Appendix I

Water Resources Technical Report

Sunset and Vine 2

Water Resources Technical Report

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1. INTRODUCTION

1.1. PROJECT DESCRIPTION

The Sunset and Vine 2 mixed-use project (Project) is proposed to be developed on an approximately 1.74-acre site (Project Site) located on the southeast corner of Vine Street and Sunset Boulevard in the City of Los Angeles within the County of Los Angeles. The Project Site is comprised of five Assessor Parcel Numbers (APNs) as summarized below:

Table 1 Project Description

APNs	PARCEL	ADDRESSES	LOT AREA (AC)	LOT AREA (SF)
5546-025-029 5546-025-030 5545-025-031 5546-025-020 5546-025-017	Parcel A	• 6151, 6257, 6263 W LELAND WAY • 6266-6270 W SUNSET BLVD	1.74 ac	75,795 sf

The Project site is currently occupied by approximately 75,795 square-feet of existing commercial uses and an existing surface parking lot that provides approximately 50 spaces. The Project site also includes uses that will remain on site but are not part of the Project. These include 64 multi-family residential units, 9,263 square-feet of commercial retail and restaurant uses within the existing Sunset Vine Tower, and two multi-family residential units within the duplex located at the southeast corner of the property line.

The Project will entail the demolition and removal of several existing commercial buildings and surface parking areas. The proposed development is described in the following paragraph.

The Project site will redevelop a portion of the site as one parcel with two drainage areas (as described below).

Parcel A: Construction of a new eight-story mixed-use residential and commercial building at the southeast corner of Vine St and Sunset Blvd. The development will include up to 150 new multi-family residential units and approximately 13,130 SF of commercial use. The new building will replace approximately 12,236 SF of existing commercial uses and existing surface parking lot.

Drainage for the onsite buildings fronting Sunset Boulevard is collected via roof drains and is discharged via sheet-flow across the sidewalk and through curb cores into W. Sunset Boulevard. The onsite drainage at the Leland Way portion of the property, which includes asphalt-paved parking area and a building, is discharged via sheet-flow toward Leland Way to the south.

1.2. SCOPE OF WORK

As part of the environmental impact report (EIR) for the Project, this report will describe the existing and proposed surface water hydrology, surface water quality, and groundwater at the Project Site and immediate surrounding areas, as well as an analysis of the Project's potential impacts on each of these water resources.

2. REGULATORY FRAMEWORK

2.1. SURFACE WATER HYDROLOGY

County of Los Angeles Hydrology Manual

Per the City of Los Angeles Municipal Code, the City has adopted the Los Angeles County (County) Department of Public Works Hydrology Manual as its basis of design for storm drainage facilities. The Hydrology Manual requires that a storm drain conveyance system be designed for a 25-year storm event and that the combined capacity of a storm drain, and street flow system accommodate flow from a 50-year storm event[1]. Areas with sump conditions are required to have a storm drain conveyance system capable of conveying flow from a 50-year storm event. The County also limits the allowable discharge into existing storm drain facilities based on the MS4 Permit which is enforced on all new developments that discharge directly into the County's storm drain system. Any proposed drainage improvements of County owned storm drain facilities such as catch basins and storm drain lines requires the approval/review from the County Flood Control District department.

The proposed Project is required to utilize the Hydrology Manual and accompanying hydrologic tools including HydroCalc Calculator to calculate existing and proposed discharges and volumes from the Project. The proposed project analyzes the 10-year, 25-year, and 50-year storm events.

Los Angeles Municipal Code

Any proposed drainage improvements within the street right of way or any other property owned by or under control of the City requires the approval of a B-permit (Section 62.105, Los Angeles Municipal Code (LAMC)). Under the B-Permit process, storm drain installation plans are subject to review and approval by the City of Los Angeles Department of Public Work, Bureau of engineering. Additionally, any connections to the City's storm drain system from a private property to a City catch basin or an underground storm drainpipe requires a storm drain connection permit from the City of Los Angeles Department of Public Works, Bureau of Engineering.

National Flood Insurance Program

The National Flood Insurance Act of 1968 and the Flood Disaster Protection Act of 1973 mandate the Federal Emergency Management Agency (FEMA) to evaluate flood hazards. FEMA provides flood insurance rates maps (FIRMS) for local and regional planners to promote sound land use and development practices by identifying potential flood areas based on the current conditions. To delineate a FIRM, FEMA conducts engineering studies, FEMA engineers and cartographers delineate special flood hazard areas (SFHA) on FIRMs.

[1] Los Angeles County Department of Public Works Hydrology Manual, January 2006. Found here: https://dpw.lacounty.gov/wrd/publication/engineering/2006_Hydrology_Manual.pdf .

2.2. SURFACE WATER QUALITY

Clean Water Act

Controlling pollution of the nation's receiving water bodies has been a major environmental concern for more than three decades. Growing public awareness of the impacts of water pollution in the United States culminated in the establishment of the federal Clean Water Act[2] (CWA) in 1972, which provided the regulatory framework for surface water quality protection.

The United States Congress amended the CWA in 1987 to specifically regulate discharges to waters of the United States from public storm drain systems and storm water flows from industrial facilities, including construction sites, and require such discharges be regulated through permits under the National Pollutant Discharge Elimination System (NPDES).[3] Rather than setting numeric effluent limitations for storm water and urban runoff, CWA regulation calls for the implementation of Best Management Practices (BMPs) to reduce or prevent the discharge of pollutants from these activities to the Maximum Extent Practicable (MEP) for urban runoff and meeting the Best Available Technology Economically achievable (BAT) and Best Conventional Pollutant Control Technology (BCT) standards for construction storm water. Regulations and permits have been implemented at the federal, state, and local level to form a comprehensive regulatory framework to serve and protect the quality of the nation's surface water resources.

In addition to reducing pollution with the regulations described above, the CWA also seeks to maintain the integrity of clean waters of the United States – in other words, to keep clean waters clean and to prevent undue degradation of others. As part of the CWA, the Federal Anti-Degradation Policy [40 Code of Federal Regulations (CFR) Section 131.12] states that each state "shall develop and adopt a statewide anti-degradation policy and identify the methods for implementing such policy…" [40 CFR Section 131.12(a)]. Three levels of protection are defined by the federal regulations:

1. Existing uses must be protected in all of the Nation's receiving waters, prohibiting any degradation that would compromise those existing uses;
2. Where existing uses are better than those needed to support propagation of aquatic wildlife and water recreation, those uses shall be maintained, unless the state finds that degradation is "…necessary to accommodate important economic or social development" [40 CFR Section 131.12(a)(2)]. Degradation, however, is not allowed to fall below the existing use of the receiving water; and
3. States must prohibit the degradation of Outstanding National Resource Waters, such as waters of national and state parks, wildlife refuges, and waters of exceptional recreation or ecological significance.

Federal Anti-Degradation Policy

The Federal Anti-Degradation Policy (40 CFR 131.12) requires states to develop statewide anti-degradation policies and identify methods for implementing them. Pursuant to the CFR, state anti-degradation policies and implementation methods shall, at a minimum, protect and maintain (1) existing in-stream water uses; (2) existing water quality, where the quality of the waters exceeds levels necessary to support existing beneficial uses, unless the state finds that allowing lower water quality is necessary to accommodate economic and social development in the area; and (3) water quality in waters considered an outstanding national resource.

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[2] Also referred to as the Federal Water Pollution Control Act of 1972.
[3] CWA Section 402(p).

Porter-Cologne Water Quality Act

In the State of California, the State Water Resources Control Board (SWRCB) and local Regional Water Quality Control Boards (RWQCBs) have assumed the responsibility of implementing the United States Environmental Protection Agency's (USEPA) NPDES Program and other programs under the CWA such as the Impaired Waters Program and the Anti-Degradation Policy. The primary quality control law in California is the Porter-Cologne Water Quality Act (Water Code Sections 13000 et seq.). Under Porter-Cologne, the SWRCB issues joint federal NPDES Storm Water permits and state Waste Discharge Requirements (WDRs) to operators of municipal separate storm sewer systems (MS4s), industrial facilities, and construction sites to obtain coverage for the storm water discharges from these operations.

California Anti-Degradation Policy

The California Anti-Degradation Policy, otherwise known as the Statement of Policy with Respect to Maintaining High Quality Water in California was adopted by the SWRCB (State Board Resolution No. 68-16) in 1968. Unlike the Federal Anti-degradation, Policy, the California Anti-Degradation Policy applies to all waters of the State, not just surface waters. The policy states that whenever the existing quality of a water body is better than the quality established in individual Basin Plans, such high quality shall be maintained and discharges to that water body shall not unreasonably affect present or anticipated beneficial use of such water resource.

California Toxic Rule

In 2000, the EPA promulgated the California Toxic Rule, which establishes water quality criteria for certain toxic substances to be applied to waters in the State. The EPA promulgated this rule based on the EPA's determination that the numeric criteria are necessary in the State to protect human health and the environment. The California Toxic Rule establishes acute (i.e., short-term) and chronic (i.e., long-term) standards for bodies of water such as inland surface waters and enclosed bays and estuaries that are designated by the Los Angeles Regional Water Quality Control Board (LARWQCB) as having beneficial uses protective of aquatic life or human health.

Board Basin Plan for the Coastal Watersheds of Los Angeles and Ventura Counties

As required by the California Water Code, the LARWQCB has adopted a plan entitled "Water Quality Control Plan, Los Angeles Region: Basin Plan for the Coastal Watersheds of Los Angeles and Ventura Counties" (Basin Plan). Specifically, the Basin Plan designates beneficial uses for surface and groundwaters, sets narrative and numerical objectives that must be attained or maintained to protect the designated beneficial uses and conform to the state's anti-degradation policy, and describes implementation programs to protect all waters in the Los Angeles Region. In addition, the Basin Plan incorporates (by reference) all applicable state and Regional Board plans and policies and other pertinent water quality policies and regulations. Those of other agencies are referenced in appropriate sections throughout the Basin Plan.

NPDES Permit Program

The NPDES permit program was first established under authority of the CWA to control the discharge of pollutants from any point source into the waters of the United States. As indicated above, in California, the NPDES stormwater permitting program is administered by the SWRCB through its nine RWQCBs.

The General Permit for Construction Activities

SWRCB Order No. 2009-0009-DWQ known as “General Permit” was adopted on September 2, 2009 and was amended by Order No 2012-0006-DWQ which became effective on July 17, 2012. This NPDES permit establishes a risk-based approach to stormwater control requirements for construction projects by identifying three project risk levels. The main objectives of the General Permit are to:

1. Reduce erosion
2. Minimize or eliminate sediment in stormwater discharges
3. Prevent materials used at a construction site from contacting stormwater
4. Implement a sampling and analysis program
5. Eliminate unauthorized non-stormwater discharges from construction sites
6. Implement appropriate measures to reduce potential impacts on waterways both during and after construction of projects
7. Establish maintenance commitments on post-construction pollution control measures

California mandates requirements for all construction activities disturbing more than one acre of land to develop and implement Stormwater Pollution Prevention Plans (SWPPP). The SWPPP documents the selection and implementation of BMPs for a specific construction project, charging Owners with stormwater quality management responsibilities. A construction site subject to the General Permit must prepare and implement a SWPPP that meets the requirements of the General Permit.

As part of the Project, preparation, and implementation of a SWPPP will be required. In addition, the Project will be required to obtain a Waste Discharger Identification Number (WDID) through the State’s Storm Water Multiple Application and Report Tracking System (S.M.A.R.T.S.).

In September 2022, the SWRCB adopted an updated version of the General Permit that goes into effect September 1, 2023. The reissuance of the permit primarily focused on the incorporation of TMDL’s into permit compliance and modifying the inspection oversight program and responsibilities of site inspectors and SWPPP developers.

Los Angeles County Municipal Storm Water System (MS4) Permit

As described previously, USEPA regulations require that MS4 permittees implement a program to monitor and control pollutants being discharged to the municipal system from both industrial and commercial projects that contribute a substantial pollutant load to the MS4.

On December 13, 2001, the LARWQCB adopted Order No. 01-182 under the CWA and the Porter-Cologne Act. This Order is the NPDES Permit or MS4 permit for municipal stormwater and urban runoff discharges within Los Angeles County. The requirements of this Order (the “Permit”) cover 84 cities and most of the unincorporated areas of Los Angeles County. Under the Permit, the LACFCD is designated as the Principal Permittee. The Permittees are the 84 Los Angeles County cities (including the City of Los Angeles) and unincorporated areas within Los Angeles County. Collectively, these are the “Co-Permittees”. The Principal Permittee helps to facilitate activities necessary to comply with the requirements outlined in the Permit but is not responsible for ensuring compliance of any of the Permittees.

Since adoption of Order No. 01-182, the LARWQCB has adopted Order No. R4-2021-0105, as amended by State Water Board Order WQ 2020-0038 NPDES Permit No. CAS004004 on July 23, 2021. This current permit will expire on September 11, 2026.

The City of Los Angeles is a Permittee of the California Regional Water Quality Control Board, Los Angeles Region, and is therefore subject to the requirements set forth in Order No. R4-2021-0105, as amended by State Water Board Order WQ 2020-0038, NPDES Permit No. CAS004004.

City of Los Angeles Stormwater Program

The City of Los Angeles supports the policies of the General Permit for Construction Activities and the Los Angeles County NPDES permit through the Development of Best Practices Handbook. Part A Construction Activities, 3rd Edition, and associated ordinances were adopted in July 2011. The Handbook provides guidance for developers in complying with the requirements of the Development Planning Program regulations of the City's Stormwater Program. Compliance with the requirements of this manual is required by the City of Los Angeles Ordinance No. 173,494. The handbook and ordinances also have specific minimum BMP requirements for all construction activities and require dischargers whose construction projects disturb one acre or more of soil to prepare a SWPPP and file a Notice of Intent (NOI) with SWRCB. The NOI informs the SWRCB of a particular project and results in the issuance if a Waste Discharger Identification (WDID) number, which is needed to demonstrate compliance with the General Permit.

The City of Los Angeles implements the requirement to incorporate stormwater BMPs through the City's plan review and approval process. During the review process projects are reviewed for compliance with the City's General Plan, zoning ordinances, and other applicable local ordinances and codes, including storm water requirements. Plans and specifications are reviewed to ensure that the appropriate BMPs are incorporated to address storm water pollution prevention goals.

Standard Urban Stormwater Mitigation Plan (SUSMP)

Under the current Los Angeles County Municipal NPDES Permit, permittees are required to implement a development planning program to address storm water pollution. These programs require project applicants for certain types of projects to implement LID BMP throughout the operational life of their projects.

The Project falls within the definition of "redevelopment" under the Los Angeles County MS4 Storm Water Permit which requires compliance with the Low Impact Development (LID) requirements.

Los Angeles County Low Impact Development (LID)

LID is a stormwater strategy that is used to mitigate the impacts of runoff and stormwater pollution as close to its source as possible. Urban runoff discharged from municipal storm drain systems is one of the principal causes of water quality impacts in most urban areas. The stormwater may contain pollutants such as trash and debris, bacteria and viruses, oil and grease, sediments, nutrients, metals, and toxic chemicals that can negatively affect the ocean, rivers, plant and animal life, and public health.

LID encompasses a set of site design approaches and BMPs that are designed to address runoff and pollution at the source. These LID practices can effectively remove nutrients, bacteria, and metals, while reducing the volume and intensity of stormwater flows.

The Project is subject to compliance with Order No. R4-2021-0105, which became effective on July 23, 2021. The main purpose of this law is to ensure that redevelopment projects mitigate runoff in a manner that captures or treats rainwater at its source, while utilizing natural resources.

In accordance with Order No. R4-2021-0105, stormwater runoff shall be infiltrated, evapotranspired, captured and used, or treated through high removal efficiency BMPs, onsite, through stormwater management techniques that comply with provisions of LA County LID standards Manual. County of Los Angeles guidelines recommend that the corrected infiltration rate be equal or greater than 0.3 inches per hour.

City of Los Angeles Low Impact Development (LID)

The City of Los Angeles has passed ordinance No. 181899, amending LAMC Chapter VI, Article 4.4, Sections 64.70.01 and 64.72 to expand the applicability of the existing SUSMP requirements by imposing rainwater LID strategies on projects that require building permits. The LID ordinance became effective on May 12, 2012. The City of Los Angeles Bureau of Sanitation, Watershed Protection division will adopt the LID standards as issued by the LARWQCB and the City of Los Angeles Department of Public Works. The LID Ordinance will conform to the regulation outlined in the NPDES permit and SUSMP.

Hydromodification

In addition to the LID requirements listed in the Permit, the Permit also addresses requirements for Hydromodification as pertaining to the project. Per Part VI.D.7.c.iv of the Permit:

> "Each Permittee shall require all New Development and Redevelopment projects located within natural drainage systems as described in Part VI.D.7.c.iv.(1)(a)(iii) to implement hydrologic control measures, to prevent accelerated downstream erosion and to protect stream habitat in natural drainage systems. The purpose of the hydrologic controls is to minimize changes in post-development hydrologic storm water runoff discharge rates, velocities, and duration. This shall be achieved by maintaining the project's pre-project stormwater runoff flow rates and durations."

However, per Part VI.D.7.c.iv.(1)(b)(iv) of the Permit, the Project is exempt from such requirements as runoff from the site is discharged directly via storm drain to a receiving water that is not susceptible to hydromodification impacts. The downstream channels for the Project Site are the Ballona Creek, Ballona Estuary, Ballona Lagoon, Ballona Wetlands, or the Marina del Rey Lagoon.

Section 3.3 of the City of Los Angeles' 2016 LID Manual requires that new development and/or redevelopment projects that drain to natural drainage systems in a small part of the Upper Los Angeles River watershed shall be required to comply with hydromodification requirements. However, based on our review, the project site is within the Ballona Creek Watershed, not the Upper Los Angeles River. Therefore, hydromodification control is not anticipated to be required for this project. The Project is not required to implement hydrologic control measures as mitigation for hydromodification impacts. In addition, implementation of the Project will result in a reduction of peak flows and volumes as compared to existing conditions, thereby satisfying hydromodification requirements in addition to the receiving water exemption.

2.3. GROUNDWATER

California Groundwater Sustainability Act

On Sept. 16, 2014, California Governor Jerry Brown signed into law a three-bill legislative package, known as the Sustainable Groundwater Management Act of 2014 (SGMA). The SGMA provides a framework for sustainable management of groundwater supplies by local authorities, with a limited role for state intervention only if necessary to protect the resource.

The SGMA requires the formation of local groundwater sustainability agencies (GSAs) that must assess conditions in their local water basins and adopt locally-based management plans. The act provides substantial time – 20 years – for GSAs to implement plans and achieve long-term groundwater sustainability. It protects existing surface water and groundwater rights and does not impact current drought response measures.

The California Water Commission (CWC) requires a statewide prioritization of California's groundwater basins using the following eight criteria:

1. Overlying population;
2. Projected growth of overlying population;
3. Public supply wells;
4. Total wells;
5. Overlying irrigated acreage;
6. Reliance on groundwater as the primary source of water;
7. Impacts on the groundwater; including overdraft, subsidence, saline intrusion, and other water quality degradation; and
8. Any other information determined to be relevant by the Department.

The Project Site is located within the Coastal Plain of Los Angeles - West Basin Subbasin. GSAs responsible for high-and medium-priority basins must adopt groundwater sustainability plans within five to seven years, depending on whether the basin is in critical overdraft. Agencies may adopt a single plan covering an entire basin or combine a number of plans created by multiple agencies. Preparation of groundwater sustainability plans is exempt from CEQA. Plans must include a physical description of the basin, including groundwater levels, groundwater quality, subsidence, information on groundwater-surface water interaction, data on historical and projected water demands and supplies, monitoring and management provisions, and a description of how the plan will affect other plans, including city and county general plans. Plans will be evaluated every five years.

Board Basin Plan for the Coastal Watersheds of Los Angeles and Ventura Counties

As required by the California Water Code, the LARWQCB has adopted a plan entitled "Water Quality Control Plan, Los Angeles Region: Basin Plan for the Coastal Watersheds of Los Angeles and Ventura Counties" (Basin Plan). Specifically, the Basin Plan designates beneficial uses for surface and groundwaters, sets narrative and numerical objectives that must be attained or maintained to protect the designated beneficial uses and conform to the state's anti-degradation policy, and describes implementation programs to protect all waters in the Los Angeles Region. In addition, the Basin Plan incorporates (by reference) all applicable state and regional board plans and policies and other pertinent water quality policies and regulations. Those of other agencies are referenced in appropriate sections throughout the Basin Plan.

The Basin Plan is a resource for the LARWQCB and others who use water and/or discharge wastewater in the Los Angeles Region. Other agencies and organizations involved in environmental permitting and resource management activities also use the Basin Plan. Finally, the Basin Plan provides valuable information to the public about local water quality issues.

Safe Drinking Water Act (SDWA)

The federal Safe Drinking Water Act (SDWA), established in 1974, sets drinking water standards throughout the country and is administered by the USEPA. The drinking water standards established in the SDWA, as set forth in the CFR, are referred to as the National Primary Drinking Water Regulations (Primary Standards, Title 40, CFR Part 141) and the National Secondary Drinking Water Regulations (Second Standards, 40 CFR Part 143). California passed its own SDWA in 1986 that authorizes the State's Department of Health Services (DHS) to protect the public from contaminants in drinking water by establishing maximum contaminants levels, as set forth in the California Code of Regulations (CCR), Title 22, Division 4, Chapter 15, that are at least as stringent as those developed by the USEPA, as required by the federal SDWA.

3. ENVIRONMENTAL SETTING

3.1. SURFACE WATER HYDROLOGY

3.1.1. Regional

The Project is located within the Ballona Creek Watershed in the County of Los Angeles. The Ballona Creek Watershed covers approximately 81,600 acres and is located in the southwestern portion of the Coastal Plain of Los Angeles Groundwater Basin. Major tributaries of Ballona Creek include Centinela Creek, Sepulveda Channel, and Benedict Canyon Channel. These tributaries flow into Ballona Creeks open concrete channel for ten miles from mid-Los Angeles through Culver City. The Estuary portion of Ballona Creek is soft-bottomed and includes the Ballona wetlands, which eventually reaches Santa Monica Bay at the Marina del Rey Harbor.

All runoff from the project drains to the Ballona Channel through City and LACFCD storm drain lines.

3.1.2. Local

Stormwater from the project site generally drains via surface flow, toward the adjacent roadways. The property is not bound by any natural bodies of water and is not subject to flooding.

- The north parcels are within Drainage Basin 2033 per the Stormwater Map from Navigate LA[4] and are currently tabled to drain toward W. Sunset Boulevard.
- The south parcels are within Drainage Basin 2034 per the Stormwater Map from Navigate LA[5] and are currently tabled to drain toward Leland Way.

The entire drainage for the site eventually comingles at Vine Street and Fountain Avenue, approximately ¼ mile south of the project site.

Please refer to **Appendix A** for the existing condition hydrology map.

3.1.3. On Site

The existing condition hydrology for the Project Site has been delineated into three Drainage Sub-Areas (see **Appendix A** for exhibits). Based on the review of the drainage areas presented on the Stormwater Map from Navigate LA, the north parcels are within Drainage Basin 2033, and are currently tabled to drain toward W. Sunset Boulevard. The south parcels are within Drainage Basin 2034 and are currently tabled to drain toward Leland Way. The entire drainage for the site eventually comingles at Vine Street and Fountain Avenue, approximately ¼ mile south of the project site. Existing drainage patterns for the Sub-Areas are described as below:

Subarea A1: This subarea drains southerly toward Leland Way, at the southerly boundary of the site. There are no storm drain facilities in Leland Way, to the south of the property, and the drainage currently discharges into the roadway via sheet-flow. The drainage is then conveyed easterly in Leland Way, toward El Centro Avenue, and then southerly in El Centro

[4] https://navigatela.lacity.org/navigatela/

[5] https://navigatela.lacity.org/navigatela/

Avenue. toward Fountain Avenue. The drainage is then conveyed within a storm drain in Fountain Avenue, before comingling with flows in the Vine Street Storm Drain.

Subarea B1 & B2: These subareas drain northerly toward W. Sunset Boulevard. There is an existing City of Los Angeles 90" Reinforced Concrete Box Culvert (RCB) storm drain in W. Sunset Boulevard that accepts drainage from the northerly retail buildings. Currently, the drainage sheet-flows from roof drains to W. Sunset Boulevard, where it is conveyed easterly to an existing catch basin located mid-block, just east of Argyle Avenue. The drainage is intercepted by the existing catch basin and conveyed to the existing 90" (RCB). The RCB drains westerly, toward Vine Street, and then southerly in Vine Street. The drainage then continues southerly in Vine Street before comingling with the northerly site flows in Fountain Avenue.

Under the existing conditions, the entire Project Site area is fully built-out with high impervious conditions (approximately 89%) and the predominant land use being surface parking lots, commercial buildings, and residential areas. See **Appendix C** for impervious percentage per the LA County Hydrology Manual Appendix D table. The topography of the site is relatively flat, draining via primarily surface flow toward adjacent roadways. See **Appendix A** for existing drainage areas and discharge points. There are no known drainage issues associated with the Project Site.

Table 2 below provides 10-year, 25-year, and 50-year storm frequency analysis for the Project Site's existing conditions. These storm frequencies are required by Los Angeles County Public Works (10-year), Hydrology Manual for Urban Flood level of protection (25-year), and the State CEQA guideline requirements (50-year). Output calculations are provided in **Appendix B**.

Table 2 Existing Condition 10-year, 25-year, and 50-year Storm Event Hydrology

Existing Conditions 10-year Storm Frequency				
Drainage Sub-Area	Acreage	Time of Concentration (min)	% Imperviousness	Q_{10} (cfs)
A	1.74	7.0	89%	3.4
Existing Conditions 25-year Storm Frequency				
Drainage Sub-Area	Acreage	Time of Concentration (min)	% Imperviousness	Q_{25} (cfs)
A	1.74	6.0	89%	4.5
Existing Conditions 50-year Storm Frequency				
Drainage Sub-Area	Acreage	Time of Concentration (min)	% Imperviousness	Q_{50} (cfs)
A	1.74	6.0	89%	5.1
Notes: See **Appendix A** for the existing hydrology exhibit and **Appendix B** for existing hydrology calculations.				

3.1.4. FEMA

The project is within Panel 1605 of 2350 (Map Number 06037C1605F, dated September 26, 2008) on Federal Emergency Management Agency's (FEMA's) Flood Insurance Rate Map (FIRM). Based on the FIRM, the project is within Zone X, which depicts areas determined to be outside of the 0.2% (500-year) annual chance floodplain and no coordination or permitting with FEMA is required.

3.2. SURFACE WATER QUALITY

3.2.1. Regional

As described previously, the Project is located within the Ballona Creek Watershed. The watershed is highly developed, with land use consisting of 64% residential use, 8% commercial, 4% industrial, and 17% vacant/open space. Overall, 76% of the watershed is covered by roads, rooftops, and other impervious surfaces.

The Ballona Creek Channel and other selected water bodies in its watershed, including the Ballona Creek Estuary, Ballona Creek Wetlands, Marina del Rey Harbor, and Santa Monica Bay are impaired by pollutants (i.e., trash, metals, bacteria, nutrients) mainly because of the watershed's large, dense population and the amount of impervious ground surface that prevents large quantities of runoff from infiltrating into the soils.

3.2.1.1. Beneficial Uses in Ballona Creek Watersheds

The Ballona Creek Watershed consists of inland surface water uses from the Ballona Creek Estuary, Lagoon, Wetlands, Creek Reach 1, Creek Reach 2 and the Marina Del Rey Lagoon. The existing and potential beneficial uses for the waters within the Ballona Creek Watershed where inland surface water flows from the southeast portion of the Project and ultimately discharge to the Marina del Rey Harbor are described below.

Table 3 Beneficial Uses of Inland Surface Waters in the Ballona Creek Watershed

MUN* - Municipal and Domestic Supply	**NAV** - Navigation
COMM - Commercial and Sport Fishing	**WARM*** - Warm Freshwater Habitat
EST - Estuarine Habitat	**MAR** - Marine Habitat
SPWN - Spawning, Reproduction, and/or Early Development	**RARE** - Rare, Threatened, or Endangered Species
MIGR - Migration or Aquatic Organisms	**WILD** - Wildlife Habitat
SPWN - Spawning, Reproduction, and/or Early Development	**LREC-1** - Limited Water Contact Recreation
WET – Wetland Habitat	**REC1** - Water Contact Recreation
SHELL - Shellfish Harvesting	**REC2** - Non-contact Water Recreation
Notes: * Potential beneficial use Source: Los Angeles Regional Water Quality Control Board Beneficial Use Table, found here: http://www.waterboards.ca.gov/losangeles/water_issues/programs/basin_plan/Beneficial_Uses/ch2/Revised%20Beneficial%20Use%20Tables.pdf	

In addition to the beneficial uses of inland surface waters the Ballona Creek Watershed also includes beneficial coastal features within the Ballona Creek Estuary, Lagoon, and Wetlands. Described below are the beneficial uses of these coastal waterbodies that receive storm drain discharges.

Table 4 Beneficial Uses of Coastal Features in the Ballona Creek Watershed

NAV - Navigation	**COMM** - Commercial and Sport Fishing
EST - Estuarine Habitat	**MAR** - Marine Habitat
WILD - Wildlife Habitat	**MIGR** - Migration or Aquatic Organisms
SPWN - Spawning, Reproduction, and/or Early Development	**RARE** - Rare, Threatened, or Endangered Species
SHELL - Shellfish Harvesting	**WETb** – Wetland Habitat
Notes : * Potential beneficial use Source: Los Angeles Regional Water Quality Control Board Beneficial Use Table, found here: http://www.waterboards.ca.gov/losangeles/water_issues/programs/basin_plan/Beneficial_Uses/ch2/Revised%20Beneficial%20Use%20Tables.pdf	

3.2.1.2. Impairments and TMDL's in the Ballona Creek Watershed

CWA 303(d) List of Water Quality Limited Segments

Under Section 303(d) of the CWA, states are required to identify water bodies that do not meet their water quality standards. Biennially, the LARWQCB prepares a list of impaired waterbodies in the region, referred to as the 303(d) list. The 303(d) list outlines the impaired waterbody and the specific pollutant(s) for which it is impaired. All waterbodies on the 303(d) list are subject to the development of a Total Daily Maximum Load (TMDL).

Storm water runoff from the Project discharges to Ballona Creek, Ballona Creek Estuary, Ballona Creek Wetlands, Marina del Rey Harbor and then eventually to the Santa Monica Bay. According to the 2018 303(d) list of Limited Water Quality Segments published by the SWRCB, the Ballona Creek Estuary, Ballona Creek Wetlands, Marina del Rey Harbor and Santa Monica Bay are listed as impaired by the constituents in **Table 5** below.

Table 5 List of 303(d) Impairments

Water Body	Listed Pollutants with TMDL 303(d) Impairment
Ballona Creek	Copper; Trash; Zinc; Lead; Viruses (enteric); Toxicity; Indicator Bacteria
Ballona Creek Estuary	PCBs (Polychlorinated biphenyls); DDT (Dichlorodiphenyltrichloroethane); Cadmium; Zinc; Chlordane; Indicator Bacteria; PAHs (Polycyclic Aromatic Hydrocarbons); Copper;
Ballona Creek Wetlands	Trash; Habitat alterations; Exotic Vegetation; Reduced Tidal Flushing
Marina del Rey Harbor	Indicator Bacteria; Toxicity; Copper; Zinc; Lead; Chlordane; PCBs (Polychlorinated biphenyls)
Santa Monica Bay	PCBs (Polychlorinated biphenyls); Trash; DDT (Dichlorodiphenyltrichloroethane)
Source: 2018 Integrated Report (Clean Water Act Section 303(d) List / 305(b) Report) – Statewide, found here: https://gispublic.waterboards.ca.gov/portal/apps/webappviewer/index.html?id=e2def63ccef54eedbee4ad726ab1552c	

Total Maximum Daily Loads (TMDLs)

Ballona Creek Watershed

Once a water body has been listed as impaired on the 303(d) list, a TMDL for the constituent of concern (pollutant) must be developed for that water body. A TMDL is an estimate of the daily load of pollutants that a water body may receive from point sources, non-point sources, and natural background conditions (including an appropriate margin of safety), without exceeding its water quality standard. Those facilities and activities that are discharging into the water body, collectively, must not exceed the TMDL. In general terms, municipal, small MS4, and other dischargers within each watershed are collectively responsible for meeting the required reductions and other TMDL requirements by the assigned deadline.

The Los Angeles RWQCB has adopted wet-weather TMDLs in Ballona Creek, Ballona Estuary, and Sepulveda Channel for Bacteria and Metals in the Ballona Creek. These TMDL pollutants include E. coli; Copper; Lead; Zinc; Selenium; 4,4'-DDE; Benzo(a)anthracene; Silver(total); 4,4'-DDT; 3,4 Benzofluoranthene; alpha-chlordane; gamma-chlordane; Benzo(a)anthracene; Benzo(a)pyrene; Benzo(k)fluoranthene; Bis(2-Ethylhexyl) phthalate; Chrysene; Indeno(1,2,3-cd)pyrene as explained in more detail below:

- **Ballona Creek Bacteria TMDL Implementation Plan (City of Beverly Hills et al., Nov 2009);** - This TMDL was adopted by: The LARWQCB on June 8, 2006. This TMDL addresses elevated bacterial indicator densities that cause impairment to water contact recreation (REC-1) beneficial use designated for Ballona Estuary, limited water contact recreation (LREC) designated for Ballona Creek Reach 2, and non-contact recreation (REC-2) beneficial uses of Ballona Creek Reach 1. Monitoring for these Bacteria TMDL has continued.

- **Multi-Pollutant TMDL Implementation Plan for the Unincorporated County Area of Ballona Creek (County of Los Angeles, 2010)** - The goal of the multi-pollutant implementation plan is to address all current TMDLs, with consideration of future potential TMDLs. The metals, bacteria, and toxics TMDLs are considered the primary focus of this implementation plan. A secondary focus is placed on trash, because reporting on progress toward TMDL implementation occurs annually and through a separate process. However, BMPs that address trash have the potential to provide added benefit in addressing other pollutants, which is assessed in this implementation plan.

- **Ballona Creek Metals TMDL Implementation Plan (City of Beverly Hills et al., Jan, 2010)** – On July 7, 2005, the Los Angeles Regional Water Quality Control Board (Los Angeles Water Board) adopted Resolution No. R05-007 amending the Basin Plan to establish a Total Maximum Daily Load (TMDL) for metals in Ballona. Ballona Creek is listed on the federal Clean Water Act section 303(d) list because it did not meet water quality standards for copper, lead, selenium, and zinc. The TMDL was approved by the State Water Resources Control Board (State Water Board) in Resolution No. 2005-0078 on October 20, 2005 and monitoring has continued in the Ballona Creek area.

- **Ballona Creek Estuary Toxic Pollutants TMDL Implementation Plan (City of Beverly Hills et al., June, 2012)** - In June 2006, the LARWQCB adopted a Basin Plan Amendment establishing the Ballona Creek and Ballona Estuary Bacteria TMDL. The Bacteria TMDL became effective in May 2008 and was amended June 2012. The requirements of the Bacteria TMDL were incorporated into the 2012 MS4 Permits to limit dry weather bacteria TMDLS in the area.

3.2.2. Local

Within the urban environment of the Project, stormwater runoff occurs during and shortly after rain events. The volume of runoff depends on the intensity and duration of the storm event and the imperviousness of the drainage area. Typical urban pollutants associated with stormwater runoff following rain events includes sediment, trash, bacteria, metals, nutrients and potentially organics and pesticides. The source of contaminants is wide ranging and includes all areas where rainfall occurs along with atmospheric deposition. Therefore, sources of contaminants within urban areas include roadways, building tops, parking lots, landscape areas and maintenance areas.

To reduce contaminant loads from entering the storm drain system, the City conducts routine street cleaning operations as well as periodic cleaning and maintenance of the catch basins to reduce stormwater pollution within the storm drain system.

3.2.3. On Site

Under the existing conditions, there are no existing water quality BMPs associated with the existing conditions. There are no drainage issues associated with the project site. Anticipated pollutants consistent with parking lots, building areas and landscaping include total suspended solids (TSS), oil/grease, heavy metals, nutrients, pesticides, and trash. See **Appendix A** for existing drainage areas and discharge points.

3.3. GROUNDWATER

3.3.1. Regional

The City of Los Angeles overlies the Los Angeles Coastal Plain Groundwater Basin (Basin) which consists of four major subbasins: Hollywood, Santa Monica, Central and West Coast. Replenishment of the Basins occurs primarily through imported water, spreading basins, recycled water, and local runoff. Injection wells are also used to pump freshwater along specific seawater barriers to prevent the intrusion of salt water. Groundwater within the Basin generally flows in a south and southwesterly direction.

3.3.2. Local

The Project resides specifically within the Hollywood Subbasin, which is located in the northeastern part of the Los Angeles Coastal Plain Groundwater Basin.

<u>Hollywood Subbasin</u>

The Hollywood Subbasin covers an area of approximately 16.4 square miles (10,500 acres) and is bounded in the north by the Santa Monica Mountains and the Hollywood fault. To the east by the Elysian Hills, west by the Inglewood fault zone, and on the south by La Brea High, a group of impermeable rock layers. The surface of the subbasin is crossed in the south by Ballona Creek which surface flows to westward to the Pacific Ocean. The storage capacity of the Hollywood Subbasin is estimated to be 200,000 acre-feet (AF) with an estimated 3,000 AF available for use. Only a fraction is available for water use to prevent physical damage to the basin. The subbasin is unadjudicated and managed through municipal ordinances that enforce sustainable groundwater use and protect water quality. The basin is low priority and does not receive artificial recharge and replenished through percolation of stream flows from higher areas to the north and aquifers to the west.

3.3.3. On Site

GPI has prepared a planning-level preliminary geotechnical investigation (December 2018) for a portion of the Project Site and Geocon West, Inc. has prepared geotechnical investigation of a neighboring site to the east of the Project Site (October 2016). The information below is in regard to both documents which are included in **Appendix D**.

In 2018 GPI conducted three cone penetration tests (CPT) within the Projects Site at depths of 52 to 75 feet below existing grades. Based on the CPTs, the site soils generally consisted of 11 to 15 feet of loose to medium dense silty sand and firm sandy silt, underlain predominantly by interbedded layers of very stiff to hard clays and silts to the depth explored. Discontinuous layers of medium dense to very dense sands and silts sands, approximately 3 to 15 feet in thickness, were encountered at depths of 37 to 59 feet below grade.

Groundwater was not encountered at the exploration’s maximum depth of 75 feet below existing grade. Because GPI's field explorations program was limited to CPTs, the presence of existing fills soils, or lack thereof, was not evaluated. At each testing location soils varied in type and friction ratio. Out of the three test a majority of soils 10 to 15 feet below grade consisted of stiff and hard clayey and silty soils, with hard or dense sandy layers dispersed between. Soil deposits may vary in type, strength, and many other important properties between points of exploration due to non-uniformity of the geologic formations or to man-made cut and fill operations.

There are no known faults within or crossing the site, it is not within an earthquake fault zone, and is not susceptible to liquefaction landslides, or methane. Overall, GPI recommends additional explorations and testing’s to accommodate any geotechnical constraint's that may occur to the design and construction at the Project Site.

Similarly, the borings done by Geocon West were located in the neighboring site approximately 60 to 200 ft away from the tests done by GPI. The soils encountered by Geocon West in the exploratory boring consisted of artificial fill and unconsolidated young alluvial deposits consisting of gravel, sand, silt, and clay. The artificial fill soils were encountered at the neighboring site to between approximately 5 ½ feet below existing ground surface; however deeper fill may exist at the neighboring site in the areas not directly explored. The fill materials generally consist of slightly moist and loose dark brown and yellowish-brown silty sand and sand with silt and some fine gravel and varying amount of asphalt debris.

