correction for fines content per Table 7.2 (SCEC 97)

(3) Corrected N_{60}

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

(5) Earthquake induced cyclic shear stress ratio calculated previously for the individual layer

(6) Factor to convert M=6.8 shear stress ratio to M=7.5 shear stress ratio, Seed, et al., 1983

(7) Corrected for Magnitude Weighting using revised Idriss factors (Fig 12, Youd and Idriss (1997) and Fig 7.2, SCEC (1997))

(8) Voumetric Strain Induced in a Liquefiable Layer, Tokimatsu and Seed, ASCE August 1987
(Strain N/A if Factor of Safety against Liquefaction > 1.2)

Appendix C

Notice of Transfer of Responsibility

Water Quality Management Plan
Notice of Transfer of Responsibility

Tracking No. Assigned by the City of Huntington Beach: _____

Submission of this Notice of Transfer of Responsibility constitutes notice to the City of Huntington Beach that responsibility for the Water Quality Management Plan ("WQMP") for the subject property identified below, and implementation of that plan, is being transferred from the Previous Owner (and his/her agent) of the site (or a portion thereof) to the New Owner, as further described below.

I. Previous Owner/Previous Responsible Party Information

Company/Individual Name		Contact Person	
Title			
Street Address			
City	State	Zip	Phone

II. Information about Site Transferred

Name of Project (if applicable)		Contact Person	
Title of WQMP Applicable to Site			
Planning Area (PA) and/or Tract Number(s) for Site Lot Numbers (if Site is a portion of a tract)			
Date WQMP Prepared (and revised if applicable)			
Street Address of Site			
City	State	Zip	Phone

III. New Owner/New Responsible Party Information

Company/Individual Name		Contact Person	
Title			
Street Address			
City	State	Zip	Phone

IV. Ownership Transfer Information

General Description of Site Transferred to New Owner
General Description of Portion of Project/Parcel Subject to WQMP Retained by Owner (if any)
Lot/Tract Numbers of Site Transferred to New Owner
Remaining Lot/Tract Numbers Subject to WQMP Still Held by Owner (if any)
Date of Ownership Transfer

Note: When the Previous Owner is transferring a Site that is a portion of a larger project/parcel addressed by the WQMP, as opposed to the entire project/parcel addressed by the WQMP, the General Description of the Site transferred and the remainder of the project/parcel not transferred shall be set forth as maps attached to

this notice. These maps shall show those portions of a project/parcel addressed by the WQMP that are transferred to the New Owner (the Transferred Site), those portions retained by the Previous Owner, and those portions previously transferred by Previous Owner. Those portions retained by Previous Owner shall be labeled "Previous Owner," and those portions previously transferred by Previous Owner shall be labeled as "Previously Transferred."

V. Purpose of Notice of Transfer

The purposes of this Notice of Transfer of Responsibility are: 1) to track transfer of responsibility for implementation and amendment of the WQMP when property to which the WQMP is transferred from the Previous Owner to the New Owner, and 2) to facilitate notification to a transferee of property subject to a WQMP that such New Owner is now the Responsible Party of record for the WQMP for those portions of the site that it owns.

VI. Certifications

A. Previous Owner

I certify under penalty of law that I am no longer the owner of the Transferred Site as described in Section II above. I have provided the New Owner with a copy of the WQMP applicable to the Transferred Site that the New Owner is acquiring from the Previous Owner.

Printed Name of Previous Owner Representative	Title
Signature of Previous Owner	Date

B. New Owner

I certify under penalty of law that I am the owner of the Transferred Site, as described in Section II above, that I have been provided a copy of the WQMP, and that I have informed myself and understand the New Owner's responsibilities related to the WQMP, its implementation, and Best Management Practices associated with it. I understand that by signing this notice, the New Owner is accepting all ongoing responsibilities for implementation and amendment of the WQMP for the Transferred Site, which the New Owner has acquired from the Previous Owner.

Printed Name of New Owner Representative	Title
Signature	Date

A complete copy of the WQMP for the Vans Center Avenue Skate Park Project, including large-format graphics, hydrologic model output data, and example BMP product information, is available for review at the City of Huntington Beach Planning Division counter.