Appendix I

Hydrology and Water Quality Report

HARVARD-WESTLAKE RIVER PARK
(4141 WHITSETT AVENUE, STUDIO CITY CA 91604)
HYDROLOGY AND WATER QUALITY REPORT
FEBRUARY 2022

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Appendix

1. INTRODUCTION

1.1. PROJECT DESCRIPTION

The Project Applicant, Harvard-Westlake School (School) is proposing to repurpose a site currently occupied by a nine-hole, 27-par golf course and tennis facility, for use as an athletic and recreational facility for its students, employees and the general public (Project). The area proposed for the Project consists of an approximately 16.1-acre parcel, owned by the School (Property), and an approximately 1.1-acre parcel the School leases from Los Angeles County (Leased Property), which collectively comprise the approximately 17.2-acre site (Project Site). The 16.1-acre Property consists of one parcel in the City of Los Angeles (City) generally bounded by Bellaire Avenue to the west, Valley Spring Lane to the north, the Los Angeles River to the south, and Whitsett Avenue to the east. Los Angeles Fire Department Station 78 is located immediately south of the Project Site along Whitsett Ave. The approximately 1.1-acre Leased Property is located between the 16.1-acre Property and the Los Angeles River.

The Project also involves off-site improvements to Valleyheart Drive, located primarily to the south of Fire Station 78, and to portions of the Zev Yaroslavsky Los Angeles River Greenway (Zev Greenway), an improved public trail along the northern edge of the Los Angeles River. The Project would implement an extensive tree and landscaping program. The Project includes a stormwater capture and reuse system for water conservation and treatment purposes. The Project would also provide approximately 6 acres of publicly accessible open space and landscaped trails connecting to the adjacent Zev Greenway and on-site landscaped areas, water features, and recreational amenities.

The Project includes two athletic fields, with Field A located in proximity to Whitsett Avenue in the southeastern portion of the Project Site, and Field B, located in proximity to Valley Spring Lane and Bellaire Avenue, in the western portion of the Project Site. Field houses for maintenance and storage are proposed at each field.

The Project would include an 80,249-square-foot multi-purpose gymnasium, located in the southern portion of the Project Site, and a 52-meter swimming pool with ancillary locker, restroom, and meeting room space. The pool would be located in the north-central portion of the Project Site to the west of eight tennis courts with seating. The Project would include a below-grade parking structure located in the eastern portion of the Project Site, with approximately 503 automobile parking spaces. Access to the parking structure would be via a two-way driveway on Whitsett Avenue. A second driveway to access the parking structure would be via a drop-off and roundabout from Valleyheart Drive at the southeastern corner of the Project Site. This vehicle entrance area would also accommodate 29 surface parking spaces. The Project would also provide three security kiosks: a 180-square-foot ground-level security kiosk to the south of the tennis courts off of the north Whitsett Avenue pedestrian entrance, a 97-square-foot underground kiosk in the parking structure, and a 70-square-foot kiosk located in proximity to the roundabout and the at-grade parking.

The original, on-site Weddington Golf & Tennis clubhouse, including its café, is located on the northeastern portion of the Project Site and would remain as part of the Project and would continue to be open to the public. An existing putting green to the northeast of the clubhouse, six existing golf ball-shaped light standards and poles, and the low brick retaining wall along the northwestern edge of the Property would also remain.

Construction of the Project would be completed over an approximately 30-month period. Activities would be phased, beginning as early as 2022. Project construction would require grading and excavation activities down to a maximum depth of 21 feet for construction of the one-level subterranean parking structure, gymnasium basement, and proposed one-million-gallon stormwater capture and reuse system. Rough grading cut volumes would be approximately 251,836 cubic yards (unadjusted), and the fill volume would be approximately 1,836 cubic yards (unadjusted), for a net cut/fill volume of approximately 250,000 cubic yards (unadjusted).[1] Because cut soils would exceed fill soils, export and disposal off-site would be required.

1.2. Scope of Work

This report provides a description of the existing surface water hydrology, surface water quality, groundwater level, and groundwater quality at the Project Site. In addition, the report includes an analysis of the Project's potential impacts related to surface water hydrology, surface water quality, groundwater level, and groundwater quality.

2. ENVIRONMENTAL SETTING

2.1. Surface Water Hydrology

2.1.1. Regional

The Project Site is located within the Los Angeles River Watershed in the Los Angeles Basin. The eastern portion spans from the Santa Monica Mountains to the Simi Hills and in the west from the Santa Susana Mountains to the San Gabriel Mountains. The watershed encompasses and is shaped by the path of the Los Angeles River, which flows from its headwaters in the mountains eastward to the northern corner of Griffith Park. Here the channel turns southward through the Glendale Narrows before it flows across the coastal plain and into San Pedro Bay near Long Beach. The Los Angeles River has evolved from an uncontrolled, meandering river providing a valuable source of water for early inhabitants, to a major flood protection waterway (see Figure 9).

[1] "Unadjusted" cut and fill is a programmed estimate that does not account for minor shrinkage from compaction, swelling, or other factors that may require final manual adjustments to achieve finished gradients/ heights.

2.1.2. Local

Underground storm drain facilities in the Project vicinity (see Figure 4) consist of the following:

- **Intersection of Whitsett Avenue and Valleyheart Drive:** There are two existing catch basins located at this intersection. They both drain south discharging via underground pipes into the adjacent Los Angeles River.

- **Intersection of Bellaire Avenue and Valleyheart Drive:** There exists one catch basin here that drains south discharging via an underground pipe into the Los Angeles River.

The underground pipes and catch basins noted above are owned and maintained by the County of Los Angeles. From the Project survey information (Figure 1), the stormwater runoff from the Project Site is collected by an offsite storm drainage inlet structure and directed to the Los Angeles River.

2.1.3. Project Site

Based on the Project survey by KPFF dated December 20, 2017, and updated February 27, 2020 (see Figure 1) and site observations, it is determined that under existing conditions the Project Site is divided into five drainage areas, which are described below and shown in Figure 3. The Project Site consists of a pervious golf course, driving range, impervious tennis courts, surface parking, buildings, and impervious pavement for pedestrian and vehicular circulation.

- Area A1 consists of the golf course along Valley Spring Lane and Bellaire Avenue.

- Area A2 consists of the driving range.

- Area A3 consists of the southern portion of the golf course, including portions of the Leased Property.

- Area A4 consists of 16 tennis courts and the surrounding area, including portions of the Leased Property.

- Area A5 consists of a surface parking lot and existing building along Whitsett Avenue.

Figure 3 shows all the input parameters used for analyzing the existing Weddington Golf & Tennis facility. Table 1 summarizes the existing volumetric flow rate generated from the recorded 85^{th} percentile (Q_{85th}) and the 50-year (Q_{50}) storm events. The Hydrocalc results for the existing Project Site can be found in Figure 5 (A1-A5).

Table 1- Existing Drainage Stormwater Runoff Calculations During 85th Percentile and 50-year Storm Events				
Drainage Area	Area (Acres)	Percent Imperviousness (%)	Q_{85th} (cfs) (volumetric flow rate measured in cubic feet per second)	Q_{50} (cfs) (volumetric flow rate measured in cubic feet per second)
A1	6.26	5%	0.19	24.20
A2	3.00	5%	0.09	11.59
A3	3.20	5%	0.11	12.37
A4	3.35	95%	1.22	12.95
A5	1.39	95%	0.60	5.37
TOTAL	17.2	**30%**	**1.4**	**54.9**

The Project proposes to collect stormwater runoff within the Project Site as well as from an offsite area to the north of the Project Site. All onsite stormwater pipe networks will be sized to handle the 50-year storm event to mitigate flooding. The Project's LID treatment system will be designed to convey the 50-year storm and treat and store the 85th percentile storm onsite.

2.2. SURFACE WATER QUALITY

2.2.1. REGIONAL

As stated above, the Project Site lies within the Los Angeles River Watershed which is broken up into six separate reaches. The Project Site is located within Los Angeles River Reach 3. Pollutants of concern listed for the Los Angeles River under California's Clean Water Act Section 303(d) List include: cadmium (dissolved), lead (dissolved), chlordane, dichloroethylene, tetrachloroethylene, trichloroethylene, coliform bacteria, copper (dissolved), total aluminum, total lead, enterococcus, fecal coliform, total coliform, algae, ammonia, oil and grease, zinc (dissolved) and trash. No Total Maximum Daily Load (TMDL) data have been recorded by EPA for this waterbody.

2.2.2. LOCAL

In general, urban stormwater runoff occurs following precipitation events, with the volume of runoff flowing into the drainage system depending on the intensity and duration of the rain event. Contaminants that may be found in stormwater from developed areas include sediments, trash, bacteria, metals, nutrients, organics and pesticides. The source of contaminants includes surface areas where precipitation falls, as well as the air through which it falls. Contaminants on surfaces such as roads, maintenance areas, parking lots, and buildings, which are usually contained in dry weather conditions, may be carried by rainfall runoff into drainage systems. The City of Los Angeles typically installs catch basins with screens to capture debris before entering the storm drain system. In addition, the City conducts routine street cleaning operations, as well as periodic cleaning and maintenance of catch basins, to reduce stormwater pollution within the City.

2.2.3. PROJECT SITE

Based on the project survey by KPFF dated December 20, 2017, and updated February 27, 2020 (see Figure 1), site observations, and the fact that the existing site was developed prior to the enforcement of storm water quality Best Management Practices (BMP) design, implementation and maintenance, it appears the Project Site currently does not implement BMPs and has no significant means of treatment for stormwater runoff.

Please refer to Table 1 "Existing Drainage Stormwater Runoff Calculations During 85th Percentile and 50-year Storm Events" for the recorded 85th percentile volumetric flow rate for each drainage area within the existing Site. The 85th percentile storm is used when sizing stormwater runoff treatment structures (BMPs).

The Project proposes to collect and treat offsite surface runoff within the proposed BMP Structure onsite, as more fully described below. Directly north of the Project is approximately 38.64 acres consisting of residential single-family uses (referred to as Area B or the Off-site Drainage Area). Due to the existing, inadequate drainage of this area the Project proposes to collect and treat the 85th percentile storm volume of the 38.64 acres area. The 38.64 acres area is bounded by Moorpark Street to the north, Whitsett Avenue to the east, Bellaire Avenue to the west, and Valley Spring Lane to the south. The existing topography of the Off-site Drainage Area slopes from north to south collecting in the southeast corner of the Off-site Drainage Area at Whitsett Avenue and Valley Spring Lane. The stormwater runoff then runs south along Whitsett Avenue to the catch basin located on the west side of the street at the intersection of Whitsett and Valleyheart Drive (see Figure 4). As stated above, this stormwater is then conveyed into the Los Angeles River.

2.3. GROUNDWATER HYDROLOGY

2.3.1. REGIONAL AND LOCAL

Groundwater use for domestic water supply is a major valuable use of groundwater basins in Los Angeles County. The Project Site lies within the SFV Groundwater Basin. Generally, groundwater flows south southwesterly within the Basin and may be restricted by natural geological features. Replenishment of groundwater basins occurs mainly by percolation of precipitation throughout the region via permeable surfaces, spreading grounds, and groundwater migration from adjacent basins,

The SFV Groundwater Basin is bounded on the north and northwest by the Santa Susana Mountains, on the north and northeast by the San Gabriel Mountains, on the east by the

San Rafael Hills, on the south by the Santa Monica Mountains and Chalk Hills, and on the west by the Simi Hills. The valley is drained by the Los Angeles River and its tributaries. Precipitation in the San Fernando Valley ranges from 15 to 23 inches per year and averages about 17 inches.[2]

The groundwater in this basin is mainly unconfined with some confinement within the Saugus Formation in the western part of the basin and in the Sylmar and Eagle Rock areas. Recharge of the basin is from a variety of sources. Spreading of imported water and runoff occurs in the Pacoima, Tujunga, and Hansen Spreading Grounds. Runoff contains natural streamflow from the surrounding mountains, precipitation falling on impervious areas, reclaimed wastewater, and industrial discharges. Water flowing in surface washes infiltrates, particularly in the eastern portion of the basin.

Groundwater flows generally from the edges of the basin toward the middle of the basin, then beneath the Los Angeles River Narrows into the Central Sub-basin of the Coastal Plain of Los Angeles Basin. In the northeastern part of the basin, groundwater moves from the La Crescenta area southward beneath the surface of Verdugo Canyon toward the Los Angeles River near Glendale, whereas the groundwater in the Tujunga area flows west following the Tujunga Wash around the Verdugo Mountains to join groundwater flowing from the west following the course of the Los Angeles River near Glendale (ULARAW 1999). Flow velocity ranges from about 5 feet per year in the western part of the basin to 1,300 feet per year beneath the Los Angeles River Narrows.

2.3.2. Project Site

The discussion below is based upon a review of relevant previous investigations and on-site explorations conducted as part of the Geotechnical Engineer Investigation Proposed Academic and Athletic Development by Geotechnologies, Inc dated July 2, 2019 and revised June 19, 2020 (Attachment 1).

Soil borings were drilled to a depth between 30 and 65 feet below the ground surface during Geotechnologies' field investigation and groundwater was encountered at varying depths between 24.5 and 49.5 feet below ground surface. Historically, highest groundwater is at the ground surface.[3]

2.4. Groundwater Quality

2.4.1. Regional

The San Fernando Valley Groundwater Basin falls under the jurisdiction of the LARWQCB. According to LARWQCB's Basin Plan, objectives applying to all ground

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2 https://water.ca.gov/LegacyFiles/groundwater/bulletin118/basindescriptions/4-12.pdf; accessed May 6, 2020

3 "Historically Highest Ground Water Contours" by California Geological Survey (2005)

waters of the region include bacteria, chemical constituents and radioactivity, mineral quality, nitrogen (nitrate, nitrite), and taste and odor.[4]

2.4.2. Local

In the western part of the Basin, calcium sulfate-bicarbonate concentration is dominant, and in the eastern part of the Basin, calcium bicarbonate character dominates. Total dissolved solids (TDS) range from 326 to 615 milligrams per liter (mg/L) Data from 125 public supply wells shows an average TDS content of 499 and a range from 176 to 1,160.[5] The average TDS content meets the secondary maximum contaminant level (SMCL) of 1000 mg/L for the San Fernando Valley Groundwater Basin. [6]

Several investigations by U.S. Geological Survey group have determined contamination of volatile organic compounds (VOCs) such as trichloroethylene (TCE), perchloroethylene (PCE), petroleum compounds, chloroform, nitrate, sulfate, and heavy metals. TCE, PCE and nitrate contamination occurs in the eastern part of the basin and elevated sulfate concentration occurs in the western part of the basin.[7]

2.4.3. Project Site

Per the Project's Phase I Environmental Site Assessment (Phase I ESA), no known groundwater contamination has been reported on the Project Site from prior uses, nor has groundwater contamination from offsite areas been reported to adversely affect groundwater beneath the Project Site.[8] However, as the majority of the Project Site is pervious, there is potential for surface water-borne contaminants associated with maintenance of the golf course, such as pesticides and fertilizers, to percolate into underlying soils and groundwater.