Groundwater was not encountered at the explorations drilled maximum depth of 45 ½ feet below the existing ground surface. Based on data from the California Division of Mines and Geology (CDMG, 1998), the historically highest groundwater level in the area is approximately 50 feet beneath the ground surface. It is unlikely that groundwater levels will ever exceed the historic high levels. Altogether the historic high ground water levels in the site vicinity, lack of groundwater in the borings and depth of proposed construction ground water is not expected to have any effect during construction or on the project.

Based on review of data available from Geocon West's exploration performed at the neighboring site (Geocon West Inc., 2016), the near surface (upper 5 feet) neighboring soils are variable artificial fill and generally consists of silty sand and sand with silt. The lower surface (under 5 feet) neighboring soils are variable alluvium soils consisting of silty sand, silt, silt with sand, sandy silt, sand, and sand with silt at the borings' deepest depth of 45 ½ feet. The near and lower surface soils at the neighbor site are suitable for the Projects uses and would likely result in feasible infiltration characteristics. Foundations were also assumed to be at or below a depth of 21 feet below the ground surface for the purpose of this report.

According to Geocon West, Inc., percolation tests performed at the neighboring site would support stormwater infiltration system. Specifically at the infiltration depths of 35 to 45 feet the percolation would support a dry well system with an average raw infiltration rate of 54 inches per hour. Geocon West further states that infiltration of stormwater will not induce excessive hydro-consolidation, will not create perched groundwater conditions, will not affect soil structure of existing or proposed foundations due to expansive soils, will not saturate soils supported by existing or proposed retaining walls, and will not increase the potential for liquefaction.

Due to the proximity of Geocon West's tests at the neighboring site, the Project Site would need more testing to confirm if stormwater infiltration systems at the site are considered feasible. Although Geocon Wests neighboring geotechnical evaluation are favorable of infiltration, the boring tests done by GPI within the Project Site indicate variability in the soils as compared to the neighboring site, within the vicinity.

Although Geocon West's percolation tests confirmed that infiltration is feasible at the neighboring site, GPIs borings indicates variable soils within the project vicinity and further testing is needed to confirm the infiltration feasibility within the Project site.

4. SIGNIFICANCE THRESHOLD

4.1. SURFACE WATER HYDROLOGY

With respect to the surface water hydrology, the State CEQA Guidelines inquire whether the Project would:

- Substantially alter the existing drainage pattern of the site or area, including through the alteration of the course of a stream or river or through the addition of impervious surfaces, in a manner which would:
    - Result in substantial erosion or siltation on- or off-site;
    - Substantially increase the rate or amount of surface runoff in a manner which would result in flooding on- or off-site;
    - Create or contribute runoff water which would exceed the capacity of existing or planned stormwater drainage systems or provide substantial additional sources of polluted runoff; or
    - Impede or redirect flood flows
    - In flood hazard, tsunami, or seiche zones, risk release of pollutants due to project inundation

4.2. SURFACE WATER QUALITY

With respect to surface water quality, the State CEQA Guidelines (Appendix G) inquire whether the Project would:

- Violate any water quality standards or waste discharge requirements or otherwise substantially degrade surface or ground water quality?
- In flood hazard, tsunami, or seiche zones, risk release of pollutants due to project inundation?
- Conflict with or obstruct implementation of a water quality control plan or sustainable groundwater management plan?

The State CEQA Guidelines include the following relevant definitions:

"Pollution" means an alteration of the quality of the waters of the state to a degree which unreasonably affects either of the following: 1) the waters for beneficial uses or 2) facilities which serve these beneficial uses. "Pollution" may include "Contamination".

"Contamination" means an impairment of the quality of the waters of the state by waste to a degree, which creates a hazard to the public health through poisoning or though the spread of disease. "Contamination" includes any equivalent effect resulting from the disposal of waste, whether or not waters of the state are affected.

"Nuisance" means anything which meets all of the following requirements: 1) is injurious to health, or is indecent or offensive to the senses, or an obstruction to the free use of property, so as to interfere with the comfortable enjoyment of life or property; 2) affects at the same time an entire community or neighborhood, or any considerable number of persons, although the extent of the annoyance or damage inflicted upon individuals may be unequal; and 3) occurs during, or as a result of, the treatment or disposal of wastes.

4.3. GROUNDWATER

With respect to groundwater quality, the State CEQA Guidelines (Appendix G) inquire whether the Project would:

- Violate any water quality standards or waste discharge requirements or otherwise substantially degrade surface or groundwater quality?
- Substantially decrease groundwater supplies or interfere substantially with groundwater recharge such that the project may impede sustainable groundwater management of the basin
- Conflict with or obstruct implementation of a water quality control plan or sustainable groundwater management plan?

5. METHODOLOGY

5.1. SURFACE WATER HYDROLOGY

The Project site is located within Los Angeles County Flood Control District (LACFD) jurisdiction therefore, the City of Los Angeles has adopted the County Department of Public Works (LACDPW) Hydrology Manual as its basis of design for storm drainage facilities. The LACDPW Hydrology Manual requires projects to have drainage facilities that meet the Urban Flood level of protection. The Urban Flood is runoff from a 25-year frequency design storm falling on a saturated watershed. A 25-year frequency design storm has a probability of 1/25 of being equaled or exceeded in any year. To provide a more conservative analysis, this report analyzed a larger storm event threshold, i.e., the 50-year frequency design storm event. However, the City of Los Angeles's CEQA Threshold Guide, establishes the 50-year frequency hydrology as a result of development. This is in part because the City of Los Angeles uses the 50-year storm event to plan the existing and planned storm water drainage systems. Consequently, the use of the 50-year frequency design storm event in this analysis is in line with the CEQA threshold to determine if the project exceeds the capacity of existing or planned storm water drainage systems or provides additional sources of polluted runoff.

The Modified Rational Method was used to calculate storm water runoff. The "peak" (maximum value) runoff for a drainage area is calculated using the formula, **Q=CIA**
Where,

Q = Volumetric flow rate (cfs)
C = Runoff coefficient (dimensionless)
I = Rainfall Intensity at a given point in time (in/hr)
A = Basin area (acres)

The Modified Rational Method assumes that a steady, uniform rainfall rate will produce maximum runoff when all parts of the basin area are contributing to outflow. This occurs when the storm event lasts longer than the time of concentration. The time of concentration (Tc) is the time it takes for rain in the most hydrologically remote part of the basin area to reach the outlet. The method assumes that the runoff coefficient (C) remains constant during a storm. The runoff coefficient is a function of both the soil characteristics and the percentage of impervious surfaces in the drainage area.

Calculations were performed utilizing the hydrologic calculator (HydroCalc) developed by the Los Angeles County Department of Public Works. HydroCalc completes the full Modified Rational Method (MODRAT) calculation process and produces the peak stormwater runoff flow rates and volumes for single subareas. Detailed calculations for the proposed treatment control BMPs, based on the HydroCalc tool, are provided in **Appendix B**.

5.2. SURFACE WATER QUALITY

5.2.1. Construction

Prior to the issuance of grading permits, the applicant is required by The City to provide of a Notice of Intent (NOI) and WDID Number issued from the SWRCB in accordance with the requirements of the General Permit to ensure the potential for soil erosion and construction impacts are minimized. In accordance with the updated General Permit (Order No 2012-0006-DWQ), the following Permit Registration Documents (PRD's) are required to be submitted to the SWRCB prior to commencement of construction activities:

- Notice of Intent (NOI);
- Risk Assessment (Standard or Site-Specific);
- Particle Size Analysis (if site-specific risk assessment is performed);
- Site Map;
- SWPPP;
- Annual Fee & Certification.

The updated General Permit uses a risk-based approach for controlling erosion and sediment discharges from construction sites, since the rates of erosion and sedimentation can vary from site to site depending on factors such as duration of construction activities, climate, topography, soil condition, and proximity to receiving water bodies. The updated General Permit identifies three levels of risk with differing requirements, designated as Risk Levels 1, 2, and 3, with Risk Level 1 having the fewest permit requirements and Risk Level 3 having the most-stringent requirements.

The Risk Assessment incorporates two risk factors for a project site: sediment risk (general amount of sediment potentially discharged from the site) and receiving water risk (the risk sediment discharges can pose to receiving waters). Based on the Risk Level a project falls under, different sets of regulatory requirements are applied to the site. The main difference between Risk Levels 1, 2, and 3 are the numeric effluent standards. In Risk Level 1, there are no numeric effluent standard requirements, as it is considered a Low sediment risk and Low receiving water risk. Instead, narrative effluent limits are prescribed. In Risk Level 2, Numeric Action Levels (NALs) of pH between 6.5-8.5 and turbidity below 250 NTU are prescribed in addition to the narrative effluent limitations found in Risk Level 1 requirements. Should the NAL be exceeded during a storm event, the discharger is required to immediately determine the source associated with the exceedance and to implement corrective actions if necessary to mitigate the exceedance. Risk Level 3 dischargers must comply with Risk Level 2 requirements for NALs in addition to more rigorous monitoring requirements such as receiving water monitoring and, in some cases, bioassessment, should NALs be exceeded.

5.2.2. Operation

The Project must comply with the requirements of the City of Los Angeles LID Handbook. The LID requirements, approved by the Regional Water Quality Control Board, call for the treatment of the peak mitigation flow rate or volume of runoff produced either by a 0.75" 24-hr rainfall event or the 85th percentile rainfall event, whichever is greater. Under section 3.2.2 of the LID Manual, this post construction stormwater runoff from the new development shall be infiltrated, evapotranspirated, captured and used, and/or treated through high efficiency BMPs onsite. The rainfall intensity of the 85th percentile rainfall for the Project Site's location is 1.0 inches; therefore, the 85th percentile rainfall event governs.

The LID Manual establishes an order of priority, as specified below. Each type of BMP shall be implemented to the maximum extent feasible when determining the appropriate BMPs for a project.

1. Infiltration Systems
2. Stormwater Capture and Use
3. High Efficiency Biofiltration/Bioretention Systems
4. Combination of Any of the Above

Feasibility screening as described in the LID Manual is to be applied to determine which BMP is best suited for a proposed development project.

5.3. GROUNDWATER

This report discusses the impact of the Project as it relates to the underlying groundwater conditions of the Hollywood Subbasin of the Los Angeles Coastal Groundwater Basin. The significance of the Project as it relates to the condition of the underlying groundwater table included a review of the following existing considerations:

- Identification of the Hollywood Subbasin of the Los Angeles Coastal Groundwater Basin as the underlying groundwater basins, and description of the level, quality, direction of flow, and existing uses for the groundwater
- Description of the location, existing uses, production capacity, quality, and other pertinent data for spreading grounds and potable water wells in the vicinity (typically within a one-mile radius) and

The analysis of the proposed Project impacts on groundwater conditions includes a review of the following proposed considerations:

- Description of the rate, duration, location and quantity of extraction, dewatering, spreading, injection or other activities;
- The projected reduction in groundwater resources and any existing wells in the vicinity (typically within one-mile radius); and
- The projected change in local or regional groundwater flow patterns

In addition, short-term groundwater quality impacts could potentially occur during construction of the Project as a result of soil or shallow groundwater being exposed to construction activities, materials, wastes, and spilled materials. These potential impacts are qualitatively assessed.

6. PROJECT IMPACT ANALYSIS

6.1. CONSTRUCTION

6.1.1. Surface Water Hydrology and Quality

Implementation of the Project would result in construction activities that includes demolition of the existing parking lots and buildings on-site and excavation of existing soils.

The project is anticipated to export approximately 46,000 cubic yards of soil.

Construction activities have the potential to temporarily alter the existing drainage patterns of the Project site and also increase the permeability of a site based on increased pervious surface coverage during construction. Exposed pervious surfaces also have the potential for erosion, scour and increased sediment and associated pollutants discharging from the site during construction activities. The main pollutant of concern during construction is typically sediment and soil particles that discharge off-site due to wind, rain, and construction patterns.

Construction Best Management Practices (BMPs)

In accordance with the existing and updated General Permit, a construction SWPPP must be prepared and implemented for the Project site, and revised as necessary, as administrative or physical conditions change. The SWPPP must be made available for review upon request, shall describe construction BMPs that address pollutant source reduction, and provide measures/controls necessary to mitigate potential pollutant sources. These measures/controls include, but are not limited to: erosion controls, sediment controls, tracking controls, non-storm water management, materials & waste management, and good housekeeping practices including the following:

- Erosion control BMPs, such as hydraulic mulch, soil binders, and geotextiles and mats, protect the soil surface by covering and/or binding the soil particles. Temporary earth dikes or drainage swales may also be employed to divert runoff away from exposed areas and into more suitable locations. When implemented correctly, erosion controls will effectively reduce the sediment loads entrained in storm water runoff from construction sites.
- Sediment controls are designed to intercept and filter out soil particles that have been detached and transported by the force of water. All storm drain inlets on the project site or within the project vicinity (i.e., along streets immediately adjacent to the project boundary) should be adequately protected with an impoundment (i.e., gravel bags) around the inlet and equipped with a sediment filter (i.e., fiber roll). Bags should also be placed around areas of soil disturbing activities, such as grading or clearing.
- Stabilize all construction entrance/exit points to reduce the tracking of sediments onto adjacent streets. Wind erosion controls should be employed in conjunction with tracking controls.
- Non-storm water management BMPs prohibit the discharge of materials other than storm water, as well as reduce the potential for pollutants from discharging at their source. Examples include avoiding paving and grinding operations during the rainy season (i.e., October 1 through April 30 each year) where feasible, and performing any vehicle equipment cleaning, fueling and maintenance in designated areas that are adequately protected and contained.

- Waste management consists of implementing procedural and structural BMPs for collecting, handling, storing, and disposing of wastes generated by a construction project to prevent the release of waste materials into storm water discharges.

Prior to commencement of construction activities, the General Permit requires the Project SWPPP to be prepared in accordance with the site-specific sediment risk analyses based on the grading plans, with erosion and sediment controls proposed for each phase of construction for the Project. The phases of construction will define the maximum amount of soil disturbed, the appropriately sized sediment basins and other control measures to accommodate all active soil disturbance areas and the appropriate monitoring and sampling plans. Major phases of the construction for the Project are described below.

Mass & Rough Grading

During mass and/or rough grading, a substantial amount of soil disturbing activities or earthwork will occur. As a consequence, soil loss potential will be at its highest risk level to exceed NALs/NELs specified in the General Permit. Therefore, an effective combination of erosion and sediment controls will be implemented during this phase of construction.

This region requires the use of sediment basins or sediment traps to control the amount of sediment discharged off-site during the rainy season. Sediment basins or sediment traps generally act as primary sediment control facilities at downstream locations that provide final polish of runoff prior to discharging off-site. Therefore, they are a major element in a project's erosion and sediment control design.

Utility and Road Installation

In addition to the erosion and sediment control BMP requirements for the grading phase, the utility and road installation phase will introduce materials to the Project site that may cause or contribute to exceedances of NALs specified in the General Permit. Materials include, but are not limited to hydrated lime, concrete, mortar, Portland cement treated base, and fly ash. For this reason, pH levels shall be controlled at this stage through non-storm water management and waste and materials management BMPs.

Vertical Construction

Once utilities and roads are in place, sediment controls (such as sediment/desilting basins) found in the rough grade phase may no longer be applicable as previously designed, due to the installment of curb and gutter, catch basins, and storm drain infrastructure to convey runoff off-site per the post-construction condition. BMPs at this stage will thus be more focused on on-site sediment control BMPs and at discharge points (i.e., catch basin inlet protection). During vertical construction, a substantial amount of construction materials will be delivered to the site, and wastes generated from the site have the potential to negatively impact pH levels. Therefore, non-storm water management and waste and materials management BMPs shall be employed regularly.

Final Stabilization and Landscaping

During final stabilization and landscaping, minimal construction will be taking place and the majority of the project site will be stabilized. The majority of activities will involve planting and landscaping lots and common areas. Sediment control at discharge locations and stockpile management will be of primary concern. Good housekeeping practices will continue in this phase of construction.

Through compliance with the General Permit including the preparation of a SWPPP, implementation of BMPs, and compliance with applicable City grading regulations, construction of the Project would not cause flooding, substantially increase or decrease the amount of surface water in a water body, or result in a permanent, adverse change to flow direction. The Project would also not result in discharges that would cause: (1) pollution that would impact the quality of waters of the State to a degree which negatively impacts beneficial uses of the waters; (2) contamination of the quality of the waters of the State by waste to a degree which creates a hazard to the public health through poisoning or through the spread of diseases; or (3) nuisance that would be injurious to health, affect an entire community or neighborhood or any considerable number of persons, and occurs during or as a result of the treatment or disposal of wastes.

At this stage in the proposed Project, a detailed, site-specific Risk Assessment cannot be performed. However, based on the Project's location and known site conditions, a preliminary erosion calculation can be performed. The Project is located in a low-risk watershed and at this stage of the project the construction schedule is not identified. See **Table 6** below highlighting the various requirements for Risk Levels 1-3 due to the unknown Risk Factor of the Project at this stage.

Table 6 Risk Level Requirements

Visual Inspection						Sample Collection	
Risk Level	Quarterly Non-Storm Water Discharge	Baseline	REAP	Daily Storm BMP	Post Storm	Storm Water Discharge	Receiving Water
1	X	X		X	X		
2	X	X	X	X	X	X	
3	X	X	X	X	X	X	X [1]

Notes
[1] When numeric effluent level (NEL) exceeded.
REAP (Rain Event Action Plan)

Through compliance with applicable regulatory requirements, including compliance with an approved SWPPP and conformance with the Project's assessed Risk Level, construction of the Project would not result in discharges that would cause surface water hydrology or water quality regulatory impacts within the Ballona Creek Watershed. Therefore, impacts to surface water hydrology and water quality during construction would be less than significant.

6.1.2. Groundwater Hydrology

Construction of the Project is not anticipated to impact any water supply wells. No water supply wells are located at or within one thousand feet of the Project and the Project will not include the construction of any water supply wells. In addition, recharge of groundwater will not be impacted.

Construction of the Project will include subgrade parking structures, two levels below the ground surface. Groundwater was not encountered at 45 ½ feet bgs as mentioned in the geotechnical investigation by Geocon West, Inc., therefore dewatering activities are not anticipated. Soils are deemed suitable for the foundational implementation of subterranean parking in addition to stormwater treatment via shallow drywell. Accordingly, impacts to groundwater hydrology during construction of the Project are anticipated to be less than significant.

6.1.3. Groundwater Quality

As previously noted above, during construction of the Project temporary dewatering practices during the construction of the subterranean parking is not anticipated.

If dewatering were to occur, to protect groundwater quality, the General NPDES Permit No CAC004004 (Order No. R4-2021-0105) covers discharges to surface waters of groundwater from dewatering operations. 40 CFR section 122.48 of the Permit requires that all NPDES permits specify requirements for recording and reporting water quality monitoring results. The Monitoring and Reporting Program establishes monitoring and reporting requirements to implement federal and State requirements. The LARWQCB evaluates the test results to determine if the water can be discharged under an NPDES dewatering permit, and if so, any treatment required to remove pollutants prior to discharge. As mentioned, the Project will acquire a dewatering permit from the LARWQCB and discharges will either go to the sewer (with separate authorization from Los Angeles City Sanitation) or to the storm drain system after water quality testing of the groundwater to ensure the quality of the water is sufficient to discharge to the adjacent storm drain system. All monitoring requirements and other provisions of the Permit will be followed.

During on-site grading and building activities, hazardous materials such as fuels, paints, solvents, and concrete additives could be used and require proper management and containment during construction activities. The presence of such materials provides an opportunity for hazardous materials to be released into groundwater. To protect groundwater resources, the Project will comply with all applicable federal, state and local requirements related to the handling, storage, application and disposal of hazardous waste which will reduce the potential for construction activities of the Project to release contaminants into groundwater that could affect existing contamination, mobilize or increase the level of groundwater contamination, or cause a violation of regulatory water quality standards at an existing production well. Therefore, the Project would not result in a significant increase in groundwater contamination though hazardous materials releases and impacts on groundwater quality during Project construction would be less than significant.

6.2. OPERATION

6.2.1. Surface Water Hydrology

Development of the Project would result in the addition of landscaped areas and building areas throughout the Project Site and would keep the amount of impervious surfaces at 89 percent. See **Appendix C** for impervious percentage per the LA County Hydrology Manual Appendix D table. **Table 7** below provides an analysis of the 10-year, 25-year, and 50-year frequency design storm events following construction of the Project. Output calculations are provided in **Appendix B**.

Table 7 Proposed Condition 10-year, 25-year, and 50-year Storm Event Hydrology

10-year Storm Event			
Area	Acreage	% Imperviousness	Q_{10} (cfs)
Total Site	1.74	89%	3.1
25-year Storm Event			
Area	Acreage	% Imperviousness	Q_{25} (cfs)
Total Site	1.74	89%	4.2
50-year Storm Event			
Area	Acreage	% Imperviousness	Q_{50} (cfs)
Total Site	1.74	89%	5.1
Notes: Calculations included in **Appendix B**.			

Table 8 provides a comparison of the existing and proposed peak flows for the 10-year, 25-year, and 50-year storm events.

Table 8 Existing versus Proposed Condition for the 10-year, 25-year, and 50-year Storm Event Hydrology

10-year Storm Event		
Condition	% Imperviousness	Q_{10} (cfs)
Existing Total Site	89%	3.4
Proposed Total Site	89%	3.1
25-year Storm Event		
Condition	% Imperviousness	Q_{25} (cfs)
Existing Total Site	89%	4.5
Proposed Total Site	89%	4.2
50-year Storm Event		
Condition	% Imperviousness	Q_{50} (cfs)
Existing Total Site	89%	5.1
Proposed Total Site	89%	5.1
Notes: Calculations included in **Appendix B**.		

Based on the above, implementation of the Project would decrease the peak flow discharge for the 10-year and 25-year events as compared to the existing condition, and the 50-year storm event would be comparable to the existing condition.

Accordingly, based on the hydrology analysis, the Project would not result in on-site or off-site flooding, impact the capacity of the existing storm drain system or street conveyance system, or worsen an existing condition flood condition. In addition, the Project would not substantially reduce or increase the amount of surface water in the local water body or result in a permanent adverse change in the drainage pattern that would result in an incremental effect on the capacity of the storm existing storm drain system. Therefore, operation of the Project would result in less than significant impact on surface water hydrology.

6.2.2. Surface Water Quality

Stormwater runoff from the Project has the potential to discharge pollutants into the City and County storm drain systems. Anticipated pollutants and typical source areas include the following:

Table 9 Potential Stormwater Pollutants and Sources

Pollutant	Source
Sediment (coarse and fine)	Parking lots, driveways, building rooftops, landscape areas, roads
Nutrients (dissolved and particulates)	Landscape areas, lawns
Pesticides	Landscape areas, lawns
Pathogens	Landscape areas, lawns, building rooftops, food serving areas
Trash/debris	Parking lots, driveways, roadways, parks
Oil/grease	Parking lots, driveways, roadways, food serving areas
Metals (dissolved and particulate)	Parking lots, driveways, roadways

To meet the local MS4 Permit and LID requirements consistent with the County's LID Ordinance and the City's LID Development BMP Handbook (February 2014), stormwater management strategies will be implemented throughout the Project Site. As discussed above, a feasibility analysis of BMP strategies has been conducted for the Project Site.

Based on review of data available from Geocon West's exploration performed at the neighboring site (Geocon West Inc., 2016), the near surface (upper 5 feet) neighboring soils are variable artificial fill and generally consists of silty sand and sand with silt. The lower surface (under 5 feet) neighboring soils are variable alluvium soils consisting of silty sand, silt, silt with sand, sandy silt, sand, and sand with silt at the borings' deepest depth of 45 ½ feet. Preliminary assessment of the neighboring Project Site indicates that infiltration is likely feasible due to groundwater depth and soil conditions. Although Geocon West's percolation tests confirmed that infiltration is feasible at the neighboring site, additional testing is necessary to confirm the infiltration feasibility within the Project site based on results from GPIs 2018 soils report which noted varying soils.

The next BMP strategy on the County list, Capture and Use, would next be evaluated if infiltration were deemed infeasible. Capture and use, commonly referred to as rainwater harvesting, collects and stores stormwater for later use, thereby offsetting potable water demand and reducing pollutant loading to the storm drain system. Therefore, sufficient landscaped area with appropriate water demand is needed for the captured runoff to be directed to. In the County of Los Angeles, the use of collected stormwater is primarily limited to irrigation of landscaped surfaces. Similar to infiltration BMPs, there are several restrictions and site constraints that can limit the use of harvesting and reuse of stormwater, including if the contemplated use of harvested stormwater would violate existing codes or ordinances, such as for those that overlap with use of reclaimed water or xeriscaping, if the demands of the project are not supported by the reuse system, and if it conflicts with any other downstream water rights or poses a significant risk to human health or environmental degradation.

The next BMP strategy studied would be the use of a high removal efficiency biofiltration/bioretention BMP. Biofiltration BMPs are landscaped facilities that capture and treat stormwater runoff through a variety of physical and biological treatment processes. These facilities, also called Bioretention Planter Boxes, provide multiple benefits, including pollutant control, peak flow control, and low amounts of volume reduction through infiltration and evapotranspiration.

Due to the Project's suitable onsite soils, groundwater depth, and neighboring site design infiltration rate, Capture and Use as well as biofiltration BMPs were not further evaluated as a treatment option for the Project at this stage in the project.

As noted previously in Section 3.3.3, Geocon West's percolation tests confirmed that infiltration is feasible at the neighboring site, but additional testing will need to be conducted to confirm infiltration feasibility within the Project site. At this phase of the Project the following infiltration design approach is modeled after the infiltration results from Geocon West report.

The results of the updated deep percolation tests found a design infiltration rate of 18 inches per hour at the eastern neighboring site from the project site. Therefore, infiltration is considered feasible, and drywells were selected for infiltrating the SWQDv for the project site.

Two (2) subsurface infiltration Maxwell IV drywell systems are proposed, to be located within the Project Site. Each drywell will be a total of 16 feet deep, with the lower 4 feet consisting of a 6' diameter infiltrating drywell, and the upper 12 feet a 4' diameter concrete settling chamber. A slurry seal is included to a depth of 12 feet to prevent lateral infiltration and ensure the safety of building foundations.

In the MaxWell® IV, preliminary treatment is provided through collection and separation in a deep, large-volume chamber where silt and other heavy particles settle to the bottom. The standard MaxWell IV System has over 1,500 gallons of capacity to contain sediment and debris carried by incoming water. Floating trash, paper, pavement oil, etc. are effectively stopped by the PureFlo® Debris Shield on top of the overflow pipe.

In order to maximize infiltration within the drywells, two (2) underground detention systems will be located upstream of the drywells. These systems will temporarily detain the SWQDv and will provide constant head to the drywells during the drawdown process. Detention gallery systems are proposed to provide detention capacity in addition to the storage capacity of the drywell settling chambers (approximately 196 cu-ft of storage per drywell). The detention galleries will have a total storage of approximately 1,881 cu-ft per gallery and the volume infiltrated within 3 hours is approximately 467 cu-ft per drywell and each drywell will have a total storage of approximately 196 cu-ft. The total amount of infiltrated volume over the 96-hour regulation is approximately 14,929 cu-ft per drywell, which exceeds the SWQDv. Refer to **Appendix C** for calculations and details on drywell sizing provided by Torrent Resources. See **Table 10** below for the Drywell summary.

Table 10 Infiltration LID Summary

Drainage Area ID	Tributary Area (ft2)	Tributary Area (ac)	Percent Impervious	SWQDv Required (cf)	Drawdown (hr)	BMP	Drywell Capacity + Infiltrating Volume within 3 hours (cf)	Additional Upstream Detention Volume (cf)
A	37,897	0.87	89%	2,544	96	One (1) Maxwell IV Drywell	663	1,881
B	37,898	0.87	89%	2,543	96	One (1) Maxwell IV Drywell	663	1,880

As noted in the existing conditions description, the existing site does not have any structural or LID BMPs onsite. Therefore, implementation of the LID features proposed as part of the Project would result in a significant improvement in surface water quality runoff as compared to existing conditions. Water quality (LID) hydrologic calculations and 85^{th} Percentile 24-hour Isohyetal (Rainfall) Map are included in **Appendix C**.

Based on the required compliance with applicable LID requirements, operation of the Project would not result in discharges that would cause: (1) an incremental increase in pollution which would alter the quality of the waters of the State (Ballona Creek Watershed) to a degree which unreasonably affects beneficial uses of the waters; (2) an incremental increase of contamination of the quality of the waters of the State by waste to a degree which creates a hazard to the public health through poisoning or through the spread of diseases; or (3) an incremental increase in the nuisance that would injurious to health; affect an entire community or neighborhood, or any considerable numbers of persons; and occurs during or as a result of the treatment or disposal of wastes. Lastly, operation of the Project would not result in discharges that would cause regulatory standards to be violated in the Ballona Creek, Ballona Estuary, Ballona Lagoon, Ballona Wetlands, or the Marina del Rey Lagoon. Thus, operational impacts on surface water quality would be less than significant.

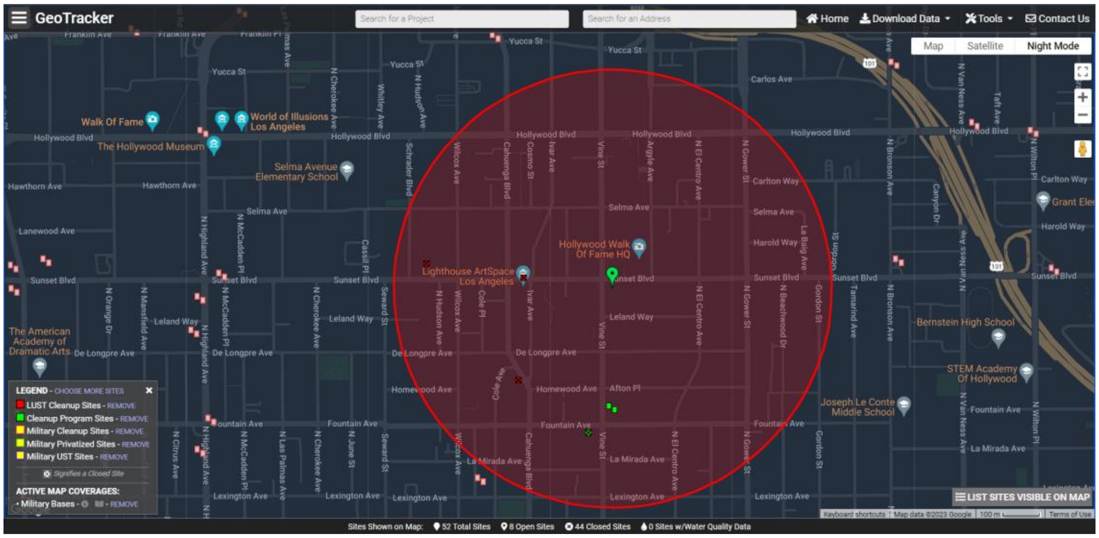
6.2.3. Groundwater Hydrology

Under the proposed conditions, region and local potable water levels and adjacent wells or well fields will not be impacted by the Project. The post-developed Project does not include any groundwater pumping and relies on the local water purveyor for water. In addition, the Project is not anticipated to adversely change the rate of direction of flow of groundwater. Accordingly, potential groundwater hydrology impacts during Project operation would be less than significant.

6.2.4. Groundwater Quality

The Geotracker website (State Water Resources Control Board) indicates there are no significant sources of soil or groundwater pollution within the project area. There are five LUST sites within a 2000 ft radius of the Project area. Four of the five LUST sites have been cleaned and removed and the remaining LUST site is in the process of open remediation. The main contaminants from the site includes Benzene, Tetrachloroethylene (PCE), Trichloroethylene (TCE) due to operations at a dry cleaners within the 2000 ft radius of the Project Area.

Accordingly, potential groundwater quality impacts during Project operation would be less than significant. See screenshot below from Geotracker.



6.3. CUMULATIVE IMPACTS

6.3.1. Surface Water Hydrology

The regional geographic context for the cumulative impact analysis on surface water hydrology is the Ballona Creek Watershed. The Project will reduce flows to this watershed due to increased perviousness as compared to the existing conditions. BMPs will be implemented during the construction phase of the Project to ensure against erosion or negative impacts to surface water hydrology. In accordance with City and County requirements, related projects and other future development projects would be required to implement BMPs to manage stormwater in accordance with applicable LID guidelines. Furthermore, the County of Los Angeles Department of Public Works would review each future development project on a case-by-case basis to ensure enough Local and regional infrastructure is available to accommodate stormwater runoff. Therefore, potential cumulative impacts associated with the Project on surface water hydrology would be less than significant.

6.3.2. Surface Water Quality

No significant impacts are anticipated regarding surface water quality during the construction or operational phases of the Project. Construction of the Project will not result in discharges that would cause regulatory water quality impacts within the Ballona Creek Watershed. In accordance with City and County requirements, related projects and other future development projects would be required to implement LID strategies and BMPs to address site runoff and prevent contaminants from entering Ballona Creek, Ballona Estuary, Ballona Lagoon, Ballona Wetlands, or the Marina del Rey Lagoon. Therefore, potential cumulative impacts associated with the Project on surface water quality would be less than significant.

6.3.3. Groundwater Hydrology

Groundwater hydrology at the Project Site is not anticipated to be impacted. In accordance with City and County requirements, related projects and other future development projects would be required to assess existing groundwater hydrology conditions and implement measures to avoid potential groundwater impacts. Therefore, potential cumulative impacts associated with the Project on groundwater hydrology would be less than significant.

6.3.4. Groundwater Quality

Groundwater quality at the Project Site is not anticipated to be impacted. The Geotracker website (State Water Resources Control Board) indicates there are no significant sources of soil or groundwater pollution within the project area and local vicinity. In accordance with City and County requirements, related projects and other future development projects would be required to assess existing groundwater quality conditions and implement measures to avoid potential impacts. Therefore, potential cumulative impacts associated with the Project on groundwater quality would be less than significant.

7. LEVEL OF SIGNIFICANCE

Based on the analysis contained in this report no significant impacts have been identified for surface water hydrology, surface water quality, or groundwater for this Project.

8. APPENDICES

Appendix A – Existing and Proposed Hydrology Exhibits

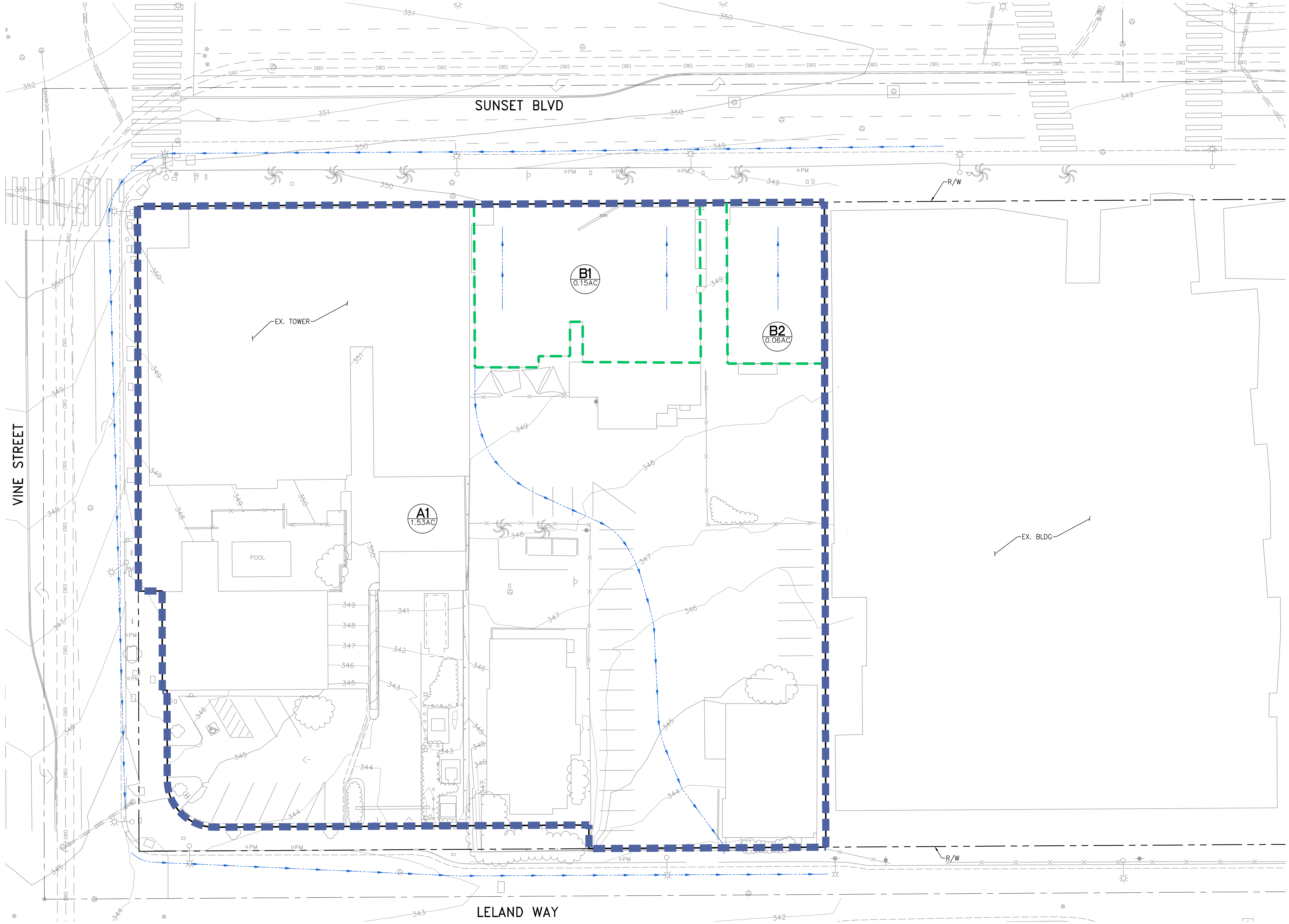
Appendix B – Existing and Proposed Hydrology Calculations

Appendix C – Water Quality Calculations

Appendix D – Geotechnical Studies

APPENDIX A

EXISTING AND PROPOSED HYDROLOGY EXHIBITS



LEGEND

HYDROLOGIC FLOWPATH/DIRECTION OF FLOW
MAJOR/PROJECT BOUNDARY
MINOR/SUB BOUNDARY
A — DRAINAGE AREA DESIGNATION
0.50AC — ACRES

SITE AREA

TOTAL AREA: 1.74AC

EXISTING CONDITIONS HYDROLOGY SUMMARY

SUBAREA	ACRES	% IMPERVIOUS	Q10 (CFS)	Q25 (CFS)	Q50 (CFS)
A	1.53	89%	2.94	3.93	4.49
B	0.21	89%	0.41	0.54	0.62
TOTAL	1.74	–	3.35	4.47	5.11

SCALE: 1" = 20'

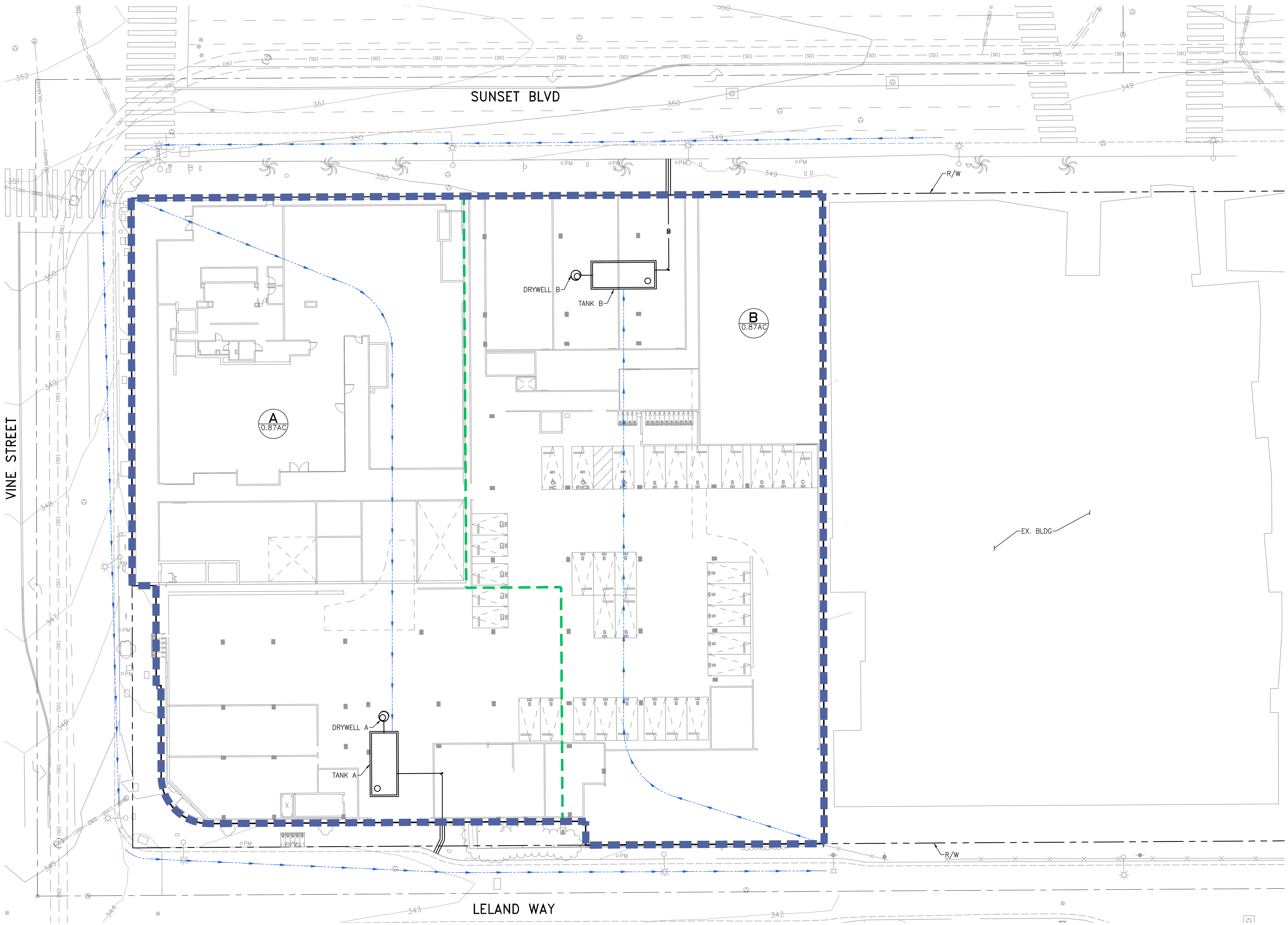
NO.	DATE	REVISION	DESCRIPTION

FUSCOE ENGINEERING
15535 Sand Canyon, Suite 100, Irvine, California 92618
tel 949.474.1960 ∘ fax 949.474.5315 ∘ www.fuscoe.com

SUNSET & VINE 2 EXISTING CONDITION HYDROLOGY MAP

DATE: 04/2023
SCALE: AS SHOWN
JOB NO.: 273-020

SHEET 1 OF 1



LEGEND

HYDROLOGIC FLOWPATH/DIRECTION OF FLOW
MAJOR/PROJECT BOUNDARY
MINOR/SUB BOUNDARY
A 0.50AC — DRAINAGE AREA DESIGNATION / ACRES

SITE AREA

TOTAL AREA: 1.74AC

PROPOSED CONDITIONS HYDROLOGY SUMMARY					
SUBAREA	ACRES	% IMPERVIOUS	Q10 (CFS)	Q25 (CFS)	Q50 (CFS)
A	0.87	89%	1.58	2.08	2.55
B	0.87	89%	1.58	2.08	2.55
TOTAL	1.74	–	3.15	4.15	5.11

NO. C63451
Exp. 09/30/24
CIVIL
STATE OF CALIFORNIA

20' 0' 10' 20'
SCALE: 1" = 20'

NO.	DATE	REVISION	DESCRIPTION

FUSCOE
ENGINEERING
15535 Sand Canyon, Suite 100, Irvine, California 92618
tel 949.474.1960 ◦ fax 949.474.5315 ◦ www.fuscoe.com

SUNSET & VINE 2 PROPOSED CONDITION HYDROLOGY MAP

DATE: 04/2023
SCALE: AS SHOWN
JOB NO.: 273–020
SHEET 1 OF 1

F:\Projects\279\020\Exhibits\Hydrology Exhibits\XH-279-020-PROPHYD.dwg (6/28/2023 2:42 PM) Plotted by: Sue Williams

APPENDIX B

EXISTING AND PROPOSED HYDROLOGY CALCULATIONS

Peak Flow Hydrologic Analysis

File location: F:/Projects/279/020/_Support Files/Reports/EIR Technical Reports/Water Resources Report/Appendices/Appendix B - Existing and Propose
Version: HydroCalc 1.0.3

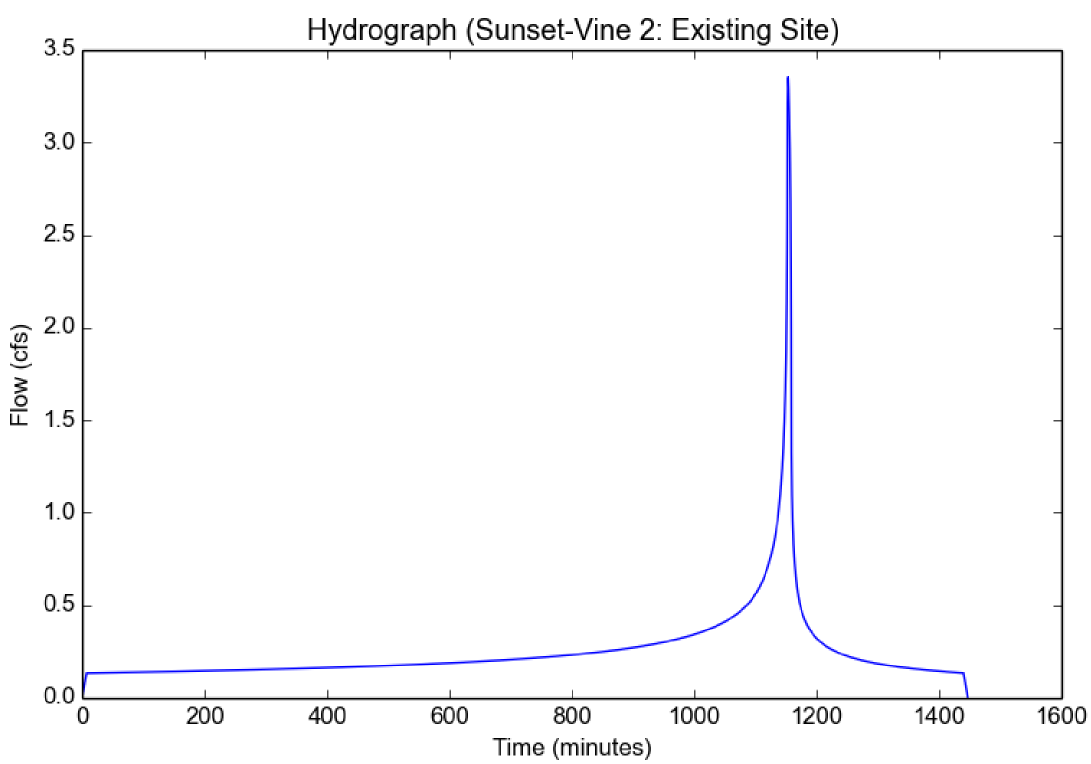
Input Parameters

Project Name	Sunset-Vine 2
Subarea ID	Existing Site
Area (ac)	1.74
Flow Path Length (ft)	450.0
Flow Path Slope (vft/hft)	0.02
50-yr Rainfall Depth (in)	6.0
Percent Impervious	0.89
Soil Type	6
Design Storm Frequency	10-yr
Fire Factor	0
LID	False

Output Results

Modeled (10-yr) Rainfall Depth (in)	4.284
Peak Intensity (in/hr)	2.1821
Undeveloped Runoff Coefficient (Cu)	0.7523
Developed Runoff Coefficient (Cd)	0.8838
Time of Concentration (min)	7.0
Clear Peak Flow Rate (cfs)	3.3555
Burned Peak Flow Rate (cfs)	3.3555
24-Hr Clear Runoff Volume (ac-ft)	0.5055
24-Hr Clear Runoff Volume (cu-ft)	22020.7003

Subarea A: 1.53 acres, 2.94 cfs, Subarea B: 0.21 acres, 0.41 cfs



Peak Flow Hydrologic Analysis

File location: F:/Projects/279/020/_Support Files/Reports/EIR Technical Reports/Water Resources Report/Appendices/Appendix B - Existing and Propose
Version: HydroCalc 1.0.3

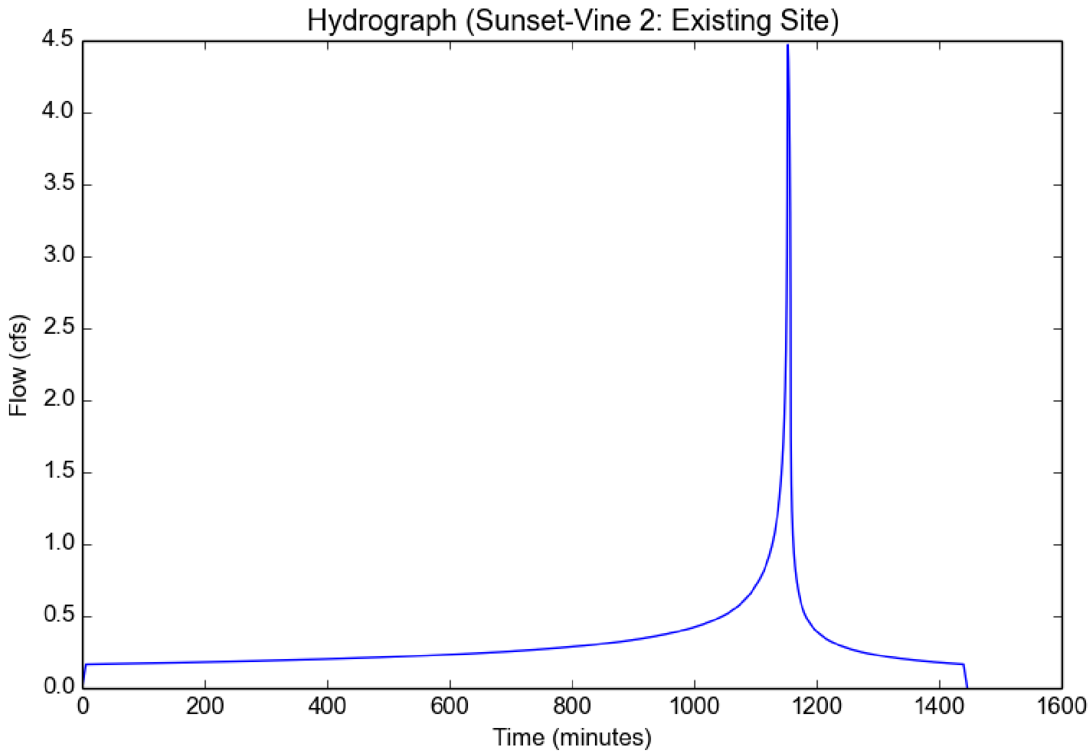
Input Parameters

Project Name	Sunset-Vine 2
Subarea ID	Existing Site
Area (ac)	1.74
Flow Path Length (ft)	450.0
Flow Path Slope (vft/hft)	0.02
50-yr Rainfall Depth (in)	6.0
Percent Impervious	0.89
Soil Type	6
Design Storm Frequency	25-yr
Fire Factor	0
LID	False

Output Results

Modeled (25-yr) Rainfall Depth (in)	5.268
Peak Intensity (in/hr)	2.8849
Undeveloped Runoff Coefficient (Cu)	0.8142
Developed Runoff Coefficient (Cd)	0.8906
Time of Concentration (min)	6.0
Clear Peak Flow Rate (cfs)	4.4704
Burned Peak Flow Rate (cfs)	4.4704
24-Hr Clear Runoff Volume (ac-ft)	0.6235
24-Hr Clear Runoff Volume (cu-ft)	27157.8985

Subarea A: 1.53 acres, 3.93 cfs, Subarea B: 0.21 acres, 0.54 cfs



Peak Flow Hydrologic Analysis

File location: F:/Projects/279/020/_Support Files/Reports/EIR Technical Reports/Water Resources Report/Appendices/Appendix B - Existing and Propose
Version: HydroCalc 1.0.3

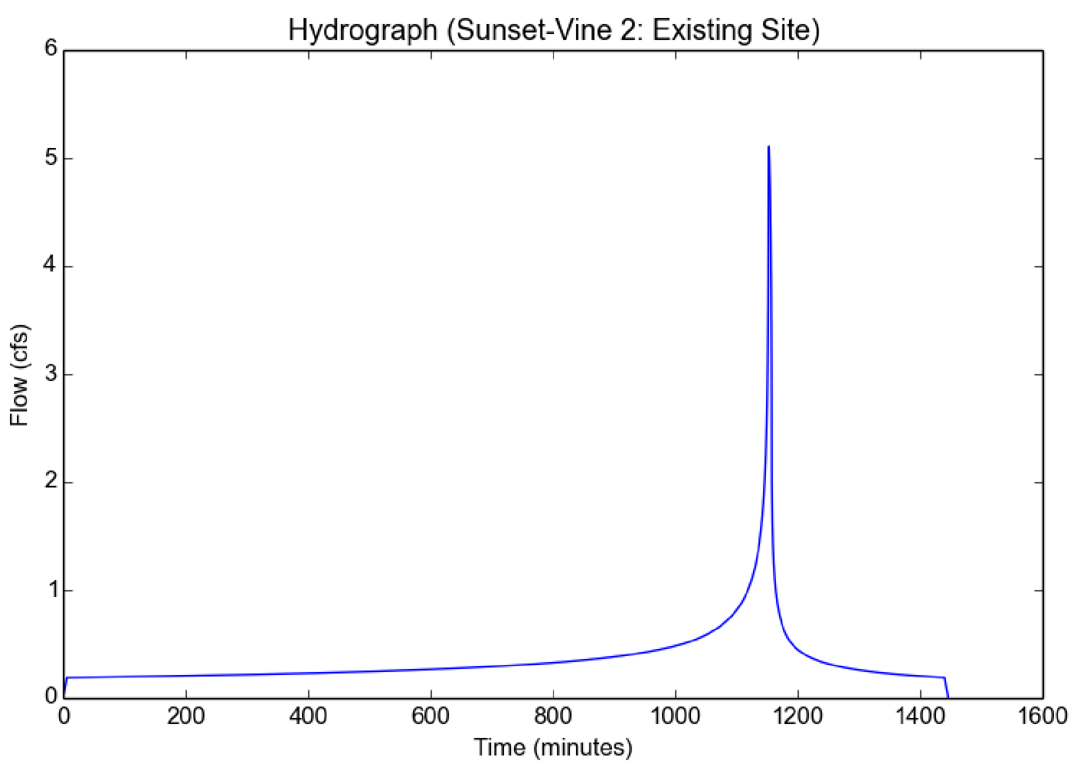
Input Parameters

Project Name	Sunset-Vine 2
Subarea ID	Existing Site
Area (ac)	1.74
Flow Path Length (ft)	450.0
Flow Path Slope (vft/hft)	0.02
50-yr Rainfall Depth (in)	6.0
Percent Impervious	0.89
Soil Type	6
Design Storm Frequency	50-yr
Fire Factor	0
LID	False

Output Results

Modeled (50-yr) Rainfall Depth (in)	6.0
Peak Intensity (in/hr)	3.2858
Undeveloped Runoff Coefficient (Cu)	0.8423
Developed Runoff Coefficient (Cd)	0.8937
Time of Concentration (min)	6.0
Clear Peak Flow Rate (cfs)	5.1093
Burned Peak Flow Rate (cfs)	5.1093
24-Hr Clear Runoff Volume (ac-ft)	0.7116
24-Hr Clear Runoff Volume (cu-ft)	30999.0233

Subarea A: 1.53 acres, 4.49 cfs,   Subarea B: 0.21 acres, 0.62 cfs



Peak Flow Hydrologic Analysis

File location: F:/Projects/279/020/_Support Files/Reports/EIR Technical Reports/Water Resources Report/Appendices/Appendix B - Existing and Propose
Version: HydroCalc 1.0.3

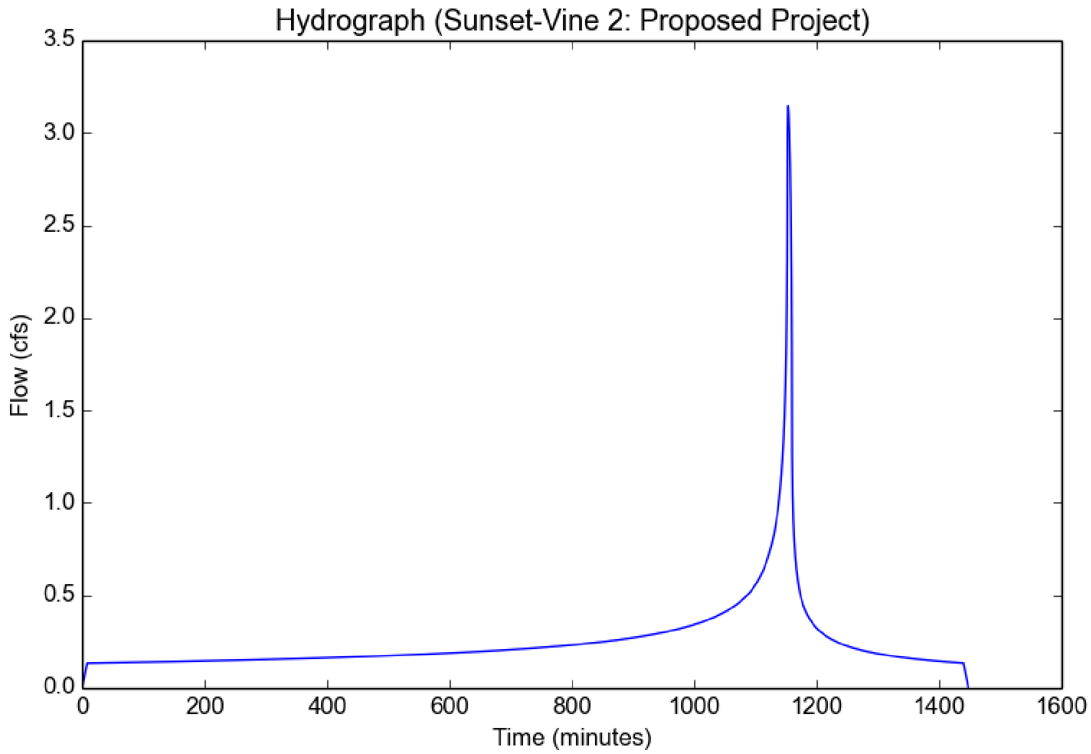
Input Parameters

Project Name	Sunset-Vine 2
Subarea ID	Proposed Project
Area (ac)	1.74
Flow Path Length (ft)	500.0
Flow Path Slope (vft/hft)	0.02
50-yr Rainfall Depth (in)	6.0
Percent Impervious	0.89
Soil Type	6
Design Storm Frequency	10-yr
Fire Factor	0
LID	False

Output Results

Modeled (10-yr) Rainfall Depth (in)	4.284
Peak Intensity (in/hr)	2.0494
Undeveloped Runoff Coefficient (Cu)	0.7377
Developed Runoff Coefficient (Cd)	0.8821
Time of Concentration (min)	8.0
Clear Peak Flow Rate (cfs)	3.1456
Burned Peak Flow Rate (cfs)	3.1456
24-Hr Clear Runoff Volume (ac-ft)	0.5055
24-Hr Clear Runoff Volume (cu-ft)	22019.9967

Subarea A: 0.87 acres, 1.58 cfs, Subarea B: 0.87 acres, 1.58 cfs



Peak Flow Hydrologic Analysis

File location: F:/Projects/279/020/_Support Files/Reports/EIR Technical Reports/Water Resources Report/Appendices/Appendix B - Existing and Propose
Version: HydroCalc 1.0.3

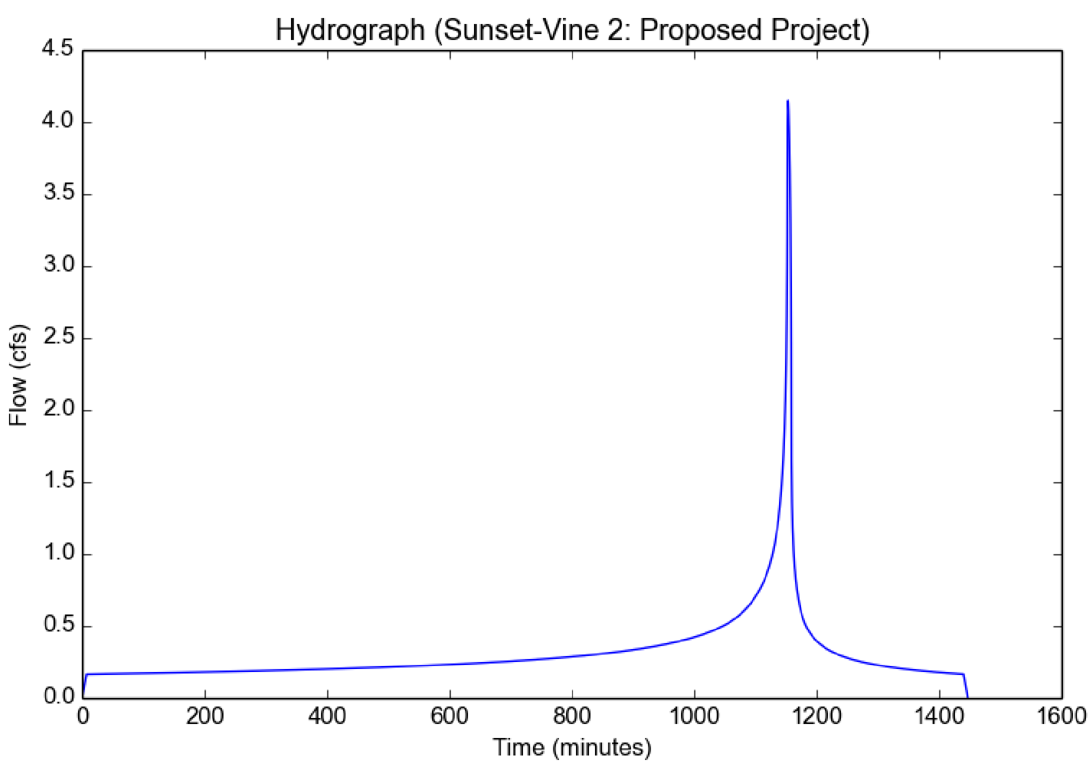
Input Parameters

Project Name	Sunset-Vine 2
Subarea ID	Proposed Project
Area (ac)	1.74
Flow Path Length (ft)	500.0
Flow Path Slope (vft/hft)	0.02
50-yr Rainfall Depth (in)	6.0
Percent Impervious	0.89
Soil Type	6
Design Storm Frequency	25-yr
Fire Factor	0
LID	False

Output Results

Modeled (25-yr) Rainfall Depth (in)	5.268
Peak Intensity (in/hr)	2.6833
Undeveloped Runoff Coefficient (Cu)	0.8
Developed Runoff Coefficient (Cd)	0.889
Time of Concentration (min)	7.0
Clear Peak Flow Rate (cfs)	4.1507
Burned Peak Flow Rate (cfs)	4.1507
24-Hr Clear Runoff Volume (ac-ft)	0.6234
24-Hr Clear Runoff Volume (cu-ft)	27157.011

Subarea A: 0.87 acres, 2.08 cfs,   Subarea B: 0.87 acres, 2.08 cfs



Peak Flow Hydrologic Analysis

File location: F:/Projects/279/020/_Support Files/Reports/EIR Technical Reports/Water Resources Report/Appendices/Appendix B - Existing and Propose
Version: HydroCalc 1.0.3

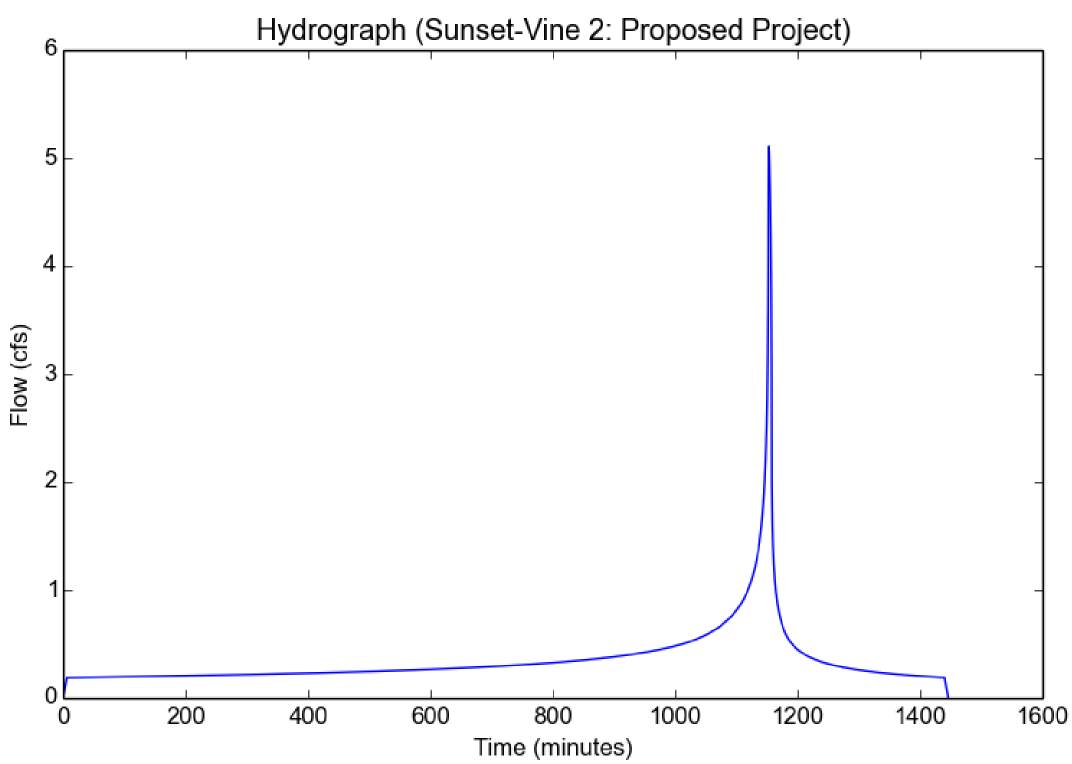
Input Parameters

Project Name	Sunset-Vine 2
Subarea ID	Proposed Project
Area (ac)	1.74
Flow Path Length (ft)	500.0
Flow Path Slope (vft/hft)	0.02
50-yr Rainfall Depth (in)	6.0
Percent Impervious	0.89
Soil Type	6
Design Storm Frequency	50-yr
Fire Factor	0
LID	False

Output Results

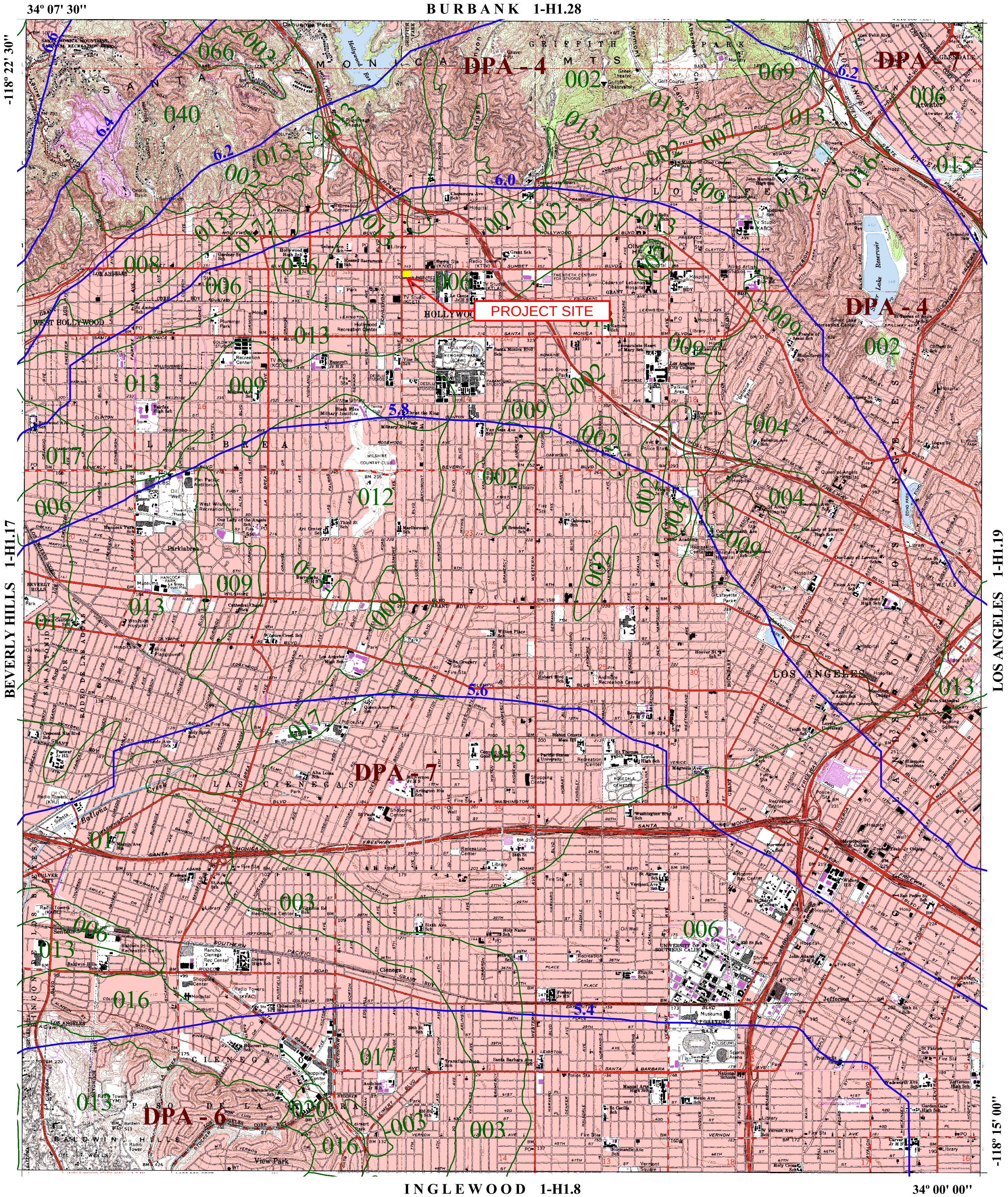
Modeled (50-yr) Rainfall Depth (in)	6.0
Peak Intensity (in/hr)	3.2858
Undeveloped Runoff Coefficient (Cu)	0.8423
Developed Runoff Coefficient (Cd)	0.8937
Time of Concentration (min)	6.0
Clear Peak Flow Rate (cfs)	5.1093
Burned Peak Flow Rate (cfs)	5.1093
24-Hr Clear Runoff Volume (ac-ft)	0.7116
24-Hr Clear Runoff Volume (cu-ft)	30999.0233

Subarea A: 0.87 acres, 2.55 cfs, Subarea B: 0.87 acres, 2.55 cfs



APPENDIX C

WATER QUALITY CALCULATIONS



016 SOIL CLASSIFICATION AREA

7.2 INCHES OF RAINFALL

DPA - 6 DEBRIS POTENTIAL AREA

N S E W

1 0 1 2 Miles

25-YEAR 24-HOUR ISOHYET REDUCTION FACTOR: 0.878
10-YEAR 24-HOUR ISOHYET REDUCTION FACTOR: 0.714

HOLLYWOOD
50-YEAR 24-HOUR ISOHYET

1-H1.18

LOS ANGELES COUNTY PUBLIC WORKS Public Service That Works

APPENDIX D

Proportion Impervious Data

Proportion Impervious Data

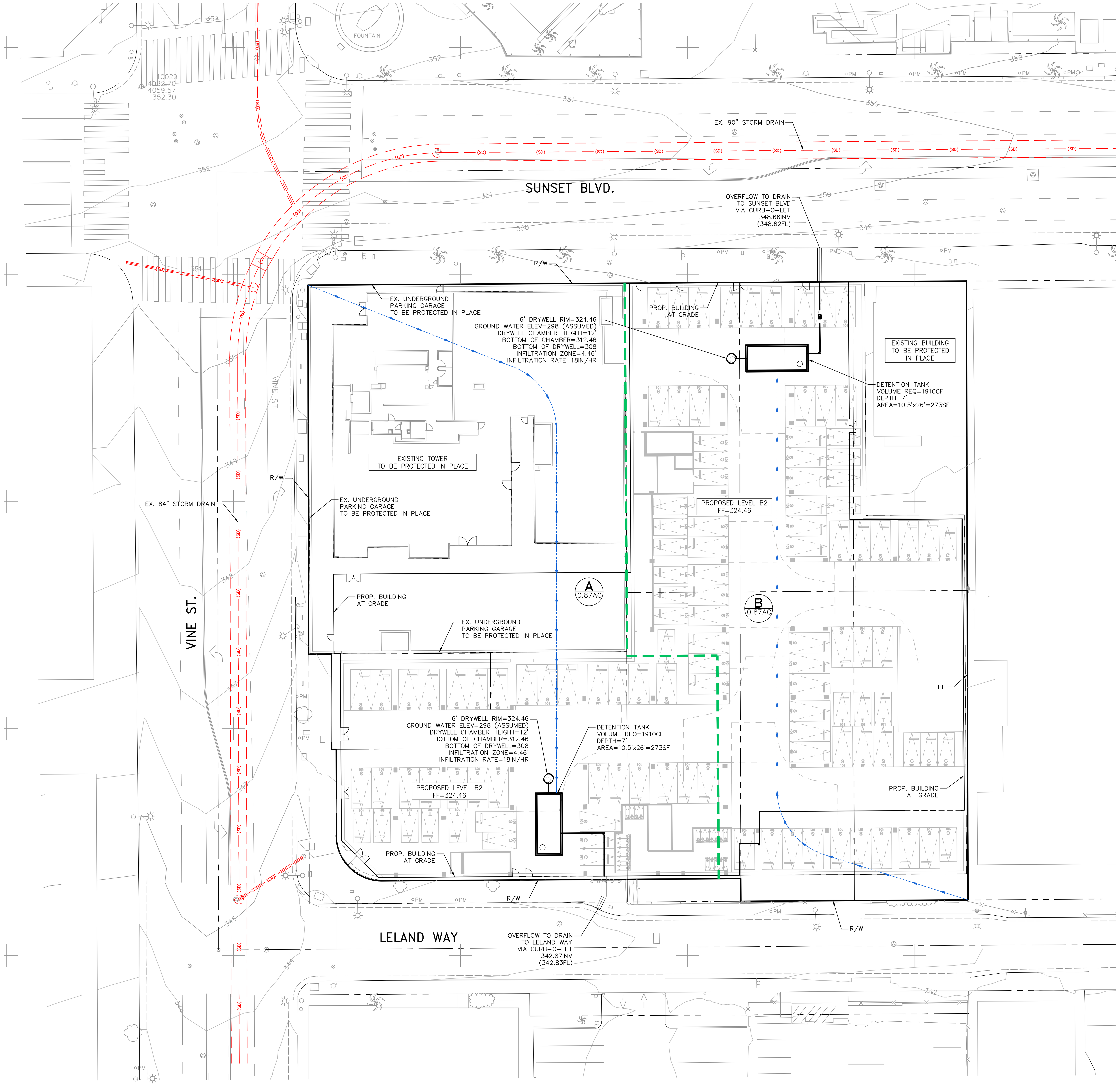
Code	Land Use Description	% Impervious
1111	High-Density Single Family Residential	42
1112	Low-Density Single Family Residential	21
1121	Mixed Multi-Family Residential	74
1122	Duplexes, Triplexes and 2-or 3-Unit Condominiums and Townhouses	55
1123	Low-Rise Apartments, Condominiums, and Townhouses	86
1124	Medium-Rise Apartments and Condominiums	86
1125	High-Rise Apartments and Condominiums	90
1131	Trailer Parks and Mobile Home Courts, High-Density	91
1132	Mobile Home Courts and Subdivisions, Low-Density	42
1140	Mixed Residential	59
1151	Rural Residential, High-Density	15
1152	Rural Residential, Low-Density	10
1211	Low- and Medium-Rise Major Office Use	91
1212	High-Rise Major Office Use	91
1213	Skyscrapers	91
1221	Regional Shopping Center	95
1222	Retail Centers (Non-Strip With Contiguous Interconnected Off-Street	96
1223	Modern Strip Development	96
1224	Older Strip Development	97
1231	Commercial Storage	90
1232	Commercial Recreation	90
1233	Hotels and Motels	96
1234	Attended Pay Public Parking Facilities	91
1241	Government Offices	91
1242	Police and Sheriff Stations	91
1243	Fire Stations	91
1244	Major Medical Health Care Facilities	74
1245	Religious Facilities	82
1246	Other Public Facilities	91
1247	Non-Attended Public Parking Facilities	91
1251	Correctional Facilities	91
1252	Special Care Facilities	74
1253	Other Special Use Facilities	86
1261	Pre-Schools/Day Care Centers	68
1262	Elementary Schools	82
1263	Junior or Intermediate High Schools	82
1264	Senior High Schools	82
1265	Colleges and Universities	47
1266	Trade Schools and Professional Training Facilities	91
1271	Base (Built-up Area)	65
1271.01	Base High-Density Single Family Residential	42
1271.02	Base Duplexes, Triplexes and 2-or 3-Unit Condominiums and T	55

Code	Land Use Description	% Impervious
1271.03	Base Government Offices	91
1271.04	Base Fire Stations	91
1271.05	Base Non-Attended Public Parking Facilities	91
1271.06	Base Air Field	45
1271.07	Base Petroleum Refining and Processing	91
1271.08	Base Mineral Extraction - Oil and Gas	10
1271.09	Base Harbor Facilities	91
1271.10	Base Navigation Aids	47
1271.11	Base Developed Local Parks and Recreation	10
1271.12	Base Vacant Undifferentiated	1
1272	Vacant Area	2
1273	Air Field	45
1274	Former Base (Built-up Area)	65
1275	Former Base Vacant Area	2
1276	Former Base Air Field	91
1311	Manufacturing, Assembly, and Industrial Services	91
1312	Motion Picture and Television Studio Lots	82
1313	Packing Houses and Grain Elevators	96
1314	Research and Development	91
1321	Manufacturing	91
1322	Petroleum Refining and Processing	91
1323	Open Storage	66
1324	Major Metal Processing	91
1325	Chemical Processing	91
1331	Mineral Extraction - Other Than Oil and Gas	10
1332	Mineral Extraction - Oil and Gas	10
1340	Wholesaling and Warehousing	91
1411	Airports	91
1411.01	Airstrip	10
1412	Railroads	15
1412.01	Railroads-Attended Pay Public Parking Facilities	91
1412.02	Railroads-Non-Attended Public Parking Facilities	91
1412.03	Railroads-Manufacturing, Assembly, and Industrial Services	91
1412.04	Railroads-Petroleum Refining and Processing	91
1412.05	Railroads-Open Storage	66
1412.06	Railroads-Truck Terminals	91
1413	Freeways and Major Roads	91
1414	Park-and-Ride Lots	91
1415	Bus Terminals and Yards	91
1416	Truck Terminals	91
1417	Harbor Facilities	91
1418	Navigation Aids	47
1420	Communication Facilities	82
1420.01	Communication Facilities-Antenna	2

Code	Land Use Description	% Impervious
1431	Electrical Power Facilities	47
1431.01	Electrical Power Facilities-Powerlines (Urban)	2
1431.02	Electrical Power Facilities-Powerlines (Rural)	1
1432	Solid Waste Disposal Facilities	15
1433	Liquid Waste Disposal Facilities	96
1434	Water Storage Facilities	91
1435	Natural Gas and Petroleum Facilities	91
1435.01	Natural Gas and Petroleum Facilities-Manufacturing, Assembly, and In	91
1435.02	Natural Gas and Petroleum Facilities-Petroleum Refining and Processing	91
1435.03	Natural Gas and Petroleum Facilities-Mineral Extraction – Oil and Gas	10
1435.04	Natural Gas and Petroleum Facilities-Vacant Undifferentiated	1
1436	Water Transfer Facilities	96
1437	Improved Flood Waterways and Structures	100
1440	Maintenance Yards	91
1450	Mixed Transportation	90
1460	Mixed Transportation and Utility	91
1460.01	Mixed Utility and Transportation-Improved Flood Waterways and Structures	100
1460.02	Mixed Utility and Transportation-Railroads	15
1460.03	Mixed Utility and Transportation-Freeways and Major Roads	91
1500	Mixed Commercial and Industrial	91
1600	Mixed Urban	89
1700	Under Construction (Use appropriate value)	91
1810	Golf Courses	3
1821	Developed Local Parks and Recreation	10
1822	Undeveloped Local Parks and Recreation	2
1831	Developed Regional Parks and Recreation	2
1832	Undeveloped Regional Parks and Recreation	1
1840	Cemeteries	10
1850	Wildlife Preserves and Sanctuaries	2
1850.01	Wildlife-Commercial Recreation	90
1850.02	Wildlife-Other Special Use Facilities	86
1850.03	Wildlife-Developed Local Parks and Recreation	10
1860	Specimen Gardens and Arboreta	15
1870	Beach Parks	10
1880	Other Open Space and Recreation	10
2110	Irrigated Cropland and Improved Pasture Land	2
2120	Non-Irrigated Cropland and Improved Pasture Land	2
2200	Orchards and Vineyards	2
2300	Nurseries	15
2400	Dairy, Intensive Livestock, and Associated Facilities	42
2500	Poultry Operations	62
2600	Other Agriculture	42
2700	Horse Ranches	42

Existing & Proposed

Code	Land Use Description	% Impervious
3100	Vacant Undifferentiated	1
3200	Abandoned Orchards and Vineyards	2
3300	Vacant With Limited Improvements (Use appropriate value)	42
3400	Beaches (Vacant)	1
4100	Water, Undifferentiated	100
4200	Harbor Water Facilities	100
4300	Marina Water Facilities	100
4400	Water Within a Military Installation	100



LEGEND

- EXISTING STORM DRAIN LINE
- PROPOSED STORM DRAIN
- PROPOSED CURB O LET
- PROPOSED SUB BOUNDARY
- HYDROLOGIC FLOWPATH/DIRECTION OF FLOW
- PROPOSED DETENTION TANK (STORMTRAP)
- PROPOSED DRYWELL
- A 0.50AC — DRAINAGE AREA DESIGNATION / ACRES

DRAINAGE AREA ID	TRIBUTARY AREA (SQFT)	TRIBUTARY AREA (AC)	PERCENT IMPERVIOUS	SWQDV REQUIRED (CF)	DRAWDOWN (HR)	BMP	DRYWELL CAPACITY + INFILTRATING VOLUME WITHIN 3 HOURS (CF)	ADDITIONAL UPSTREAM DETENTION VOLUME (CF)
A	37,897	0.87	89%	2,544	96	MAXWELL IV DRYWELL	663	1,881
B	37,897	0.87	89%	2,544	96	MAXWELL IV DRYWELL	663	1,880

CONCEPTUAL LID EXHIBIT

SUNSET & VINE 2

MAY 12, 2023

N

20' 0' 10' 20'
SCALE: 1" = 20'

FUSCOE ENGINEERING
FULL CIRCLE THINKING®

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Irvine, California 92618 fuscoe.com

F:\Projects\279\020\Exhibits\279-020-XH-Conceptual LID Exhibit.dwg (5/15/2023 2:55 PM) Plotted by: May Kyi

NOTES TO USERS

This map is for use in administering the National Flood Insurance Program. It does not necessarily identify all areas subject to flooding, particularly from local drainage sources of small size. The **community map repository** should be consulted for possible updated or additional flood hazard information.