3. SIGNIFICANCE THRESHOLDS

In accordance with Appendix G of the CEQA Guidelines, a project would have a significant impact related to hydrology and water quality if it would :

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4 Los Angeles Regional Water Quality Control Board, Basin Plan, March 2013, https://www.waterboards.ca.gov/losangeles/water_issues/programs/basin_plan/2019/chap3updatedMay2019.pdf accessed May 6, 2020.

5 B118 Basin Boundary Description – San Fernando Valley Groundwater Basin

6 USGS Status of Groundwater Quality in San Fernando-San Gabriel Study Unit, 2005: California GAMA Priority Basin Project

7 https://water.ca.gov/LegacyFiles/groundwater/bulletin118/basindescriptions/4-12.pdf accessed May 6, 2020.

8 Citadel EHS, Phase I Environmental Site Assessment Report, April 30, 2020, revised October 13, 2020. Provided in Appendix H-1 of this Draft EIR.

- 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;
- 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 offsite;
  - 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; or
- Conflict with or obstruct implementation of a water quality control plan or sustainable groundwater management plan.

3.1. SURFACE WATER HYDROLOGY

In the context of these questions from Appendix G of the CEQA Guidelines, the City of Los Angeles CEQA Thresholds Guide (*L.A. CEQA Thresholds Guide*) states that a project would normally have a significant impact on surface water hydrology if it would:

- Cause flooding during the projected 50-year developed storm event, which would have the potential to harm people or damage property or sensitive biological resources;
- Substantially reduce or increase the amount of surface water in a water body; or
- Result in a permanent, adverse change to the movement of surface water sufficient to produce a substantial change in the current or direction of water flow.

3.2. SURFACE WATER QUALITY

In the context of the above questions from Appendix G, the *L.A. CEQA Thresholds Guide* states that a project would normally have a significant impact on surface water quality if it would result in discharges that would create pollution, contamination or nuisance, as defined in Section 13050 of the California Water Code (CWC) or that cause regulatory standards to be violated, as defined in the applicable NPDES stormwater permit or Water Quality Control Plan for the receiving water body.

The CWC includes the following 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.[9]

3.3. GROUNDWATER HYDROLOGY

In the context of the above questions from Appendix G, the *L.A. CEQA Thresholds Guide* states that a project would normally have a significant impact on groundwater if it would:

- Change potable water levels sufficiently to:
    - Reduce the ability of a water utility to use the groundwater basin for public water supplies, conjunctive use purposes, storage of imported water, summer/winter peaking, or to respond to emergencies and drought;
    - Reduce yields of adjacent wells or well fields (public or private); or
    - Adversely change the rate or direction of flow of groundwater; or
- Result in demonstrable and sustained reduction of groundwater recharge capacity.

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9 City of Los Angeles.LA. CEQA Thresholds Guide. 2006 http://www.environmentla.org/programs/Thresholds/Complete%20Threshold%20Guide%202006.pdf

3.4. GROUNDWATER QUALITY

With respect to groundwater quality, and in the context of the above question from Appendix G pertaining to groundwater, the *L.A. CEQA Thresholds Guide* states that a project would normally have a significant impact on groundwater quality if it would:

- Affect the rate or change the direction of movement of existing contaminants;
- Expand the area affected by contaminants;
- Result in an increased level of groundwater contamination (including that from direct percolation, injection or saltwater intrusion); or
- Cause regulatory water quality standards at an existing production well to be violated, as defined in the California Code of Regulations (CCR), Title 22, Division 4, and Chapter 15 and in the Safe Drinking Water Act.

4. METHODOLOGY

4.1. SURFACE WATER HYDROLOGY

The Project Site is located within the City of Los Angeles, and drainage collection, treatment and conveyance are regulated by the City. Per the City's Special Order No. 007-1299, December 3, 1999, the City has adopted the Los Angeles 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. The *L.A. CEQA Thresholds Guide*, however, establishes the 50-year frequency design storm event as the threshold to analyze potential impacts on surface water hydrology as a result of development. To provide a more conservative analysis, this report analyzes the larger storm event threshold, i.e., the 50-year frequency design storm event.

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.

LACDPW has developed a time of concentration calculator, Hydrocalc, to automate time of concentration calculations as well as the peak runoff rates and volumes using the Modified Rational Method design criteria as outlined in the Hydrology Manual. The data input requirements include: sub-area size, soil type, land use, flow path length, flow path slope and rainfall isohyet. The Hydrocalc Calculator was used to calculate the storm water peak runoff flow rate for the Project conditions by evaluating an individual sub-area independent of all adjacent subareas. See Figure 4 for the Hydrocalc Calculator results for the Offsite Drainage Area and Project Site and Figure 8 for the Isohyet Map.

4.2. SURFACE WATER QUALITY

4.2.1. CONSTRUCTION

Construction BMPs will be designed and maintained as part of the implementation of the SWPPP in compliance with the Construction General Permit. The SWPPP shall begin when construction commences, before any site clearing and grubbing or demolition activity. During construction, the SWPPP will be referred to regularly and amended as changes occur throughout the construction process. As the total area of ground disturbance is less than one acre, the Project will not be required to file with the State; however, it will be required to comply with the requirements of the Construction General Permit and local regulations.

4.2.2. OPERATION

The Project must comply with the requirements of the City's LID standards.[10] Under section 3.1.3. of the LID Manual, post-construction stormwater runoff from a new development must be infiltrated, evapotranspirated, captured and used, and/or treated through high efficiency BMPs onsite for at least the volume of water produced by the greater of the 85th percentile storm or the 0.75 inch storm event. The LID Manual prioritized the selection of BMPs used to comply with stormwater mitigation requirement. The order of priority is:

1. Infiltration Systems
2. Stormwater Capture and Use
3. High Efficient Biofiltration/Bioretention Systems

[10] The Development Best Management Practices Handbook, Part B Planning Activities, 5th edition was adopted by the City of Los Angeles, Board of Public Works on July 1, 2011 to reflect Low Impact Development (LID) requirements that took effect May 12, 2012.

4. Combination of Any of the Above

Feasibility screening delineated in the LID manual is applied to determine which BMP will best suit the Project. Specifically, LID guidelines require that infiltration systems maintain at least 10 feet of clearance to the groundwater, property line, and any building structure. Per the Project's Geotechnical Report, groundwater was encountered during substructure investigation.

According to the Geotechnical investigation prepared for the project site (Attachment 1), groundwater infiltration is not feasible for the Project Site[11]. The next tier in the City of Los Angeles LID Manual, and stated above, is a Stormwater Capture and Use System. The remainder of this report analyzes the installation of a Capture and Use System in the Project Site.

4.3. GROUNDWATER

The significance of the Project as it relates to the level of the underlying groundwater table of the San Fernando Valley Groundwater Basin included a review of the following considerations:

Analysis and Description of the Project's Existing Condition

- Identification of the San Fernando Valley Basin as the underlying groundwater basin, and description of the level, quality, direction of flow, and existing uses for the water;
- Description of the location, existing uses, production capacity, quality, and other pertinent data for spreading grounds and potable water wells in the vicinity (usually within a one-mile radius), and;
- Area and degree of permeability of soils on the Project Site, and;

Analysis of the Proposed Project Impact on Groundwater Level

- 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 (usually within a one-mile radius); and
- The projected change in local or regional groundwater flow patterns.

[11] Page 46 Attachment 1.

In addition, this report discusses the impact of both existing and proposed activities at the Project Site on the groundwater quality of the underlying San Fernando Valley Groundwater Basin.

5. PROPOSED IMPACT ANALYSIS

5.1. CONSTRUCTION

5.1.1. SURFACE WATER HYDROLOGY

Construction activities for the Project would include site preparation and demolition of existing on-site facilities, with the exception of the Weddington Golf & Tennis clubhouse, café, and putting green. Excavation is expected to occur to a maximum depth of 21 feet below grade for the stormwater Capture and Use structure, the Project's development footings, and the hardscape and landscape around the Project's gymnasium and subterranean parking structures. These activities have the potential to temporarily alter existing drainage patterns on the Project Site by exposing the underlying soils, modifying flow direction, and making the Project Site temporarily more permeable. Also, exposed and stockpiled soils could be subject to erosion and conveyance into nearby storm drains during storm events. In addition, construction activities such as earth moving, maintenance/operation of construction equipment, and handling/storage/disposal of materials could contribute to pollutant loading in stormwater runoff.

As the construction site would be greater than one acre, the Project would be required to obtain coverage under the NPDES General Construction stormwater permit. In accordance with the requirements of this permit, the Project would implement a Stormwater Pollution Prevention Plan (SWPPP) that specifies BMPs and erosion control measures to be used during construction to manage runoff flows and prevent pollution. BMPs would be designed to reduce runoff and pollutant levels in runoff during construction. The NPDES and SWPPP measures are designed to (and would) contain and treat, as necessary, stormwater or construction watering for dust reduction on the Project Site so runoff does not impact off-site drainage facilities or receiving waters. Construction activities would be temporary, and flow directions and runoff volumes during construction would be controlled.

In addition, the Project would be required to comply with all applicable City grading permit regulations that require necessary measures, plans, and inspections to reduce sedimentation and erosion. Thus, through compliance with all NPDES General Construction Permit requirements, including preparation of a SWPPP, implementation of BMPs, and compliance with applicable City grading regulations, the Project would not substantially alter the Project Site drainage patterns in a manner that would result in substantial erosion, siltation, or flooding on- or off-site. Similarly, adherence to standard compliance measurements in construction activities would avoid flooding, substantially increasing or decreasing the amount of surface water flow from the Project Site into a water body, or a permanent, adverse change to the movement of surface water.

5.1.2. SURFACE WATER QUALITY

Construction activities such as earth moving, maintenance/operation of construction equipment, potential dewatering, and handling/storage/disposal of materials could contribute to pollutant loading in stormwater runoff. However, as previously discussed, the Project would be required to obtain coverage under the NPDES General Construction Permit (order No. 2009-0009-SWQ). In accordance with the requirements of the permit, the Project Applicants would prepare and implement a site-specific SWPPP adhering to the California Stormwater Quality Association (CASQA) BMP Handbook. The SWPPP would specify BMPs to be used during construction. BMPs would include but would not necessarily be limited to: erosion control, sediment control, non-stormwater management, and materials management BMPs. Refer to Exhibit 1 for typical SWPPP BMPs implemented during the construction of development projects.

With the implementation of site-specific BMPs included as part of the required SWPPP, the Project would reduce or eliminate the discharge of potential pollutants from the stormwater runoff. In addition, the Project Applicant would be required to comply with City grading permit regulations, which require implementation of necessary measures, plans (including a wet weather erosion control plan if construction occurs during the rainy season), and inspection to reduce sedimentation and erosion. Therefore, with compliance with NPDES requirements and City grading regulations, construction of the Project would not result in discharge that would cause: (1) pollution which would alter the quality of the water of the State (i.e. Los Angeles River) to a degree which unreasonably affects beneficial uses of the waters; (2) contamination of the quality of the water 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. Furthermore, construction of the Project would not result in discharges that would cause regulatory standards to be violated in the Los Angeles River.

5.1.3. GROUNDWATER HYDROLOGY

While recent geotechnical investigations on the Project Site encountered groundwater beginning at a depth of 24.5 feet below grade[12], the City requires the use of the highest historical level for design and engineering purposes. That level is at the ground surface, and, as such, dewatering initiatives should be part of construction planning and deployed if conditions warrant. Dewatering operations are practices that discharge non-stormwater, such as groundwater, that must be removed from a work location and discharged into the storm drain system to proceed with construction. Discharges from dewatering operations

[12] Page 46 Attachement 1.

can contain high levels of fine sediments, which, if not properly treated, could lead to exceedance of the NPDES requirements. If groundwater is encountered during construction, temporary pumps and filtration would be utilized in compliance with the NPDES permit. The temporary system would comply with all relevant NPDES requirements related to construction and discharges from dewatering operations. If dewatering is required, the treatment and disposal of the dewatered water would occur in accordance with the requirements of LARWQCB's Waste Discharge Requirements for Discharges of Groundwater from Construction and Project Dewatering to Surface Waters in Coastal Watersheds of Los Angeles and Ventura Counties.

5.1.4. GROUNDWATER QUALITY

As discussed above, the Project would include excavations to a maximum depth of approximately 21 feet below ground surface. The Project would also result in a net export of existing soil material. While not anticipated, any contaminated soils found would be captured within that volume of excavated material, removed from the Project Site, and remediated at an approved disposal facility in accordance with regulatory requirements and site-specific mitigation, as required.

During on-site grading and building construction, hazardous materials, such as fuels, paints, solvents, and concrete additives, could be used and would therefore require proper management and, in some cases, disposal. The management of any resultant hazardous wastes could increase the opportunity for hazardous materials releases into groundwater. Compliance with all applicable federal, state, and local requirements concerning the handling, storage and disposal of hazardous waste, such as those applicable provisions of the California Code of Regulations (CCR) Title 22, would reduce the potential for the construction of the Project to release contaminants into groundwater that could affect existing contaminants, expand the area 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 any substantial increase in groundwater contamination through hazardous materials releases and impacts on groundwater quality would be less than significant.

5.2. OPERATION

5.2.1. SURFACE WATER HYDROLOGY

The Project is expected to increase the overall percentage of impervious area from the current condition of the Project Site. Currently, the Project Site is comprised of a golf course, driving range, putting green, tennis courts, parking lot, auxiliary buildings, and club house, at 30% impervious overall. The Project will develop a gymnasium, parking garage, other recreation facilities, and paved area, creating a post-Project condition of approximately 59% impervious area overall.

A comparison of the pre and post peak flow rates indicate an insignificant increase in stormwater runoff. The post construction runoff would change with an approximate overall 0.01 percent increase. Ultimately, the Project would not cause flooding during the 50-year developed storm event and would not create runoff which would exceed the capacity of the planned drainage systems as they are sized to convey the 50-year storm event.