To obtain more detailed information in areas where **Base Flood Elevations** (BFEs) and/or **floodways** have been determined, users are encouraged to consult the Flood Profiles and Floodway Data and/or Summary of Stillwater Elevations tables contained within the Flood Insurance Study (FIS) report that accompanies this FIRM. Users should be aware that BFEs shown on the FIRM represent rounded whole-foot elevations. These BFEs are intended for flood insurance rating purposes only and should not be used as the sole source of flood elevation information. Accordingly, flood elevation data presented in the FIS report should be utilized in conjunction with the FIRM for purposes of construction and/or floodplain management.

Coastal Base Flood Elevations shown on this map apply only landward of 0.0' North American Vertical Datum of 1988 (NAVD 88). Users of this FIRM should be aware that coastal flood elevations are also provided in the Summary of Stillwater Elevations table in the Flood Insurance Study report for this jurisdiction. Elevations shown in the Summary of Stillwater Elevations table should be used for construction and/or floodplain management purposes when they are higher than the elevations shown on this FIRM.

Boundaries of the **floodways** were computed at cross sections and interpolated between cross sections. The floodways were based on hydraulic considerations with regard to requirements of the National Flood Insurance Program. Floodway widths and other pertinent floodway data are provided in the Flood Insurance Study report for this jurisdiction.

Certain areas not in Special Flood Hazard Areas may be protected by **flood control structures.** Refer to Section 2.4 "Flood Protection Measures" of the Flood Insurance Study report for information on flood control structures for this jurisdiction.

The **projection** used in the preparation of this map was Universal Transverse Mercator (UTM) zone 11. The **horizontal datum** was NAD83, GRS1980 spheroid. Differences in datum, spheroid, projection or UTM zones used in the production of FIRMs for adjacent jurisdictions may result in slight positional differences in map features across jurisdiction boundaries. These differences do not affect the accuracy of this FIRM.

Flood elevations on this map are referenced to the North American Vertical Datum of 1988. These flood elevations must be compared to structure and ground elevations referenced to the same **vertical datum.** For information regarding conversion between the National Geodetic Vertical Datum of 1929 and the North American Vertical Datum of 1988, visit the National Geodetic Survey website at http://www.ngs.noaa.gov/ or contact the National Geodetic Survey at the following address:

NGS Information Services
NOAA, N/NGS12
National Geodetic Survey
SSMC–3, #9202
1315 East–West Highway
Silver Spring, MD 20910–3282

To obtain current elevation, description, and/or location information for **bench marks** shown on this map, please contact the Information Services Branch of the National Geodetic Survey at **(301) 713–3242,** or visit its website at http://www.ngs.noaa.gov/.

Base map information shown on this FIRM was derived from U.S. Geological Survey Digital Orthophoto Quadrangles produced at a scale of 1:12,000 from photography dated 1994 or later and from National Geospatial Intelligence Agency imagery produced at a scale of 1:4,000 from photography dated 2003 or later.

This map reflects more detailed and up–to–date **stream channel configurations** than those shown on the previous FIRM for this jurisdiction. The floodplains and floodways that were transferred from the previous FIRM may have been adjusted to conform to these new stream channel configurations. As a result, the Flood Profiles and Floodway Data tables *in the Flood Insurance Study report (which contains authoritative hydraulic data*) may reflect stream channel distances that differ from what is shown on this map.

Corporate limits shown on this map are based on the best data available at the time of publication. Because changes due to annexations or de–annexations may have occurred after this map was published, map users should contact appropriate community officials to verify current corporate limit locations.

Please refer to the separately printed **Map Index** for an overview map of the county showing the layout of map panels; community map repository addresses; and a Listing of Communities table containing National Flood Insurance Program dates for each community as well as a listing of the panels on which each community is located.

Contact the **FEMA Map Service Center** at 1–800–358–9616 for information on available products associated with this FIRM. Available products may include previously issued Letters of Map Change, *a Flood Insurance Study report*, and/or digital versions of this map. The FEMA Map Service Center may also be reached by Fax at 1–800–358–9620 and its website at http://www.msc.fema.gov/.

If you have **questions about this map** or questions concerning the National Flood Insurance Program in general, please call **1–877–FEMA MAP** (1–877–336–2627) or visit the FEMA website at http://www.fema.gov/.

LEGEND

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

The 1% annual chance 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.

COASTAL BARRIER RESOURCES SYSTEM (CBRS) AREAS

OTHERWISE PROTECTED AREAS (OPAs)

CBRS areas and OPAs are normally located within or adjacent to Special Flood Hazard Areas.

1% annual chance floodplain boundary
0.2% annual chance floodplain boundary
Floodway boundary
Zone D boundary
CBRS and OPA boundary
Boundary dividing Special Flood Hazard Areas of different Base Flood Elevations, flood depths or flood velocities.

~~ *513* ~~~~	Base Flood Elevation line and value; elevation in feet*
(EL 987)	Base Flood Elevation value where uniform within zone; elevation in feet*

* Referenced to the North American Vertical Datum of 1988 (NAVD 88)

(A)———(A)	Cross section line
(23)- - - - - -(23)	Transect line
97°07'30", 32°22'30"	Geographic coordinates referenced to the North American Datum of 1983 (NAD 83)
$^{42}75^{000m}$N	1000-meter Universal Transverse Mercator grid values, zone 11
6000000 FT	5000-foot grid ticks: California State Plane coordinate system, V zone (FIPSZONE 0405), Lambert Conformal Conic
DX5510 ×	Bench mark (see explanation in Notes to Users section of this FIRM panel)
• M1.5	River Mile

MAP REPOSITORIES
Refer to Map Repositories list on Map Index

EFFECTIVE DATE OF COUNTYWIDE
FLOOD INSURANCE RATE MAP
September 26, 2008
EFFECTIVE DATE(S) OF REVISION(S) TO THIS PANEL

For community map revision history prior to countywide mapping, refer to the Community Map History table located in the Flood Insurance Study report for this jurisdiction.

To determine if flood insurance is available in this community, contact your insurance agent or call the National Flood Insurance Program at 1–800–638–6620.

MAP SCALE 1" = 1000'

500 0 1000 2000 FEET
300 0 300 600 METERS

NFIP

NATIONAL FLOOD INSURANCE PROGRAM

PANEL 1605F

FIRM
FLOOD INSURANCE RATE MAP
LOS ANGELES COUNTY, CALIFORNIA
AND INCORPORATED AREAS

PANEL 1605 OF 2350
(SEE MAP INDEX FOR FIRM PANEL LAYOUT)

CONTAINS:

COMMUNITY	NUMBER	PANEL	SUFFIX
LOS ANGELES COUNTY	065043	1605	F
BEVERLY HILLS, CITY OF	060655	1605	F
LOS ANGELES, CITY OF	060137	1605	F
WEST HOLLYWOOD, CITY OF	060720	1605	F

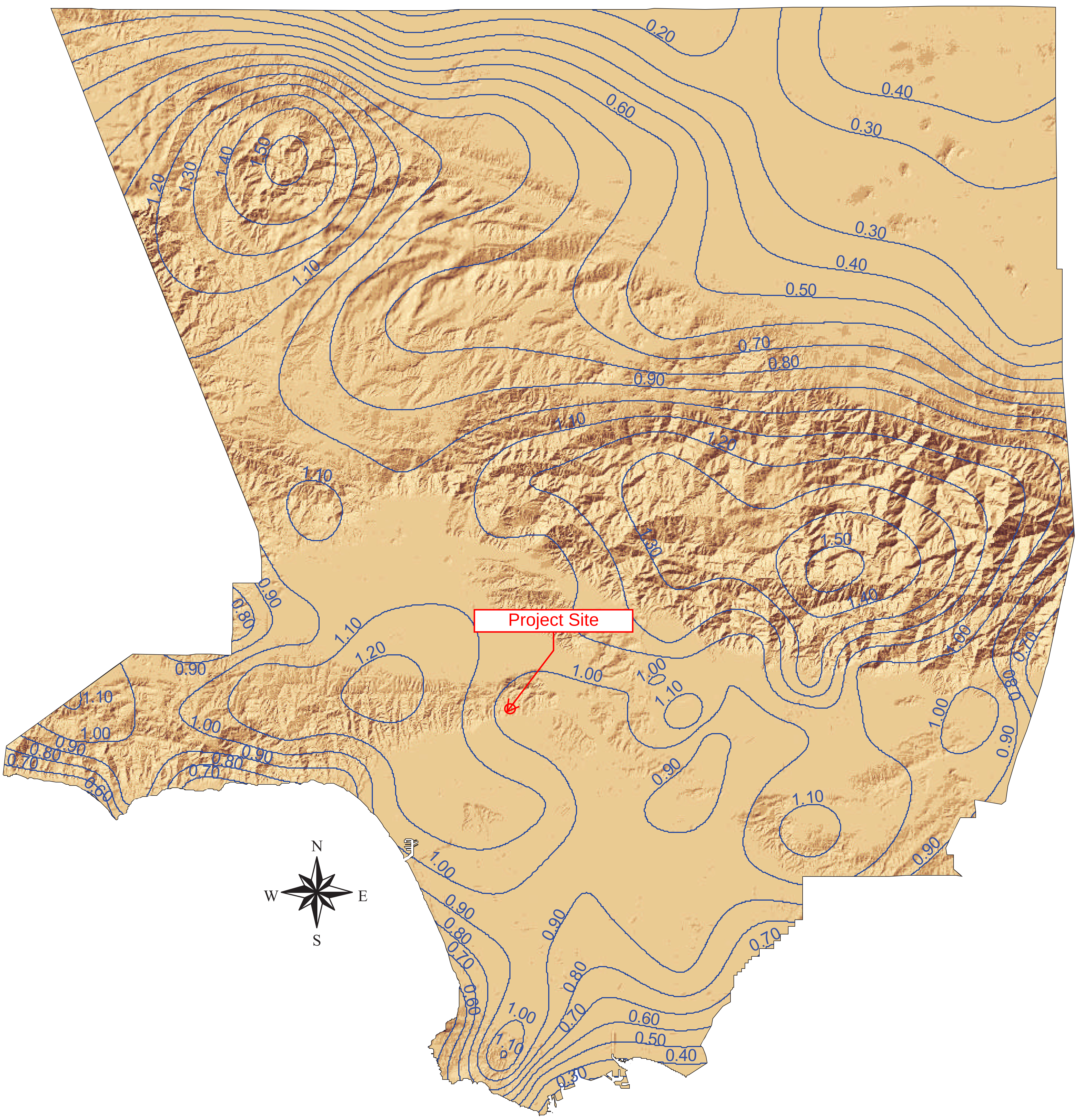
Notice to User: The **Map Number** shown below should be used when placing map orders; the **Community Number** shown above should be used on insurance applications for the subject community.

MAP NUMBER
06037C1605F

EFFECTIVE DATE
SEPTEMBER 26, 2008

Federal Emergency Management Agency

85th Percentile 24-hr Rainfall Isohyetal Map



10 0 10 20 30 40 Miles

85th Percentile 24-hr Rainfall Depth

Peak Flow Hydrologic Analysis

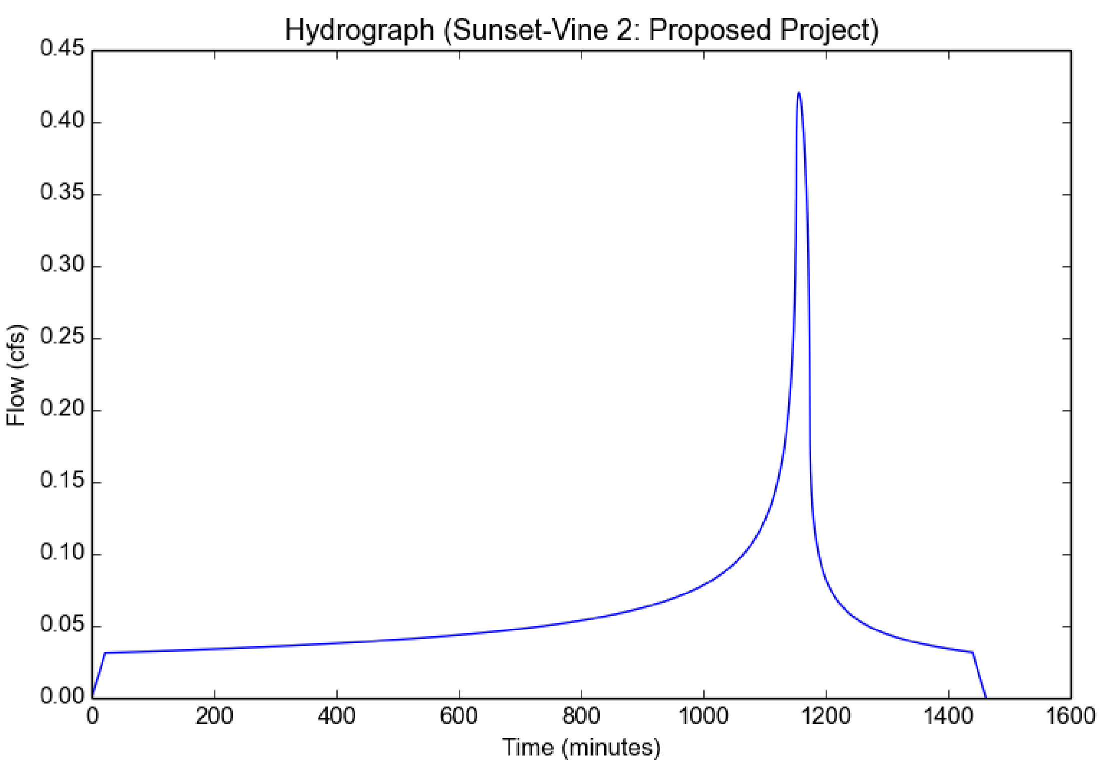
File location: F:/Projects/279/020/_Support Files/Reports/EIR Technical Reports/Water Resources Report/Appendices/Appendix C - Water Quality Calcul
Version: HydroCalc 1.0.3

Input Parameters

Project Name	Sunset-Vine 2
Subarea ID	Proposed Project
Area (ac)	1.74
Flow Path Length (ft)	500.0
Flow Path Slope (vft/hft)	0.02
85th Percentile Rainfall Depth (in)	1.0
Percent Impervious	0.89
Soil Type	6
Design Storm Frequency	85th percentile storm
Fire Factor	0
LID	True

Output Results

Modeled (85th percentile storm) Rainfall Depth (in)	1.0
Peak Intensity (in/hr)	0.2974
Undeveloped Runoff Coefficient (Cu)	0.1
Developed Runoff Coefficient (Cd)	0.812
Time of Concentration (min)	22.0
Clear Peak Flow Rate (cfs)	0.4201
Burned Peak Flow Rate (cfs)	0.4201
24-Hr Clear Runoff Volume (ac-ft)	0.1168
24-Hr Clear Runoff Volume (cu-ft)	5086.3992

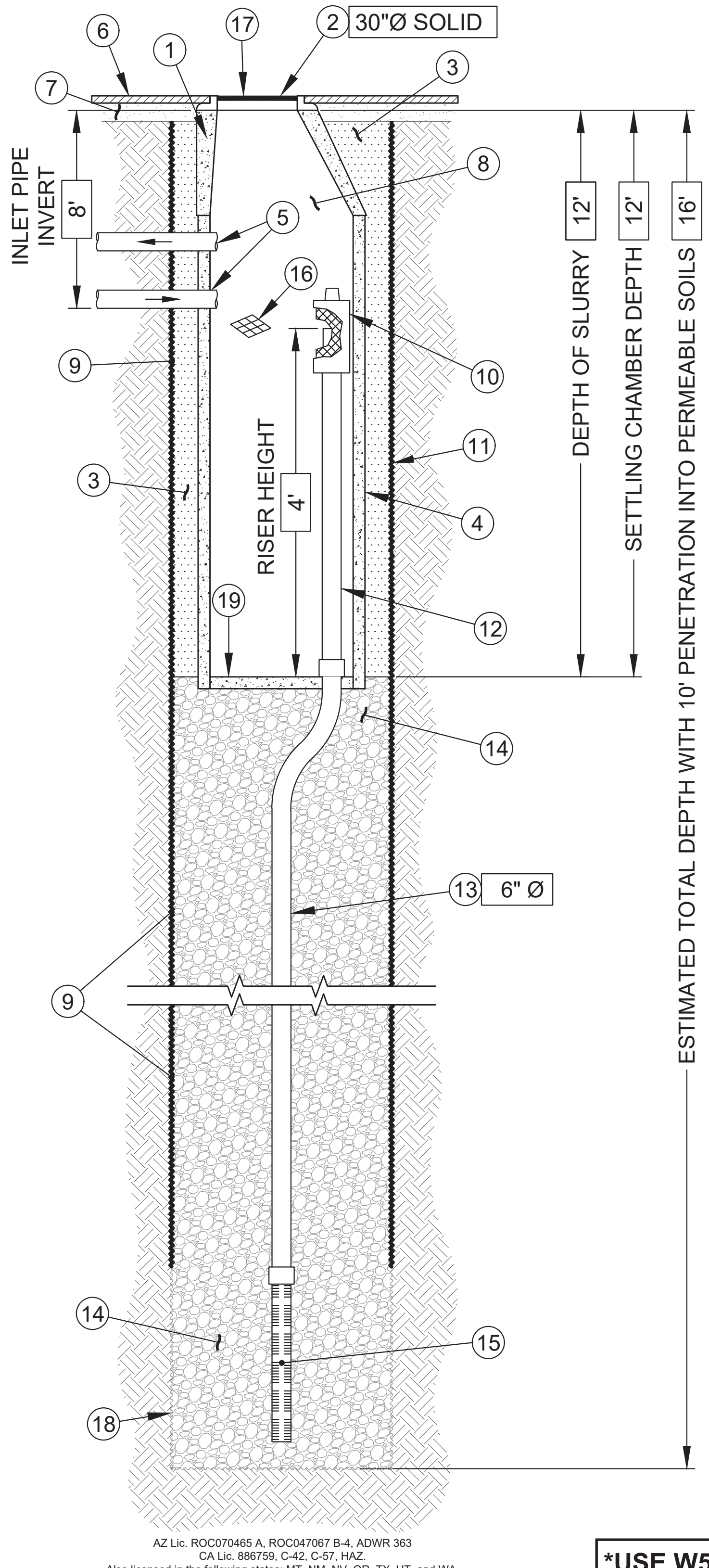


The MaxWell® IV

DRAINAGE SYSTEM DETAILS AND SPECIFICATIONS

Sunset & Vine 2

Los Angeles, CA



AZ Lic. ROC070465 A, ROC047067 B-4, ADWR 363
CA Lic. 886759, C-42, C-57, HAZ.
Also licensed in the following states: MT, NM, NV, OR, TX, UT, and WA.

U.S. Patent No. 4,923,330 - TM Trademark 1974, 1990, 2004

ITEM NUMBERS

1. **MANHOLE CONE** - MODIFIED FLAT BOTTOM.
2. **BOLTED RING & COVER** - DIAMETER & TYPE AS SHOWN. CLEAN CAST IRON PRESSURIZED COVER WITH GASKET (NEENAH R-6462-HH). BOLTED. RIM ELEVATION±0.02' OF PLANS.
3. **STABILIZED BACKFILL** - TWO-SACK SLURRY MIX.
4. ***PRE-CAST LINER -** 4000 PSI CONCRETE 48" ID. X 54" OD. **CENTER IN HOLE** AND ALIGN SECTIONS **TO MAXIMIZE BEARING SURFACE.**
5. **INLET PIPE/OUTLET PIPE (BY OTHERS).** SEE SEPARATE PLAN FOR INVERT ELEVATIONS.
6. **GRADED BASIN OR PAVING** (BY OTHERS).
7. **COMPACTED BASE MATERIAL**, IF REQUIRED (BY OTHERS).
8. **FREEBOARD DEPTH VARIES** WITH INLET PIPE ELEVATION. INCREASE SETTLING CHAMBER DEPTH AS NEEDED TO MAINTAIN ALL INLET PIPE ELEVATIONS ABOVE RISER PIPE.
9. **NON-WOVEN GEOTEXTILE SLEEVE** - MIRAFI 140 NL. MIN. 6 FT Ø. HELD APPROX. 10 FEET OFF THE BOTTOM OF EXCAVATION.
10. **PUREFLO® DEBRIS SHIELD** - ROLLED 16 GA. STEEL X 24" LENGTH WITH VENTED ANTI-SIPHON AND INTERNAL 0.265" MAX. SWO FLATTENED EXPANDED STEEL SCREEN X 12" LENGTH. **FUSION BONDED EPOXY COATED**.
11. **MIN. 6' Ø DRILLED SHAFT.**
12. **RISER PIPE** - SCH. 40 PVC MATED TO DRAINAGE PIPE AT BASE SEAL.
13. **DRAINAGE PIPE** - ADS HIGHWAY GRADE OR SCH. 40 PVC WITH TRI-A COUPLER. SUSPEND PIPE DURING BACKFILL OPERATIONS. DIAMETER AS NOTED.
14. **ROCK** - WASHED, SIZED BETWEEN 3/8" AND 1-1/2".
15. **FLOFAST® DRAINAGE SCREEN** - SCH. 40 PVC 0.120" SLOTTED WELL SCREEN WITH 32 SLOTS PER ROW/FT. OVERALL LENGTH VARIES, UP TO 120" WITH TRI-B COUPLER.
16. **ABSORBENT** - HYDROPHOBIC PETROCHEMICAL SPONGE. MIN. 128 OZ. CAPACITY. TYPICAL, 2 PER CHAMBER.
17. **FABRIC SEAL** - U.V. RESISTANT GEOTEXTILE - **TO BE REMOVED BY CUSTOMER** AT PROJECT COMPLETION. GRATED ONLY.
18. **MIN. 6' Ø DRILLED SHAFT.**
19. **BASE SEAL** - GEOTEXTILE

***USE W5 WWF AT 5" ON CENTER.**

DETAIL: IV-6-CS-CA	REVISED BY: RJA	
DRAWN ON: 09-24-20	REVISED DATE: 05-05-23	SCALE: N.T.S

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ARIZONA 602-268-0785

DRAFT

Maxwell® IV Drainage System Calculations Prepared on May 12, 2023

Project: **Sunset & Vine 2 - North - Los Angeles,CA**

Contact: Sue Williams at Fuscoe - Irvine, CA

Given: *(All depths measured from subterranean garage basement floor rim elevation which is ~25' bgs)*

Measured Infiltration Rate	54.00	in/hr
Safety Factor	3.00	
Design Infiltration Rate	18.00	in/hr
Mitigated Volume	2544	cf
Required Drawdown Time	96	hours
Depth to Emergency Overflow	0	ft
Min. Depth to Infiltration	12	ft
Groundwater Depth for Design	26	ft

Proposed:

Drywell Rock Shaft Diameter	6	ft
Drywell Chamber Depth	12	ft
Rock Porosity	40	%
Depth to Infiltration	12	ft
Drywell Bottom Depth	16	ft

Apply Safety Factor to get Design Rate.

$54.00 \frac{in}{hr} \div 3 = 18.00 \frac{in}{hr}$

Convert Design Rate from in/hr to ft/sec.

$18.00 \frac{in}{hr} \times \frac{1\ ft}{12\ in} \times \frac{1\ hr}{3600\ sec} = 0.000417 \frac{ft}{sec}$

A 6 foot diameter drywell provides 18.85 SF of infiltration area per foot of depth, plus 28.27 SF at the bottom.

For a 16 foot deep drywell, infiltration occurs between 12 feet and 16 feet below grade. This provides 4 feet of infiltration depth in addition to the bottom area. Infiltration area per drywell is calculated below.

$(4\ ft \times 18.85 \frac{ft^2}{ft}) + 28.27\ ft^2 = 104\ ft^2$

Combine design rate with infiltration area to get infiltration flowrate for each drywell.

$0.000417 \frac{ft}{sec} \times 104\ ft^2 = 0.04320 \frac{ft^3}{sec}$

Infiltration volume for each drywell based on various time frames are included below.

96 hrs: 0.0432 CFS x 96 hours x $\frac{3600\ sec}{1\ hr}$ = 14,929 cubic feet of water infiltrated.

3 hrs: 0.0432 CFS x 3 hours x $\frac{3600\ sec}{1\ hr}$ = 467 cubic feet of water infiltrated.

Chamber diameter = 4 feet. Drywell rock shaft diameter = 6 feet.

Volume provided in each drywell with chamber depth of 12 feet.

$(12\ ft \times 12.57\ ft^2) + (4\ ft \times 28.27\ ft^2 \times 40\ \%) = 196\ ft^3$

The proposed MaxWell System is composed of 1 drywell(s) .

Total volume provided = 196 ft^3

Total 3 hour infiltration volume = 467 ft^3

Total 96 hour infiltration volume = 14,929 ft^3

Total infiltration flowrate = 0.04320 $\frac{ft^3}{sec}$

Based on the total mitigated volume of 2544 CF, after subtracting the volume stored in the MaxWell System 196 CF and the volume infiltrated within 3 hours 467 CF, the residual volume of 1881 CF could be stored in a StormCapture or similar detention system and connected to the drywell system.

For any questions, please contact Ryan Adaya at 951-202-1037 or via email at Ryan.Adaya@Oldcastle.com

Torrent Resources (CA) Incorporated
9950 Alder Avenue
Bloomington, CA 92316
Phone 909-829-0740

DRAFT

Maxwell® IV Drainage System Calculations Prepared on May 12, 2023

Project: **Sunset & Vine 2 - South - Los Angeles,CA**

Contact: Sue Williams at Fuscoe - Irvine, CA

Given: *(All depths measured from subterranean garage basement floor rim elevation which is ~25' bgs)*

Measured Infiltration Rate	54.00	in/hr
Safety Factor	3.00	
Design Infiltration Rate	18.00	in/hr
Mitigated Volume	2543	cf
Required Drawdown Time	96	hours
Depth to Emergency Overflow	0	ft
Min. Depth to Infiltration	12	ft
Groundwater Depth for Design	26	ft

Proposed:

Drywell Rock Shaft Diameter	6	ft
Drywell Chamber Depth	12	ft
Rock Porosity	40	%
Depth to Infiltration	12	ft
Drywell Bottom Depth	16	ft

Apply Safety Factor to get Design Rate.

$54.00 \frac{in}{hr} \div 3 = 18.00 \frac{in}{hr}$

Convert Design Rate from in/hr to ft/sec.

$18.00 \frac{in}{hr} \times \frac{1\ ft}{12\ in} \times \frac{1\ hr}{3600\ sec} = 0.000417 \frac{ft}{sec}$

A 6 foot diameter drywell provides 18.85 SF of infiltration area per foot of depth, plus 28.27 SF at the bottom.

For a 16 foot deep drywell, infiltration occurs between 12 feet and 16 feet below grade. This provides 4 feet of infiltration depth in addition to the bottom area. Infiltration area per drywell is calculated below.

$(4\ ft \times 18.85 \frac{ft^2}{ft}) + 28.27\ ft^2 = 104\ ft^2$

Combine design rate with infiltration area to get infiltration flowrate for each drywell.

$0.000417 \frac{ft}{sec} \times 104\ ft^2 = 0.04320 \frac{ft^3}{sec}$

Infiltration volume for each drywell based on various time frames are included below.

96 hrs: 0.0432 CFS x 96 hours x $\frac{3600\ sec}{1\ hr}$ = 14,929 cubic feet of water infiltrated.

3 hrs: 0.0432 CFS x 3 hours x $\frac{3600\ sec}{1\ hr}$ = 467 cubic feet of water infiltrated.

Chamber diameter = 4 feet. Drywell rock shaft diameter = 6 feet.

Volume provided in each drywell with chamber depth of 12 feet.

$(12\ ft \times 12.57\ ft^2) + (4\ ft \times 28.27\ ft^2 \times 40\ \%) = 196\ ft^3$

The proposed MaxWell System is composed of 1 drywell(s) .

Total volume provided = 196 ft^3

Total 3 hour infiltration volume = 467 ft^3

Total 96 hour infiltration volume = 14,929 ft^3

Total infiltration flowrate = 0.04320 $\frac{ft^3}{sec}$

Based on the total mitigated volume of 2543 CF, after subtracting the volume stored in the MaxWell System 196 CF and the volume infiltrated within 3 hours 467 CF, the residual volume of 1880 CF could be stored in a StormCapture or similar detention system and connected to the drywell system.

For any questions, please contact Ryan Adaya at 951-202-1037 or via email at Ryan.Adaya@Oldcastle.com

Torrent Resources (CA) Incorporated
9950 Alder Avenue
Bloomington, CA 92316
Phone 909-829-0740

APPENDIX D

Geotechnical Studies

GEOTECHNICAL INVESTIGATION

PROPOSED MULTI-FAMILY RESIDENTIAL DEVELOPMENT
6250 SUNSET PROJECT
6234-6258 WEST SUNSET & 6235-6249 WEST LELAND WAY
LOS ANGELES, CALIFORNIA
TRACT: TR 5840, LOTS: 1-8

GEOCON
WEST, INC.

GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

PREPARED FOR

ESSEX PROPERTY TRUST
LOS ANGELES, CALIFORNIA

PROJECT NO. A9202-06-01

OCTOBER 6, 2016

Project No. A9202-06-01
October 6, 2016

Mr. Bob Linder
Essex Property Trust
5141 California Avenue, Suite 250
Irvine, California 92617

Subject: GEOTECHNICAL INVESTIGATION
PROPOSED MULTI-FAMILY RESIDENTIAL DEVELOPMENT
6250 SUNSET PROJECT
6234-6258 WEST SUNSET & 6235-6249 WEST LELAND WAY
LOS ANGELES, CALIFORNIA
TRACT: TR 5840, LOTS: 1-8

Dear Mr. Linder:

In accordance with your authorization of our proposal dated September 12, 2014 (*Revised October 21, 2014*), we have performed a geotechnical investigation for the proposed multi-family residential development located at 6250 West Sunset Boulevard in the City of Los Angeles, California. The accompanying report presents the findings of our study, and our conclusions and recommendations pertaining to the geotechnical aspects of proposed design and construction. Based on the results of our investigation, it is our opinion that the site can be developed as proposed, provided the recommendations of this report are followed and implemented during design and construction.

If you have any questions regarding this report, or if we may be of further service, please contact the undersigned.

Very truly yours,

GEOCON WEST, INC.

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GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

This report presents the results of a geotechnical investigation for the proposed multi-family residential development known as the 6250 Sunset Project and located at 6234-6258 West Sunset and 6235-6249 West Leland Way in the City of Los Angeles, California (see Vicinity Map, Figure 1). The purpose of the investigation was to evaluate subsurface soil and geologic conditions underlying the site and, based on conditions encountered, to provide conclusions and recommendations pertaining to the geotechnical aspects of design and construction.

The scope of this investigation included a review of a draft geotechnical report prepared for the site, a site reconnaissance, field exploration, laboratory testing, engineering analysis, and the preparation of this report. The site was explored on September 6 and September 7, 2016, by excavating two 8-inch diameter borings to depths of approximately 45½ feet below the existing ground surface utilizing a truck-mounted hollow-stem auger drilling machine. The approximate locations of the exploratory borings are depicted on the Site Plan (see Figure 2). A detailed discussion of the field investigation, including boring logs, is presented in Appendix A.

Laboratory tests were performed on selected soil samples obtained during the investigation to determine pertinent physical and chemical soil properties. Appendix B presents a summary of the laboratory test results.

The recommendations presented herein are based on analysis of the data obtained during the investigation and our experience with similar soil and geologic conditions. References reviewed to prepare this report are provided in the *List of References* section.

If project details vary significantly from those described herein, Geocon should be contacted to determine the necessity for review and possible revision of this report.

2. SITE AND PROJECT DESCRIPTION

The subject site is located at 6234-6258 West Sunset and 6235-6249 West Leland Way in the City of Los Angeles, California. The site is a rectangular-shaped parcel and is currently occupied by a paved parking lot. The site is bounded by paved surface parking and two-story mixed-use structures to the west, by West Sunset Boulevard to the north, by Leland Way to the south and by a three-story mixed-use structure (the Earl Carrol Theatre) to the east. The site is relatively level, with no pronounced highs or lows. Surface water drainage at the site appears to be by sheet flow along the existing ground contours to the city streets. Vegetation onsite consists of grass and trees, which are located in isolated planter areas.

Based on the information provided by the Client, it is our understanding that the proposed development will consist of a new 200-unit, seven-story multi-family residential development constructed over two levels of subterranean parking. The proposed development is depicted on the Site Plan and Cross-Section (see Figures 2 and 3).

Based on the preliminary nature of the design at this time, wall and column loads were not available. It is anticipated that column loads for the proposed structure will be up to 1,000 kips, and wall loads will be up to 10 kips per linear foot.

Once the design phase and foundation loading configuration proceeds to a more finalized plan, the recommendations within this report should be reviewed and revised, if necessary. Any changes in the design, location or elevation of any structure, as outlined in this report, should be reviewed by this office. Geocon should be contacted to determine the necessity for review and possible revision of this report.

3. GEOLOGIC SETTING

The site is located in the northern portion of the Los Angeles Basin, a coastal plain bounded by the Santa Monica Mountains on the north, the Elysian Hills and Repetto Hills on the northeast, the Puente Hills and Whittier Fault on the east, the Palos Verdes Peninsula and Pacific Ocean on the west and south, and the Santa Ana Mountains and San Joaquin Hills on the southeast. The basin is underlain by a deep structural depression which has been filled by both marine and continental sedimentary deposits underlain by a basement complex of igneous and metamorphic composition (Yerkes, et al., 1965). The basement surface within the central portion of the basin extends to a maximum depth of approximately 32,000 feet below sea level. Regionally, the site is located within the northern portion of the Peninsular Ranges geomorphic province. This geomorphic province is characterized by northwest-trending physiographic and geologic features such as the Newport-Inglewood Fault Zone located approximately 5.5 miles west of the site.

4. SOIL AND GEOLOGIC CONDITIONS

Based on our field investigation and published geologic maps of the area, the site is underlain by artificial fill and unconsolidated Holocene age alluvial deposits consisting of gravel, sand, silt and clay derived from the Santa Monica Mountains to the north (Dibblee, 1991; California Geological Survey, 2010). Detailed stratigraphic profiles of the materials encountered at the site are provided on the boring logs in Appendix A.

4.1 Artificial Fill

Artificial fill was encountered in our field explorations to a maximum depth of 5½ feet below existing ground surface. The artificial fill generally consists of dark brown and yellowish brown silty sand and sand with silt some fine gravel and varying amount of asphalt debris. The artificial fill is characterized as slightly moist and loose. The fill is likely the result of past grading or construction activities at the site. Deeper fill may exist between excavations and in other portions of the site that were not directly explored.

4.2 Alluvium

Holocene age alluvium was encountered beneath the fill. The alluvium generally consists of yellowish brown to dark yellowish brown sand, silty sand, sandy silt, silt with sand and silt with varying amounts of gravel and cobbles in the granular soils. The alluvial soils are primarily fine- to medium-grained, slightly moist and very loose to very dense or firm to stiff.

5. GROUNDWATER

Review of the Seismic Hazard Zone Report for the Hollywood Quadrangle (California Division of Mines and Geology [CDMG], 1998) indicates that the historically highest groundwater level in the area is approximately 50 feet beneath the ground surface. Groundwater information presented in this document is generated from data collected in the early 1900's to the late 1990s. Based on current groundwater basin management practices, it is unlikely that groundwater levels will ever exceed the historic high levels.

Groundwater was not encountered in our field explorations drilled to a maximum depth of 45½ feet below the existing ground surface. Based on the historic high groundwater levels in the site vicinity, the lack of groundwater in our borings, and the depth of proposed construction, groundwater is neither expected to be encountered during construction, nor have a detrimental effect on the project. However, it is not uncommon for groundwater levels to vary seasonally or for groundwater seepage conditions to develop where none previously existed, especially in impermeable fine-grained soils which are heavily irrigated or after seasonal rainfall. In addition, recent requirements for stormwater infiltration could result in shallower seepage conditions in the immediate site vicinity. Proper surface drainage of irrigation and precipitation will be critical for future performance of the project. Recommendations for drainage are provided in the S*urface Drainage* section of this report (see Section 7.25).