Table 2 below shows the proposed peak flow rates stormwater runoff calculation for the 50-year frequency design storm event (Figure 6). The table also slows the proposed increase in stormwater runoff due to the Project which is less than significant.

Table 2 – Pre and Post Drainage Stormwater Runoff Comparison – 50-year Storm Event

Pre-Project Q_{50} (cfs)	Post-Project Q_{50} (cfs)	Increase of Stormwater Runoff from Existing to Proposed Condition
60.93	60.94	0.01%

The LID requirements for the Project would outline the stormwater treatment post-construction BMPs required to control pollutants associated with storm events up to the 85th percentile storm event. The Project's proposed BMP will mitigate the stormwater runoff quality and quantity.

With the Project's BMPs in place, such as Rainwater Harvesting Cisterns, the Project would not cause flooding during the 50-year developed storm event, would not create runoff which would exceed the capacity of existing or planned drainage systems, would not substantially reduce or increase the amount of surface water in a water body, or result in a permanent adverse change to the movement of surface water.

Additionally, the Project Site is located within Zone C identified by FEMA and published in the FIRM.[13] This is an area of minimal flood hazard, usually depicted on FIRMs as above the 500-year flood level. Zone C may have ponding and local drainage problems that do not warrant a detailed study or designation as base floodplain due to the low risk.[14]

5.2.2. SURFACE WATER QUALITY

The Project would not increase concentrations of the items listed as constituents of concern for the Los Angeles River Watershed.

13 FIRMs depict the 100-year floodplain as Zone A, Zone AO, Zone AH, Zones A1-A30, Zone AE, Zone A99, Zone AR, Zone AR/AE, Zone AR/AO, Zone AR/A1-A30, Zone AR/A, Zone V, Zone VE, and Zones V1-V30. FIRMs depict the 500-year floodplain as Zone B or Zone X (shaded).

14 https://snmapmod.snco.us/fmm/document/fema-flood-zone-definitions.pdf, accessed on May 6, 2020.

The Project would meet the requirements of the City's LID standards. Under section 3.1.3. of the LID Manual, post-construction stormwater runoff from a new development must be infiltrated, evapotranspirated, captured and used, and/or treated through high efficiency BMPs onsite for at least the volume of water produced by the greater of the 85th percentile storm or the 0.75 inch storm event. The LID Manual prioritized the selection of BMPs used to comply with stormwater mitigation requirement. The order of priority is:

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

Feasibility screening delineated in the LID manual is applied to determine which BMP will best suit the Project.

Based on Project's Geotechnical Report (Attachment 1), infiltration is not feasible at the Project Site. Per the geotechnical investigation, groundwater was encountered at depths between 24.5 and 49.5 feet below grade. On-site infiltration of stormwater would acute the existing perched water condition. In addition, native alluvial soils are susceptible to liquefaction when saturated. Therefore, the Project would be subject to priority tier 2 – Stormwater Capture and Use.

Per the City of LA LID Standards, the Project would be required to capture and treat the 85^{th} percentile storm event volume. The proposed 85^{th} percentile storm event volume is approximately 40,708 cubic feet (304,517 gallons).

The Project proposes to install a 1 million-gallon Stormwater Capture and Use system within the Project Site, which will substantially exceed the City's LID standards of capturing and using the Project's 85^{th} percentile storm event volume which is stated above and equals 304,517 gallons. In addition to treating runoff from the Project Site, the Project proposes to capture and treat stormwater runoff from the 38.64-acre residential neighborhood north of the Project Site (Area B). Currently, this Off-site Drainage Area, during rainfall events and even with dry weather flows (such as residential landscape irrigation and car washing), sheet flows untreated and polluted water to an inlet that directs water into the Los Angeles River.

Under the Project's Stormwater Capture and Use system design, illustrated in Figure 4, the Project would capture, treat, and use stormwater and other urban water runoff from the onsite drainage area (A) and the Off-site Drainage Area (B).

- **Project Site**
    - Area A consists of the entire Project Site, 17.2 acres. The general drainage would enter various catch basins and area drains to be designed and located by the Civil Engineer, Architect, Landscape and Plumbing

Engineer. The captured stormwater would be routed via building and site conveyance pipes and would be connected to the LID BMP system.

- **Off-site**

    - Area B consists of the Project's Off-site Drainage Area consisting of 38.64 acres of residential single-family homes. The Off-site Drainage Area is bounded by Moorpark Street to the north, Whitsett Avenue to the east, Valley Spring Lane and the Project to the south, and Bellaire Avenue to the west. The Project proposes to install a new curb inlet at the southwestern corner of Whitsett Avenue and Valley Spring Lane to intercept the offsite runoff before it drains into the County storm drain system. From this new inlet, stormwater would be conveyed onsite and through a below grade hydrodynamic separator to clean the water of particles and contaminants, such as sediment, oil and grease, pesticides and other toxics. The water would then be conveyed and stored in an underground Capture and Use system, a cistern with a maximum capacity of one million gallons, to be used for onsite irrigation and water features. See Exhibit 4 for typical details of a rainwater harvesting cistern system.

The Project's proposed and existing mitigated volume was calculated using HydroCalc for the 85th percentile storm event (Figure 7). Table 3 below shows both the proposed Project's (A) and existing Offsite (B) mitigated volume (M_{v85th}).

Table 3 – Proposed –Drainage Conditions During 85th Percentile Storm Event			
Drainage Area	Area (acres)	Percent Imperviousness (%)	M_{V85th} (cf) (volume cubic feet/gallons)
Onsite (Proposed)			
A	17.2	59.0%	40,708/304,517
Offsite			
B	38.64	80%	118,380/885,544

Compliance with the LID requirements for the Project Site would ensure stormwater treatment with post-construction BMPs that are required to control pollutants associated with storm events up to the 85th percentile storm event, per the City's Stormwater Program. In order to meet the LID requirements of the Project's 85th percentile storm mitigated volume, an estimated total of 40,708 cubic feet (304,517 gallons) of stormwater would need to be treated within the Project site. In addition, it is estimated that the Off-site Drainage Area 85th percentile storm event volume would be 118,380 cubic feet (885,544 gallons). A portion of this expected volume will be directed to the onsite 1-million-gallon Cistern depending on available capacity during any given storm event. The cistern, consisting of a series of detention tanks, would temporarily store the captured stormwater and reuse it for Project Site irrigation.

A study has been conducted by Studio-MLA (Landscape Architect) [15] to estimate the irrigation demand of the Project. Based on that analysis, the demand is estimated to be 3.3 million gallons annually. Depending on storm frequency, a third or more of the total annual irrigation supply demand could be provided by the Project's Stormwater Capture and Use system. Please refer to Exhibit 4 for a standard rainwater harvesting (cistern) system design.

As discussed above, the Project would implement a 1-million-gallon Stormwater Capture and Use (cistern) system as a BMP for managing stormwater runoff in accordance with current LID requirements. Since it appears there are currently no existing onsite BMPs, stormwater run-off during post-Project conditions would result in improved surface water quality.

Due to the incorporation of the required LID BMP(s), operation of the Project would not result in discharges that would cause: (1) pollution which would alter the quality of the waters of the State (i.e., Los Angeles River) to a degree which unreasonably affects 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.

As is typical of most urban existing uses and proposed developments, stormwater runoff from the Project Site has the potential to introduce pollutants into the stormwater system. Anticipated and potential pollutants generated by the Project are sediment, nutrients, pesticides, metals, pathogens, and oil and grease. Release of such pollutants would be minimized by implementation of the 1-million-gallon cistern system and pretreatment filtration unit.

Furthermore, operation of the Project would not result in discharges that would cause regulatory standards to be violated. As stated above, it appears that the existing conditions on the Project Site discharge without any means of treatment. Runoff from the Project Site (A) and Off-site Drainage Area (B) will be directed to the Project's capture, treat and reuse system in compliance with LID BMP requirements to control and treat stormwater runoff to mitigate the 85th percentile storm event. The installed BMP systems will be designed with an internal bypass overflow system to prevent upstream flooding during major storm events. Implementation of LID BMPs will reduce operational impacts on surface water quality and would further improve runoff into the Los Angeles River by treating runoff

[15] Landscape Water Use Worksheet by Mia Lehrer and Associates dated February 21, 2020 – Exhibit 5

from the Off-site Drainage Area before entering the River. Therefore, the Project would not result in any substantial increase in concentration of items listed as constituents of concern for the Los Angeles River Watershed and impacts on surface water quality would be less than significant.

5.2.3. GROUNDWATER HYDROLOGY

Regarding groundwater recharge, the majority of the Project Site is pervious in the existing condition, and there is limited groundwater recharge potential. Per the Project's Preliminary Geotechnical Report infiltration is not feasible at the Project Site. As stated in the Preliminary Geotechnical Report, it is the opinion of Geotechnologies Inc. that the groundwater encountered during soil borings at varying depths between 24.5 and 49.5 feet bgs is water perched on top of the underlying clay soils and bedrock, which are relatively impervious layers. The amount of impervious area on the Project Site would increase from the existing 30 percent to 59 percent upon Project buildout. However, the Project would capture, treat and store up to 1-million gallons of stormwater at a time from the developed portions of the Project Site through the stormwater LID capture and reuse cistern system, which will then use the treated stormwater for irrigation or water features on the Project Site. Stormwater that is captured from the Off-site Drainage Area would also be conveyed to the Project's cistern system and ultimately used for irrigation or water features. It is acknowledged that during heavy or sustained rain events when the cistern storage tanks are at capacity, treated water would bypass the storage cisterns and discharge to the Los Angeles River. However, even with the Project's increase in impervious area, the amount of water percolating into the underlying soils would largely be similar to existing conditions because of the Project's capture and reuse system which would return captured and treated stormwater into the on-site soils during irrigation. Because the Project Site's underlying soils and geologic characteristics do not allow for significant groundwater recharge and because there would not be a substantial change to the amount of water that would percolate into the underlying soils compared to existing conditions, the Project would not substantially decrease groundwater supplies or interfere substantially with groundwater recharge.

Also, the Project would not include the installation or operation of water wells, or any extraction or recharge system that is in the vicinity of the coast, an area of known groundwater contamination or seawater intrusion, a municipal supply well or spreading round facility. Furthermore, the Project would not introduce activities that would impede sustainable groundwater management of the SFV Groundwater Basin. Therefore, Project operation would not substantially decrease groundwater supplies or interfere substantially with groundwater recharge such that the Project may impede sustainable groundwater management of the SFV Groundwater Basin.

5.2.4. GROUNDWATER QUALITY

The Project does not include the installation or operation of water wells, or any extraction or recharge system that is in the vicinity of the coast, an area of known groundwater contamination or seawater intrusion, a municipal supply well or spreading ground facility.

Operational activities which could affect groundwater quality include spills of hazardous materials and leaking underground storage tanks. No underground storage tanks are currently operated at the Project Site or will be operated by the Project. Source control measures per the City's LID requirements, including good housekeeping, removal of trash and maintenance of driveways and parking areas, and proper use and storage of pesticides, would also reduce surface water quality impacts and would prevent pollutants from entering the groundwater by percolation within landscaped areas or other permeable surfaces. Any on-site use of hazardous materials to be used in association with operation of the Project, such as small quantities of potentially hazardous materials in the form of cleaning solvents, painting supplies, pesticides for landscaping, and pool maintenance, as well as fuel storage associated with maintenance and/or emergency equipment, would be contained, stored, and used in accordance with manufacturers' instructions and handled in compliance with applicable standards and regulations such that no hazardous materials be exposed to or otherwise would adversely impact groundwater quality. Furthermore, the elimination of large grass areas associated with the golf course, and use of artificial turf and native plantings under the Project, would reduce levels of pesticide use and fertilizers compared to existing conditions and thereby reduce the potential for contaminates to enter into surface runoff or groundwater.

Additionally, the Project would include the installation of capture and use system as a means of treatment and disposal of the volume of water produced by the greater of the 85th percentile storm or the 0.75-inch storm event, which would allow for treatment of the on-site stormwater prior to using it for irrigation or discharge into the Los Angeles River if more than storage capacity is collected.

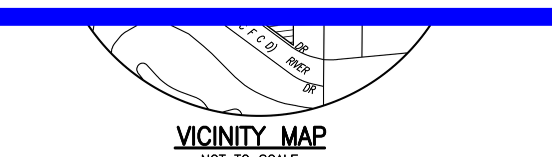
6. LEVEL OF SIGNIFICANCE

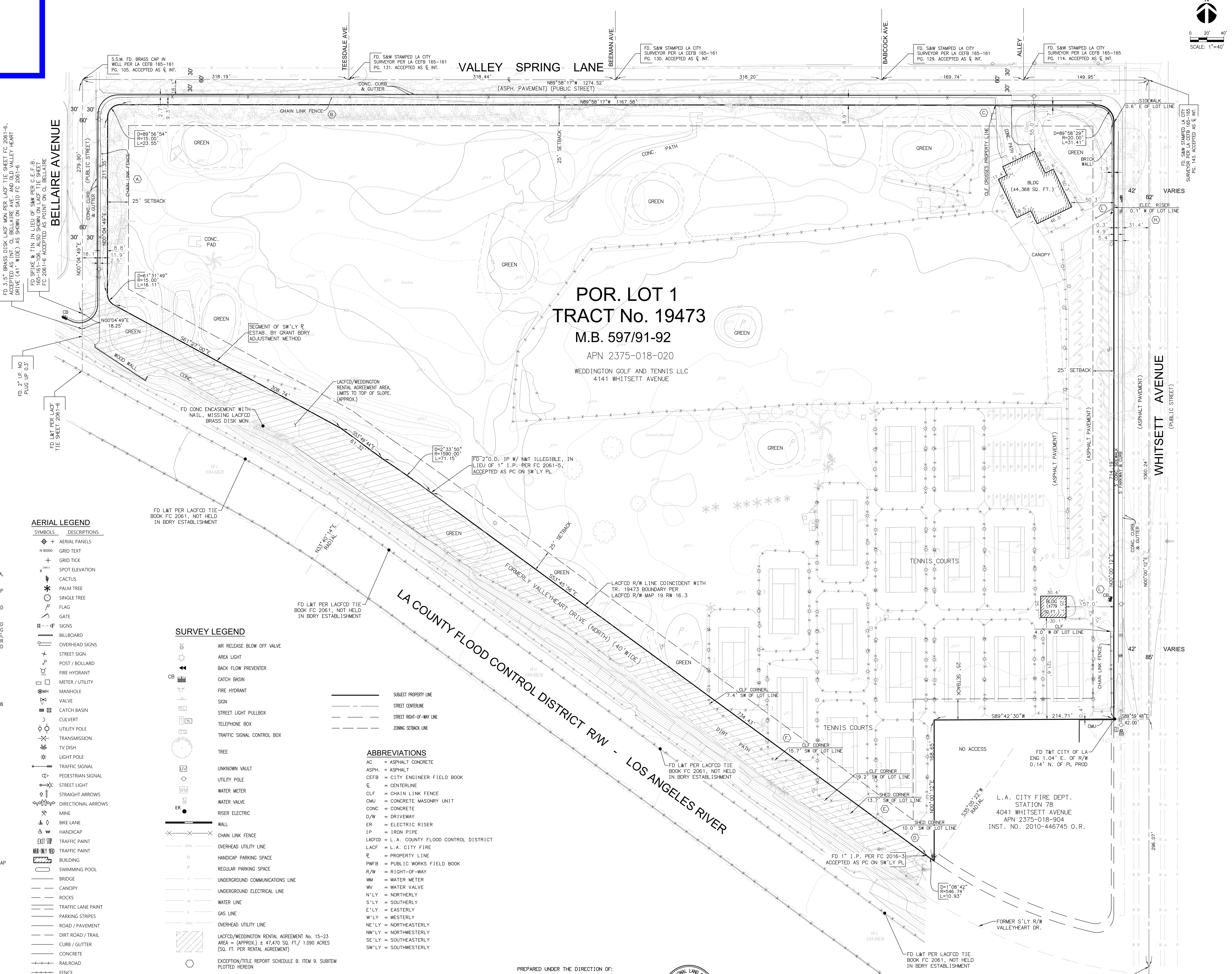
In conclusion, the Project would improve the Project Site's hydrologic function. The Project design would include implementation of a capture and use system that would exceed the City's LID requirements. Whereas stormwater from the Project Site currently sheet flows without treatment into the Los Angeles River, implementation of the Project would capture and use stormwater from the Project Site as well as a portion of the 38.64-acre neighborhood to the north of the Project Site, improving water quality and reducing the amount of water discharged from these areas. Overall, hydrology and water quality impacts would be less than significant.

APPENDICES

FIGURE 1
Project Survey

ALTA/NSPS LAND TITLE SURVEY





COMMENTS

BOUNDARY LINES . . . WERE ESTABLISHED FROM THE RECOVERED CITY, COUNTY AND/OR PRIVATE ENGINEER MONUMENTS WHOSE CHARACTER AND SOURCE ARE SO NOTED ON THE SURVEY.

SITE ADDRESS . . . . 4141 WHITSETT AVENUE, STUDIO CITY, CA 91604

APN NO. 2375-018-020

DATES OF SURVEY . . . NOVEMBER 08, 11, 21, AND DECEMBER 05 - 2017

BASIS OF BEARINGS . . THE BEARING OF N 89°58'17" W ALONG THE CENTERLINE OF VALLEY SPRING LANE AS SHOWN ON THE MAP OF TRACT NO. 19437, AS RECORDED IN MAP BOOK 597, PAGES 91 & 92, WAS TAKEN AS THE BASIS OF BEARINGS FOR THIS SURVEY.

GROSS LAND AREA . . 701,539 SQ. FT. OR 16.105 ACRES

BENCH MARK . . . . CITY OF LA BM# 08-07350, SQ SPK IN W CURB WHITSETT AVE; 60FT N OF CTR LINE PRODUCED; OF VALLEY HEART DR FROM EAST; S END OF CB

ELEV. = 618.956 FT; NAVD 1988, RECORDED 2000

FLOOD INSURANCE RATE MAP. . . . . . . ZONE "X" AREAS DETERMINED TO BE OUTSIDE THE 0.2% ANNUAL CHANCE FLOOD PER FLOOD INSURANCE RATE MAP (FIRM) MAP PANEL MAP NO. 06037C1320F EFFECTIVE DATE SEPTEMBER 26, 2008.

UTILITIES . . . . ALL VISIBLE ABOVE-GROUND UTILITY FEATURES SHOWN ON THIS MAP WERE OBTAINED BY CONVENTIONAL AND AERIAL PHOTOGRAMMETRIC MEANS. ABOVE-GROUND UTILITIES WERE COMBINED WITH CITY OF LA SUBSTRUCTURE MAPS TO PLOT UNDERGROUND UTILITY LINES SHOWN HEREON. NO REPRESENTATION IS MADE AS TO THE COMPLETENESS OF SAID UTILITY INFORMATION AND ANY USER OF THIS INFORMATION SHOULD CONTACT THE UTILITY OR GOVERNMENT AGENCY DIRECTLY.

PUBLIC ACCESS . . . PUBLIC ACCESS TO SITE FROM VALLEY SPRING LANE AND WHITSETT AVENUE.

PARKING . . . . THERE WERE APPROXIMATELY 87 VEHICULAR PARKING SPACES & 2 HANDICAPPED STALLS MARKED ON THE SITE AT TIME OF THE SURVEY.

PARKING SPACES REQUIRED: 1 PARKING SPACE PER 100 SQUARE FEET (SECTION 12.21) (APPROXIMATELY 5,331 SQUARE FEET / 100 SQUARE FEET = 53.31 PARKING SPACES)

ADA REQUIREMENTS: THE ADA REQUIREMENTS REQUIRE 4 OF THE EXISTING 89 PARKING SPACES TO BE DESIGNATED HANDICAP. THERE ARE CURRENTLY 2 EXISTING HANDICAP PARKING SPACES ON SITE. IN ORDER TO COMPLY WITH ADA REQUIREMENTS, 2 ADDITIONAL PARKING SPACES MUST BE DESIGNATED HANDICAP.

ZONING . . . . . . PER GLOBAL ZONING PRELIMINARY ZONING REPORT No. GZ3892 DATED DECEMBER 5, 2017

A1-1XL-RIO AGRICULTURE ZONE

GENERAL PLAN: OPEN SPACE
RIVER IMPROVEMENT OVERLAY

SETBACKS:
FRONT - NO LESS THAN 20% OF LOT DEPTH, NOT TO EXCEED 25 FEET.
SIDE - NO LESS THAN 10% OF LOT DEPTH, NOT TO EXCEED 25 FEET.
REAR - NO LESS THAN 25% OF LOT DEPTH, NOT TO EXCEED 25 FEET.

MAX HEIGHT: 30 FEET - 2 STORIES

MAXIMUM DENSITY: NONE

STREET RIGHT-OF-WAY LINES. . . . . . WHITSETT AVENUE RIGHT-OF-WAY PROPOSED TO BE WIDENED TO 86' PER CITY STANDARD STREET PLANS (AVENUE II). WESTERLY RIGHT-OF-WAY LINE OF SAID WHITSETT AVENUE PROPOSED TO SHIFT WEST 1.0 FOOT. WHITSETT AVENUE EXISTING STREET ½ WIDTH (CL TO FACE OF CURB) MEETS/EXCEEDS CITY STANDARD ½ WIDTH (AVENUE II, 28').

LEGAL DESCRIPTION

(PER CHICAGO TITLE INSURANCE COMPANY PRO FORMA POLICY No. **Pro Forma-CA-FBSC-IMP-72306-1-17-00077530** ORDER No. 00077530-016-NJ-DB DATED DECEMBER 8, 2017)

THE LAND REFERRED TO HEREIN BELOW IS SITUATED IN STUDIO CITY, IN THE COUNTY OF LOS ANGELES, STATE OF CALIFORNIA, AND IS DESCRIBED AS FOLLOWS:

LOT 1 OF TRACT NO. 19437, IN THE CITY OF LOS ANELES, COUNTY OF LOS ANGELES, STATE OF CALIFORNIA, AS PER MAP RECORDED IN BOOK 597 PAGES 91 AND 92 OF MAPS, IN THE OFFICE OF THE COUNTY RECORDER OF SAID COUNTY.

EXCEPT THEREFROM THAT PORTION OF SAID LAND DESCRIBED IN THAT CERTAIN FINAL ORDER OF CONDEMNATION, A CERTIFIED COPY OF WHICH WAS RECORDED APRIL 1, 2010 AS INSTRUMENT NO. 2010-446745, OF OFFICIAL RECORDS OF SAID COUNTY.

ALSO EXCEPT THEREFROM ALL MINERALS, OILS, PETROLEUM AND KINDRED SUBSTANCES AND NATURAL GAS UNDER OR IN SAID LANDS, IT BEING THE INTENT OF THIS RESERVATION TO PREVENT ALL EXPLORATION OR USE OF SAID LANDS FOR OIL OR MINING PURPOSES, AND PARTY OF THE FIRST PART AGREES THAT IT WILL NOT BORE FOR OIL, GAS, OR PETROLEUM ON SAID LANDS, OR MINE THE SAME FOR MINERALS, NOR WILL IT GRANT TO ANY OTHER PERSON OR CORPORATION, PERMISSION TO MINE OR BORE ON SAID LANDS AT ANY TIME, AS RESERVED BY TITLE INSURANCE AND TRUST COMPANY, A CORPORATION, IN DEED RECORDED IN BOOK 6119 PAGE 209, OF DEEDS.

APN: 2375-018-020

EXCEPTIONS

(PER CHICAGO TITLE INSURANCE COMPANY PRO FORMA POLICY No. **Pro Forma-CA-FBSC-IMP-72306-1-17-00077530** ORDER No. 00077530-016-NJ-DB DATED DECEMBER 8, 2017)

1. WATER RIGHTS, CLAIMS OR TITLE TO WATER, WHETHER OR NOT DISCLOSED BY THE PUBLIC RECORDS.
2. PROVISIONS IN A DEED PROHIBITING THE BUYING, SELLING OR HANDLING OF INTOXICATING LIQUORS ON SAID LAND:

   RECORDING DATE: IN BOOK 6119 PAGE 209, OF DEEDS (BLANKET, NOT PLOTTABLE)
3. (TO BE DELETED IF NOT DISCLOSED AS A TENANT ON SELLER'S OWNER'S DECLARATION AT CLOSING)
4. MATTERS CONTAINED IN THAT CERTAIN DOCUMENT

   ENTITLED: COVENANT AND AGREEMENT
   RECORDING DATE: AUGUST 17, 1966
   RECORDING NO: 2833, OF OFFICIAL RECORDS

   REFERENCE IS HEREBY MADE TO SAID DOCUMENT FOR FULL PARTICULARS. (BLANKET, NOT PLOTTABLE)
5. WAIVER OF ANY CLAIMS FOR DAMAGES TO SAID LAND BY REASON OF THE LOCATION, CONSTRUCTION, LANDSCAPING OR MAINTENANCE OF THE STREET OR HIGHWAY ADJOINING SAID LAND, AS CONTAINED IN THE DEED TO

   RECORDING DATE: NOVEMBER 13, 1973
   RECORDING NO: 2143, OF OFFICIAL RECORDS
   (BLANKET, NOT PLOTTABLE)

6-8 ITEMS DELETED

9. ANY RIGHTS, CLAIMS OR INTERESTS THAT MAY EXIST OR ARISE BY REASON OF THE FOLLOWING FACTS DISCLOSED BY A MAP DATED DECEMBER 6, 2017 OF AN ALTA/NSPS LAND TITLE SURVEY COMPLETED DECEMBER 5, 2017 PREPARED BY OR UNDER THE RESPONSIBLE CHARGE OF JASON J. STILLER LS 9271, OF/FOR KPFF, PROJECT/JOB NO. 1700870.

   A. A CHAIN LINK FENCE LIES 8.8 FEET WEST OF THE WEST LINE OF SURVEYED PROPERTY WITHIN BELLAIRE AVENUE.
   B. A CHAIN LINK FENCE LIES FROM 8.9 FEET TO 9.3 FEET NORTH OF THE NORTH LINE OF SURVEYED PROPERTY WITHIN VALLEY SPRING LANE.
   C. A CHAIN LINK FENCE CROSSES THE NORTH LINE OF SURVEYED PROPERTY.
   D. A SHED CORNER LIES 10.0 FEET SOUTHWEST OF THE SOUTHWEST LINE OF SURVEYED PROPERTY.
   E. A SHED CORNER LIES 13.7 FEET SOUTHWEST OF THE SOUTHWEST LINE OF SURVEYED PROPERTY.
   F. CHAIN LINK FENCE CORNERS ENCLOSING TENNIS COURTS ON THE SOUTHWEST SIDE OF SURVEYED PROPERTY EXTEND 19.2 FEET, 15.7 FEET, AND 7.4 FEET ONTO THE SOUTHWESTERLY ADJOINING PROPERTY.
   G. A FENCE EXTENDS ACROSS THE SOUTHWEST LINE OF SURVEYED PROPERTY.
   H. AN ELECTRIC RISER LIES 0.1 FEET WEST OF THE EAST LINE OF SURVEYED PROPERTY.
   I. THERE ARE VALVES AND A CATCH BASIN ON THE EAST SIDE OF SURVEYED PROPERTY.

   (9.A.-9.I. PLOTTED HEREON)

10. ANY RIGHTS OF THE PARTIES IN POSSESSION OF A PORTION OF, OR ALL OF, SAID LAND, WHICH RIGHTS ARE NOT DISCLOSED BY THE PUBLIC RECORDS, TO THE EXTENT SUCH PARTIES ARE SHOWN ON THE RENT ROLL ATTACHED HERETO AND MADE A PART HEREOF.

*NO PLOTTABLE EASEMENTS OF RECORD

SURVEYOR'S NOTES

NO OBSERVABLE EVIDENCE OF CEMETERY OR BURIAL GROUNDS ON SUBJECT PROPERTY.

NO OBSERVABLE EVIDENCE OF RECENT EARTH MOVING WORK OR BUILDING CONSTRUCTION.

NO OBSERVABLE EVIDENCE OF SITE USE AS A SOLID WASTE DUMP, SUMP OR SANITARY LANDFILL.

NO OBSERVABLE EVIDENCE OF STREAMS OR TRAILS ON SUBJECT PROPERTY.

THERE WERE NOT ANY DEMARCATIONS OF WETLAND AREAS ON THE SUBJECT PROPERTY AT THE TIME OF THE SURVEY.