6. GEOLOGIC HAZARDS

6.1 Surface Fault Rupture

The numerous faults in Southern California include active, potentially active, and inactive faults. The criteria for these major groups are based on criteria developed by the California Geological Survey (CGS, formerly known as CDMG) for the Alquist-Priolo Earthquake Fault Zone Program (CGS, 2016; Bryant and Hart, 2007). By definition, an active fault is one that has had surface displacement within Holocene time (about the last 11,000 years). A potentially active fault has demonstrated surface displacement during Quaternary time (approximately the last 1.6 million years), but has had no known Holocene movement. Faults that have not moved in the last 1.6 million years are considered inactive.

The site is not within a state-designated Alquist-Priolo Earthquake Fault Zone (CGS, 2014b, CGS, 2016) or a city-designated Preliminary Fault Rupture Study Area (City of Los Angeles, 2016) for surface fault rupture hazards. No active or potentially active faults with the potential for surface fault rupture are known to pass directly beneath the site. Therefore, the potential for surface rupture due to faulting occurring beneath the site during the design life of the proposed development is considered low. However, the site is located in the seismically active Southern California region, and could be subjected to moderate to strong ground shaking in the event of an earthquake on one of the many active Southern California faults. The faults in the vicinity of the site are shown in Figure 4, Regional Fault Map.

The closest surface trace of an active fault to the site is the Hollywood Fault located approximately 0.5 mile to the north (CGS, 2014b). Other nearby active faults include the Raymond Fault, the Newport-Inglewood Fault, the Santa Monica Fault, and the Verdugo Fault located approximately 3.9 miles northeast, 5.5 miles west, 5.6 miles west, and 6.2 miles northeast of the site, respectively (Ziony and Jones, 1989). The active San Andreas Fault Zone is located approximately 33 miles northeast of the site.

The closest potentially active fault to the site is the MacArthur Park Fault located approximately 1.3 miles to the southeast (Ziony and Jones, 1989). Other nearby potentially active faults are the Overland Avenue Fault, the Charnock Fault, and the Coyote Pass Fault located approximately 6.3 miles southwest, 7.7 miles southwest, and 8.3 miles southeast of the site, respectively (Ziony and Jones, 1989).

Several buried thrust faults, commonly referred to as blind thrusts, underlie the Los Angeles Basin at depth. These faults are not exposed at the ground surface and are typically identified at depths greater than 3.0 kilometers. The October 1, 1987 M_w 5.9 Whittier Narrows earthquake and the January 17, 1994 M_w 6.7 Northridge earthquake were a result of movement on the Puente Hills Blind Thrust and the Northridge Thrust, respectively. These thrust faults and others in the Los Angeles area are not exposed at the surface and do not present a potential surface fault rupture hazard at the site; however, these deep thrust faults are considered active features capable of generating future earthquakes that could result in moderate to significant ground shaking at the site.

6.2 Seismicity

As with all of Southern California, the site has experienced historic earthquakes from various regional faults. The seismicity of the region surrounding the site was formulated based on research of an electronic database of earthquake data. The epicenters of recorded earthquakes with magnitudes equal to or greater than 5.0 in the site vicinity are depicted on Figure 5, Regional Seismicity Map. A partial list of moderate to major magnitude earthquakes that have occurred in the Southern California area within the last 100 years is included in the following table.

LIST OF HISTORIC EARTHQUAKES

Earthquake (Oldest to Youngest)	Date of Earthquake	Magnitude	Distance to Epicenter (Miles)	Direction to Epicenter
San Jacinto-Hemet area	April 21, 1918	6.8	80	ESE
Near Redlands	July 23, 1923	6.3	62	E
Long Beach	March 10, 1933	6.4	39	SE
Tehachapi	July 21, 1952	7.5	74	NW
San Fernando	February 9, 1971	6.6	22	NNW
Whittier Narrows	October 1, 1987	5.9	14	E
Sierra Madre	June 28, 1991	5.8	22	ENE
Landers	June 28, 1992	7.3	108	E
Big Bear	June 28, 1992	6.4	86	E
Northridge	January 17, 1994	6.7	14	WNW
Hector Mine	October 16, 1999	7.1	122	ENE

The site could be subjected to strong ground shaking in the event of an earthquake. However, this hazard is common in Southern California and the effects of ground shaking can be mitigated if the proposed structures are designed and constructed in conformance with current building codes and engineering practices.

6.3 Seismic Design Criteria

The following table summarizes summarizes site-specific design criteria obtained from the 2013 California Building Code (CBC; Based on the 2012 International Building Code [IBC] and ASCE 7-10), Chapter 16 Structural Design, Section 1613 Earthquake Loads. The data was calculated using the computer program *U.S. Seismic Design Maps*, provided by the USGS. The short spectral response uses a period of 0.2 second. The values presented below are for the risk-targeted maximum considered earthquake (MCE_R).

2013 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value	2013 CBC Reference
Site Class	D	Table 1613.3.2
MCE_R Ground Motion Spectral Response Acceleration – Class B (short), S_S	2.372g	Figure 1613.3.1(1)
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.880g	Figure 1613.3.1(2)
Site Coefficient, F_A	1.0	Table 1613.3.3(1)
Site Coefficient, F_V	1.5	Table 1613.3.3(2)
Site Class Modified MCE_R Spectral Response Acceleration (short), S_{MS}	2.372g	Section 1613.3.3 (Eqn 16-37)
Site Class Modified MCE_R Spectral Response Acceleration – (1 sec), S_{M1}	1.320g	Section 1613.3.3 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	1.582g	Section 1613.3.4 (Eqn 16-39)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.880g	Section 1613.3.4 (Eqn 16-40)

The table below presents the mapped maximum considered geometric mean (MCE_G) seismic design parameters for projects located in Seismic Design Categories of D through F in accordance with ASCE 7-10.

ASCE 7-10 PEAK GROUND ACCELERATION

Parameter	Value	ASCE 7-10 Reference
Mapped MCE_G Peak Ground Acceleration, PGA	0.918g	Figure 22-7
Site Coefficient, F_{PGA}	1.0	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.918g	Section 11.8.3 (Eqn 11.8-1)

The Maximum Considered Earthquake Ground Motion (MCE) is the level of ground motion that has a 2 percent chance of exceedance in 50 years, with a statistical return period of 2,475 years. According to the 2013 California Building Code and ASCE 7-10, the MCE is to be utilized for the evaluation of liquefaction, lateral spreading, seismic settlements, and it is our understanding that the intent of the Building code is to maintain "Life Safety" during a MCE event. The Design Earthquake Ground Motion (DE) is the level of ground motion that has a 10 percent chance of exceedance in 50 years, with a statistical return period of 475 years.

Deaggregation of the MCE peak ground acceleration was performed using the USGS 2008 Probabilistic Seismic Hazard Analysis (PSHA) Interactive Deaggregation online tool. The result of the deaggregation analysis indicates that the predominant earthquake contributing to the MCE peak ground acceleration is characterized as a 6.68 magnitude event occurring at a hypocentral distance of 5.1 kilometers from the site.

Deaggregation was also performed for the Design Earthquake (DE) peak ground acceleration, and the result of the analysis indicates that the predominant earthquake contributing to the DE peak ground acceleration is characterized as a 6.66 magnitude occurring at a hypocentral distance of 9.3 kilometers from the site.

Conformance to the criteria in the above tables for seismic design does not constitute any kind of guarantee or assurance that significant structural damage or ground failure will not occur if a large earthquake occurs. The primary goal of seismic design is to protect life, not to avoid all damage, since such design may be economically prohibitive.

6.4 Liquefaction Potential

Liquefaction is a phenomenon in which loose, saturated, relatively cohesionless soil deposits lose shear strength during strong ground motions. Primary factors controlling liquefaction include intensity and duration of ground motion, gradation characteristics of the subsurface soils, in-situ stress conditions, and the depth to groundwater. Liquefaction is typified by a loss of shear strength in the liquefied layers due to rapid increases in pore water pressure generated by earthquake accelerations.

The current standard of practice, as outlined in the "Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Liquefaction in California" and "Special Publication 117A, Guidelines for Evaluating and Mitigating Seismic Hazards in California" requires liquefaction analysis to a depth of 50 feet below the lowest portion of the proposed structure. Liquefaction typically occurs in areas where the soils below the water table are composed of poorly consolidated, fine to medium-grained, primarily sandy soil. In addition to the requisite soil conditions, the ground acceleration and duration of the earthquake must also be of a sufficient level to induce liquefaction.

The State of California Seismic Hazard Zone Map for the Hollywood Quadrangle (CDMG, 1999; CGS, 2014b) indicates that the site is not located in an area designated as "liquefiable." However, a review of the County of Los Angeles Seismic Safety Element (Leighton, 1990) indicates that the site is located within an area identified as having a potential for liquefaction. It is our opinion, due to the depth of the historic high groundwater levels in the site vicinity (50 feet) and the relatively well-consolidated nature of the alluvial soils (see Figures B3 and B4), that the potential for liquefaction and associated ground deformations beneath the site is very low.

6.5 Slope Stability

The topography at the site is relatively level and the topography in the immediate site vicinity slopes gently to the southwest. The site is not located within a City of Los Angeles Hillside Grading Area and not within a Hillside Ordinance Area (City of Los Angeles, 2016). The County of Los Angeles Safety Element (Leighton, 1990), indicates the site is not within an area identified as having a potential for slope instability. Additionally, the site is not within an area identified as having a potential for seismic slope instability (CDMG, 1999). There are no known landslides near the site, nor is the site in the path of any known or potential landslides. Therefore, the potential for slope stability hazards to adversely affect the proposed development is considered low.

6.6 Earthquake-Induced Flooding

Earthquake-induced flooding is inundation caused by failure of dams or other water-retaining structures due to earthquakes. The Los Angeles County Safety Element (Leighton, 1990) indicates that the site is located within the Mulholland Dam inundation area. However, this reservoir, as well as others in California, are continually monitored by various governmental agencies (such as the State of California Division of Safety of Dams and the U.S. Army Corps of Engineers) to guard against the threat of dam failure. Current design, construction practices, and ongoing programs of review, modification, or total reconstruction of existing dams are intended to ensure that all dams are capable of withstanding the maximum considered earthquake (MCE) for the site. Therefore, the potential for inundation at the site as a result of an earthquake-induced dam failure is considered low.

6.7 Tsunamis, Seiches, and Flooding

The site is not located within a coastal area. Therefore, tsunamis, seismic sea waves, are not considered a significant hazard at the site.

Seiches are large waves generated in enclosed bodies of water in response to ground shaking. No major water-retaining structures are located immediately up gradient from the project site. Therefore, flooding resulting from a seismically-induced seiche is considered unlikely.

The site is within an area of minimal flooding (Zone X) as defined by the Federal Emergency Management Agency (LACDPW, 2016b).

6.8 Oil Fields & Methane Potential

Based on a review of the California Division of Oil, Gas and Geothermal Resources (DOGGR) Oil and Gas Well Location Map W1-5, the site is not located within the limits of an oilfield and oil or gas wells are not located in the immediate site vicinity. However, due to the voluntary nature of record reporting by the oil well drilling companies, wells may be improperly located or not shown on the location map and undocumented wells could be encountered during construction. Any wells encountered during construction will need to be properly abandoned in accordance with the current requirements of the DOGGR.

The site is not located within the boundaries of a city-designated Methane Zone or Methane Buffer Zone (City of Los Angeles, 2016). Since the site is not located within the boundaries of a known oil field, the potential for the presence of methane or other volatile gases at the site is considered low. However, should it be determined that a methane study is required for the proposed development it is recommended that a qualified methane consultant be retained to perform the study and provide mitigation measures as necessary.

6.9 Subsidence

Subsidence occurs when a large portion of land is displaced vertically, usually due to the withdrawal of groundwater, oil, or natural gas. Soils that are particularly subject to subsidence include those with high silt or clay content. The site is not located within an area of known ground subsidence. No large-scale extraction of groundwater, gas, oil, or geothermal energy is occurring or planned at the site or in the general site vicinity. There appears to be little or no potential for ground subsidence due to withdrawal of fluids or gases at the site.

7. CONCLUSIONS AND RECOMMENDATIONS

7.1 General

7.1.1 It is our opinion that neither soil nor geologic conditions were encountered during the investigation that would preclude the construction of the proposed development provided the recommendations presented herein are followed and implemented during design and construction.

7.1.2 Up to 5½ feet of existing artificial fill was encountered during the site investigation. The existing fill encountered is believed to be the result of past grading and construction activities at the site. Deeper fill may exist in other areas of the site that were not directly explored. The existing fill and site soils are suitable for re-use as engineered fill, if needed, provided the recommendations in the *Grading* section of this report are followed (see Section 7.4). Excavation for the proposed subterranean levels are anticipated to penetrate through the existing fill and expose competent alluvial soils throughout the excavation bottom.

7.1.3 Based on these considerations, the proposed structure may be supported on a conventional foundation system deriving support in the competent alluvial soils found at the excavation bottom. For the purposes of this report, is has been assumed that foundations will be at or below a depth of 21 feet below the ground surface. Foundations should be deepened as necessary to penetrate through any unsuitable soils and derive support in competent alluvial soils. All foundation excavations must be observed and approved in writing by the Geotechnical Engineer prior to placement of steel or concrete. Recommendations for *Foundation Design* are provided in Section 7.6 of this report.

7.1.4 The concrete slab-on-grade and ramp for the subterranean level may bear in newly placed engineered fill or directly on the competent undisturbed alluvial soil at the excavation bottom. Any soils that are disturbed should be properly compacted for slab and ramp support, as necessary.

7.1.5 Foundations for small outlying structures, such as block walls up to 6 feet in height, planter walls or trash enclosures, which will not be tied to the proposed structure, may be supported on conventional foundations deriving support on a minimum of 12 inches of newly placed engineered fill which extends laterally at least 12 inches beyond the foundation area. Where excavation and compaction cannot be performed or is undesirable, foundations may derive support directly in the competent undisturbed alluvial soils. Due to the depth to alluvial soils, special foundation recommendations may be required and can be provided under separate cover, if needed. If the soils exposed in the excavation bottom are soft or loose, compaction of the soils will be required prior to placing steel or concrete. Compaction of the foundation excavation bottom is typically accomplished with a compaction wheel or mechanical whacker and must be observed and approved by a Geocon representative.

7.1.6 Excavations on the order of 25 feet in vertical height may be required for construction of the subterranean level, including the excavations for the foundation system, as indicated on Cross-Sections A-A' and B-B' (see Figure 2). Due to the depth of the excavation and the proximity to the property lines, city streets, and adjacent offsite structures, excavation of the proposed subterranean level will require sloping and/or shoring measures in order to provide a stable excavation. Where shoring is required it is recommended that a soldier pile shoring system be utilized. In addition, where the proposed excavation will be deeper than and adjacent to an offsite structure, the proposed retaining wall and shoring systems should be designed to resist the surcharge imposed by the adjacent offsite structure. Recommendations for *Shoring* are provided in Section 7.18.

7.1.7 Due to the nature of the proposed design and intent for subterranean levels, waterproofing of subterranean walls and slabs is suggested. Particular care should be taken in the design and installation of waterproofing to avoid moisture problems, or actual water seepage into the structure through any normal shrinkage cracks which may develop in the concrete walls, floor slab, foundations and/or construction joints. The design and inspection of the waterproofing is not the responsibility of the geotechnical engineer. A waterproofing consultant should be retained in order to recommend a product or method, which would provide protection to subterranean walls, floor slabs and foundations.

7.1.8 Based on the results of percolation testing performed at the site, a stormwater infiltration system is considered feasible for this project. Recommendations for infiltration are provided in the *Stormwater Infiltration* section of this report (see Section 7.24).

7.1.9 Once the design and foundation loading configuration for the proposed structure proceeds to a more finalized plan, the recommendations within this report should be reviewed and revised, if necessary. Based on the final foundation loading configurations, the potential for settlement should be re-evaluated by this office.

7.1.10 Any changes in the design, location or elevation, as outlined in this report, should be reviewed by this office. Geocon should be contacted to determine the necessity for review and possible revision of this report.

7.1.11 The most recent ASTM standards apply to this project and must be utilized, even if older ASTM standards are indicated in this report.

7.2 Soil and Excavation Characteristics

7.2.1 The in-situ soils can be excavated with moderate effort using conventional excavation equipment. Some caving should be anticipated in unshored excavations, especially where granular soils are encountered.

7.2.2 It is the responsibility of the contractor to ensure that all excavations and trenches are properly shored and maintained in accordance with applicable OSHA rules and regulations to maintain safety and maintain the stability of existing adjacent improvements.

7.2.3 All onsite excavations must be conducted in such a manner that potential surcharges from existing structures, construction equipment, and vehicle loads are resisted. The surcharge area may be defined by a 1:1 projection down and away from the bottom of an existing foundation or vehicle load. Penetrations below this 1:1 projection will require special excavation measures such as sloping or shoring. Excavation recommendations are provided in the *Temporary Excavations* section of this report (see Section 7.17).

7.2.4 Based on depth of the proposed subterranean levels, the proposed structure would not be prone to the effects of expansive soils.

7.3 Minimum Resistivity, pH, and Water-Soluble Sulfate

7.3.1 Potential of Hydrogen (pH) and resistivity testing as well as chloride content testing were performed on representative samples of soil to generally evaluate the corrosion potential to surface utilities. The tests were performed in accordance with California Test Method Nos. 643 and 422 and indicate that the soils are considered “severely corrosive” with respect to corrosion of buried ferrous metals on site. The results are presented in Appendix B (Figure B5) and should be considered for design of underground structures. Due to the corrosive potential of the soils, it is suggested that ABS pipes be considered in lieu of cast-iron for subdrains and retaining wall drains beneath the structure.

7.3.2 Laboratory tests were performed on representative samples of the site materials to measure the percentage of water-soluble sulfate content. Results from the laboratory water-soluble sulfate tests are presented in Appendix B (Figure B5) and indicate that the on-site materials possess “negligible” sulfate exposure to concrete structures as defined by 2013 CBC Section 1904 and ACI 318-11 Sections 4.2 and 4.3.

7.3.3 Geocon West, Inc. does not practice in the field of corrosion engineering and mitigation. If corrosion sensitive improvements are planned, it is recommended that a corrosion engineer be retained to evaluate corrosion test results and incorporate the necessary precautions to avoid premature corrosion of buried metal pipes and concrete structures in direct contact with the soils.

7.4 Grading

7.4.1 Grading is anticipated to include excavation of site soils for the proposed subterranean structure, foundations, and utility trenches, as well as placement of backfill for walls and trenches.

7.4.2 Earthwork should be observed, and compacted fill tested by representatives of Geocon West, Inc. The existing fill and alluvial soil encountered during exploration is suitable for re-use as engineered fill, provided any encountered oversize material (greater than 6 inches) and any encountered deleterious debris are removed.

7.4.3 A preconstruction conference should be held at the site prior to the beginning of grading operations with the owner, contractor, civil engineer, geotechnical engineer, and building official in attendance. Special soil handling requirements can be discussed at that time.

7.4.4 Grading should commence with the removal of all existing vegetation and existing improvements from the area to be graded. Deleterious debris such as wood and root structures should be exported from the site and should not be mixed with the fill soils. Asphalt and concrete should not be mixed with the fill soils unless approved by the Geotechnical Engineer. All existing underground improvements planned for removal should be completely excavated and the resulting depressions properly backfilled in accordance with the procedures described herein. Once a clean excavation bottom has been established it must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon West, Inc.) and the City of Los Angeles Inspector.

7.4.5 The foundation system for the proposed structure may derive support in the competent undisturbed alluvial soils found at and below a depth of 21 feet. Foundations should be deepened as necessary to extend into satisfactory soils and must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon West, Inc.).

7.4.6 The concrete slab-on-grade and ramp for the subterranean portion of the proposed structure may bear directly on newly placed engineered fill or the undisturbed alluvial soils found at the excavation bottom. Any disturbed soils should be properly compacted for slab and ramp support, as necessary.

7.4.7 The City of Los Angeles Department of Building and Safety requires a minimum compactive effort of 95 percent of the laboratory maximum dry density in accordance with ASTM D 1557 (latest edition) where the soils to be utilized in the fill have less than 15 percent finer than 0.005 millimeters. Soils with more than 15 percent finer than 0.005 millimeters may be compacted to 90 percent of the laboratory maximum dry density in accordance with ASTM D 1557 (latest edition). All fill and backfill soils should be placed in horizontal loose layers approximately 6 to 8 inches thick, moisture conditioned to optimum moisture content, and properly compacted to the required degree of compaction in accordance with ASTM D 1557 (latest edition).

7.4.8 Although not anticipated for this project, all imported fill shall be observed, tested, and approved by Geocon West, Inc. prior to bringing soil to the site. Rocks larger than 6 inches in diameter shall not be used in the fill. If necessary, import soils used as structural fill should have an expansion index less than 20 and corrosivity properties that are equally or less detrimental to that of the existing onsite soils (see Figure B5).

7.4.9 Foundations for small outlying structures, such as block walls up to 6 feet high, planter walls or trash enclosures, which will not be tied to the proposed building, may be supported on conventional foundations deriving support on a minimum of 12 inches of newly placed engineered fill which extends laterally at least 12 inches beyond the foundation area. Where excavation and proper compaction cannot be performed or is undesirable, foundations may derive support directly in the undisturbed alluvial soils. Due to the depth to alluvial soils, special foundation recommendations may be required and can be provided under separate cover, if needed. If the soils exposed in the excavation bottom are soft or loose, compaction of the soils will be required prior to placing steel or concrete. Compaction of the foundation excavation bottom is typically accomplished with a compaction wheel or mechanical whacker and must be observed and approved by a Geocon representative.

7.4.10 Utility trenches should be properly backfilled in accordance with the requirements of the Green Book (latest edition). The pipe should be bedded with clean sands (Sand Equivalent greater than 30) to a depth of at least 1 foot over the pipe, and the bedding material must be inspected and approved in writing by the Geotechnical Engineer (a representative of Geocon). The use of gravel is not acceptable unless used in conjunction with filter fabric to prevent the gravel from having direct contact with soil. The remainder of the trench backfill may be derived from onsite soil or approved import soil, compacted as necessary, until the required compaction is obtained. The use of minimum 2-sack slurry is also acceptable as backfill (see Section 7.5). Prior to placing any bedding materials or pipes, the excavation bottom must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon).

7.4.11 All trench and foundation excavation bottoms must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon), prior to placing bedding materials, fill, steel, gravel, or concrete.

7.5 Controlled Low Strength Material (CLSM)

7.5.1 Controlled Low Strength Material (CLSM) may be utilized in lieu of compacted soil as engineered fill where approved in writing by the Geotechnical Engineer. Where utilized within the City of Los Angeles use of CLSM is subject to the following requirements:

Standard Requirements

1. CLSM shall be ready-mixed by a City of Los Angeles approved batch plant;
2. CLSM shall not be placed on uncertified fill, on incompetent natural soil, nor below water;
3. CLSM shall not be placed on a sloping surface with a gradient steeper than 5:1 (horizontal to vertical);
4. Placement of the CLSM shall be under the continuous inspection of a concrete deputy inspector;
5. The excavation bottom shall be accepted by the soil engineer and the City Inspector prior to placing CLSM.

Requirements for CLSM that will be used for support of footings

1. The cement content of the CLSM shall not be less than 188 pounds per cubic yard (min. 2 sacks);
2. The excavation bottom must be level, cleaned of loose soils and approved in writing by Geocon prior to placement of the CLSM;
3. The ultimate compressive strength of the CLSM shall be no less than 100 pounds per square inch (psi) when tested on the 28th-day per ASTM D4832 (latest edition), Standard Test Method for Preparation and Testing of Controlled Low Strength Material Test Cylinders. Compression testing will be performed in accordance with ASTM C39 and City of Los Angeles requirements;
4. Samples of the CLSM will be collected during placement, a minimum of one test (two cylinders) for each 50 cubic yards or fraction thereof;
5. Overexcavation for CLSM placement shall extend laterally beyond the footprint of any proposed footings as required for placement of compacted fill, unless justified otherwise by the soil engineer that footings will have adequate vertical and horizontal bearing capacity.

7.6 Foundation Design

7.6.1 Once the subterranean design and foundation loading configuration for the proposed structure proceeds to a more finalized plan, the recommendations within this report should be reviewed and revised, if necessary.

7.6.2 The proposed structure may be supported on a conventional foundation system deriving support in the competent alluvial soils found at or below a depth of 21 feet below the existing ground surface. Foundations should be deepened as necessary to penetrate through any unsuitable soils and derive support in competent alluvial soils. Any soils unintentionally disturbed should be removed from the foundation excavation.

7.6.3 Continuous footings may be designed for an allowable bearing capacity of 3,500 pounds per square foot (psf), and should be a minimum of 12 inches in width, 18 inches in depth below the lowest adjacent grade, and 18 inches into the recommended bearing materials.

7.6.4 Isolated spread foundations may be designed for an allowable bearing capacity of 4,000 psf, and should be a minimum of 24 inches in width, 18 inches in depth below the lowest adjacent grade, and 18 inches into the recommended bearing materials.

7.6.5 The allowable soil bearing pressure above may be increased by 100 psf and 300 psf for each additional foot of foundation width and depth, respectively, up to maximum allowable bearing value of 6,500 psf.

7.6.6 The allowable bearing pressures may be increased by one-third for transient loads due to wind or seismic forces.

7.6.7 Continuous footings should be reinforced with a minimum of four No. 4 steel reinforcing bars, two placed near the top of the footing and two near the bottom. The reinforcement for foundations should be designed by the project structural engineer.

7.6.8 If depth increases are utilized for the exterior wall footings, this office should be provided a copy of the final construction plans so that the excavation recommendations presented herein could be properly reviewed and revised if necessary.

7.6.9 The above foundation dimensions and minimum reinforcement recommendations are based on soil conditions and building code requirements only, and are not intended to be used in lieu of those required for structural purposes.

7.6.10 No special subgrade presaturation is required prior to placement of concrete. However, the foundation subgrade should be sprinkled as necessary to maintain a moist condition at the time of concrete placement.

7.6.11 Foundation excavations should be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon West, Inc.), to verify that the excavations and exposed soil conditions are consistent with those anticipated. If unanticipated soil conditions are encountered, foundation modifications may be required.

7.6.12 This office should be provided a copy of the final construction plans so that the foundation recommendations presented herein could be properly reviewed and revised if necessary.

7.7 Foundation Settlement

7.7.1 The maximum expected static settlement for a structure supported on a conventional foundation system deriving support in the recommended bearing materials and designed with a maximum bearing pressure of 6,500 psf is estimated to be less than 1 inch and occur below the heaviest loaded structural element. Settlement of the foundation system is expected to occur on initial application of loading. Differential settlement is not expected to exceed ½ inch over a distance of 20 feet.

7.7.2 Once the design and foundation loading configurations for the proposed structures proceeds to a more finalized plan, the estimated settlements presented in this report should be reviewed and revised, if necessary. If the final foundation loading configurations are greater than the assumed loading conditions, the potential for settlement should be reevaluated by this office.

7.8 Miscellaneous Foundations

7.8.1 Foundations for small outlying structures, such as block walls up to 6 feet in height, planter walls or trash enclosures which will not be tied to the proposed structure may be supported on conventional foundations bearing on a minimum of 12 inches of newly placed engineered fill which extends laterally at least 12 inches beyond the foundation area. Where excavation and compaction cannot be performed or is undesirable, such as adjacent to property lines, foundations may derive support in the undisturbed alluvial soils. Due to the depth to alluvial soils, special foundation recommendations may be required and can be provided under separate cover, if needed.

7.8.2 If the soils exposed in the excavation bottom are soft, compaction of the soft soils will be required prior to placing steel or concrete. Compaction of the foundation excavation bottom is typically accomplished with a compaction wheel or mechanical whacker and must be observed and approved by a Geocon representative. Miscellaneous foundations may be designed for a bearing value of 1,500 psf, and should be a minimum of 12 inches in width, 24 inches in depth below the lowest adjacent grade and 12 inches into the recommended bearing material. The allowable bearing pressure may be increased by up to one-third for transient loads due to wind or seismic forces.

7.8.3 Foundation excavations should be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon West, Inc.), prior to the placement of reinforcing steel and concrete to verify that the excavations and exposed soil conditions are consistent with those anticipated.

7.9 Lateral Design

7.9.1 Resistance to lateral loading may be provided by friction acting at the base of foundations, slabs and by passive earth pressure. An allowable coefficient of friction of 0.35 may be used with the dead load forces in the competent alluvial soils or in properly compacted engineered fill near the ground surface, and 0.25 may be used in the competent alluvial soils found at the subterranean level.

7.9.2 Passive earth pressure for the sides of foundations and slabs poured against properly compacted engineered fill or competent alluvial soils near the ground surface may be computed as an equivalent fluid having a density of 270 pcf with a maximum earth pressure of 2,700 psf; and at the subterranean level may be taken as 180 pcf with a maximum earth pressure of 1,800 psf When combining passive and friction for lateral resistance, the passive component should be reduced by one-third.

7.10 Concrete Slabs-on-Grade

7.10.1 Unless specifically evaluated and designed by a qualified structural engineer, the slab-on-grade in the parking garage subject to vehicle loading should be a minimum of 5 inches of concrete reinforced with No. 3 steel reinforcing bars placed 18 inches on center in both horizontal directions and positioned vertically near the slab midpoint. The concrete slab-on-grade and ramp may bear directly on the undisturbed alluvium at the excavation bottom and/or newly placed engineered fill. Any disturbed soils should be removed and or properly compacted for slab support. The upper 12 inches of subgrade exposed along the ramp should be properly compacted to at least 95 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition) for ramp support.

7.10.2 Slabs-on-grade at the ground surface that may receive moisture-sensitive floor coverings or may be used to store moisture-sensitive materials should be underlain by a vapor retarder placed directly beneath the slab. The vapor retarder and acceptable permeance should be specified by the project architect or developer based on the type of floor covering that will be installed. The vapor retarder design should be consistent with the guidelines presented in Section 9.3 of the American Concrete Institute's (ACI) *Guide for Concrete Slabs that Receive Moisture-Sensitive Flooring Materials* (ACI 302.2R-06) and should be installed in general conformance with ASTM E 1643 (latest edition) and the manufacturer's recommendations. A minimum thickness of 15 mils extruded polyolefin plastic is recommended; vapor retarders which contain recycled content or woven materials are not recommended. The vapor retarder should have a permeance of less than 0.01 perms demonstrated by testing before and after mandatory conditioning. The vapor retarder should be installed in direct contact with the concrete slab with proper perimeter seal. If the Los Angeles Green Building Code requirements apply to this project, the vapor retarder should be underlain by 4 inches of clean aggregate. It is important that the vapor retarder be puncture resistant since it will be in direct contact with angular gravel. As an alternative to the clean aggregate suggested in the Los Angeles Green Building Code, it is our opinion that the concrete slab-on-grade may be underlain by a vapor retarder over 4 inches of clean sand (sand equivalent greater than 30), since the sand will serve a capillary break and will minimize the potential for punctures and damage to the vapor barrier.

7.10.3 Due to the nature of the proposed design and intent for a subterranean level, waterproofing of subterranean walls and slabs is suggested. Particular care should be taken in the design and installation of waterproofing to avoid moisture problems, or actual water seepage into the structure through any normal shrinkage cracks which may develop in the concrete walls, floor slab, foundations and/or construction joints. The design and inspection of the waterproofing is not the responsibility of the geotechnical engineer. A waterproofing consultant should be retained in order to recommend a product or method, which would provide protection to subterranean walls, floor slabs and foundations.

7.10.4 For seismic design purposes, a coefficient of friction of 0.35 may be utilized between concrete slabs and subgrade soils near the ground surface without a moisture barrier, 0.25 may be utilized between concrete slabs and subgrade soils at the subterranean level without a moisture barrier, and 0.15 for slabs underlain by a moisture barrier.

7.10.5 Exterior slabs, not subject to traffic loads, should be at least 4 inches thick and reinforced with No. 3 steel reinforcing bars placed 18 inches on center in both horizontal directions, positioned near the slab midpoint. Prior to construction of slabs, the upper 12 inches of subgrade should be moistened to near optimum moisture content and properly compacted to at least 95 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition). Crack control joints should be spaced at intervals not greater than 10 feet and should be constructed using saw-cuts or other methods as soon as practical following concrete placement. Crack control joints should extend a minimum depth of one-fourth the slab thickness. The project structural engineer should design construction joints as necessary.

7.10.6 The recommendations of this report are intended to reduce the potential for cracking of slabs due to settlement. However, even with the incorporation of the recommendations presented herein, foundations, stucco walls, and slabs-on-grade may exhibit some cracking due to minor soil movement and/or concrete shrinkage. The occurrence of concrete shrinkage cracks is independent of the supporting soil characteristics. Their occurrence may be reduced and/or controlled by limiting the slump of the concrete, proper concrete placement and curing, and by the placement of crack control joints at periodic intervals, in particular, where re-entrant slab corners occur.

7.11 Preliminary Pavement Recommendations

7.11.1 Where new paving is to be placed, it is recommended that all existing fill and soft or unsuitable alluvial materials be excavated and properly recompacted for paving support. The client should be aware that excavation and compaction of all existing artificial fill and soft alluvium in the area of new paving is not required; however, paving constructed over existing unsuitable material may experience increased settlement and/or cracking, and may therefore have a shorter design life and increased maintenance costs. As a minimum, the upper twelve inches of paving subgrade should be scarified, moisture conditioned to optimum moisture content, and properly compacted to at least 95 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition).

7.11.2 The following pavement sections are based on an assumed R-Value of 35. Once site grading activities are complete an R-Value should be obtained by laboratory testing to confirm the properties of the soils serving as paving subgrade, prior to placing pavement.

7.11.3 The Traffic Indices listed below are estimates. Geocon does not practice in the field of traffic engineering. The actual Traffic Index for each area should be determined by the project civil engineer. If pavement sections for Traffic Indices other than those listed below are required, Geocon should be contacted to provide additional recommendations. Pavement thicknesses were determined following procedures outlined in the *California Highway Design Manual* (Caltrans). It is anticipated that the majority of traffic will consist of automobile and large truck traffic.

PRELIMINARY PAVEMENT DESIGN SECTIONS

Location	Estimated Traffic Index (TI)	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)
Automobile Parking And Driveways	4.0	3.0	4.0
Trash Truck & Fire Lanes	7.0	3.5	9.0

7.11.4 Asphalt concrete should conform to Section 203-6 of the "*Standard Specifications for Public Works Construction"* (Green Book). Class 2 aggregate base materials should conform to Section 26-1.02A of the "*Standard Specifications of the State of California, Department of Transportation"* (Caltrans). The use of Crushed Miscellaneous Base in lieu of Class 2 aggregate base is acceptable. Crushed Miscellaneous Base should conform to Section 200-2.4 of the "*Standard Specifications for Public Works Construction"* (Green Book).

7.11.5 Unless specifically designed and evaluated by the project structural engineer, where exterior concrete paving will be utilized for support of vehicles, it is recommended that the concrete be a minimum of 6 inches of concrete reinforced with No. 3 steel reinforcing bars placed 18 inches on center in both horizontal directions. Concrete paving supporting vehicular traffic should be underlain by a minimum of 4 inches of aggregate base and a properly compacted subgrade. The subgrade and base material should be compacted to 95 percent relative compactions determined by ASTM Test Method D 1557 (latest edition).

7.11.6 The performance of pavements is highly dependent upon providing positive surface drainage away from the edge of pavements. Ponding of water on or adjacent to the pavement will likely result in saturation of the subgrade materials and subsequent cracking, subsidence and pavement distress. If planters are planned adjacent to paving, it is recommended that the perimeter curb be extended at least 12 inches below the bottom of the aggregate base to minimize the introduction of water beneath the paving.

7.12 Retaining Walls Design

7.12.1 The recommendations presented below are generally applicable to the design of rigid concrete or masonry retaining walls having a maximum height of 23 feet. In the event that walls significantly higher than 23 feet are planned, Geocon should be contacted for additional recommendations.

7.12.2 Retaining wall foundations may be designed in accordance with the recommendations provided in the *Foundation Design* sections of this report (see Section 7.6).

7.12.3 Retaining walls with a level backfill surface that are not restrained at the top should be designed utilizing a triangular distribution of pressure (active pressure) of 45 pcf.

7.12.4 Restrained walls are those that are not allowed to rotate more than 0.001H (where H equals the height of the retaining portion of the wall in feet) at the top of the wall. Where walls are restrained from movement at the top, walls may be designed utilizing a triangular distribution of pressure (at-rest pressure) of 66 pcf. Calculation of the recommended earth pressures is provided as Figure 6.

7.12.5 The wall pressures provided above assume that the proposed retaining walls will support relatively undisturbed alluvial soils. If sloping techniques are to be utilized for construction of proposed walls, which would result in a wedge of engineered fill behind the retaining walls, revised earth pressures may be required. This should be evaluated once the use of sloping measures is established and once the geotechnical characteristics of the engineered backfill soils can be further evaluated.

7.12.6 The wall pressures provided above assume that the retaining wall will be properly drained preventing the buildup of hydrostatic pressure. If retaining wall drainage is not implemented, the equivalent fluid pressure to be used in design of undrained walls is 90 pcf. The value includes hydrostatic pressures plus buoyant lateral earth pressures.

7.12.7 Additional active pressure should be added for a surcharge condition due to sloping ground, vehicular traffic, or adjacent structures. Recommendations for the incorporation of surcharges are provided in section 7.23 of this report. Once the design becomes more finalized, an addendum letter can be prepared revising recommendations and addressing specific surcharge conditions throughout the project, if necessary.

7.12.8 In addition to the recommended earth pressure, the upper ten feet of the subterranean wall adjacent to the street or driveway areas should be designed to resist a uniform lateral pressure of 100 psf, acting as a result of an assumed 300 psf surcharge behind the wall due to normal street traffic. If the traffic is kept back at least ten feet from the wall, the traffic surcharge may be neglected.

7.12.9 Seismic lateral forces should be incorporated into the design as necessary, and recommendations for seismic lateral forces are presented below.

7.13 Dynamic (Seismic) Lateral Forces

7.13.1 The structural engineer should determine the seismic design category for the project in accordance with Section 1613 of the CBC. If the project possesses a seismic design category of D, E, or F, proposed retaining walls in excess of 6 feet in height should be designed with seismic lateral pressure (Section 1803.5.12 of the 2013 CBC).

7.13.2 A seismic load of 10 pcf should be used for design of walls that support more than 6 feet of backfill in accordance with Section 1803.5.12 of the 2013 CBC. The seismic load is applied as an equivalent fluid pressure along the height of the wall and the calculated loads result in a maximum load exerted at the base of the wall and zero at the top of the wall. This seismic load should be applied in addition to the active earth pressure. The earth pressure is based on half of two-thirds of PGA_M calculated from ASCE 7-10 Section 11.8.3.

7.14 Retaining Wall Drainage

7.14.1 Retaining walls not designed for hydrostatic pressure should be provided with a drainage system. At the base of the drain system, a subdrain covered with a minimum of 12 inches of gravel should be installed, and a compacted fill blanket or other seal placed at the surface (see Figure 7). The clean bottom and subdrain pipe, behind a retaining wall, should be observed by the Geotechnical Engineer (a representative of Geocon), prior to placement of gravel or compacting backfill.

7.14.2 As an alternative, a plastic drainage composite such as Miradrain or equivalent may be installed in continuous, 4-foot wide columns along the entire back face of the wall, at 8 feet on center. The top of these drainage composite columns should terminate approximately 18 inches below the ground surface, where either hardscape or a minimum of 18 inches of relatively cohesive material should be placed as a cap (see Figure 8). These vertical columns of drainage material would then be connected at the bottom of the wall to a collection panel or a 1-cubic-foot rock pocket drained by a 4-inch subdrain pipe.

7.14.3 Subdrainage pipes at the base of the retaining wall drainage system should outlet to an acceptable location via controlled drainage structures. Drainage should not be allowed to flow uncontrolled over descending slopes.