SURVEYOR'S CERTIFICATE

TO (i) Harvard-Westlake School, a California non-profit corporation (BUYER), (ii) 4141 Whitsett LLC, a California limited liability company, and (iii) CHICAGO TITLE INSURANCE COMPANY (TITLE COMPANY):

THIS IS TO CERTIFY THAT THIS MAP OR PLAT AND THE SURVEY ON WHICH IT IS BASED WERE MADE IN ACCORDANCE WITH THE 2016 MINIMUM STANDARD DETAIL REQUIREMENTS FOR ALTA/NSPS LAND TITLE SURVEYS, JOINTLY ESTABLISHED AND ADOPTED BY ALTA AND NSPS, AND INCLUDES ITEMS 2,3,4,6(a),7(a),7(b1),8,9,11,13,14,16,17,19 AND 20 OF TABLE A THEREOF. THE FIELD WORK WAS COMPLETED ON DECEMBER 5, 2017.

PREPARED UNDER THE DIRECTION OF:

JASON J. STILLER, LS 9271
JASON.STILLER@KPFF.COM

PROFESSIONAL LAND SURVEYOR
JASON J. STILLER
No.9271
STATE OF CALIFORNIA

NO.	DATE	REVISIONS
6		
5		
4		
3		
2	12/14/17	UPDATED TITLE, SCHEDULE B, UPDATED OBS. CALLOUTS
1	12/06/17	UPDATED S'LY PL, ADDED ZONING INFO & SETBACKS

PROJECT #	1700870
DATE PREPARED	12/14/2017
DRAWN BY	DA
CHECKED BY	JS

WEDDINGTON GOLF AND TENNIS

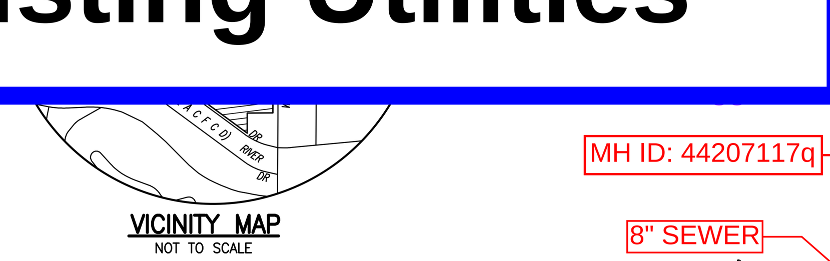
PREPARED FOR:
MR. DAVID WEIL
HARVARD WESTLAKE SCHOOL
700 NORTH FARING ROAD
LOS ANGELES, CA 90077

kpff
700 FLOWER ST., Suite 2100
Los Angeles, CA 90017
O: 213.418.0201
F: 213.266.5294
www.kpff.com

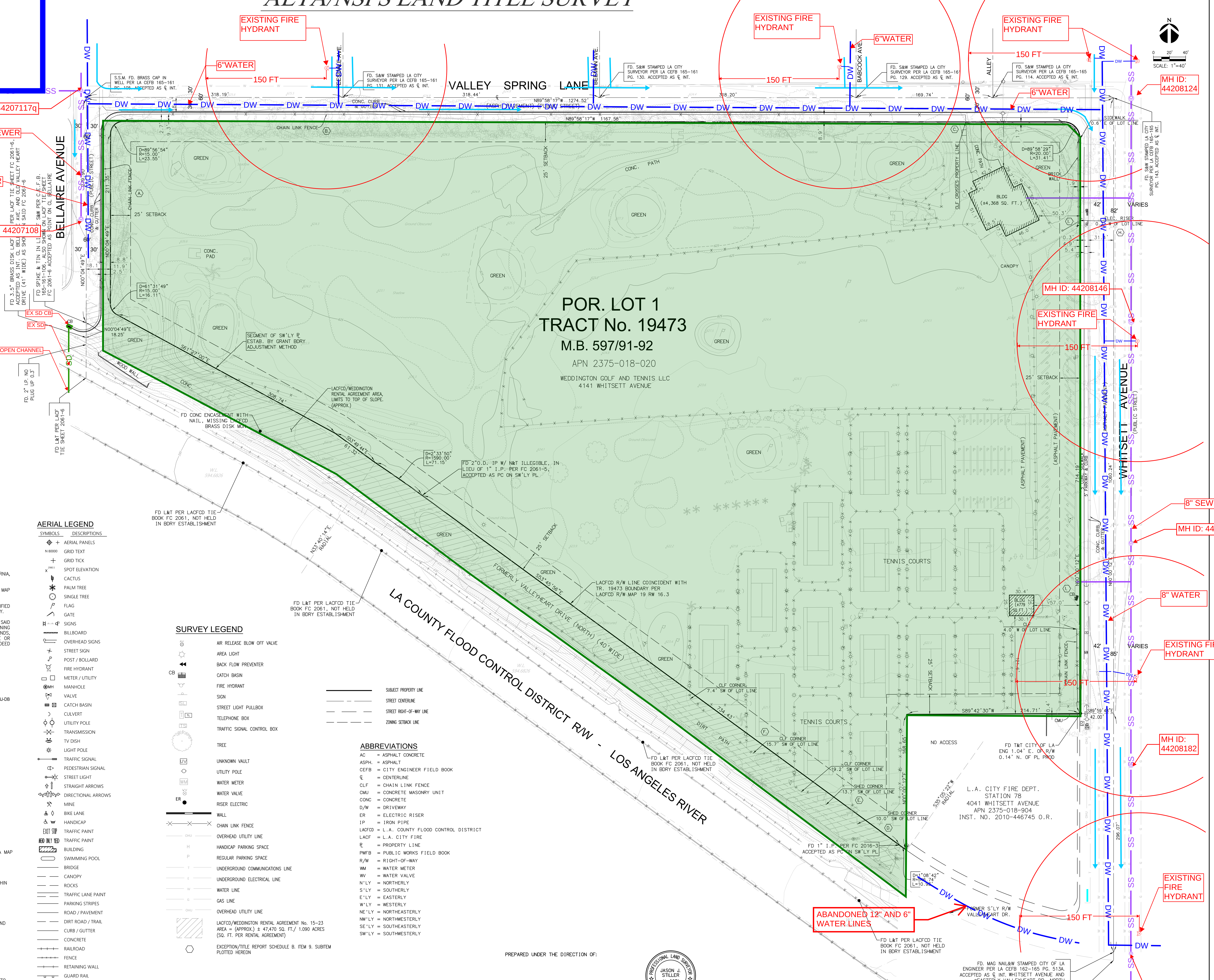
SHEET 1 OF 1

FIGURE 2
Existing Utilities

ALTA/NSPS LAND TITLE SURVEY



VICINITY MAP
NOT TO SCALE



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BENCH MARK . . . . CITY OF LA BM# 08–07350, SQ SPK IN W CURB WHITSETT AVE; 60FT N OF CTR LINE PRODUCED; OF VALLEY HEART DR FROM EAST; S END OF CB

ELEV. = 618.956 FT; NAVD 1988, RECORDED 2000

FLOOD INSURANCE RATE MAP. . . . . . . ZONE "X" AREAS DETERMINED TO BE OUTSIDE THE 0.2% ANNUAL CHANCE FLOOD PER FLOOD INSURANCE RATE MAP (FIRM) MAP PANEL MAP NO. 06037C1320F EFFECTIVE DATE SEPTEMBER 26, 2008.

UTILITIES . . . . ALL VISIBLE ABOVE–GROUND UTILITY FEATURES SHOWN ON THIS MAP WERE OBTAINED BY CONVENTIONAL AND AERIAL PHOTOGRAMMETRIC MEANS. ABOVE–GROUND UTILITIES WERE COMBINED WITH CITY OF LA SUBSTRUCTURE MAPS TO PLOT UNDERGROUND UTILITY LINES SHOWN HEREON. NO REPRESENTATION IS MADE AS TO THE COMPLETENESS OF SAID UTILITY INFORMATION AND ANY USER OF THIS INFORMATION SHOULD CONTACT THE UTILITY OR GOVERNMENT AGENCY DIRECTLY.

PUBLIC ACCESS . . . PUBLIC ACCESS TO SITE FROM VALLEY SPRING LANE AND WHITSETT AVENUE.

PARKING . . . . THERE WERE APPROXIMATELY 87 VEHICULAR PARKING SPACES & 2 HANDICAPPED STALLS MARKED ON THE SITE AT TIME OF THE SURVEY.

PARKING SPACES REQUIRED: 1 PARKING SPACE PER 100 SQUARE FEET (SECTION 12.21) (APPROXIMATELY 5,331 SQUARE FEET / 100 SQUARE FEET = 53.31 PARKING SPACES)

ADA REQUIREMENTS: THE ADA REQUIREMENTS REQUIRE 4 OF THE EXISTING 89 PARKING SPACES TO BE DESIGNATED HANDICAP. THERE ARE CURRENTLY 2 EXISTING HANDICAP PARKING SPACES ON SITE. IN ORDER TO COMPLY WITH ADA REQUIREMENTS, 2 ADDITIONAL PARKING SPACES MUST BE DESIGNATED HANDICAP.

ZONING . . . . . . PER GLOBAL ZONING PRELIMINARY ZONING REPORT No. GZ3892 DATED DECEMBER 5, 2017

A1–1XL–RIO AGRICULTURE ZONE

GENERAL PLAN: OPEN SPACE
RIVER IMPROVEMENT OVERLAY

SETBACKS:
FRONT – NO LESS THAN 20% OF LOT DEPTH, NOT TO EXCEED 25 FEET.
SIDE – NO LESS THAN 10% OF LOT DEPTH, NOT TO EXCEED 25 FEET.
REAR – NO LESS THAN 25% OF LOT DEPTH, NOT TO EXCEED 25 FEET.

MAX HEIGHT: 30 FEET – 2 STORIES

MAXIMUM DENSITY: NONE

STREET RIGHT–OF–WAY LINES. . . . . . WHITSETT AVENUE RIGHT–OF–WAY PROPOSED TO BE WIDENED TO 86' PER CITY STANDARD STREET PLANS (AVENUE II). WESTERLY RIGHT–OF–WAY LINE OF SAID WHITSETT AVENUE PROPOSED TO SHIFT WEST 1.0 FOOT. WHITSETT AVENUE EXISTING STREET ½ WIDTH (CL TO FACE OF CURB) MEETS/EXCEEDS CITY STANDARD ½ WIDTH (AVENUE II, 28').

LEGAL DESCRIPTION

(PER CHICAGO TITLE INSURANCE COMPANY PRO FORMA POLICY No. **Pro Forma-CA-FBSC-IMP-72306-1-17-00077530** ORDER No. 00077530-016-NJ-DB DATED DECEMBER 8, 2017)

THE LAND REFERRED TO HEREIN BELOW IS SITUATED IN STUDIO CITY, IN THE COUNTY OF LOS ANGELES, STATE OF CALIFORNIA, AND IS DESCRIBED AS FOLLOWS:

LOT 1 OF TRACT NO. 19437, IN THE CITY OF LOS ANELES, COUNTY OF LOS ANGELES, STATE OF CALIFORNIA, AS PER MAP RECORDED IN BOOK 597 PAGES 91 AND 92 OF MAPS, IN THE OFFICE OF THE COUNTY RECORDER OF SAID COUNTY.

EXCEPT THEREFROM THAT PORTION OF SAID LAND DESCRIBED IN THAT CERTAIN FINAL ORDER OF CONDEMNATION, A CERTIFIED COPY OF WHICH WAS RECORDED APRIL 1, 2010 AS INSTRUMENT NO. 2010–446745, OF OFFICIAL RECORDS OF SAID COUNTY.

ALSO EXCEPT THEREFROM ALL MINERALS, OILS, PETROLEUM AND KINDRED SUBSTANCES AND NATURAL GAS UNDER OR IN SAID LANDS, IT BEING THE INTENT OF THIS RESERVATION TO PREVENT ALL EXPLORATION OR USE OF SAID LANDS FOR OIL OR MINING PURPOSES, AND PARTY OF THE FIRST PART AGREES THAT IT WILL NOT BORE FOR OIL, GAS, OR PETROLEUM ON SAID LANDS, OR MINE THE SAME FOR MINERALS, NOR WILL IT GRANT TO ANY OTHER PERSON OR CORPORATION, PERMISSION TO MINE OR BORE ON SAID LANDS AT ANY TIME, AS RESERVED BY TITLE INSURANCE AND TRUST COMPANY, A CORPORATION, IN DEED RECORDED IN BOOK 6119 PAGE 209, OF DEEDS.

APN: 2375–018–020

EXCEPTIONS

(PER CHICAGO TITLE INSURANCE COMPANY PRO FORMA POLICY No. **Pro Forma-CA-FBSC-IMP-72306-1-17-00077530** ORDER No. 00077530-016-NJ-DB DATED DECEMBER 8, 2017)

1. WATER RIGHTS, CLAIMS OR TITLE TO WATER, WHETHER OR NOT DISCLOSED BY THE PUBLIC RECORDS.
2. PROVISIONS IN A DEED PROHIBITING THE BUYING, SELLING OR HANDLING OF INTOXICATING LIQUORS ON SAID LAND:

   RECORDING DATE: IN BOOK 6119 PAGE 209, OF DEEDS (BLANKET, NOT PLOTTABLE)
3. (TO BE DELETED IF NOT DISCLOSED AS A TENANT ON SELLER'S OWNER'S DECLARATION AT CLOSING)
4. MATTERS CONTAINED IN THAT CERTAIN DOCUMENT

   ENTITLED: COVENANT AND AGREEMENT
   RECORDING DATE: AUGUST 17, 1966
   RECORDING NO: 2833, OF OFFICIAL RECORDS

   REFERENCE IS HEREBY MADE TO SAID DOCUMENT FOR FULL PARTICULARS. (BLANKET, NOT PLOTTABLE)
5. WAIVER OF ANY CLAIMS FOR DAMAGES TO SAID LAND BY REASON OF THE LOCATION, CONSTRUCTION, LANDSCAPING OR MAINTENANCE OF THE STREET OR HIGHWAY ADJOINING SAID LAND, AS CONTAINED IN THE DEED TO

   RECORDING DATE: NOVEMBER 13, 1973
   RECORDING NO: 2143, OF OFFICIAL RECORDS
   (BLANKET, NOT PLOTTABLE)

6–8 ITEMS DELETED

9. ANY RIGHTS, CLAIMS OR INTERESTS THAT MAY EXIST OR ARISE BY REASON OF THE FOLLOWING FACTS DISCLOSED BY A MAP DATED DECEMBER 6, 2017 OF AN ALTA/NSPS LAND TITLE SURVEY COMPLETED DECEMBER 5, 2017 PREPARED BY OR UNDER THE RESPONSIBLE CHARGE OF JASON J. STILLER LS 9271, OF/FOR KPFF, PROJECT/JOB NO. 1700870.
   - A. A CHAIN LINK FENCE LIES 8.8 FEET WEST OF THE WEST LINE OF SURVEYED PROPERTY WITHIN BELLAIRE AVENUE.
   - B. A CHAIN LINK FENCE LIES FROM 8.9 FEET TO 9.3 FEET NORTH OF THE NORTH LINE OF SURVEYED PROPERTY WITHIN VALLEY SPRING LANE.
   - C. A CHAIN LINK FENCE CROSSES THE NORTH LINE OF SURVEYED PROPERTY.
   - D. A SHED CORNER LIES 10.0 FEET SOUTHWEST OF THE SOUTHWEST LINE OF SURVEYED PROPERTY.
   - E. A SHED CORNER LIES 13.7 FEET SOUTHWEST OF THE SOUTHWEST LINE OF SURVEYED PROPERTY.
   - F. CHAIN LINK FENCE CORNERS ENCLOSING TENNIS COURTS ON THE SOUTHWEST SIDE OF SURVEYED PROPERTY EXTEND 19.2 FEET, 15.7 FEET, AND 7.4 FEET ONTO THE SOUTHWESTERLY ADJOINING PROPERTY.
   - G. A FENCE EXTENDS ACROSS THE SOUTHWEST LINE OF SURVEYED PROPERTY.
   - H. AN ELECTRIC RISER LIES 0.1 FEET WEST OF THE EAST LINE OF SURVEYED PROPERTY.
   - I. THERE ARE VALVES AND A CATCH BASIN ON THE EAST SIDE OF SURVEYED PROPERTY.