7.14.4 Moisture affecting below grade walls is one of the most common post-construction complaints. Poorly applied or omitted waterproofing can lead to efflorescence or standing water. Particular care should be taken in the design and installation of waterproofing to avoid moisture problems, or actual water seepage into the structure through any normal shrinkage cracks which may develop in the concrete walls, floor slab, foundations and/or construction joints. The design and inspection of the waterproofing is not the responsibility of the geotechnical engineer. A waterproofing consultant should be retained in order to recommend a product or method, which would provide protection to subterranean walls, floor slabs and foundations.

7.15 Elevator Pit Design

7.15.1 The elevator pit slab and retaining wall should be designed by the project structural engineer. Elevator pit walls may be designed in accordance with the recommendations in the *Foundation Design* and *Retaining Wall Design* sections of this report (see Sections 7.6 and 7.12).

7.15.2 Additional active pressure should be added for a surcharge condition due to sloping ground, vehicular traffic or adjacent foundations and should be designed for each condition as the project progresses.

7.15.3 If retaining wall drainage is to be provided, the drainage system should be designed in accordance with the *Retaining Wall Drainage* section of this report (see Section 7.14).

7.15.4 It is suggested that the exterior walls and slab be waterproofed to prevent excessive moisture inside of the elevator pit. Waterproofing design and installation is not the responsibility of the geotechnical engineer.

7.16 Elevator Piston

7.16.1 If a plunger-type elevator piston is installed for this project, a deep drilled excavation will be required. It is important to verify that the drilled excavation is not situated immediately adjacent to a foundation or shoring pile, or the drilled excavation could compromise the existing foundation or pile support, especially if the drilling is performed subsequent to the foundation or pile construction.

7.16.2 Some caving is expected and the contractor should be prepared to use casing and should have it readily available at the commencement of drilling activities. Continuous observation of the drilling and installation of the elevator piston by the Geotechnical Engineer (a representative of Geocon West, Inc.) is required.

7.16.3 The annular space between the piston casing and drilled excavation wall should be filled with a minimum of 1½-sack slurry pumped from the bottom up. As an alternative, pea gravel may be utilized. The use of soil to backfill the annular space is not acceptable.

7.17 Temporary Excavations

7.17.1 Excavations on the order of 25 feet in height may be required during excavation and construction of the proposed subterranean levels and foundation system. The excavations are expected to expose artificial fill and alluvial soils, which are suitable for vertical excavations up to 5 feet in height where loose soils or caving sands are not present, and where not surcharged by adjacent traffic or structures.

7.17.2 Vertical excavations greater than 5 feet or where surcharged by existing structures will require sloping or shoring measures in order to provide a stable excavation. Where sufficient space is available, temporary unsurcharged embankments could be sloped back at a uniform 1:1 slope gradient or flatter up to maximum height of 10 feet. A uniform slope does not have a vertical portion.

7.17.3 Where temporary construction slopes are utilized, the top of the slope should be barricaded to prevent vehicles and storage loads at the top of the slope within a horizontal distance equal to the height of the slope. If the temporary slopes are to be maintained during the rainy season, berms are suggested along the tops of the slopes where necessary to prevent runoff water from entering the excavation and eroding the slope faces. Geocon personnel should inspect the soils exposed in the cut slopes during excavation so that modifications of the slopes can be made if variations in the soil conditions occur. All excavations should be stabilized within 30 days of initial excavation.

7.18 Shoring – Soldier Pile Design and Installation

7.18.1 The following information on the design and installation of shoring is preliminary. Review of the final shoring plans and specifications should be made by this office prior to bidding or negotiating with a shoring contractor.

7.18.2 One method of shoring would consist of steel soldier piles, placed in drilled holes and backfilled with concrete. The steel soldier piles may also be installed utilizing high frequency vibration. Where maximum excavation heights are less than 12 feet the soldier piles are typically designed as cantilevers. Where excavations exceed 12 feet or are surcharged, soldier piles may require lateral bracing utilizing drilled tie-back anchors or raker braces to maintain an economical steel beam size and prevent excessive deflection. The size of the steel beam, the need for lateral bracing, and the acceptable shoring deflection should be determined by the project shoring engineer.

7.18.3 The design embedment of the shoring pile toes must be maintained during excavation activities. The toes of the perimeter shoring piles should be deepened to take into account any required excavations necessary for foundations and/or adjacent drainage systems.

7.18.4 Drilled cast-in-place soldier piles should be placed no closer than 2 diameters on center. The minimum diameter of the piles is 18 inches. Structural concrete should be used for the soldier piles below the excavation; lean-mix concrete may be employed above that level. As an alternative, lean-mix concrete may be used throughout the pile where the reinforcing consists of a wideflange section. The slurry must be of sufficient strength to impart the lateral bearing pressure developed by the wideflange section to the soil. For design purposes, an allowable passive value for the soils below the bottom plane of excavation may be assumed to be 250 psf per foot. Where piles are installed by vibration techniques, the passive pressure may be assumed to mobilize across a width equal to the 2 times the dimension of the beam flange. The allowable passive value may be doubled for isolated piles, spaced a minimum of three times the pile diameter. To develop the full lateral value, provisions should be implemented to assure firm contact between the soldier piles and the undisturbed alluvium.

7.18.5 Groundwater was not encountered during site exploration. However, local seepage may be encountered during excavations for the proposed soldier piles, especially if conducted during the rainy season. If more than 6 inches of water is present in the bottom of the excavation, a tremie is required to place the concrete into the bottom of the hole. A tremie should consist of a rigid, water-tight tube having a diameter of not less than 6 inches with a hopper at the top. The tube should be equipped with a device that will close the discharge end and prevent water from entering the tube while it is being charged with concrete. The tremie should be supported so as to permit free movement of the discharge end over the entire top surface of the work and to permit rapid lowering when necessary to retard or stop the flow of concrete. The discharge end should be closed at the start of the work to prevent water entering the tube and should be entirely sealed at all times, except when the concrete is being placed. The tremie tube should be kept full of concrete. The flow should be continuous until the work is completed and the resulting concrete seal should be monolithic and homogeneous. The tip of the tremie tube should always be kept about 5 feet below the surface of the concrete and definite steps and safeguards should be taken to insure that the tip of the tremie tube is never raised above the surface of the concrete.

7.18.6 A special concrete mix should be used for concrete to be placed below water. The design should provide for concrete with an unconfined compressive strength psi of 1,000 pounds per square inch (psi) over the initial job specification. An admixture that reduces the problem of segregation of paste/aggregates and dilution of paste should be included. The slump should be commensurate to any research report for the admixture, provided that it should also be the minimum for a reasonable consistency for placing when water is present.

7.18.7 Some caving is anticipated to occur especially if granular soils are encountered and the contractor should have casing available prior to commencement of pile excavation. When casing is used, extreme care should be employed so that the pile is not pulled apart as the casing is withdrawn. At no time should the distance between the surface of the concrete and the bottom of the casing be less than 5 feet. Continuous observation of the drilling and pouring of the piles by the Geotechnical Engineer (a representative of Geocon West, Inc.), is required.

7.18.8 If a vibratory method of solider pile installation is utilized, predrilling may be performed prior to installation of the steel beams. If predrilling is performed, it is recommended that the bore diameter be at least 2 inches smaller than the largest dimension of the pile to prevent excessive loss in the frictional component of the pile capacity. Predrilling should not be conducted below the proposed excavation bottom.

7.18.9 If a vibratory method is utilized, the owner should be aware of the potential risks associated with vibratory efforts, which typically involve inducing settlement within the vicinity of the pile which could result in a potential for damage to existing improvements in the area.

7.18.10 The level of vibration that results from the installation of the piles should not exceed a threshold where occupants of nearby structures are disturbed, despite higher vibration tolerances that a building may endure without deformation or damage. The main parameter used for vibration assessment is peak particle velocity in units of inch per second (in/sec). The acceptable range of peak particle velocity should be evaluated based on the age and condition of adjacent structures, as well as the tolerance of human response to vibration.

7.18.11 Based on Table 19 of the *Transportation and Construction Induced Vibration Guidance Manual* (Caltrans 2013), a continuous source of vibrations (ex. vibratory pile driving) which generates a maximum peak particle velocity of 0.5 in/sec is considered tolerable for modern industrial/commercial buildings and new residential structures. The Client should be aware that a lower value may be necessary if older or fragile structures are in the immediate vicinity of the site.

7.18.12 Vibrations should be monitored and record with seismographs during pile installation to detect the magnitude of vibration and oscillation experienced by adjacent structures. If the vibrations exceed the acceptable range during installation, the shoring contractor should modify the installation procedure to reduce the values to within the acceptable range. Vibration monitoring is not the responsibility of the Geotechnical Engineer.

7.18.13 Geocon does not practice in the field of vibration monitoring. If construction techniques will be implemented, it is recommended that qualified consultant be retained to provide site specific recommendations for vibration thresholds and monitoring.

7.18.14 The frictional resistance between the soldier piles and retained soil may be used to resist the vertical component of the load. The coefficient of friction may be taken as 0.3 based on uniform contact between the steel beam and lean-mix concrete and the alluvium found below the excavation bottom. The portion of soldier piles below the plane of excavation may also be employed to resist the downward loads. The downward capacity may be determined using a frictional resistance of 550 psf.

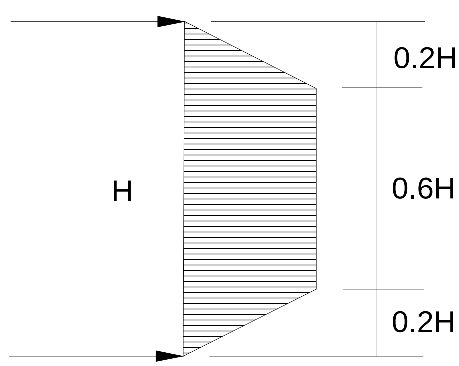
7.18.15 Due to the nature of the site soils, it is expected that continuous lagging between soldier piles will be required. However, it is recommended that the exposed soils be observed by the Geotechnical Engineer (a representative of Geocon West, Inc.), to verify the presence of any cohesive soils and the areas where lagging may be omitted.

7.18.16 The time between lagging excavation and lagging placement should be as short as possible. Soldier piles should be designed for the full-anticipated pressures. Due to arching in the soils, the pressure on the lagging will be less. It is recommended that the lagging be designed for the full design pressure but be limited to a maximum of 400 psf.

7.18.17 For the design of shoring, it is recommended that an equivalent fluid pressure based on the following table, be utilized for design. A diagram depicting the trapezoidal pressure distribution of lateral earth pressure is provided below the table. Calculation of the recommended shoring pressures is provided as Figure 9.

HEIGHT OF SHORING (FEET)	**EQUIVALENT FLUID PRESSURE (Pounds Per Cubic Foot) (ACTIVE PRESSURE)**	**EQUIVALENT FLUID PRESSURE (Pounds Per Square Foot per Foot) Trapezoidal –Active (Where H is the height of the shoring in feet)**
Up to 25	37	23H

Trapezoidal Distribution of Pressure



7.18.18 It is very important to note that active pressures can only be achieved when movement in the soil (earth wall) occurs. If movement in the soil is not acceptable, such as adjacent to an existing structure, or the pile is restrained from movement by bracing or a tie back anchor, an at-rest pressure of 58 pcf should be considered for design purposes.

7.18.19 Where a combination of sloped embankment and shoring is utilized, the pressure will be greater and must be determined for each combination. Additional active pressure should be added for a surcharge condition due to slopes, vehicular traffic or adjacent structures and should be designed for each condition. The surcharge pressure should be evaluated in accordance with the recommendations in Section 7.23 of this report.

7.18.20 In addition to the recommended earth pressure, the upper ten feet of the shoring adjacent to the street or driveway areas should be designed to resist a uniform lateral pressure of 100 psf, acting as a result of an assumed 300 psf surcharge behind the shoring due to normal street traffic. If the traffic is kept back at least 10 feet from the shoring, the traffic surcharge may be neglected.

7.18.21 It is difficult to accurately predict the amount of deflection of a shored embankment. It should be realized that some deflection will occur. It is recommended that the deflection be minimized to prevent damage to existing structures and adjacent improvements. Where public right-of-ways are present or adjacent offsite structures do not surcharge the shoring excavation, the shoring deflection should be limited to less than 1 inch at the top of the shored embankment. Where offsite structures are within the shoring surcharge area it is recommended that the beam deflection be limited to less than ½ inch at the elevation of the adjacent offsite foundation, and no deflection at all if deflections will damage existing structures. The allowable deflection is dependent on many factors, such as the presence of structures and utilities near the top of the embankment, and will be assessed and designed by the project shoring engineer.

7.18.22 Because of the depth of the excavation, some means of monitoring the performance of the shoring system is suggested. The monitoring should consist of periodic surveying of the lateral and vertical locations of the tops of all soldier piles and the lateral movement along the entire lengths of selected soldier piles.

7.18.23 Due to the depth of the depth of the excavation and proximity to adjacent structures, it is suggested that prior to excavation the existing improvements be inspected to document the present condition. For documentation purposes, photographs should be taken of preconstruction distress conditions and level surveys of adjacent grade and pavement should be considered. During excavation activities, the adjacent structures and pavement should be periodically inspected for signs of distress. In the even that distress or settlement is noted, an investigation should be performed and corrective measures taken sot that continued or worsened distress or settlement is mitigated. Documentation and monitoring of the offsite structures and improvements is not the responsibility of the geotechnical engineer.

7.19 Tie-Back Anchors

7.19.1 Tie-back anchors may be used with the soldier pile wall system to resist lateral loads. Post-grouted friction anchors are recommended. For design purposes, it may be assumed that the active wedge adjacent to the shoring is defined by a plane drawn 35 degrees with the vertical through the bottom plane of the excavation. Friction anchors should extend a minimum of 20 feet beyond the potentially active wedge and to greater lengths if necessary to develop the desired capacities. The locations and depths of all offsite utilities should be thoroughly checked and incorporated into the drilling angle design for the tie-back anchors.

7.19.2 The capacities of the anchors should be determined by testing of the initial anchors as outlined in a following section. Only the frictional resistance developed beyond the active wedge would be effective in resisting lateral loads. Anchors should be placed at least 6 feet on center to be considered isolated. Based on the height of the proposed excavation, it is anticipated that two rows of anchors may be required. For preliminary design purposes, it is estimated that drilled friction anchors constructed without utilizing post-grouting techniques will develop average skin frictions as follows:

- 8 feet below the top of the excavation – 950 pounds per square foot

7.19.3 Depending on the techniques utilized, and the experience of the contractor performing the installation, a maximum allowable friction capacity of 3.0 kips per linear foot for post-grouted anchors (for a minimum 20-foot length beyond the active wedge) may be assumed for design purposes. Only the frictional resistance developed beyond the active wedge should be utilized in resisting lateral loads.

7.20 Anchor Installation

7.20.1 Tied-back anchors are typically installed between 20 and 40 degrees below the horizontal; however, occasionally alternative angles are necessary to avoid existing improvements and utilities. The locations and depths of all offsite utilities should be thoroughly checked prior to design and installation of the tie-back anchors. Caving of the anchor shafts, particularly within sand and gravel deposits or seepage zones, should be anticipated during installation and provisions should be implemented in order to minimize such caving. It is suggested that hollow-stem auger drilling equipment be used to install the anchors. The anchor shafts should be filled with concrete by pumping from the tip out, and the concrete should extend from the tip of the anchor to the active wedge. In order to minimize the chances of caving, it is recommended that the portion of the anchor shaft within the active wedge be backfilled with sand before testing the anchor. This portion of the shaft should be filled tightly and flush with the face of the excavation. The sand backfill should be placed by pumping; the sand may contain a small amount of cement to facilitate pumping.

7.21 Anchor Testing

7.21.1 All of the anchors should be tested to at least 150 percent of design load. The total deflection during this test should not exceed 12 inches. The rate of creep under the 150 percent test load should not exceed 0.1 inch over a 15-minute period in order for the anchor to be approved for the design loading.

7.21.2 At least ten percent of the anchors should be selected for "quick" 200 percent tests and three additional anchors should be selected for 24-hour 200 percent tests. The purpose of the 200 percent tests is to verify the friction value assumed in design. The anchors should be tested to develop twice the assumed friction value. These tests should be performed prior to installation of additional tiebacks. Where satisfactory tests are not achieved on the initial anchors, the anchor diameter and/or length should be increased until satisfactory test results are obtained.

7.21.3 The total deflection during the 24-hour 200 percent test should not exceed 12 inches. During the 24-hour tests, the anchor deflection should not exceed 0.75 inches measured after the 200 percent test load is applied.

7.21.4 For the "quick" 200 percent tests, the 200 percent test load should be maintained for 30 minutes. The total deflection of the anchor during the 200 percent quick tests should not exceed 12 inches; the deflection after the 200 percent load has been applied should not exceed 0.25 inch during the 30-minute period.

7.21.5 After a satisfactory test, each anchor should be locked-off at the design load. This should be verified by rechecking the load in the anchor. The load should be within 10 percent of the design load. The installation and testing of the anchors should be observed by a representative of this firm.

7.22 Internal Bracing

7.22.1 Rakers may be utilized to brace the soldier piles in lieu of tieback anchors. The raker bracing could be supported laterally by temporary concrete footings (deadmen) or by the permanent, interior footings. For design of such temporary footings or deadmen, poured with the bearing surface normal to rakers inclined at 45 degrees, a bearing value of 2,500 psf in competent alluvial soil, provided the shallowest point of the footing is at least one foot below the lowest adjacent grade.

7.22.2 The client should be aware that the utilization of rakers could significantly impact the construction schedule do to their intrusion into the construction site and potential interference with equipment. In addition, the raker footing plan should be checked by the project structural engineer to verify if there are any conflicts with the proposed structural foundations, and resolve any issues prior to commencement of construction activities.

7.23 Surcharge from Adjacent Structures and Improvements

7.23.1 Additional active pressure should be added for a surcharge condition due to sloping ground, vehicular traffic or adjacent structures and should be designed for each condition as the project progresses.

7.23.2 It is recommended that line-load surcharges from adjacent wall footings, use horizontal pressures generated from NAV-FAC DM 7.2. The governing equations are:

$$For \quad x/H \leq 0.4$$

$$\sigma_H(z) = \frac{0.20\left(\frac{z}{H}\right)}{\left[0.16 + \left(\frac{z}{H}\right)^2\right]^2} \frac{Q_L}{H}$$

and

$$For \quad x/H > 0.4$$

$$\sigma_H(x,z) = \frac{1.26\left(\frac{x}{H}\right)^2\left(\frac{z}{H}\right)}{\left[\left(\frac{x}{H}\right)^2 + \left(\frac{z}{H}\right)^2\right]^2} \frac{Q_L}{H}$$

where x is the distance from the face of the excavation to the vertical line-load, H is the distance from the bottom of the footing to the bottom of excavation, z is the depth at which the horizontal pressure is desired, QL is the vertical line-load and σH is the horizontal pressure at depth z.

7.23.3 It is recommended that vertical point-loads, from construction equipment outriggers or adjacent building columns use horizontal pressures generated from NAV-FAC DM 7.2. The governing equations are:

$$For \quad {}^{x}\!/_{H} \leq 0.4$$

$$\sigma(z) = \frac{0.28 \times \left(\frac{z}{H}\right)^2}{\left[0.16 + \left(\frac{z}{H}\right)^2\right]^3} \times \frac{Q_p}{H^2}$$

and

$$For \quad {}^{x}\!/_{H} > 0.4$$

$$\sigma(z) = \frac{1.77 \times \left(\frac{x}{H}\right)^2 \times \left(\frac{z}{H}\right)^2}{\left[\left(\frac{x}{H}\right)^2 + \left(\frac{z}{H}\right)^2\right]^3} \times \frac{Q_p}{H^2}$$

then

$$\sigma_H^{'}(z) = \sigma_H(z)\cos^2(1.1\theta)$$

where x is the distance from the face of the excavation to the vertical point-load, H is distance from the outrigger/bottom of column footing to the bottom of excavation, z is the depth at which the horizontal pressure is desired, Q_p is the vertical point-load, σ is the vertical pressure at depth z, Θ is the angle between a line perpendicular to the bulkhead and a line from the point-load to half the pile spacing at the bulkhead, and σ_H is the horizontal pressure at depth z.

7.24 Stormwater Infiltration

7.24.1 During the site exploration, boring B1 was utilized to perform percolation testing. The boring was advanced to the depth listed in the table below. Slotted casing was placed in the boring, and the annular space between the casing and excavation was filled with filter pack. The boring was then filled with water to pre-saturate the soils. On September 7, 2016, the casing was refilled with water and percolation test readings were performed after repeated flooding of the cased excavation. Based on the test results, the average percolation rate for the earth materials encountered, is provided in the following table. The percolation rate is intended to be used for the design of a drywell system; if a different type of infiltration system is proposed, and adjusted rate may be required. Additional correction factors may be required and should be applied by the engineer in responsible charge of the design of the stormwater infiltration system and based on applicable guidelines

Boring	Infiltration Depth (ft.)	Average Percolation Rate (in / hour)
B1	35-45	54

7.24.2 The results of the percolation testing indicate that the soils at depths in the above table are conductive to infiltration. It is our opinion that the soil zone encountered at the depth and location as listed in the table above are suitable for infiltration of stormwater and will not induce excessive hydro-consolidation, will not create a perched groundwater condition, will not affect soil structure interaction of existing or proposed foundations due to expansive soils, will not saturate soils supported by existing or proposed retaining walls, and will not increase the potential for liquefaction. Resulting settlements are anticipated to be less than ¼ inch, if any.

7.24.3 The infiltration system must be located such that the closest distance between an adjacent foundation is at least 10 feet in all directions from the zone of saturation. The zone of saturation may be assumed to project downward from the discharge of the infiltration facility at a gradient of 1:1. Additional property line or foundation setbacks may be required by the governing jurisdiction and should be incorporated into the stormwater infiltration system design as necessary.

7.24.4 Where the 10-foot horizontal setback cannot be maintained between the infiltration system and an adjacent footing, and the infiltration system penetrates below the foundation influence line, the proposed stormwater infiltration system must be designed to resist the surcharge from the adjacent foundation. The foundation surcharge line may be assumed to project down away from the bottom of the foundation at a 1:1 gradient. The stormwater infiltration system must still be sufficiently deep to maintain the 10-foot vertical offset between the bottom of the footing and the zone of saturation.

7.24.5 Subsequent to the placement of the infiltration system, it is acceptable to backfill the resulting void space between the excavation sidewalls and the infiltration system with minimum two-sack slurry provided the slurry is not placed in the infiltration zone. It is recommended that pea gravel be utilized adjacent to the infiltration zone so communication of water to the soil is not hindered.

7.24.6 Due to the preliminary nature of the project at this time, the type of stormwater infiltration system and location of the stormwater infiltration systems has not yet been determined. The design drawings should be reviewed and approved by the Geotechnical Engineer. The installation of the stormwater infiltration system should be observed and approved by the Geotechnical Engineer (a representative of Geocon).

7.25 Surface Drainage

7.25.1 Proper surface drainage is critical to the future performance of the project. Uncontrolled infiltration of irrigation excess and storm runoff into the soils can adversely affect the performance of the planned improvements. Saturation of a soil can cause it to lose internal shear strength and increase its compressibility, resulting in a change in the original designed engineering properties. Proper drainage should be maintained at all times.

7.25.2 All site drainage should be collected and controlled in non-erosive drainage devices. Drainage should not be allowed to pond anywhere on the site, and especially not against any foundation or retaining wall. The site should be graded and maintained such that surface drainage is directed away from structures in accordance with 2013 CBC 1804.3 or other applicable standards. In addition, drainage should not be allowed to flow uncontrolled over any descending slope. Discharge from downspouts, roof drains and scuppers are not recommended onto unprotected soils within 5 feet of the building perimeter. Planters which are located adjacent to foundations should be sealed to prevent moisture intrusion into the soils providing foundation support. Landscape irrigation is not recommended within 5 feet of the building perimeter footings except when enclosed in protected planters.

7.25.3 Positive site drainage should be provided away from structures, pavement, and the tops of slopes to swales or other controlled drainage structures. The building pad and pavement areas should be fine graded such that water is not allowed to pond.

7.25.4 Landscaping planters immediately adjacent to paved areas are not recommended due to the potential for surface or irrigation water to infiltrate the pavement's subgrade and base course. Either a subdrain, which collects excess irrigation water and transmits it to drainage structures, or an impervious above-grade planter boxes should be used. In addition, where landscaping is planned adjacent to the pavement, it is recommended that consideration be given to providing a cutoff wall along the edge of the pavement that extends at least 12 inches below the base material.

7.26 Plan Review

7.26.1 Grading, foundation, and shoring plans should be reviewed by the Geotechnical Engineer (a representative of Geocon West, Inc.), prior to finalization to verify that the plans have been prepared in substantial conformance with the recommendations of this report and to provide additional analyses or recommendations.

LIMITATIONS AND UNIFORMITY OF CONDITIONS

1. The recommendations of this report pertain only to the site investigated and are based upon the assumption that the soil conditions do not deviate from those disclosed in the investigation. If any variations or undesirable conditions are encountered during construction, or if the proposed construction will differ from that anticipated herein, Geocon West, Inc. should be notified so that supplemental recommendations can be given. The evaluation or identification of the potential presence of hazardous or corrosive materials was not part of the scope of services provided by Geocon West, Inc.

2. This report is issued with the understanding that it is the responsibility of the owner, or of his representative, to ensure that the information and recommendations contained herein are brought to the attention of the architect and engineer for the project and incorporated into the plans, and the necessary steps are taken to see that the contractor and subcontractors carry out such recommendations in the field.

3. The findings of this report are valid as of the date of this report. However, changes in the conditions of a property can occur with the passage of time, whether they are due to natural processes or the works of man on this or adjacent properties. In addition, changes in applicable or appropriate standards may occur, whether they result from legislation or the broadening of knowledge. Accordingly, the findings of this report may be invalidated wholly or partially by changes outside our control. Therefore, this report is subject to review and should not be relied upon after a period of three years.

4. The firm that performed the geotechnical investigation for the project should be retained to provide testing and observation services during construction to provide continuity of geotechnical interpretation and to check that the recommendations presented for geotechnical aspects of site development are incorporated during site grading, construction of improvements, and excavation of foundations. If another geotechnical firm is selected to perform the testing and observation services during construction operations, that firm should prepare a letter indicating their intent to assume the responsibilities of project geotechnical engineer of record. A copy of the letter should be provided to the regulatory agency for their records. In addition, that firm should provide revised recommendations concerning the geotechnical aspects of the proposed development, or a written acknowledgement of their concurrence with the recommendations presented in our report. They should also perform additional analyses deemed necessary to assume the role of Geotechnical Engineer of Record.

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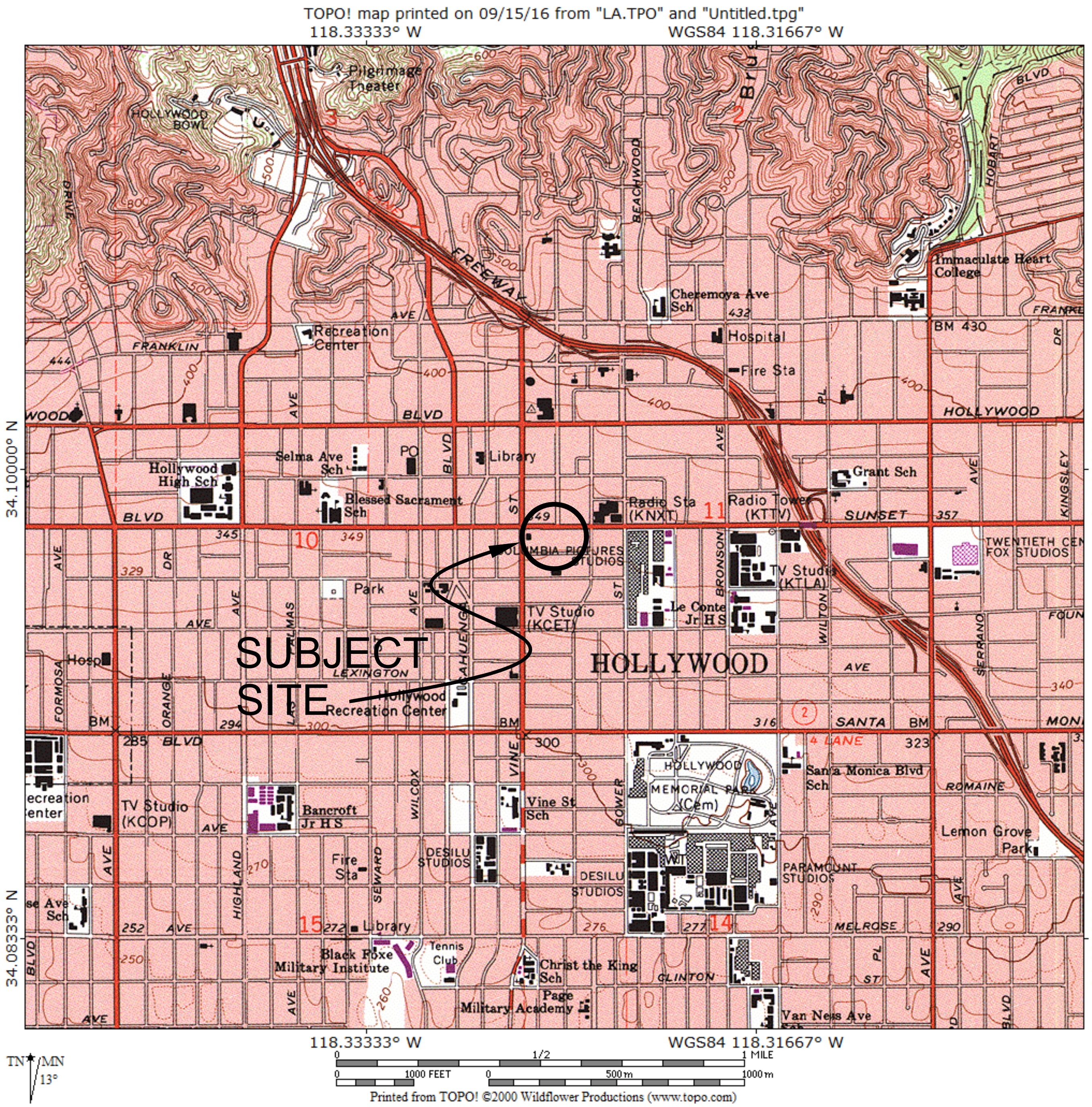

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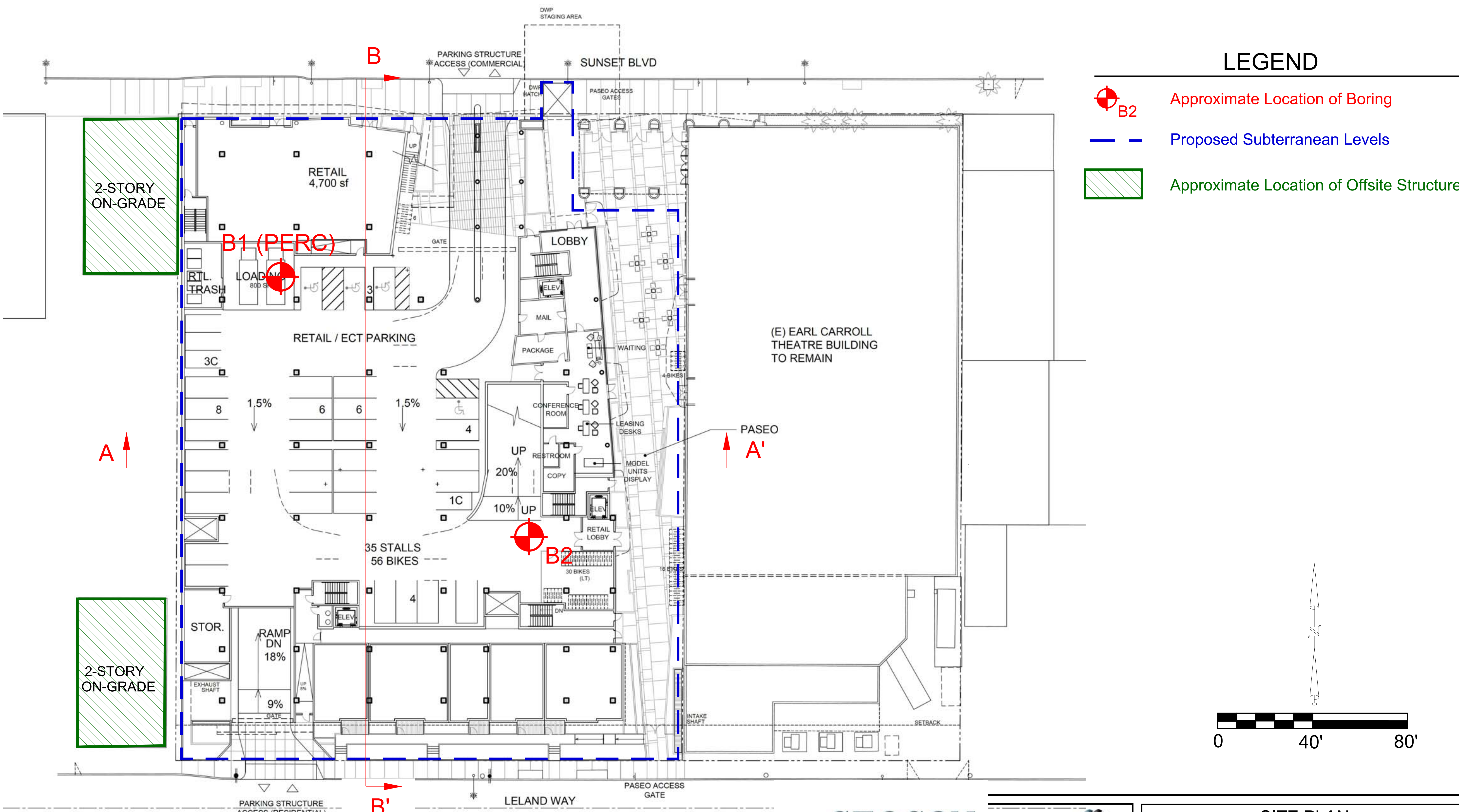
VICINITY MAP

ESSEX PROPERTY TRUST
6250 SUNSET PROJECT
LOS ANGELES, CALIFORNIA

OCT 2016	PROJECT NO. A9202-06-01	FIG. 1



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SITE PLAN

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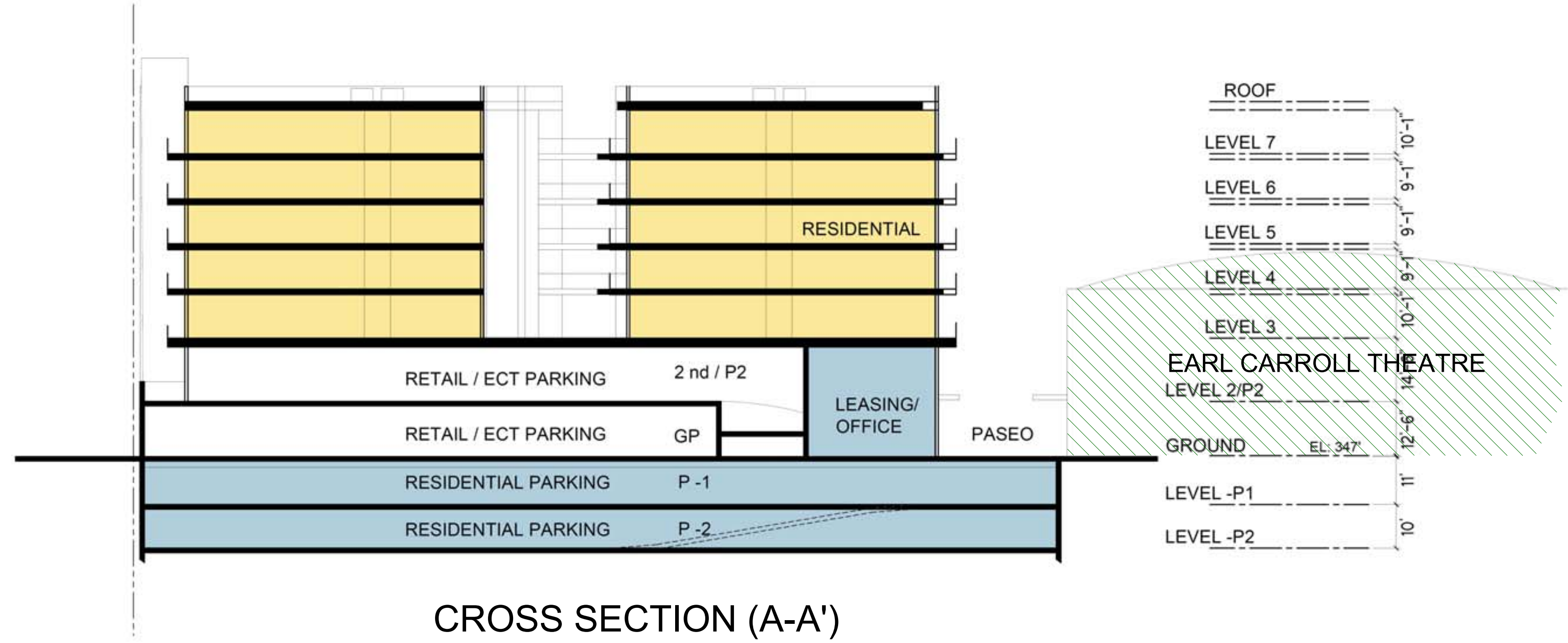
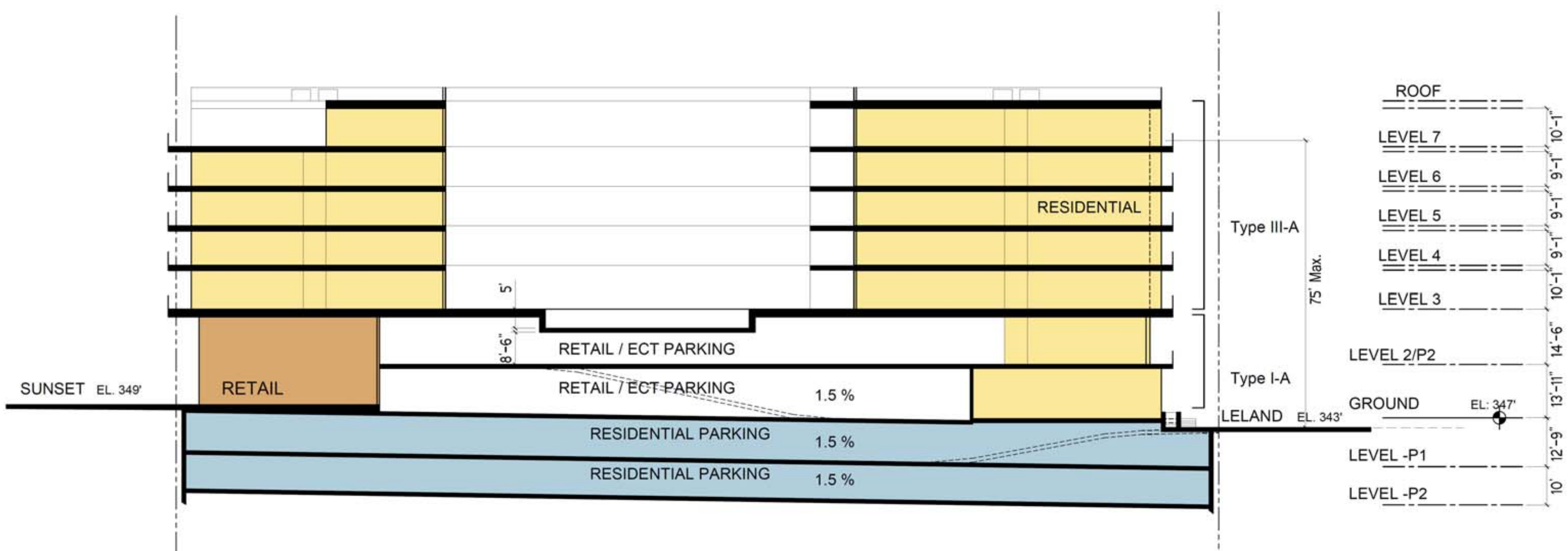
OCT 2016 | PROJECT NO. A9202-06-01 | FIG. 2



CROSS SECTION (A-A')



CROSS SECTION (B-B')

SCALE: 1" = 40' (H&V)

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CROSS-SECTION

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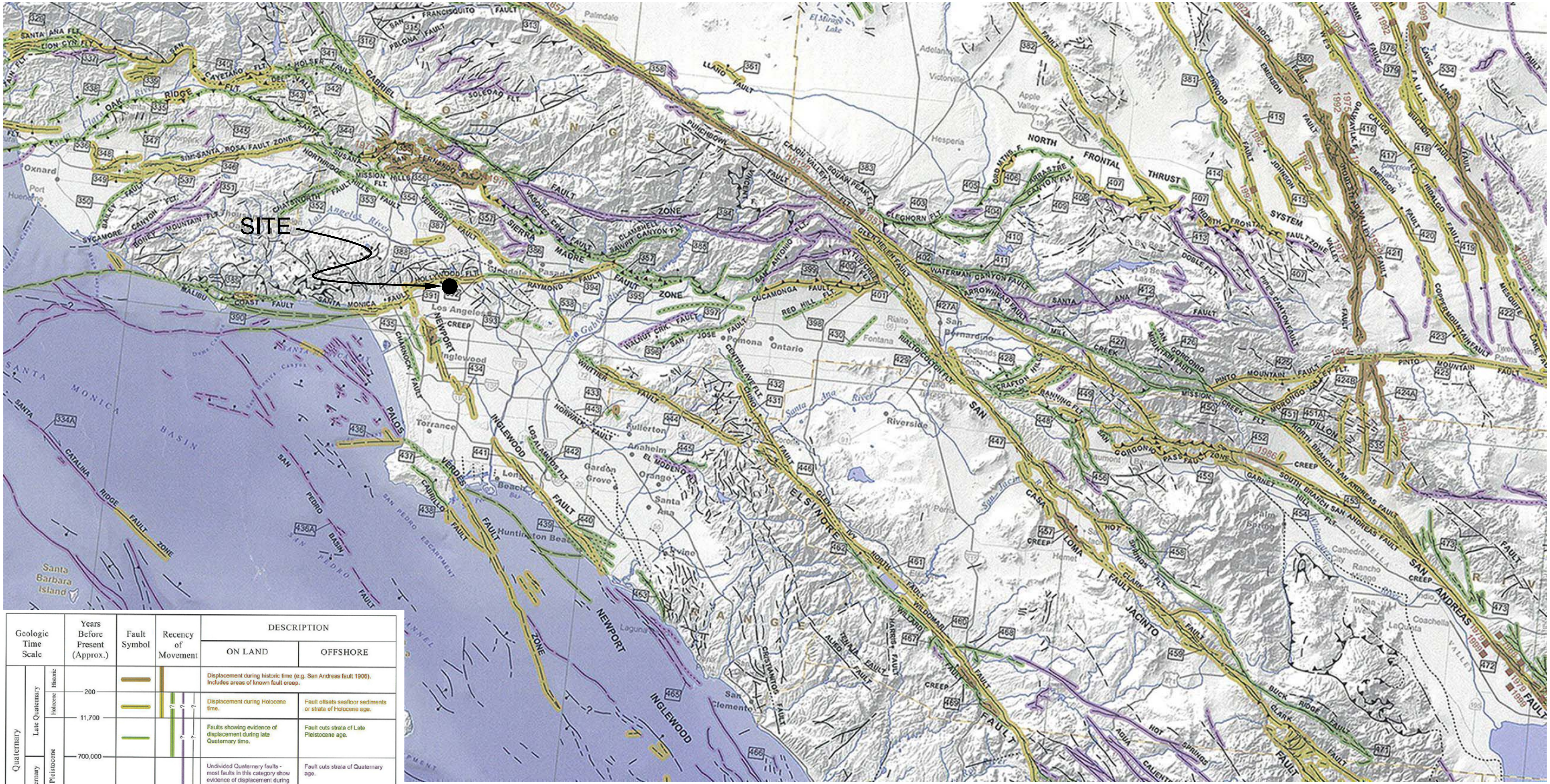
OCT 2016 | PROJECT NO. A9202-06-01 | FIG. 3

Reference: Jennings, C.W. and Bryant, W. A., 2010, Fault Activity Map of California, California Geological Survey Geologic Data Map No. 6.