   (9.A.–9.I. PLOTTED HEREON)
10. ANY RIGHTS OF THE PARTIES IN POSSESSION OF A PORTION OF, OR ALL OF, SAID LAND, WHICH RIGHTS ARE NOT DISCLOSED BY THE PUBLIC RECORDS, TO THE EXTENT SUCH PARTIES ARE SHOWN ON THE RENT ROLL ATTACHED HERETO AND MADE A PART HEREOF.

*NO PLOTTABLE EASEMENTS OF RECORD

SURVEYOR'S NOTES

NO OBSERVABLE EVIDENCE OF CEMETERY OR BURIAL GROUNDS ON SUBJECT PROPERTY.

NO OBSERVABLE EVIDENCE OF RECENT EARTH MOVING WORK OR BUILDING CONSTRUCTION.

NO OBSERVABLE EVIDENCE OF SITE USE AS A SOLID WASTE DUMP, SUMP OR SANITARY LANDFILL.

NO OBSERVABLE EVIDENCE OF STREAMS OR TRAILS ON SUBJECT PROPERTY.

THERE WERE NOT ANY DEMARCATIONS OF WETLAND AREAS ON THE SUBJECT PROPERTY AT THE TIME OF THE SURVEY.

ABBREVIATIONS

AC = ASPHALT CONCRETE
ASPH. = ASPHALT
CEFB = CITY ENGINEER FIELD BOOK
℄ = CENTERLINE
CLF = CHAIN LINK FENCE
CMU = CONCRETE MASONRY UNIT
CONC = CONCRETE
D/W = DRIVEWAY
ER = ELECTRIC RISER
IP = IRON PIPE
LACFCD = L.A. COUNTY FLOOD CONTROL DISTRICT
LACF = L.A. CITY FIRE
PL = PROPERTY LINE
PWFB = PUBLIC WORKS FIELD BOOK
R/W = RIGHT–OF–WAY
WM = WATER METER
WV = WATER VALVE
N'LY = NORTHERLY
S'LY = SOUTHERLY
E'LY = EASTERLY
W'LY = WESTERLY
NE'LY = NORTHEASTERLY
NW'LY = NORTHWESTERLY
SE'LY = SOUTHEASTERLY
SW'LY = SOUTHWESTERLY

SURVEYOR'S CERTIFICATE

TO (i) Harvard–Westlake School, a California non–profit corporation (BUYER), (ii) 4141 Whitsett LLC, a California limited liability company, and (iii) CHICAGO TITLE INSURANCE COMPANY (TITLE COMPANY):

THIS IS TO CERTIFY THAT THIS MAP OR PLAT AND THE SURVEY ON WHICH IT IS BASED WERE MADE IN ACCORDANCE WITH THE 2016 MINIMUM STANDARD DETAIL REQUIREMENTS FOR ALTA/NSPS LAND TITLE SURVEYS, JOINTLY ESTABLISHED AND ADOPTED BY ALTA AND NSPS, AND INCLUDES ITEMS 2,3,4,6(a),7(a),7(b1),8,9,11,13,14,16,17,19 AND 20 OF TABLE A THEREOF. THE FIELD WORK WAS COMPLETED ON DECEMBER 5, 2017.

PREPARED UNDER THE DIRECTION OF:

PROFESSIONAL LAND SURVEYOR JASON J. STILLER No.9271 STATE OF CALIFORNIA

JASON J. STILLER, LS 9271
JASON.STILLER@KPFF.COM

NO.	DATE	REVISIONS
6		
5		
4		
3		
2	12/14/17	UPDATED TITLE, SCHEDULE B, UPDATED OBS. CALLOUTS
1	12/06/17	UPDATED S'LY PL, ADDED ZONING INFO & SETBACKS

PROJECT #	1700870
DATE PREPARED	12/14/2017
DRAWN BY	DA
CHECKED BY	JS

WEDDINGTON GOLF AND TENNIS

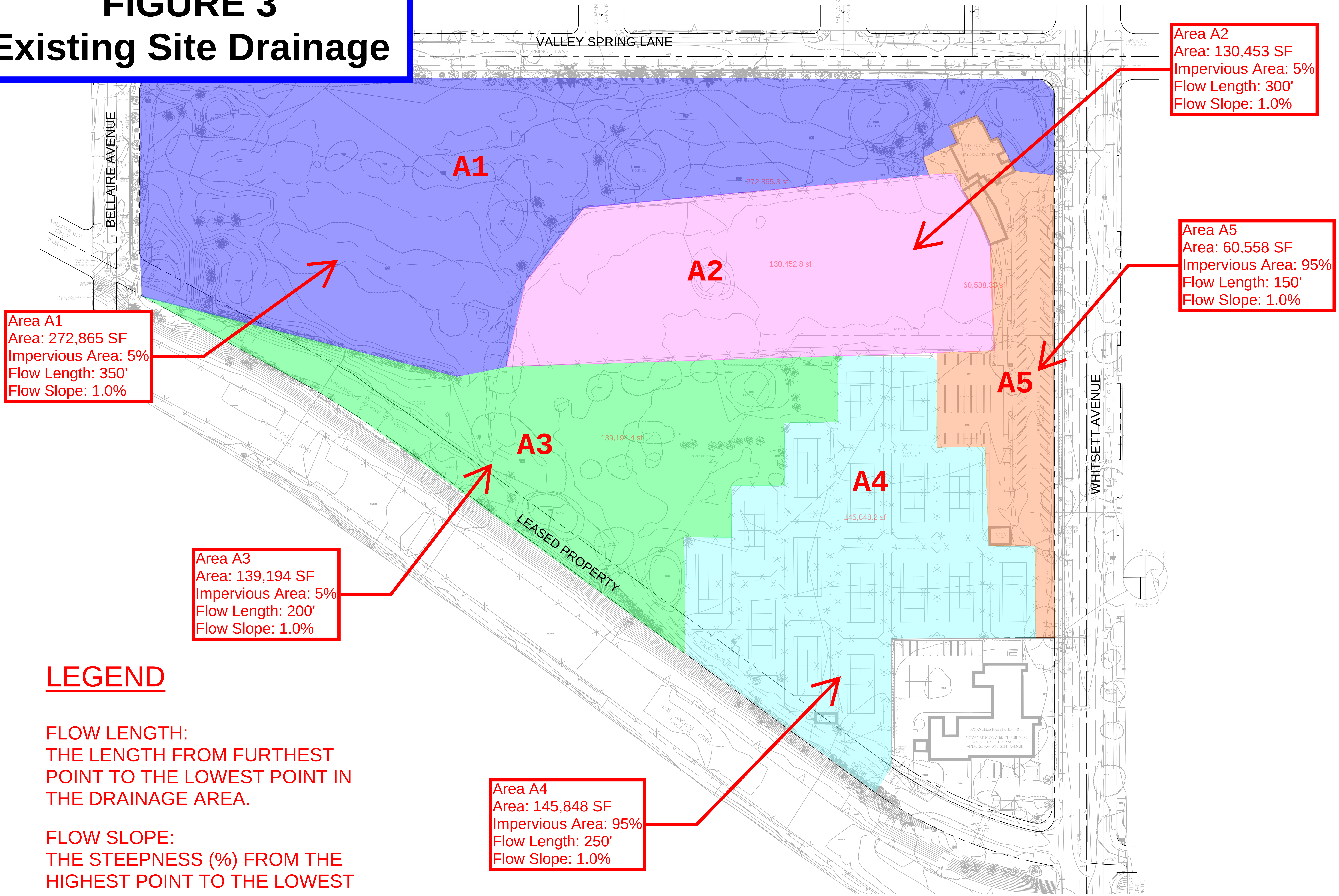
PREPARED FOR:

MR. DAVID WEIL
HARVARD WESTLAKE SCHOOL
700 NORTH FARING ROAD
LOS ANGELES, CA 90077

kpff
700 FLOWER ST., Suite 2100
Los Angeles, CA 90017
O: 213.418.0201
F: 213.266.5294
www.kpff.com

SHEET 1 OF 1

FIGURE 3
Existing Site Drainage



LEGEND

FLOW LENGTH:
THE LENGTH FROM FURTHEST POINT TO THE LOWEST POINT IN THE DRAINAGE AREA.

FLOW SLOPE:
THE STEEPNESS (%) FROM THE HIGHEST POINT TO THE LOWEST POINT OF THE DRAINAGE AREA.

HARVARD WESTLAKE RIVER PARK
4141 Whitsett Ave
Studio City, CA 91604

HARVARD WESTLAKE SCHOOL

Gensler

STUDIO-MLA

kpff

NOT FOR CONSTRUCTION

Project Name
HARVARD-WESTLAKE WEDDINGTON COMPLEX

Project Number

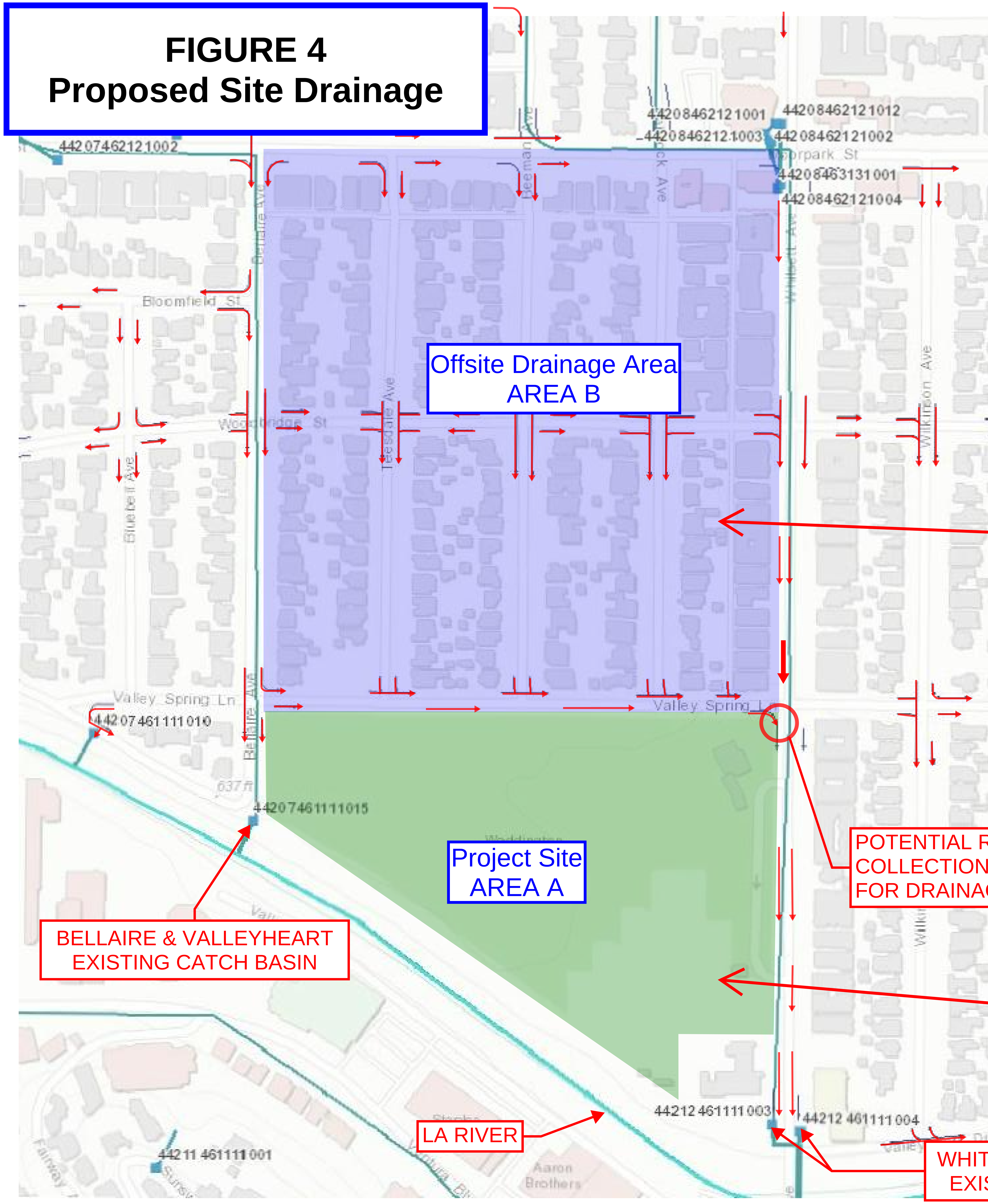
Description
EXHIBIT

Scale
PER PLAN

SCALE: 1"=60'

C102

FIGURE 4
Proposed Site Drainage



LEGEND

→	GUTTER FLOW PATH
—	STORM DRAIN PIPE TO REMAIN

N

0 50' 100'

SCALE: 1"=100'

kpff

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Suite 2100
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F: 213.266.5294
www.kpff.com