0 12 24 Miles

Geologic Time Scale			Years Before Present (Approx.)	Fault Symbol	Recency of Movement	DESCRIPTION: ON LAND	DESCRIPTION: OFFSHORE
Quaternary	Late Quaternary	Historic	200			Displacement during historic time (e.g. San Andreas fault 1906). Includes areas of known fault creep.	
		Holocene	11,700		? ? ?	Displacement during Holocene time.	Fault offsets seafloor sediments or strata of Holocene age.
		Pleistocene	700,000		? ?	Faults showing evidence of displacement during late Quaternary time.	Fault cuts strata of Late Pleistocene age.
	Early Quaternary		1,600,000*		?	Undivided Quaternary faults - most faults in this category show evidence of displacement during the last 1,600,000 years; possible exceptions are faults which displace rocks of undifferentiated Plio-Pleistocene age.	Fault cuts strata of Quaternary age.
Pre-Quaternary			4.5 billion (Age of Earth)			Faults without recognized Quaternary displacement or showing evidence of no displacement during Quaternary time. Not necessarily inactive.	Fault cuts strata of Pliocene or older age.

* Quaternary now recognized as extending to 2.6 Ma (Walker and Geissman, 2009). Quaternary faults in this map were established using the previous 1.6 Ma criterion.

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REGIONAL FAULT MAP

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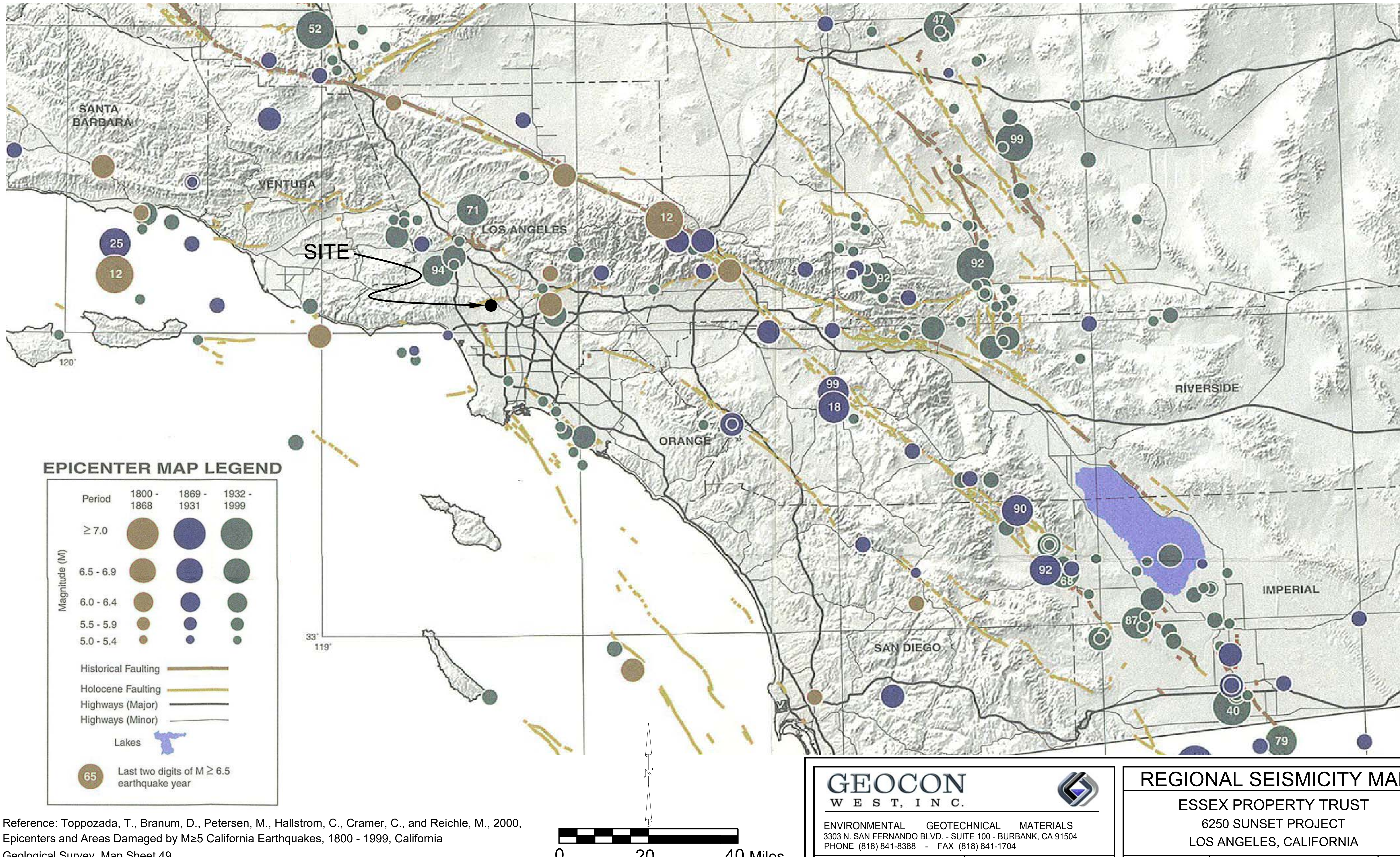
OCT 2016 | PROJECT NO. A9202-06-01 | FIG. 4



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REGIONAL SEISMICITY MAP

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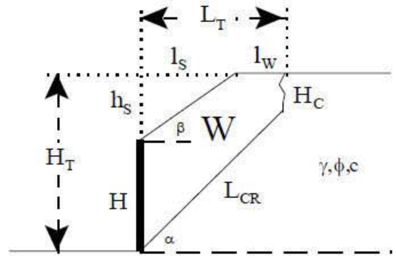
OCT 2016 | PROJECT NO. A9202-06-01 | FIG. 5

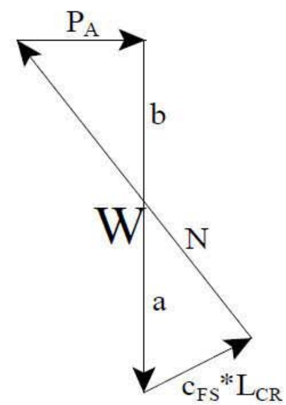
Retaining Wall Design with Transitioned Backfill
(Vector Analysis)

Input:

Retaining Wall Height	(H)	23.00 feet
Slope Angle of Backfill	(β)	0.0 degrees
Height of Slope above Wall	(h_s)	0.0 feet
Horizontal Length of Slope	(l_s)	0.0 feet
Total Height (Wall + Slope)	(H_T)	23.0 feet
Unit Weight of Retained Soils	(γ)	125.0 pcf
Friction Angle of Retained Soils	(φ)	32.0 degrees
Cohesion of Retained Soils	(c)	150.0 psf
Factor of Safety	(FS)	1.50
Factored Parameters:	(ϕ_{FS})	22.6 degrees
	(c_{FS})	100.0 psf



Failure Angle (α) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) $feet^2$	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
45	2.7	261	32621.2	28.7	6947.8	25673.4	10573.6
46	2.7	252	31522.8	28.3	6573.7	24949.2	10788.4
47	2.6	244	30457.2	27.9	6232.8	24224.4	10980.7
48	2.6	235	29422.5	27.5	5921.4	23501.2	11151.3
49	2.5	227	28417.2	27.1	5636.0	22781.2	11300.9
50	2.5	220	27439.7	26.8	5373.9	22065.7	11430.2
51	2.5	212	26488.4	26.4	5132.6	21355.8	11539.5
52	2.4	204	25561.9	26.1	4909.9	20652.0	11629.4
53	2.4	197	24658.8	25.8	4703.9	19955.0	11700.2
54	2.4	190	23777.9	25.5	4512.9	19265.0	11752.2
55	2.4	183	22917.9	25.2	4335.6	18582.3	11785.6
56	2.4	177	22077.5	24.9	4170.5	17907.0	11800.5
57	2.4	170	21255.7	24.6	4016.7	17239.1	11796.9
58	2.4	164	20451.5	24.3	3873.0	16578.5	11774.9
59	2.4	157	19663.8	24.0	3738.5	15925.3	11734.4
60	2.4	151	18891.7	23.8	3612.5	15279.2	11675.2
61	2.5	145	18134.3	23.5	3494.2	14640.1	11597.1
62	2.5	139	17390.7	23.3	3382.8	14007.9	11499.8
63	2.5	133	16660.1	23.0	3277.9	13382.2	11382.9
64	2.5	128	15941.8	22.8	3178.8	12762.9	11245.9
65	2.6	122	15234.9	22.5	3085.0	12149.8	11088.3
66	2.6	116	14538.8	22.3	2996.1	11542.7	10909.4
67	2.7	111	13852.7	22.1	2911.5	10941.2	10708.5
68	2.8	105	13176.1	21.8	2830.9	10345.1	10484.9
69	2.8	100	12508.2	21.6	2753.9	9754.3	10237.5
70	2.9	95	11848.4	21.4	2679.9	9168.5	9965.2



Design Equations (Vector Analysis):

$a = c_{FS}*L_{CR}*\sin(90+\phi_{FS})/\sin(\alpha-\phi_{FS})$

$b = W-a$

$P_A = b*\tan(\alpha-\phi_{FS})$

$EFP = 2*P_A/H^2$

Maximum Active Pressure Resultant

$P_{A,\ max}$ 11800.47 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of wall)

$EFP = 2*P_A/H^2$

	Active	At-Rest
EFP	44.6 pcf	65.9 pcf
Design Wall for an Equivalent Fluid Pressure:	**45 pcf**	**66 pcf**

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RETAINING WALL PRESSURE CALCULATION

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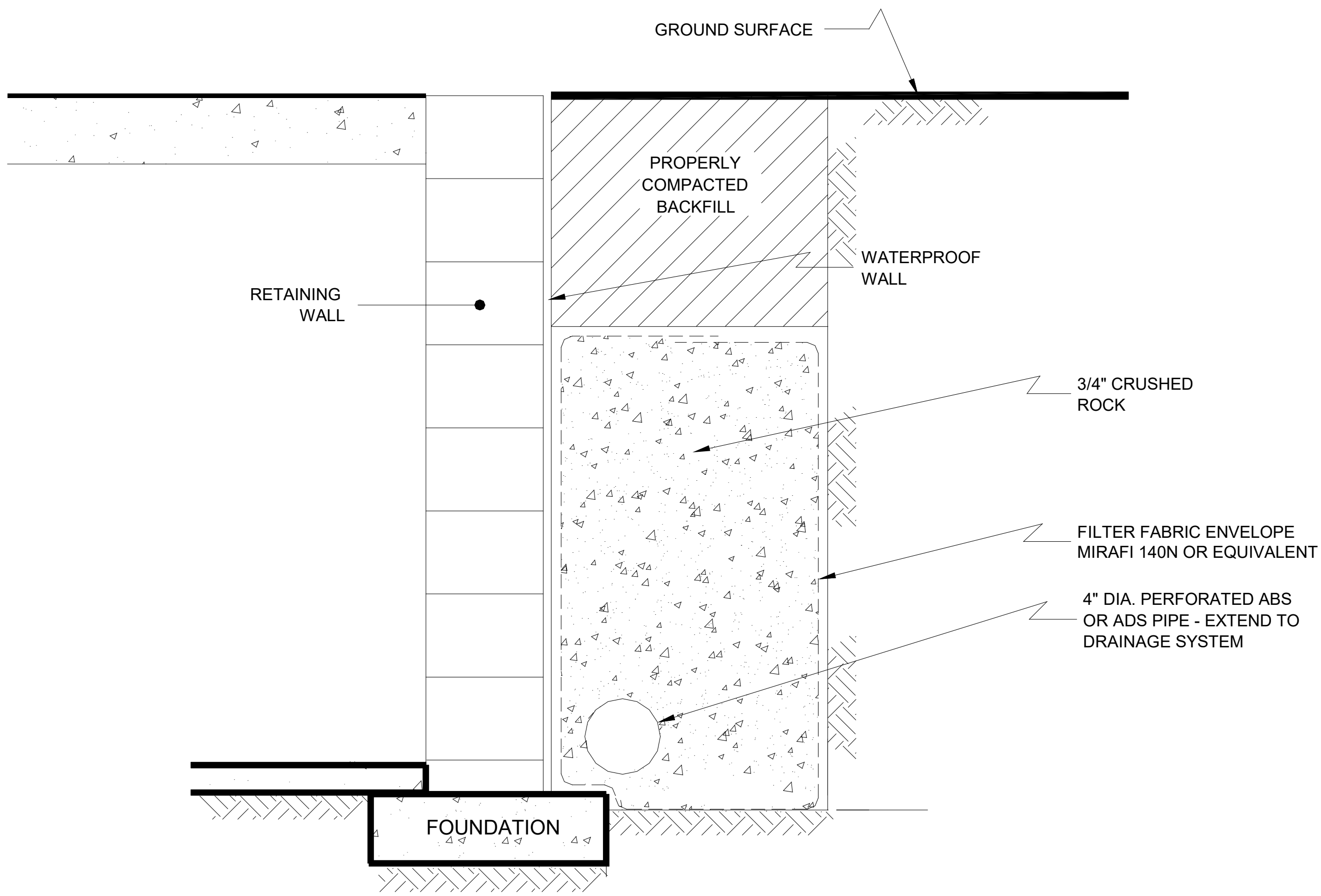
OCT 2016 | PROJECT NO. A9202-06-01 | FIG. 6



NO SCALE

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RETAINING WALL DRAIN DETAIL

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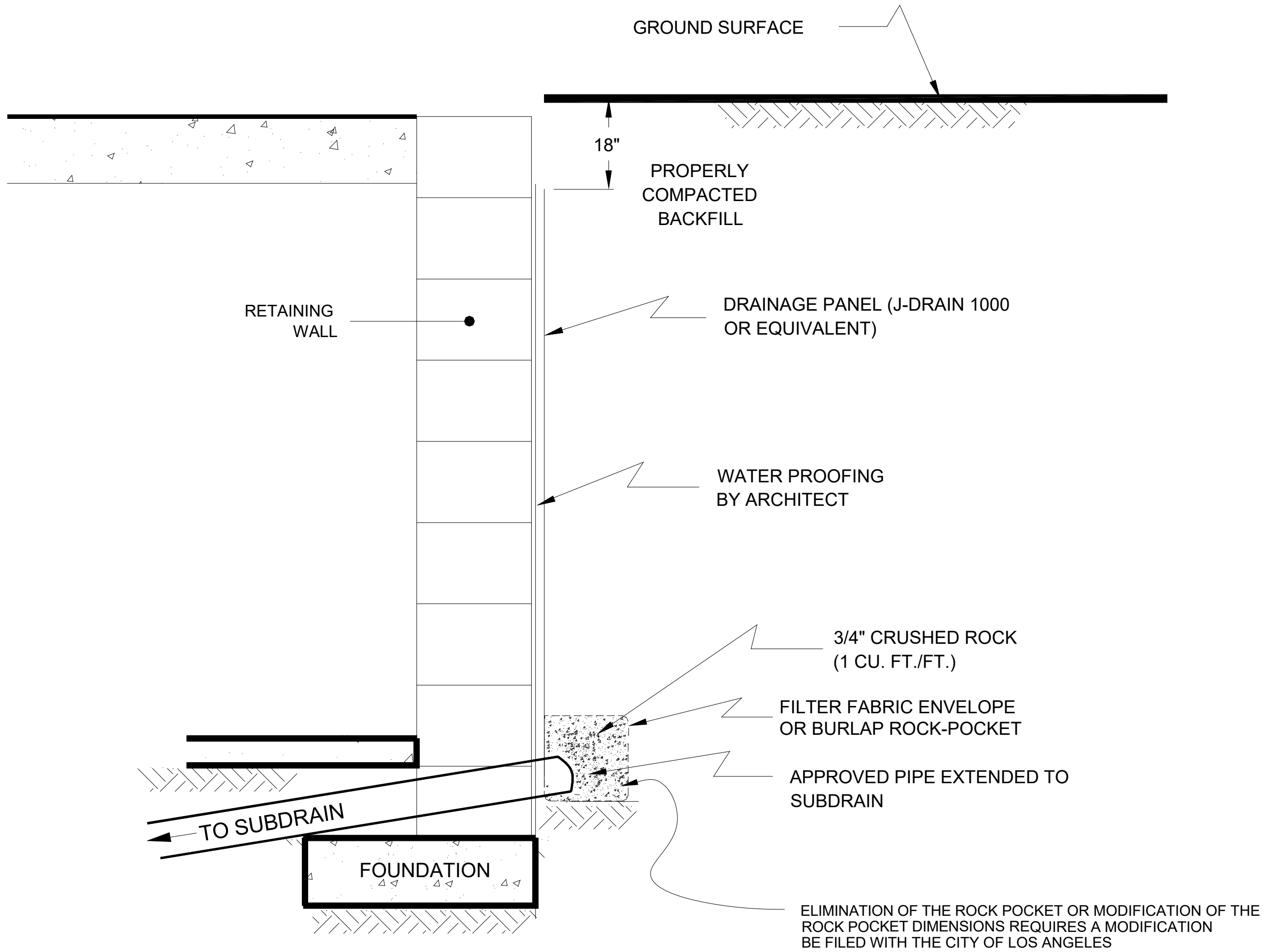

NO SCALE

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RETAINING WALL DRAIN DETAIL

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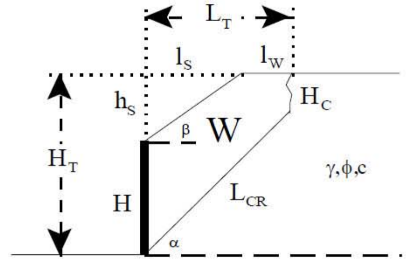
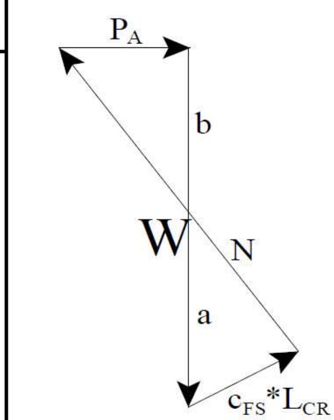
Shoring Design with Transitioned Backfill
(Vector Analysis)

Input:

Shoring Height	(H)	25.00 feet
Slope Angle of Backfill	(β)	0.0 degrees
Height of Slope above Shoring	(h_s)	0.0 feet
Horizontal Length of Slope	(l_s)	0.0 feet
Total Height (Shoring + Slope)	(H_T)	25.0 feet
Unit Weight of Retained Soils	(γ)	125.0 pcf
Friction Angle of Retained Soils	(ϕ)	32.0 degrees
Cohesion of Retained Soils	(c)	150.0 psf
Factor of Safety	(FS)	1.25
Factored Parameters:	(ϕ_{FS})	26.6 degrees
	(c_{FS})	120.0 psf



Failure Angle (α) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) feet^2	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
45	3.8	305	38172.5	29.9	10159.9	28012.7	9340.2
46	3.7	295	36919.8	29.6	9547.8	27372.0	9660.6
47	3.6	286	35697.9	29.3	8995.5	26702.5	9951.6
48	3.5	276	34506.5	28.9	8495.3	26011.2	10214.5
49	3.4	267	33344.9	28.6	8040.9	25304.0	10450.1
50	3.4	258	32212.1	28.3	7626.7	24585.4	10659.3
51	3.3	249	31107.0	27.9	7248.0	23859.0	10842.9
52	3.2	240	30028.6	27.6	6900.9	23127.7	11001.5
53	3.2	232	28975.6	27.3	6581.9	22393.7	11135.7
54	3.2	224	27946.9	27.0	6287.9	21659.0	11246.0
55	3.1	216	26941.3	26.7	6016.4	20924.9	11332.8
56	3.1	208	25957.6	26.4	5765.0	20192.6	11396.4
57	3.1	200	24994.7	26.1	5531.8	19462.9	11437.0
58	3.1	192	24051.6	25.8	5314.9	18736.7	11454.8
59	3.1	185	23127.1	25.6	5112.8	18014.3	11449.7
60	3.1	178	22220.3	25.3	4924.1	17296.2	11421.9
61	3.1	171	21330.2	25.0	4747.6	16582.6	11371.3
62	3.2	164	20455.9	24.8	4582.1	15873.8	11297.5
63	3.2	157	19596.4	24.5	4426.5	15169.8	11200.4
64	3.2	150	18750.8	24.2	4280.0	14470.8	11079.7
65	3.3	143	17918.4	24.0	4141.8	13776.6	10934.8
66	3.3	137	17098.3	23.7	4011.0	13087.3	10765.3
67	3.4	130	16289.8	23.5	3886.9	12402.9	10570.5
68	3.5	124	15492.0	23.2	3768.8	11723.1	10349.8
69	3.6	118	14704.2	23.0	3656.1	11048.1	10102.3
70	3.7	111	13925.7	22.7	3548.2	10377.5	9827.2



Design Equations (Vector Analysis):

$a = c_{FS}*L_{CR}*\sin(90+\phi_{FS})/\sin(\alpha-\phi_{FS})$

$b = W-a$

$P_A = b*\tan(\alpha-\phi_{FS})$

$EFP = 2*P_A/H^2$

Maximum Active Pressure Resultant

$P_{A,\ max}$ 11454.75 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of shoring)

$EFP = 2*P_A/H^2$

EFP 36.7 pcf 58.0 pcf

Design Shoring for an Equivalent Fluid Pressure: **37 pcf** Active **58 pcf** At-Rest

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SHORING PRESSURE CALCULATION

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APPENDIX A

APPENDIX A

FIELD INVESTIGATION

The site was explored on September 6 and September 7, 2016, by excavating two 8-inch-diameter borings utilizing a truck-mounted hollow-stem auger drilling machine. The borings were excavated to depths of approximately 45½ feet below the existing ground surface. Percolation testing for the design of a stormwater infiltration system was performed in boring B2. Representative and relatively undisturbed samples were obtained by driving a 3-inch, O. D., California Modified Sampler into the "undisturbed" soil mass with blows from a 140-pound auto-hammer falling 30 inches. The California Modified Sampler was equipped with 1-inch high by 2³/₈-inch diameter brass sampler rings to facilitate soil removal and testing. Bulk samples were also obtained.

The soil conditions encountered in the borings were visually examined, classified and logged in general accordance with the Unified Soil Classification System (USCS). Logs of the borings are presented on Figures A1 and A2. The logs depict the soil and geologic conditions encountered and the depth at which samples were obtained. The locations of the borings are shown on Figure 2.

PROJECT NO. A9202-06-01

BORING 1

ELEV. (MSL.) ________ DATE COMPLETED **9/6/16**

EQUIPMENT **HOLLOW STEM AUGER** BY: **RMA**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	BULK 0-5'				**AC: 3.5" BASE: NONE**			
					ARTIFICIAL FILL			
					Silty Sand, loose, slightly moist, dark brown, fine- to meduim-grained, trace asphalt fragments (to 4").			
2					Sand with Silt, poorly graded, loose, slightly moist, yellowish brown, fine- to medium-grained, some coarse-grained, some fine gravel (to 4"). - asphalt chunks			
4					Silty Sand, very loose, slightly moist, dark brown with yellowish oxidation mottles, fine- to medium-grained, trace fine gravel.			
	B1@5'					6	112.0	13.9
6					**ALLUVIUM**			
					Silty Sand, very loose, slightly moist, dark brown, fine- to medium-grained, trace coarse-grained.			
8				SM	- loose, yellowish brown, fine-grained, trace medium-grained			
10	B1@10'					14	111.5	16.9
12								
14					Silt, stiff, slightly moist, yellowish brown, trace fine- to medium-grained sand.			
	B1@15'					31	106.2	21.3
16				ML				
18	B1@18'				- stiff	33	109.7	20.1
20	BULK 20-25'				Silty Sand, medium dense, slightly moist, yellowish brown, fine- to medium-grained.			
	B1@21'					20	107.4	16.7
22				SM	- trace medium-grained			
24	B1@24'					15	103.8	14.5
26					Silt with Sand, firm, slightly moist, dark yellowish brown, fine- to medium-grained, trace clay.			
	B1@27'			ML		21	108.0	16.7
28								
				ML	Sandy Silt, stiff, slightly moist, dark brown, fine-grained, trace clay.			

A9202-06-01 BORING LOGS.GPJ

Figure A1,
Log of Boring 1, Page 1 of 2

SAMPLE SYMBOLS			
	... SAMPLING UNSUCCESSFUL	... STANDARD PENETRATION TEST	... DRIVE SAMPLE (UNDISTURBED)
	... DISTURBED OR BAG SAMPLE	... CHUNK SAMPLE	... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

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BORING 1

ELEV. (MSL.) ________ DATE COMPLETED **9/6/16**

EQUIPMENT **HOLLOW STEM AUGER** BY: **RMA**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B1@30'			ML		22	117.6	15.3
32								
34				SP	Sand, poorly graded, dense, slightly moist, yellowish brown, fine- to medium-grained, trace silt.			
36	B1@35'					45	123.4	9.0
38								
40	B1@40'			SP-SM	Sand with Silt, poorly graded, medium dense, slightly moist, yellowish brown, fine- to medium-grained, trace coarse-grained.	28	119.4	10.8
42								
44	B1@45'				- dense, trace clay	43	121.3	14.4

Total depth of boring: 45.5 feet
Fill to 5.5 feet.
No groundwater encountered.
Percolation testing performed on 9/7/16.
Backfilled with cutting and tampled.
Surface patched with asphalt.

*Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.

Figure A1,
Log of Boring 1, Page 2 of 2

A9202-06-01 BORING LOGS.GPJ

SAMPLE SYMBOLS
- ... SAMPLING UNSUCCESSFUL
- ... STANDARD PENETRATION TEST
- ... DRIVE SAMPLE (UNDISTURBED)
- ... DISTURBED OR BAG SAMPLE
- ... CHUNK SAMPLE
- ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

PROJECT NO. A9202-06-01

BORING 2

ELEV. (MSL.) ______ DATE COMPLETED **9/6/16**

EQUIPMENT **HOLLOW STEM AUGER** BY: **RMA**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	BULK 0-5'				**AC: 6" BASE: NONE** **ARTIFICIAL FILL** Silty Sand, loose, slightly moist, dark brown, fine- to medium-grained, some asphalt chunks (to 2").			
2					Sand with Silt, poorly graded, loose, slightly moist, yellowish brown, fine- to medium-grained, some coarse-grained, some fine gravel (to 1").			
4	B2@5'				Silty Sand, very loose, slightly moist, dark brown, fine- to medium-grained, trace fine gravel.	6	113.7	9.6
6				SM	**ALLUVIUM** Silty Sand, very loose, slightly moist, dark yellowish brown, fine- to medium-grained.			
8					- loose, increase in silt content			
10	B2@10'					14	119.1	6.5
12								
14	B2@15'			ML	Sandy Silt, stiff, slightly moist, yellowish brown, fine-grained, trace clay.	25	109.0	18.5
16								
18	B2@18'			ML	Silt, stiff, slightly moist, yellowish brown, trace fine-grained sand, trace clay.	34	110.8	20.3
20	BULK 20-25' B2@21'					33	112.0	18.5
22								
24	B2@24'				- stiff	21	107.3	19.4
26	B2@27'			ML	Silt with Sand, firm, slightly moist, yellowish brown, fine-grained, trace clay.	17	113.2	10.0
28				ML	Sandy Silt, stiff, slightly moist, dark brown, fine-grained, trace clay.			

Figure A2,
Log of Boring 2, Page 1 of 2

A9202-06-01 BORING LOGS.GPJ

SAMPLE SYMBOLS

- ... SAMPLING UNSUCCESSFUL
- ... STANDARD PENETRATION TEST
- ... DRIVE SAMPLE (UNDISTURBED)
- ... DISTURBED OR BAG SAMPLE
- ... CHUNK SAMPLE
- ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

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DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	**BORING 2** ELEV. (MSL.) ______ DATE COMPLETED **9/6/16** EQUIPMENT **HOLLOW STEM AUGER** BY: **RMA** — MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B2@30'			ML		22	117.6	15.3
32								
34				SM	Silty Sand, medium dense, slightly moist, yellowish brown, fine- to medium-grained, trace coarse-grained, trace fine gravel (to 1").			
36	B2@35'					40	123.4	9.0
38					- very dense			
40	B2@40'					55	123.0	13.4
42								
44				SP	Sand, poorly graded, dense, slightly moist, grayish brown, fine- to medium-grained, some coarse-grained, some fine gravel (to 1").			
	B2@45'					74	122.7	5.6

Total depth of boring: 45.5 feet
Fill to 5.5 feet.
No groundwater encountered.
Backfilled with soil cuttings and tamped.
Surface patched with asphalt.

*Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.

Figure A2,
Log of Boring 2, Page 2 of 2

A9202-06-01 BORING LOGS.GPJ

SAMPLE SYMBOLS
... SAMPLING UNSUCCESSFUL
... STANDARD PENETRATION TEST
... DRIVE SAMPLE (UNDISTURBED)
... DISTURBED OR BAG SAMPLE
... CHUNK SAMPLE
... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

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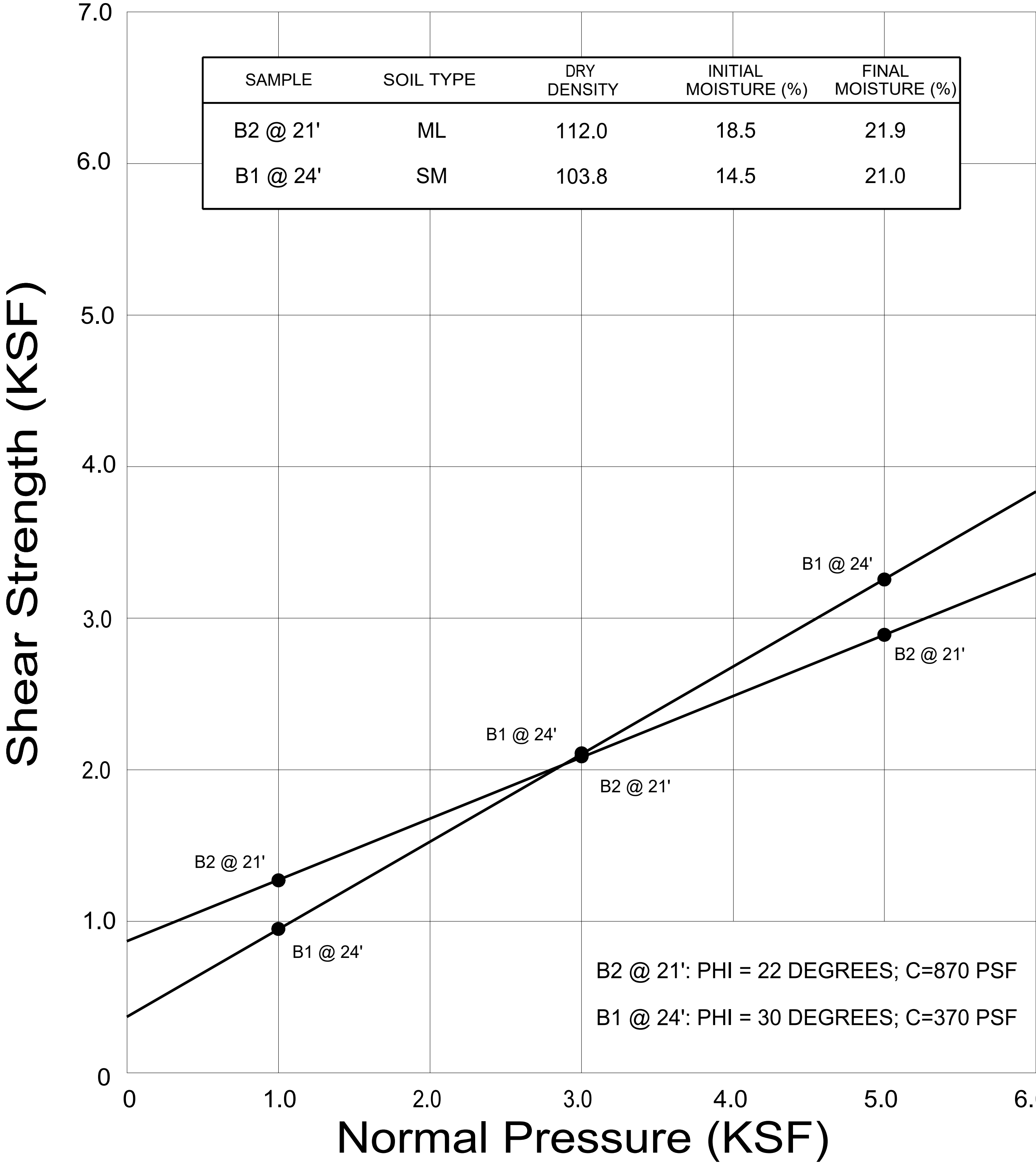
APPENDIX B

APPENDIX B

LABORATORY TESTING

Laboratory tests were performed in accordance with generally accepted test methods of the "American Society for Testing and Materials (ASTM)", or other suggested procedures. Selected samples were tested for direct shear strength, consolidation, corrosivity, in-place dry density and moisture content. The results of the laboratory tests are summarized in Figures B1 through B5. The in-place dry density and moisture content of the samples tested are presented on the boring logs, Appendix A.



SAMPLE	SOIL TYPE	DRY DENSITY	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B2 @ 21'	ML	112.0	18.5	21.9
B1 @ 24'	SM	103.8	14.5	21.0

● Direct Shear, Saturated

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DIRECT SHEAR TEST RESULTS

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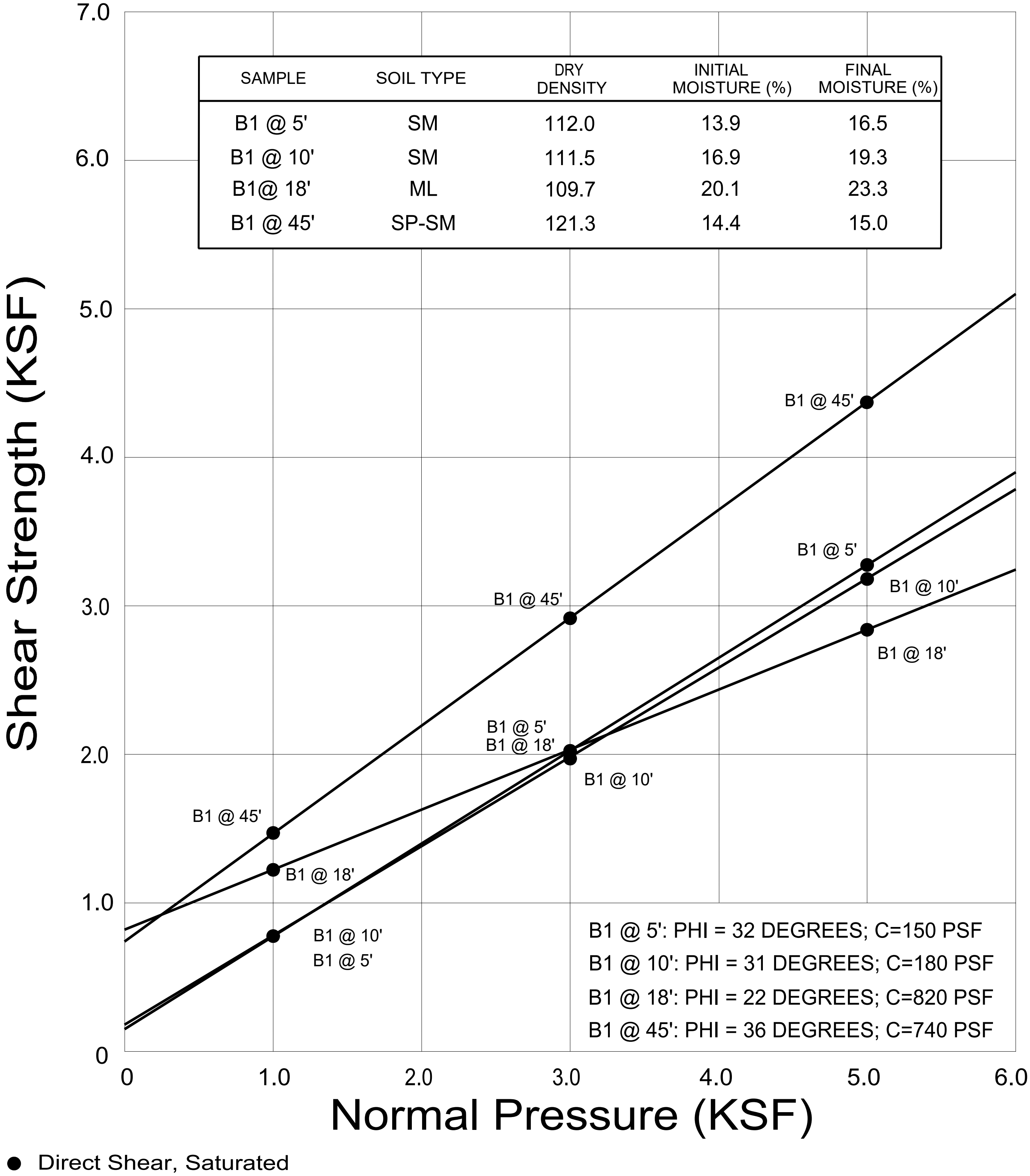
OCT 2016 | PROJECT NO. A9202-06-01 | FIG. B1



SAMPLE	SOIL TYPE	DRY DENSITY	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1 @ 5'	SM	112.0	13.9	16.5
B1 @ 10'	SM	111.5	16.9	19.3
B1@ 18'	ML	109.7	20.1	23.3
B1 @ 45'	SP-SM	121.3	14.4	15.0

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DIRECT SHEAR TEST RESULTS

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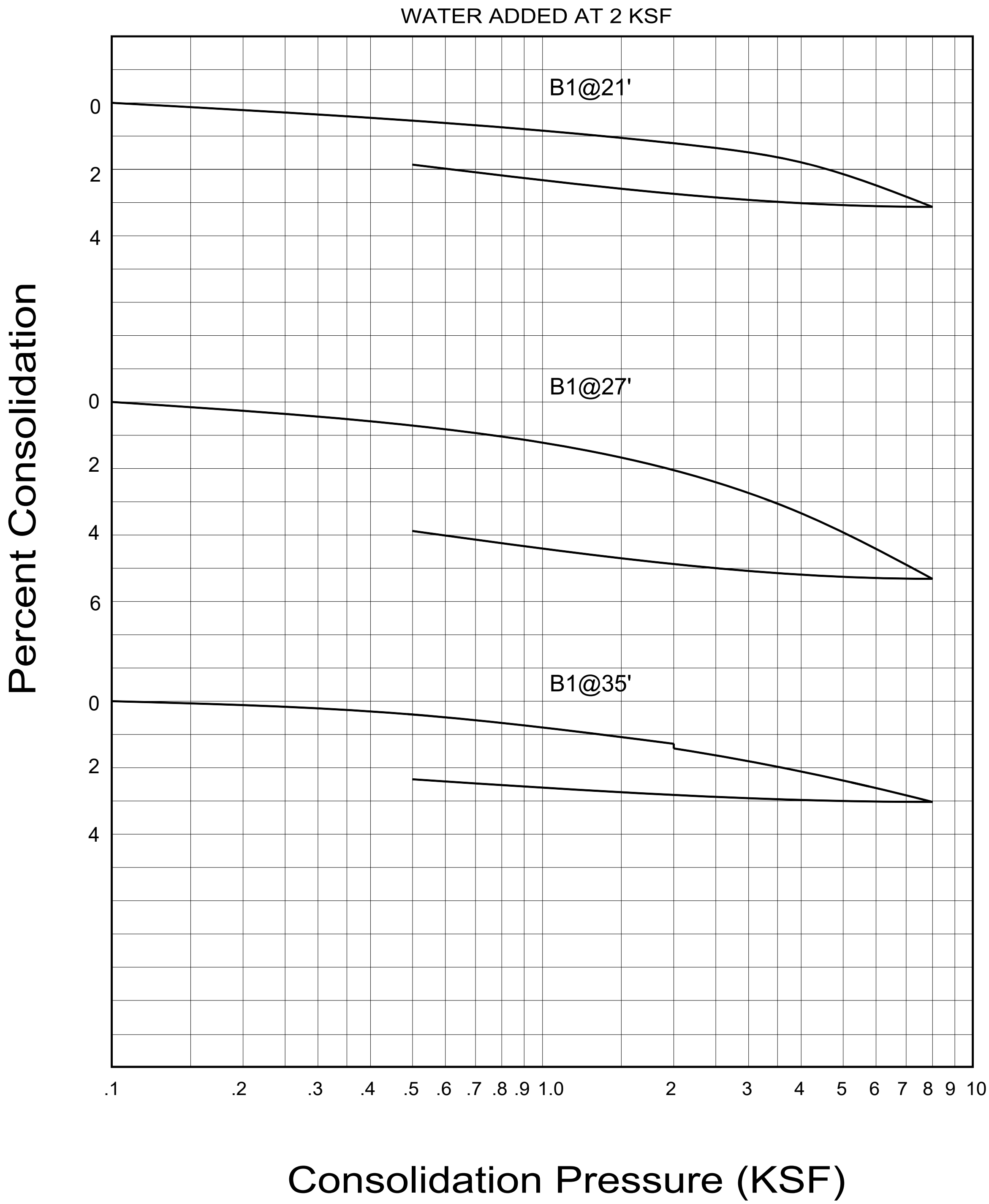

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CONSOLIDATION TEST RESULTS

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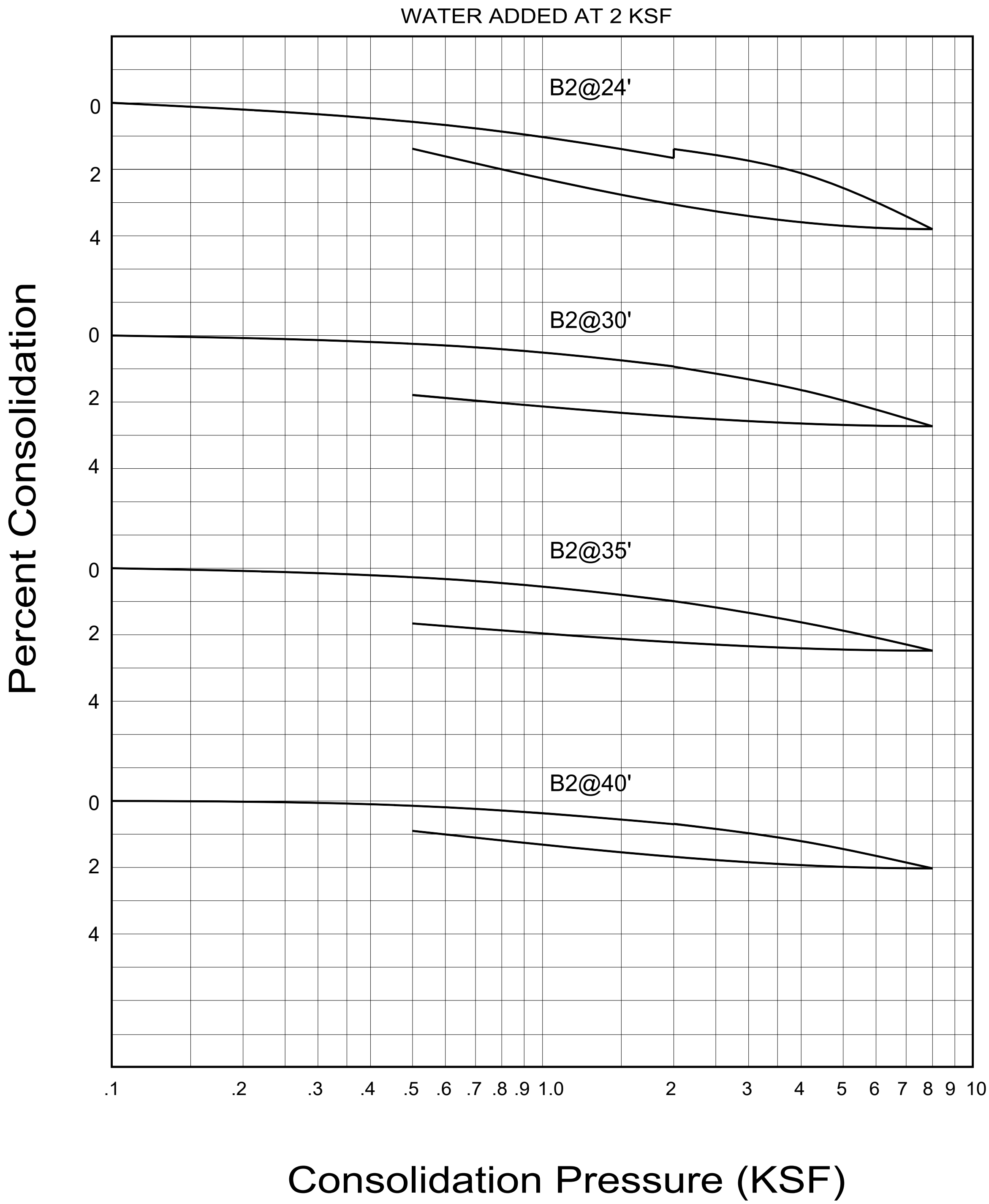

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CONSOLIDATION TEST RESULTS

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OCT 2016 | PROJECT NO. A9202-06-01 | FIG. B4

SUMMARY OF LABORATORY POTENTIAL OF HYDROGEN (pH) AND RESISTIVITY TEST RESULTS CALIFORNIA TEST NO. 643

Sample No.	pH	Resistivity (ohm centimeters)
B1 @ 20-25'	8.11	790 (Severely Corrosive)

SUMMARY OF LABORATORY CHLORIDE CONTENT TEST RESULTS EPA NO. 325.3

Sample No.	Chloride Ion Content (%)
B1 @ 20-25'	0.004

SUMMARY OF LABORATORY WATER SOLUBLE SULFATE TEST RESULTS CALIFORNIA TEST NO. 417

Sample No.	Water Soluble Sulfate (% SO_4)	Sulfate Exposure*
B1 @ 20-25'	0.03	Negligible

* Reference: 2013 California Building Code, Section 1904.3 and ACI 318-11 Section 4.3.

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CORROSIVITY TEST RESULTS

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GPI GEOTECHNICAL PROFESSIONALS INC.

December 19, 2018

Sares-Regis Group
18802 Bardeen Avenue
Irvine, California 92612

Attention: Mr. John Pinnell
Development Manager

Subject: Planning-Level Preliminary Geotechnical Investigation
Proposed Apartment Development
6266 West Sunset Boulevard
Los Angeles, California
GPI Project No. 2910.I

Dear Mr. Pinnell:

In accordance with your request, this report presents the results of our planning-level preliminary geotechnical investigation for the proposed apartment and retail development at the subject site. The purpose of our investigation was to determine, based on existing published data and limited subsurface exploration, if significant geotechnical constraints currently affect the site.

INTRODUCTION

We understand the proposed project may consist of a podium type structure with 5 to 7 levels of residential units over 2 to 3 levels of parking below grade. The lowest parking level is anticipated to be on the order of 24 to 36 feet below existing grades. The project will include 100 to 150 apartments. Column loads are not available at this preliminary stage, but we anticipate the maximum loads to be on the order of 700 to 1000 kips.

The project site is located at 6266 West Sunset Boulevard in the Hollywood Area of the City of Los Angeles, California. The location of the site is shown on the Site Location Map, Figure 1. The approximate limits of proposed development are shown on the Exploration Location Plan, Figure 2.

SCOPE OF WORK

Our scope of work included review of published information, limited subsurface exploration, engineering evaluations, and preparation of this planning-level preliminary geotechnical letter report. We reviewed the Special Studies Fault Zone maps and Seismic Hazard Zone maps as part of our study.

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We performed three cone penetration tests (CPTs) to evaluate subsurface conditions at the site. The approximate exploration locations are shown on Figure 2. The CPT's were performed to depths of approximately 51½ to 75 feet below existing grade. A description of field procedures and logs of the CPTs are presented in the attached Appendix. The approximate locations of the subsurface explorations are shown on the Site Plan, Figure 2.

Engineering evaluations were performed to provide planning-level recommendations and an assessment of seismic hazards. The results of our evaluations are presented in the remainder of the report.

SURFACE CONDITIONS

The site is approximately 27,250 square feet in plan and currently occupied by 1 and 2 story retail/restaurant buildings and asphalt paved parking and drives in the southern portion of the site. The site is bounded on the north by West Sunset Boulevard and on the south by Leland Way. The Sunset Vine Tower building at 1480 Vine Avenue is located due west of the project site. The Sunset Vine Tower building is a 20-story building with 2 levels of below grade parking. The property due east of the project site is current under construction. It appears the future building to the east will have at least one subterranean level. Ground surface elevations gently slope downward from approximate elevation 349 feet along West Sunset Boulevard to 343 feet along Leland Way.

SUBSURFACE CONDITIONS

Our preliminary field investigation was performed with CPTs; as such the presence of fill soils, or lack thereof, could not be evaluated. With the planned construction including subterranean parking levels, the upper 24 to 36 feet of materials would be excavated and exported off-site.

Based on the CPTs conducted for this study, the site soils generally consist of 11 to 15 feet of loose to medium dense silty sand and firm sandy silt, underlain predominantly by interbedded layers of very stiff to hard clays and silts to the depth explored. Discontinuous layers of medium dense to very dense sands and silty sands, approximately 3 to 15 feet in thickness, were encountered at depths of 37 to 59 feet below grade.

The natural soils in the site vicinity are geologically mapped as Quaternary sediments that include older and younger alluvial-fan deposits.

The holes resulting from our CPTs caved back to depths of approximately 37 to 46 feet below grade. Groundwater was not encountered above those depths. Historical high groundwater levels are reported to be on the order of 50 to 60 feet below the existing ground surface (CDMG, 2001).

FINDINGS AND RECOMMENDATIONS

Based on the results of our investigation and experience with nearby, similar projects, it is our opinion that from a geotechnical engineering viewpoint it is feasible to develop the site as proposed. The most significant geotechnical issues that will affect the design and construction of the proposed structures are as follows:

- There are no known faults crossing or projecting through the site. The development does not lie within an Alquist-Priolo Earthquake Fault Zone as designated by the California Geologic Survey (CGS) or within a Preliminary Fault Rupture Study Area (PFRSA) as designated by the City of Los Angeles, which would require further studies of the fault. Therefore, ground rupture due to faulting is considered unlikely at this site. The closest fault to the site is the Hollywood fault, which is mapped about 0.34 miles to the north. The site is located approximately 0.25 miles south of the Alquist-Priolo Earthquake Fault Zone for the Hollywood fault.

- Typical of Southern California, there is a potential that the site development will be subjected to strong ground motion during its life. We computed that the site could be subjected to a site modified peak ground acceleration (PGA_M) of 0.92g for a magnitude 6.8 earthquake. This acceleration was computed using the mapped Maximum Considered Geometric Mean peak ground acceleration from ASCE 7-10 (ASCE, 2010) and a site coefficient (F_{PGA}) based on Site Class D conditions. The predominant earthquake magnitude (6.8) was determined using a 2 percent probability of exceedance in a 50-year period, or an average return period of 2,475 years. The structural design will need to incorporate measures to mitigate the effects of strong ground motion. The PGA value above is used to evaluate the potential for soil liquefaction and seismic settlement.

- The site is not located in a Seismic Hazards Zone for liquefaction nor earthquake induced landslide zones. The site was likely excluded from a liquefaction hazard zone because historical high groundwater is deeper than 50 feet below grade.

- The site is not located in either a Methane Zone or Methane Buffer Zone as mapped by the City of Los Angeles (NavigateLA; LADPW, 2004).

- We evaluated the potential for seismic-induced (dry-sand) settlement of the soils to be ¼-inch or less below the planned subterranean levels. This settlement value is considered to be tolerable for most structures of this type. The potential seismic induced settlement of the upper 11 to 15 of loose silty sand materials is expected to exceed tolerable levels if left in place.

- The proposed project should be designed in accordance with the current version of the California Building Code (currently the 2016 CBC) and current version of the LABC (currently the 2017 LABC). The available data indicates Site Class D (Stiff Soil) would be appropriate for use in design according to the 2016 CBC. The Project Structural Engineer can determine the remaining seismic code values by using the value above and the pertinent websites and tables from the building code.

- Because the field explorations program was limited to CPTs, the presence of existing fills soils, or lack thereof, was not evaluated. It is likely, due to past development activities, some shallow fill exists at the site. Existing fills will need to be removed within building areas and replaced with properly compacted fill, where not removed by subterranean construction.

- The planned excavation for subterranean parking levels will likely remove existing undocumented fills and low density upper soils across the majority of the site. Removals are anticipated for remedial grading to support minor at-grade structures on spread footings. For planning purposes, we anticipate that it will be required to remove and recompact the materials within the upper 5 feet prior to constructing minor at-grade structures (screen walls, etc.).

- Based on our preliminary findings the earthwork can be performed using conventional rubber-tired equipment. This should be further evaluated during a design level geotechnical investigation.

- Additional laboratory testing should be conducted during the design level geotechnical investigation to evaluate the expansion potential of the clay soils that will be encountered at the lowest subterranean level. These potentially expansive soils are not anticipated to impact the structure if a subterranean level is planned but may need to be considered in the support of at grade floor slab and exterior hardscape. Because of the poor drainage characteristics of the fine-grained clays and silts, these soils are not considered suitable for use as backfill behind retaining walls.

- Based on our findings, it appears that the proposed building may be supported on either spread footings or a mat foundation underlain by the undisturbed, very stiff to hard clays and silts encountered at the anticipated subterranean levels. An allowable net bearing capacity for spread footings on the order of 4 kips per square foot (ksf) for the undisturbed natural materials (subterranean levels) or 2 ksf for properly compacted fill (minor at-grade foundations) is anticipated. We anticipate a mat foundation would have bearing pressures ranging from approximately 1,200 to 2,500 pounds per square foot (psf). Settlement analyses based on the actual design loads and the configurations of the proposed building foundations relative to the adjacent existing structures should be conducted during the design level investigation to assess the feasibility of these two foundation types further.

- Details regarding adjacent structures and foundations should be considered in site planning and design so as not to remove support of adjacent structures during construction or impose additional surcharge loading and/or settlement on the adjacent structures. Deep foundations such as auger cast piles or drilled piers would be required to support the proposed structure if the proposed structure supported on spread or mat foundations surcharges the adjacent structures or induces excessive settlement on the adjacent properties.

- Where there is not sufficient space for sloped embankments, which appears to be the majority of the site, shoring will be required. One method of shoring would consist of steel soldier piles placed in drilled holes, backfilled with concrete, and tied-back with earth anchors. Driven or vibrated soldier piles may also be a considered if potential vibration impacts on adjacent structures is mitigated. The tie-back anchors may require permission and be subject to limitations from the adjacent property owners and the City of Los Angeles. Rakers providing support to the soldier piles from inside the excavation would be an option if tie-backs are not allowed.

- Groundwater is not anticipated to impact the construction or long-term maintenance of the development.

- The upper silty sands and deeper clays are anticipated to have R-values on the order of 30 and 5, respectively. Where pavements are supported by the upper silty sands recompacted as engineered fill, we anticipate an asphalt pavement section of 3 inches of asphalt concrete over 7 inches of aggregate base for access driveways and portland cement concrete drives on the order of 7 inches thick. Where pavements are supported by the deeper clay soils, we anticipate an asphalt pavement section of 3½ inches of asphalt concrete over 10 inches of aggregate base for access driveways and portland cement concrete drives on the order of 8 inches thick.

- The existing buildings on site are anticipated to be supported on shallow spread foundations. If the buildings are supported on piles, care should be taken during site demolition to reduce the disturbance of the in-place soils. Removal of the piles should only include the upper portion of the piles (to within 5 feet of the bottom of proposed foundations), after excavating the adjacent soils and cutting the concrete and steel. Removal should not include bending or breaking the piles or pulling in an attempt to remove the entire element.

In general, the subject site is favorable from a geotechnical standpoint with some geotechnical constraints that will impact the design and construction of the planned project. Additional explorations and testing, as well as structural loads and details, will be required for the design-level geotechnical study to provide detailed recommendations for design of foundations, temporary shoring, and walls below grade.

LIMITATIONS

The geotechnical investigation reported herein was performed for the exclusive use by Sares-Regis Group and their consultants in evaluating the feasibility of constructing the proposed improvements. This report should not be used for evaluating the feasibility of developing the site for other uses or for the detailed design of the proposed project, because this report does not contain sufficient or appropriate information for such use.

Soil deposits may vary in type, strength, and many other important properties between points of exploration due to non-uniformity of the geologic formations or to man-made cut and fill operations. While we cannot evaluate the consistency of the properties of materials in areas not explored, the conclusions drawn in this report are based on the assumption that the data obtained in the field are reasonably representative of field conditions and are conducive to interpolation and extrapolation.

As noted previously, additional geotechnical investigations will be needed for design and construction. Furthermore, our recommendations were developed with the assumption that a proper level of field observation and construction review will be provided by a qualified geotechnical consulting firm during grading, excavation, and foundation construction. If design- and construction-phase geotechnical services are performed by others they must accept full responsibility for all geotechnical aspects of the project.

Our investigation and evaluations were performed using generally accepted engineering approaches and principles available at this time and the degree of care and skill ordinarily exercised under similar circumstances by reputable Geotechnical Engineers practicing in this area. No other representation, either express or implied, is included or intended in our report.

Respectfully submitted,
Geotechnical Professionals Inc.

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Enclosures: References
Figure 1 - Site Location Map
Figure 2 - Exploration Location Plan
Appendix - Cone Penetration Tests

Distribution: (3) Addressee
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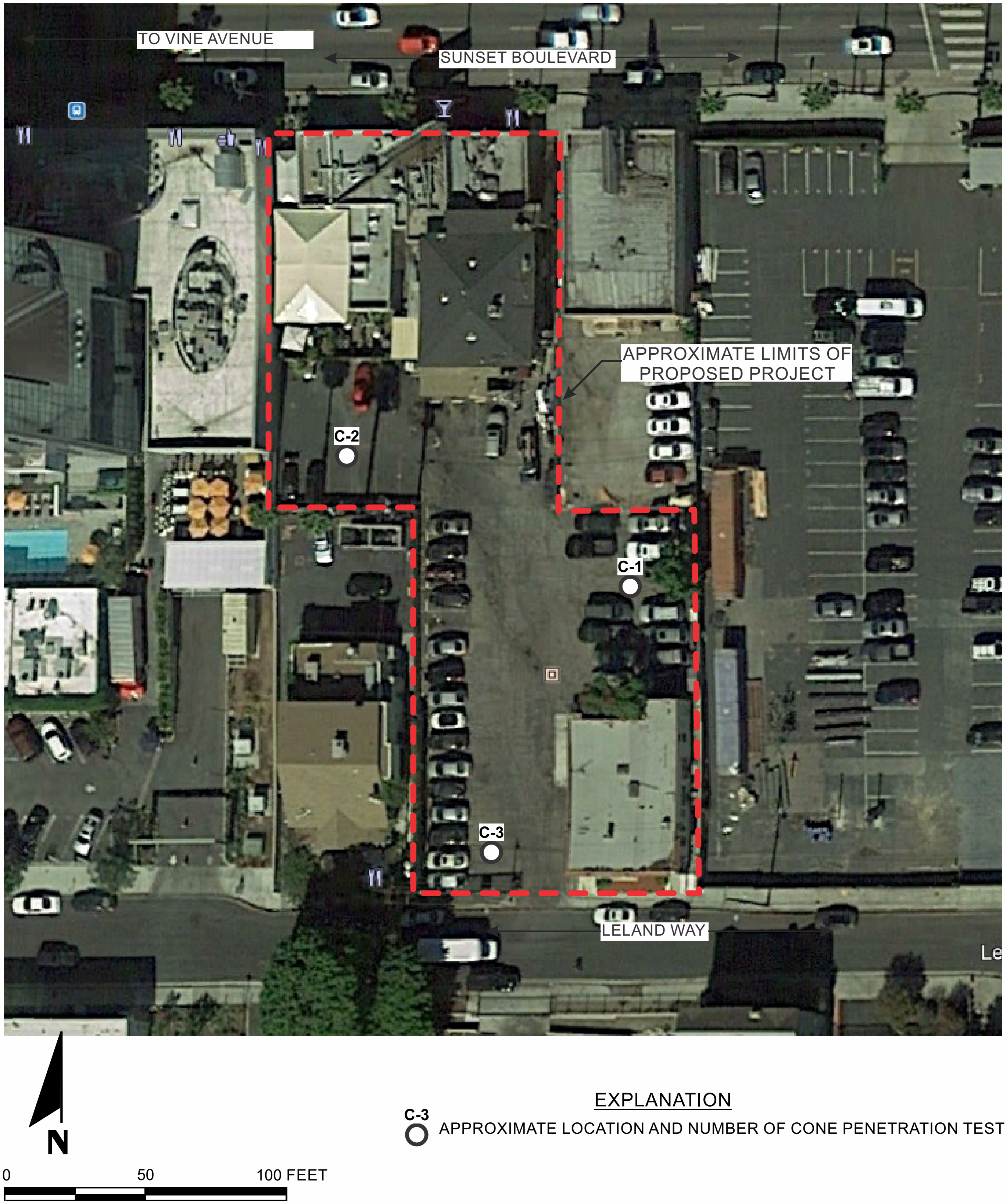
GPI GEOTECHNICAL PROFESSIONALS, INC.		SITE LOCATION MAP
SRG 6266 WEST SUNSET BLVD		
GPI PROJECT NO. 2910.I	SCALE: 1" = 2000'	FIGURE 1



EXPLANATION

C-3 APPROXIMATE LOCATION AND NUMBER OF CONE PENETRATION TEST

BASE MAP REPRODUCED FROM GOOGLE EARTH © 2018

GEOTECHNICAL PROFESSIONALS, INC.

SRG 6266 WEST SUNSET BLVD

GPI PROJECT NO. 2910.I | SCALE: 1" = 50'

EXPLORATION LOCATION PLAN

FIGURE 2

APPENDIX

APPENDIX

CONE PENETRATION TESTS

The subsurface conditions were investigated by performing three cone penetration tests (CPT’s) at the site. The soundings were advanced to depths of 52 to 75 feet below existing grades. The approximate locations of the CPT’s are shown on the Exploration Location Plan, Figure 2.

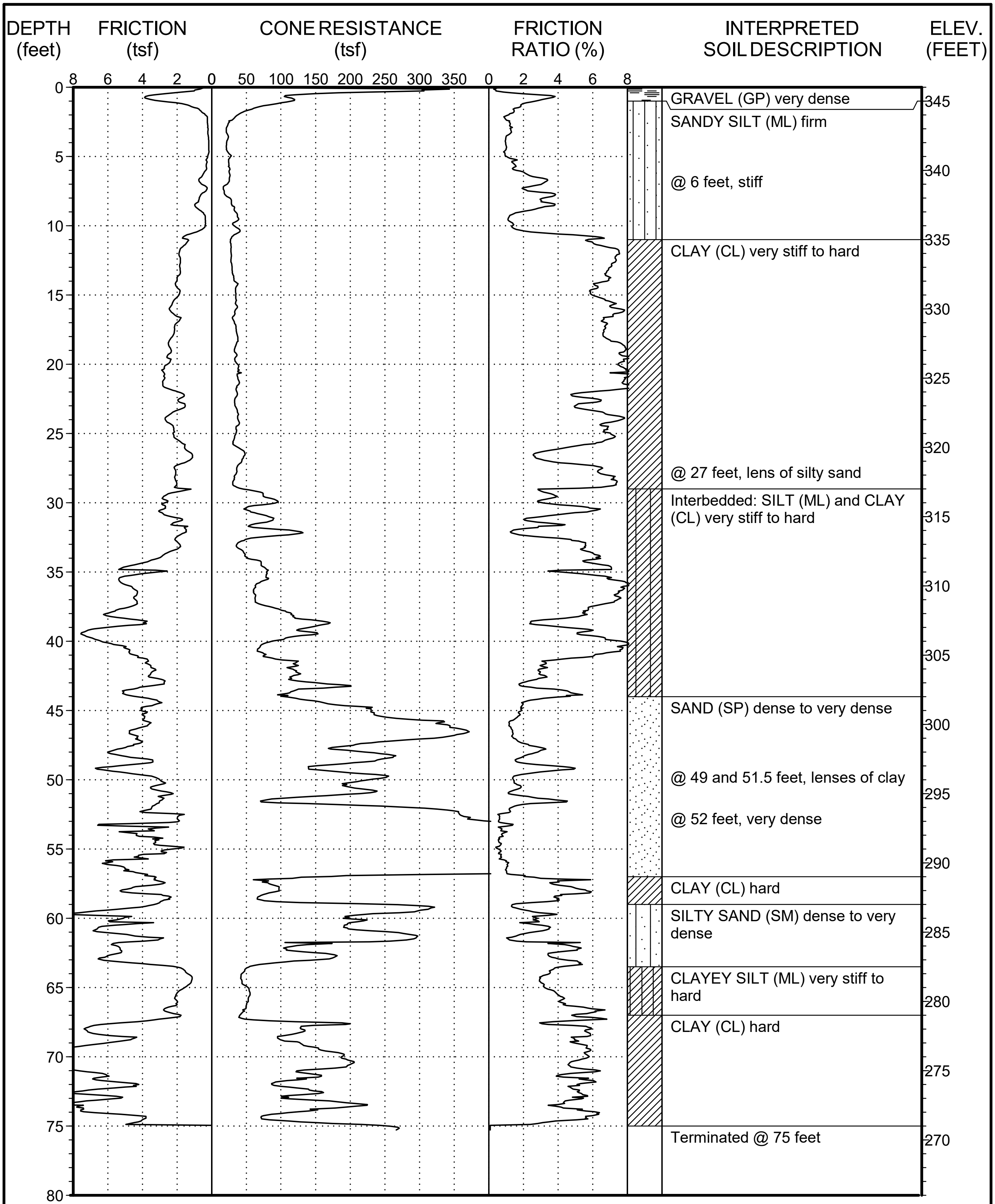
The Cone Penetration Test consists of pushing a cone-tipped probe into the soil deposit while simultaneously recording the cone tip resistance and side friction resistance of the soil to penetration (refer to Figure A-1). The CPT's described in this report were conducted in general accordance with ASTM specifications (ASTM D 5778) using an electric cone penetrometer.

The CPT equipment consists of a cone assembly mounted at the end of a series of hollow sounding rods. A set of hydraulic rams is used to push the cone and rods into the soil while a continuous record of cone and friction resistance versus depth is obtained in both analog and digital form at the ground surface.

Data obtained during a CPT consists of continuous stratigraphic information with close vertical resolution. Stratigraphic interpretation is based on relationships between cone tip resistance and friction resistance. The calculated friction ratio (CPT friction sleeve resistance divided by cone tip resistance) is used as an indicator of soil type. Granular soils typically have low friction ratios and high cone resistance, while cohesive or organic soils have high friction ratios and low cone resistance. These stratigraphic material categories form the basis for all subsequent calculations, which utilize the CPT data.

Computer plots of the reduced CPT data acquired for this investigation are presented in Figures A-2 through A-4 of this appendix. The field testing and computer processing for the current investigation was performed by Kehoe Testing under subcontract to Geotechnical Professionals Inc. (GPI). The interpreted soil descriptions were prepared by GPI.

The CPT locations were laid out in the field by measuring from existing features at the site. Upon completion, the CPT hole was backfilled above caving with a bentonite plug and capped with quick set grout. The ground surface elevations at the CPT locations were estimated from Google Earth and should be considered very approximate.



Date performed: 11-29-18

This summary applies only at the location of this cone penetration test and at the time of the exploration. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The interpreted soil description is derived from the friction ratio and cone resistance and is a simplification of actual conditions encountered.

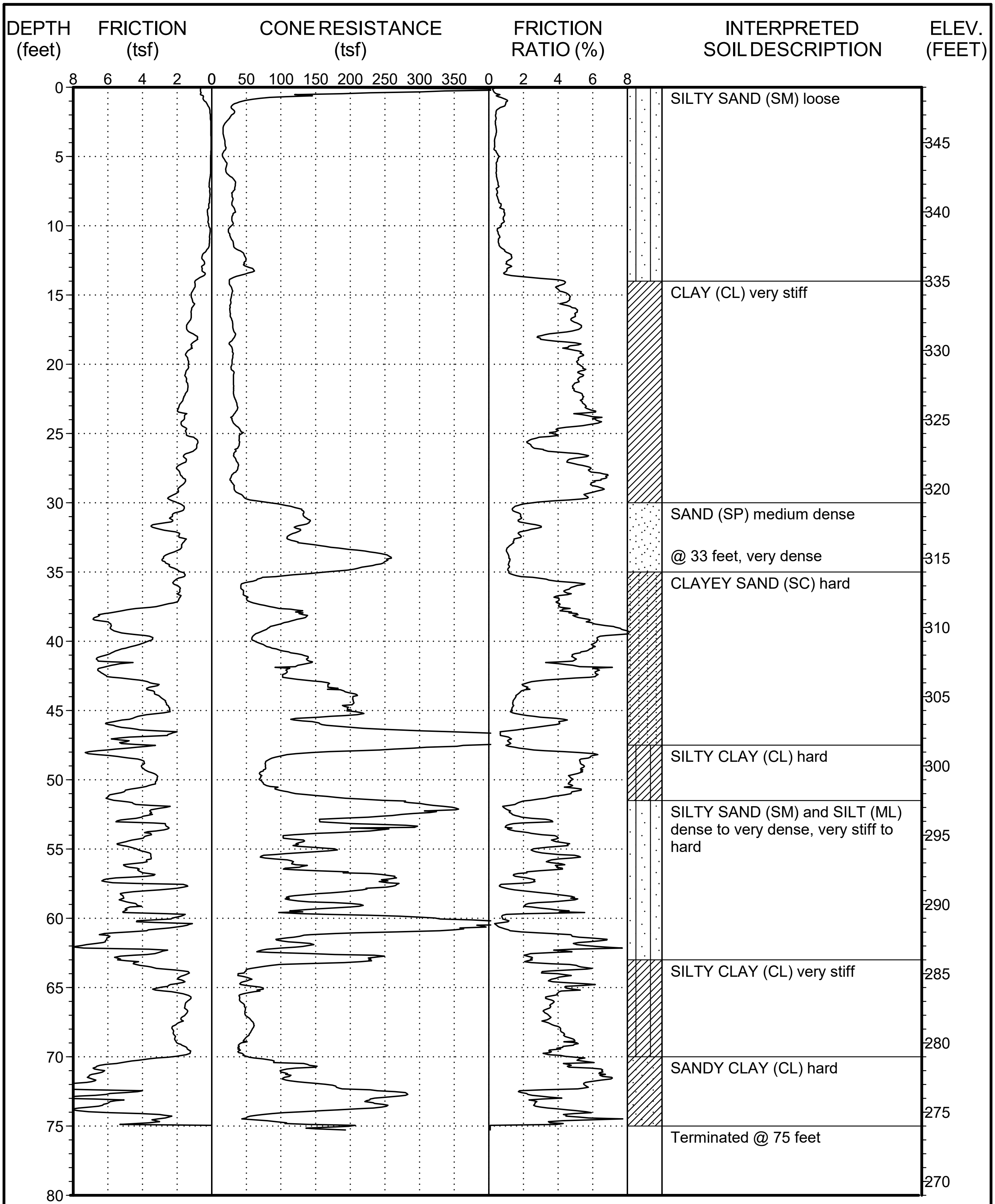
GPI

PROJECT NO.: 2910.I

SRG SUNSET

LOG OF CPT NO. C-1

FIGURE A-1



Date performed: 11-29-18

This summary applies only at the location of this cone penetration test and at the time of the exploration. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The interpreted soil description is derived from the friction ratio and cone resistance and is a simplification of actual conditions encountered.

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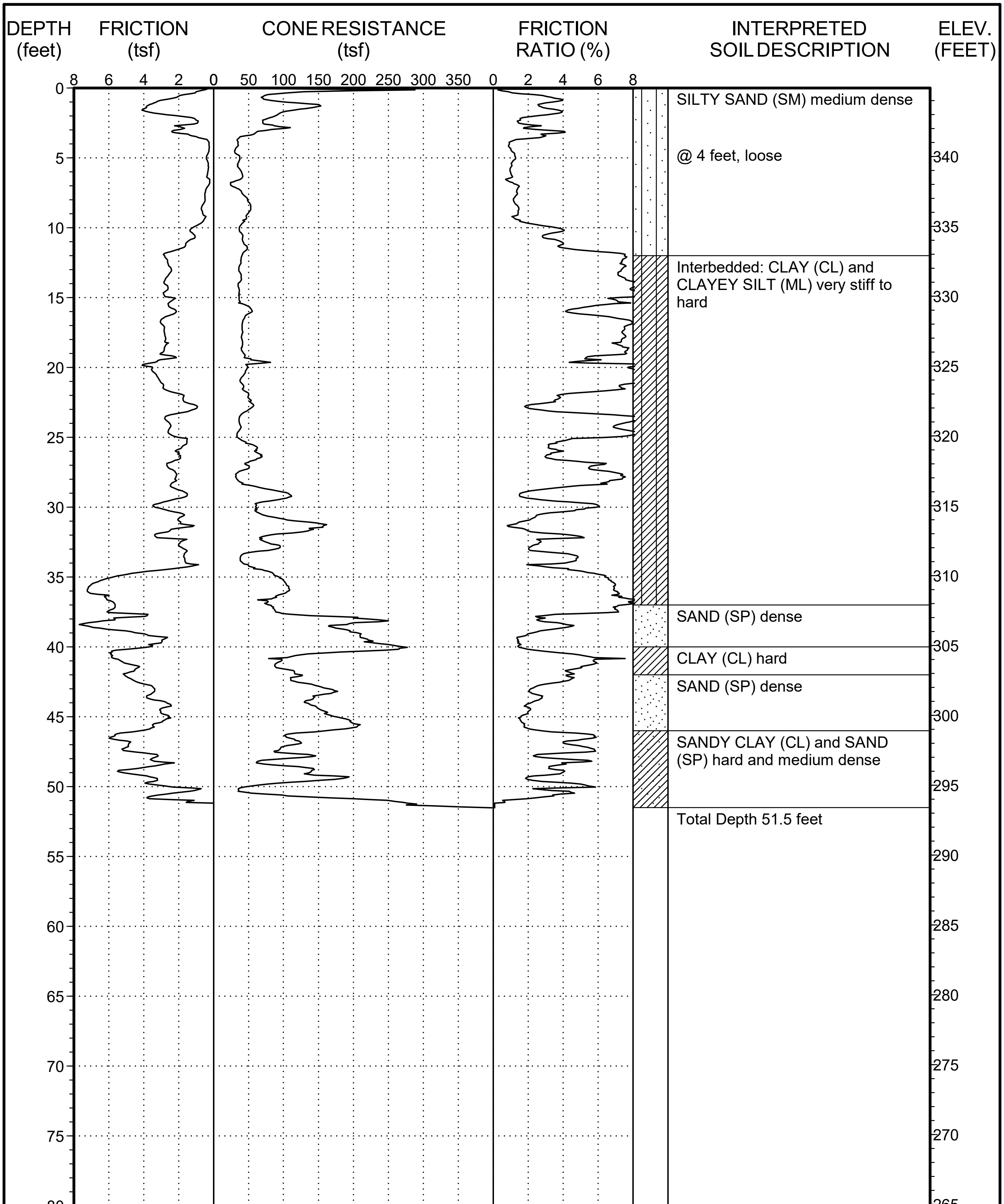
PROJECT NO.: 2910.I

SRG SUNSET

LOG OF CPT NO. C-2

FIGURE A-2



Date performed:11-29-18

This summary applies only at the location of this cone penetration test and at the time of the exploration. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The interpreted soil description is derived from the friction ratio and cone resistance and is a simplification of actual conditions encountered.

PROJECT NO.: 2910.I

SRG SUNSET

LOG OF CPT NO. C-3

FIGURE A-3