CONSULTANTS

STAMP

REVISIONS

DATE	△	ISSUED FOR

DATE	11.16.2018
PROJECT NUMBER	1800807
DESIGNED BY	IB
DRAWN BY	MN
CHECKED BY	RD
SCALE	AS SPECIFIED

KEY MAP

PROJECT DESCRIPTION

HARVARD-WESTLAKE
RIVER PARK

4141 WHITSETT AVE
STUDIO CITY, CA 91604

DRAWING TITLE

PROPOSED SITE
DRAINAGE

SHEET NUMBER (EXHIBIT NUMBER)

EX-1

P:\2018\1800807 Harvard-Westlake Weddington Complex\DWG\SHEET\EXHIBIT\1800807-C1.01EXBT.dwg, Oct 08, 2021 - 8:18am

FIGURE 5

Peak Flow Hydrology Analysis, 85th and 50yr Storm Events (Pre-Project)

FIGURE 5 - A1 85th
Peak Flow Hydrology Analysis
(Pre-Project)

Peak Flow

File location: P:/2018/1800807 Harvard-Westlake Weddington Complex/ENGR/WATER/Reports/CEQA Hydrology Report/Appendices/EJ HYDROCALCS
Version: HydroCalc 1.0.3

Input Parameters

Project Name	1800809
Subarea ID	A1
Area (ac)	6.26
Flow Path Length (ft)	350.0
Flow Path Slope (vft/hft)	0.01
85th Percentile Rainfall Depth (in)	1.15
Percent Impervious	0.05
Soil Type	16
Design Storm Frequency	85th percentile storm
Fire Factor	0
LID	True

Output Results

Modeled (85th percentile storm) Rainfall Depth (in)	1.15
Peak Intensity (in/hr)	0.2117
Undeveloped Runoff Coefficient (Cu)	0.1
Developed Runoff Coefficient (Cd)	0.14
Time of Concentration (min)	61.0
Clear Peak Flow Rate (cfs)	0.1856
Burned Peak Flow Rate (cfs)	0.1856
24-Hr Clear Runoff Volume (ac-ft)	0.0833
24-Hr Clear Runoff Volume (cu-ft)	3628.4718

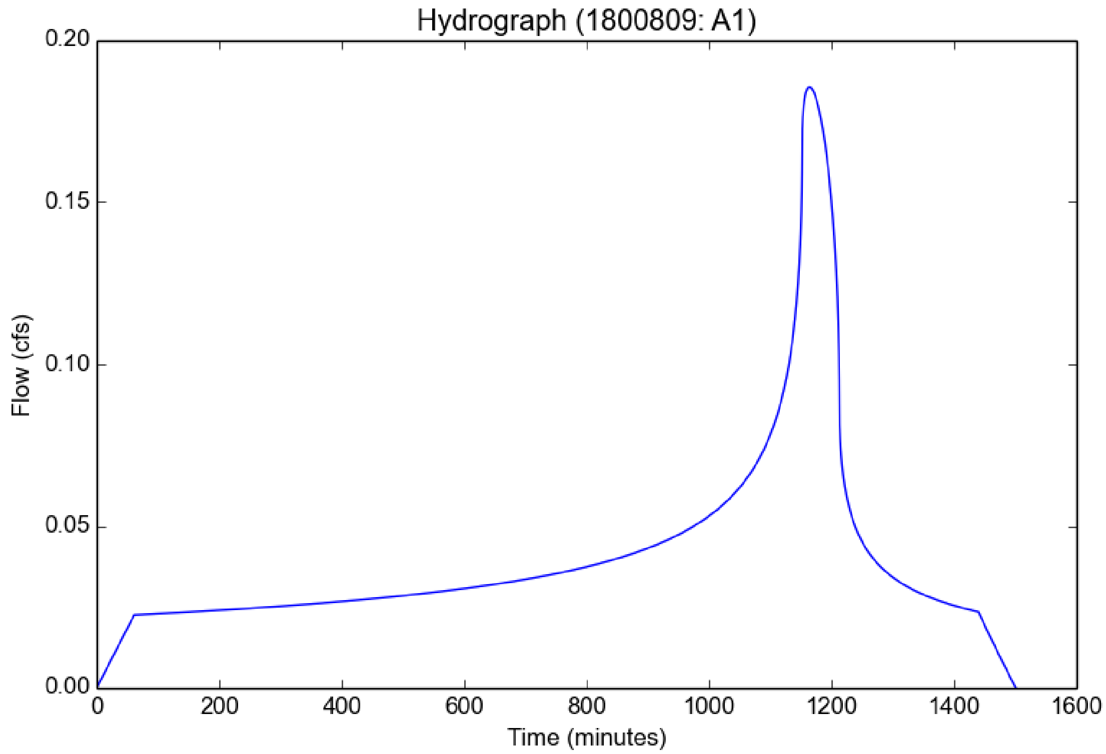


FIGURE 5 - A1 50yr
Peak Flow Hydrology Analysis
(Pre-Project)

Peak Flow

File location: P:/2018/1800807 Harvard-Westlake Weddington Complex/ENGR/EIR/Technical Reports/Hydrology and Water Quality/Appendices/HydroCa
Version: HydroCalc 1.0.3

Input Parameters

Project Name	Harvard Westlake
Subarea ID	Subarea 1A
Area (ac)	6.26
Flow Path Length (ft)	350.0
Flow Path Slope (vft/hft)	0.01
50-yr Rainfall Depth (in)	7.2
Percent Impervious	0.05
Soil Type	16
Design Storm Frequency	50-yr
Fire Factor	0
LID	False

Output Results

Modeled (50-yr) Rainfall Depth (in)	7.2
Peak Intensity (in/hr)	4.2957
Undeveloped Runoff Coefficient (Cu)	0.9
Developed Runoff Coefficient (Cd)	0.9
Time of Concentration (min)	5.0
Clear Peak Flow Rate (cfs)	24.2021
Burned Peak Flow Rate (cfs)	24.2021
24-Hr Clear Runoff Volume (ac-ft)	0.9528
24-Hr Clear Runoff Volume (cu-ft)	41504.0839

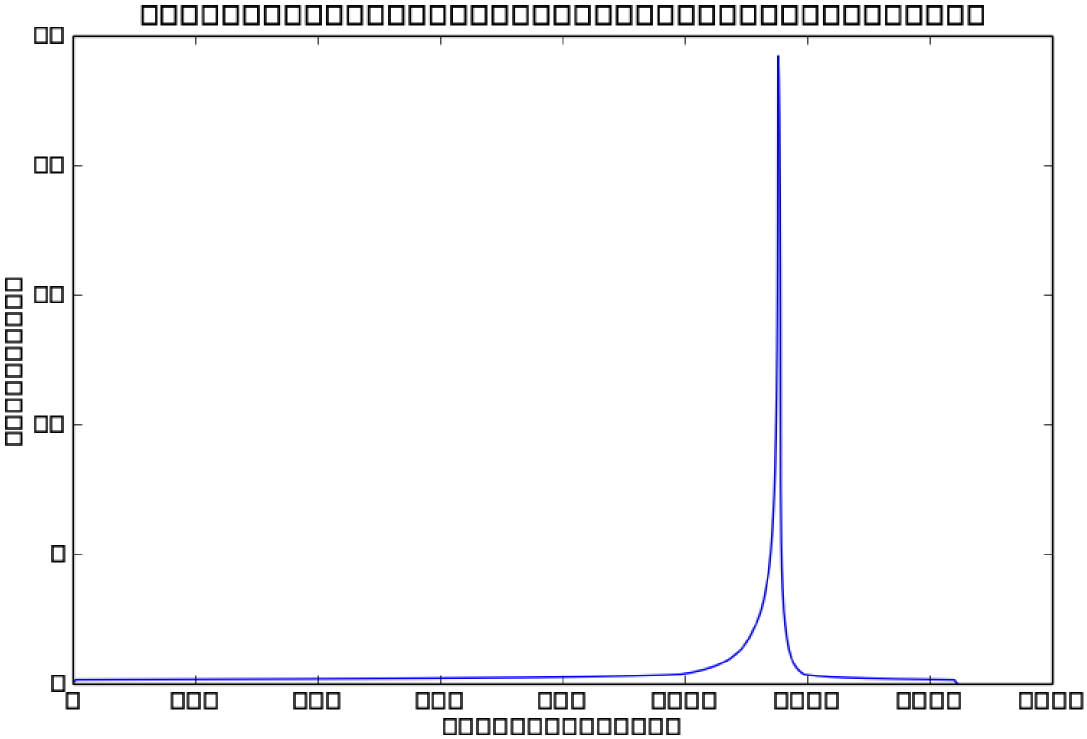


FIGURE 5 - A2 85th Peak Flow Hydrology Analysis (Pre-Project)

Peak Flow

File location: P:/2018/1800807 Harvard-Westlake Weddington Complex/ENGR/WATER/Reports/CEQA Hydrology Report/Appendices/EJ HYDROCALCS
Version: HydroCalc 1.0.3

Input Parameters

Project Name	1800809
Subarea ID	A2
Area (ac)	3.0
Flow Path Length (ft)	300.0
Flow Path Slope (vft/hft)	0.01
85th Percentile Rainfall Depth (in)	1.15
Percent Impervious	0.05
Soil Type	16
Design Storm Frequency	85th percentile storm
Fire Factor	0
LID	True

Output Results

Modeled (85th percentile storm) Rainfall Depth (in)	1.15
Peak Intensity (in/hr)	0.2223
Undeveloped Runoff Coefficient (Cu)	0.1
Developed Runoff Coefficient (Cd)	0.14
Time of Concentration (min)	55.0
Clear Peak Flow Rate (cfs)	0.0934
Burned Peak Flow Rate (cfs)	0.0934
24-Hr Clear Runoff Volume (ac-ft)	0.0399
24-Hr Clear Runoff Volume (cu-ft)	1738.8682

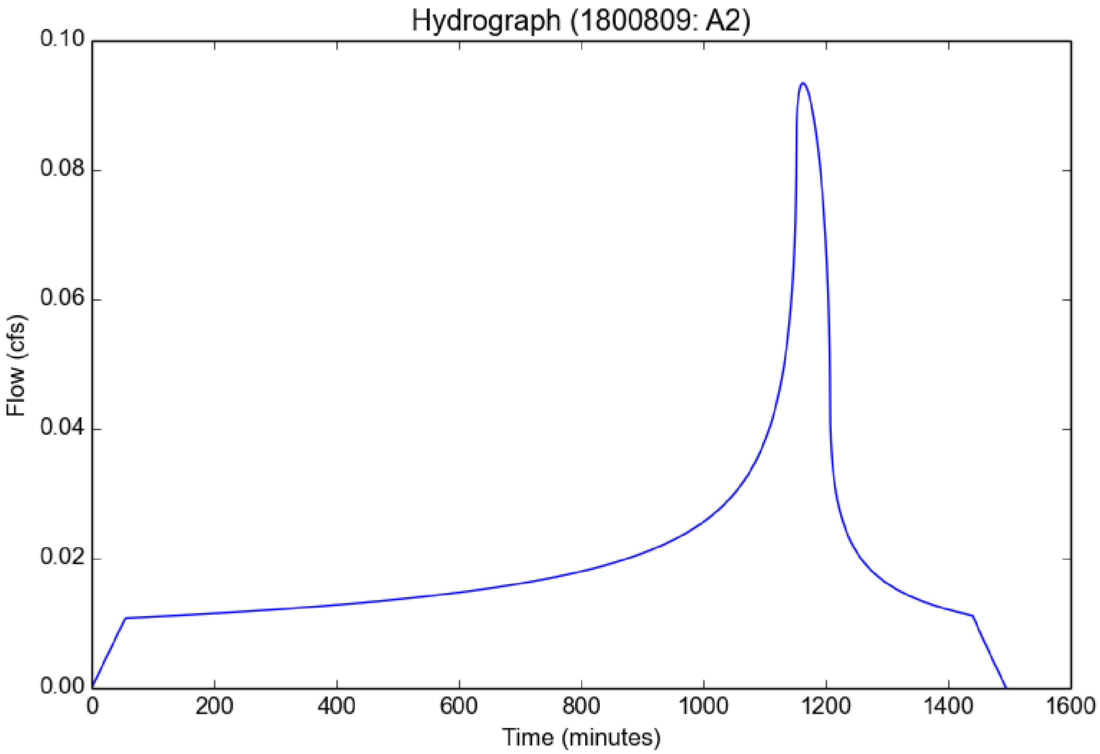


FIGURE 5 - A2 50yr
Peak Flow Hydrology Analysis
(Pre-Project)

Peak Flow

File location: P:/2018/1800807 Harvard-Westlake Weddington Complex/ENGR/EIR/Technical Reports/Hydrology and Water Quality/Appendices/HydroCa
Version: HydroCalc 1.0.3

Input Parameters

Project Name	Harvard Westlake
Subarea ID	A2
Area (ac)	3.0
Flow Path Length (ft)	300.0
Flow Path Slope (vft/hft)	0.01
50-yr Rainfall Depth (in)	7.2
Percent Impervious	0.05
Soil Type	16
Design Storm Frequency	50-yr
Fire Factor	0
LID	False

Output Results

Modeled (50-yr) Rainfall Depth (in)	7.2
Peak Intensity (in/hr)	4.2957
Undeveloped Runoff Coefficient (Cu)	0.9
Developed Runoff Coefficient (Cd)	0.9
Time of Concentration (min)	5.0
Clear Peak Flow Rate (cfs)	11.5984
Burned Peak Flow Rate (cfs)	11.5984
24-Hr Clear Runoff Volume (ac-ft)	0.4566
24-Hr Clear Runoff Volume (cu-ft)	19890.1361

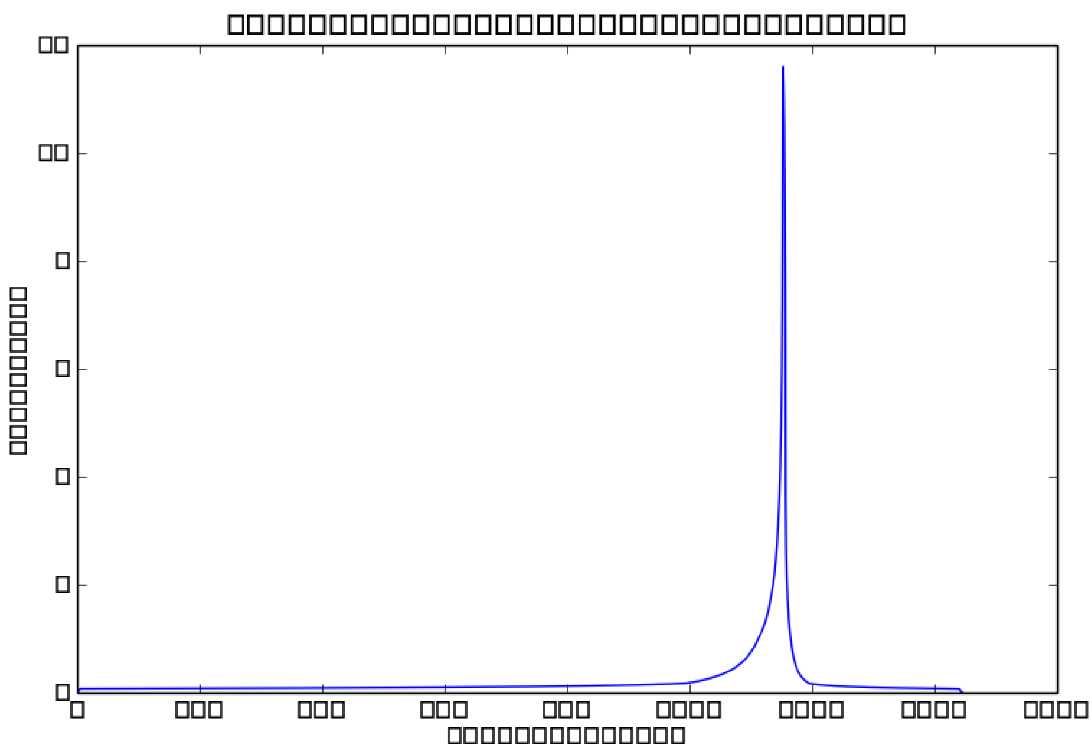


FIGURE 5 - A3 85th
Peak Flow Hydrology Analysis
(Pre-Project)

Peak Flow

File location: P:/2018/1800807 Harvard-Westlake Weddington Complex/ENGR/WATER/Reports/CEQA Hydrology Report/Appendices/EJ HYDROCALCS
Version: HydroCalc 1.0.3

Input Parameters

Project Name	1800809
Subarea ID	A3
Area (ac)	3.2
Flow Path Length (ft)	200.0
Flow Path Slope (vft/hft)	0.01
85th Percentile Rainfall Depth (in)	1.15
Percent Impervious	0.05
Soil Type	16
Design Storm Frequency	85th percentile storm
Fire Factor	0
LID	True

Output Results

Modeled (85th percentile storm) Rainfall Depth (in)	1.15
Peak Intensity (in/hr)	0.2523
Undeveloped Runoff Coefficient (Cu)	0.1
Developed Runoff Coefficient (Cd)	0.14
Time of Concentration (min)	42.0
Clear Peak Flow Rate (cfs)	0.113
Burned Peak Flow Rate (cfs)	0.113
24-Hr Clear Runoff Volume (ac-ft)	0.0426
24-Hr Clear Runoff Volume (cu-ft)	1854.7621

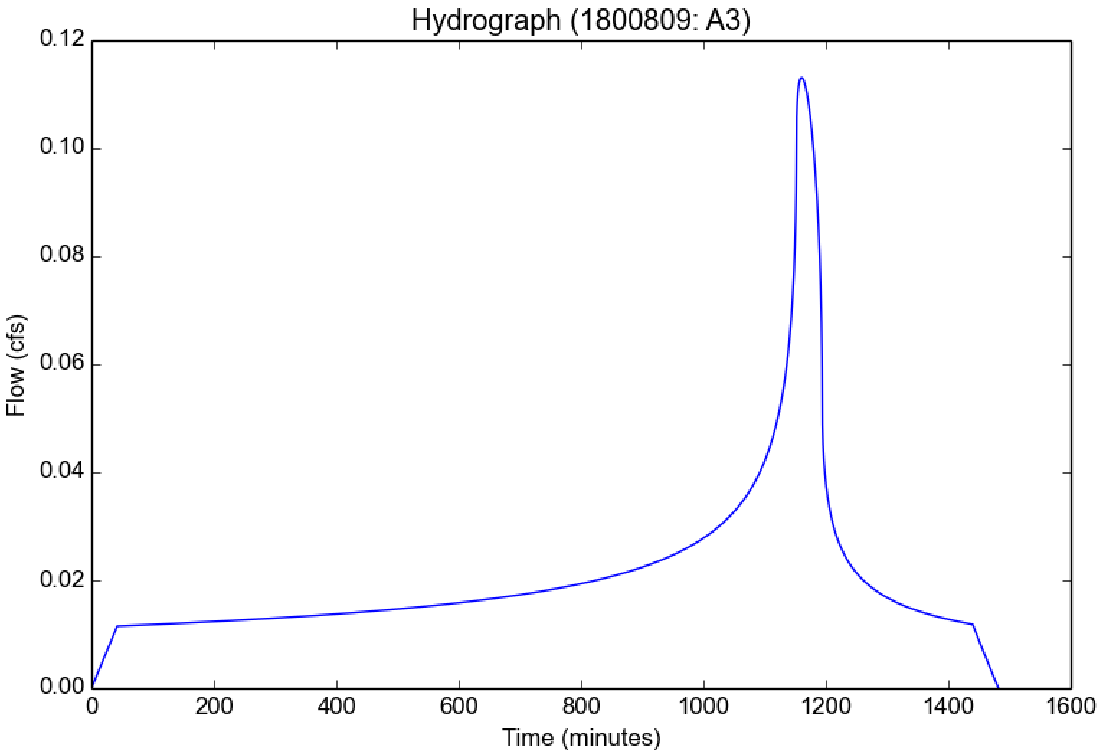


FIGURE 5 - A3 50yr
Peak Flow Hydrology Analysis
(Pre-Project)

Peak Flow I

File location: P:/2018/1800807 Harvard-Westlake Weddington Complex/ENGR/EIR/Technical Reports/Hydrology and Water Quality/Appendices/HydroCa
Version: HydroCalc 1.0.3

Input Parameters

Project Name	Harvard Westlake
Subarea ID	A3
Area (ac)	3.2
Flow Path Length (ft)	200.0
Flow Path Slope (vft/hft)	0.01
50-yr Rainfall Depth (in)	7.2
Percent Impervious	0.05
Soil Type	16
Design Storm Frequency	50-yr
Fire Factor	0
LID	False

Output Results

Modeled (50-yr) Rainfall Depth (in)	7.2
Peak Intensity (in/hr)	4.2957
Undeveloped Runoff Coefficient (Cu)	0.9
Developed Runoff Coefficient (Cd)	0.9
Time of Concentration (min)	5.0
Clear Peak Flow Rate (cfs)	12.3717
Burned Peak Flow Rate (cfs)	12.3717
24-Hr Clear Runoff Volume (ac-ft)	0.4871
24-Hr Clear Runoff Volume (cu-ft)	21216.1451

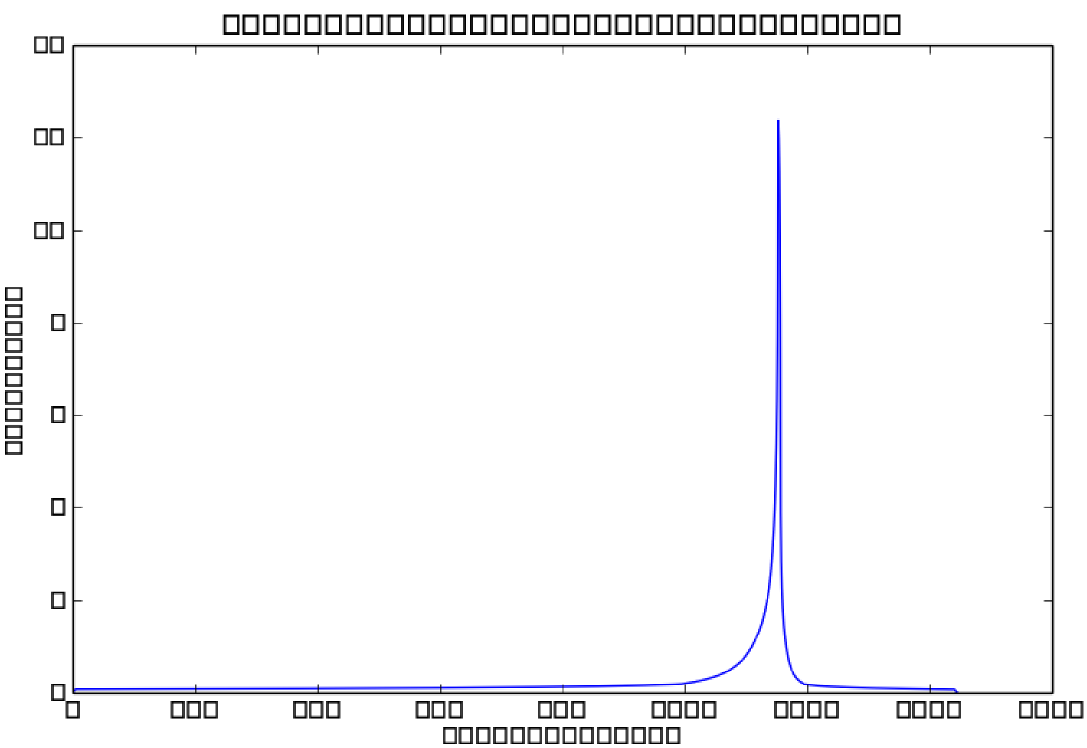


FIGURE 5 - A4 85th
Peak Flow Hydrology Analysis
(Pre-Project)

Peak Flow

File location: P:/2018/1800807 Harvard-Westlake Weddington Complex/ENGR/EIR/Technical Reports/Hydrology and Water Quality/Appendices/HWRP -
Version: HydroCalc 1.0.3

Input Parameters

Project Name	HWRP
Subarea ID	A4
Area (ac)	3.35
Flow Path Length (ft)	250.0
Flow Path Slope (vft/hft)	0.01
85th Percentile Rainfall Depth (in)	1.15
Percent Impervious	0.95
Soil Type	16
Design Storm Frequency	85th percentile storm
Fire Factor	0
LID	True

Output Results

Modeled (85th percentile storm) Rainfall Depth (in)	1.15
Peak Intensity (in/hr)	0.4229
Undeveloped Runoff Coefficient (Cu)	0.1208
Developed Runoff Coefficient (Cd)	0.861
Time of Concentration (min)	14.0
Clear Peak Flow Rate (cfs)	1.2198
Burned Peak Flow Rate (cfs)	1.2198
24-Hr Clear Runoff Volume (ac-ft)	0.2738
24-Hr Clear Runoff Volume (cu-ft)	11927.7834

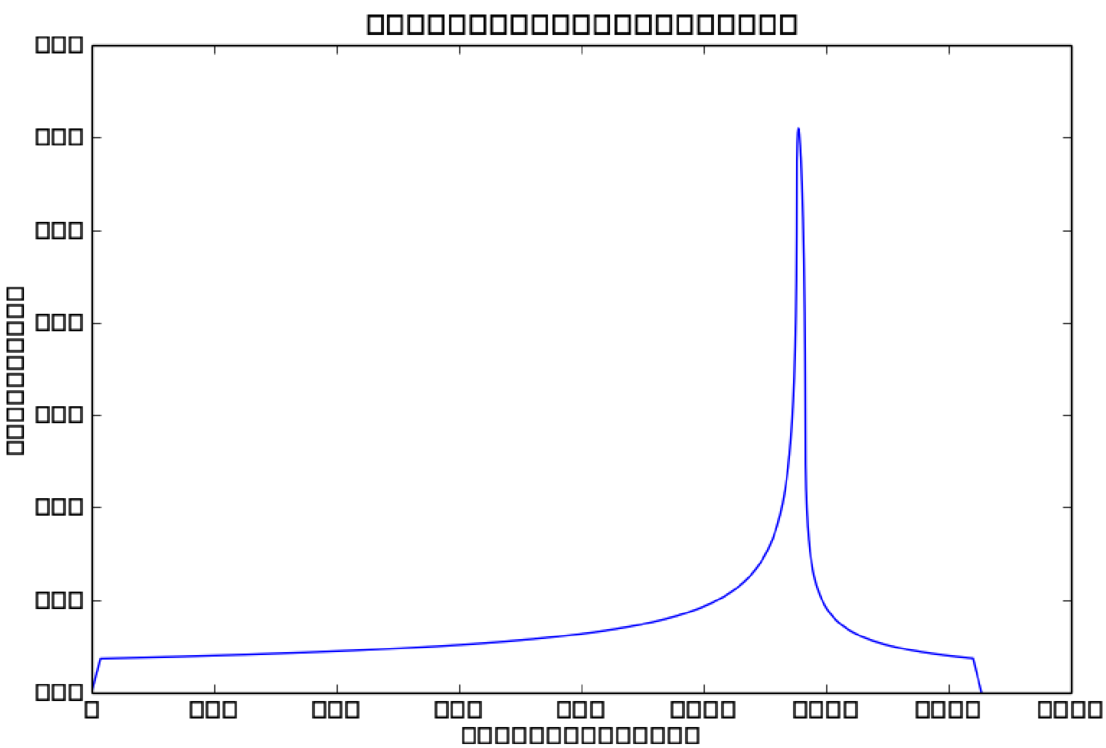


FIGURE 5 - A4 85th
Peak Flow Hydrology Analysis
(Pre-Project)

Peak Flow

File location: P:/2018/1800807 Harvard-Westlake Weddington Complex/ENGR/WATER/Reports/CEQA Hydrology Report/Appendices/EJ HYDROCALCS
Version: HydroCalc 1.0.3

Input Parameters

Project Name	1800809
Subarea ID	A4
Area (ac)	3.35
Flow Path Length (ft)	250.0
Flow Path Slope (vft/hft)	0.01
85th Percentile Rainfall Depth (in)	1.15
Percent Impervious	0.95
Soil Type	16
Design Storm Frequency	85th percentile storm
Fire Factor	0
LID	True

Output Results

Modeled (85th percentile storm) Rainfall Depth (in)	1.15
Peak Intensity (in/hr)	0.4229
Undeveloped Runoff Coefficient (Cu)	0.1208
Developed Runoff Coefficient (Cd)	0.861
Time of Concentration (min)	14.0
Clear Peak Flow Rate (cfs)	1.2198
Burned Peak Flow Rate (cfs)	1.2198
24-Hr Clear Runoff Volume (ac-ft)	0.2738
24-Hr Clear Runoff Volume (cu-ft)	11927.7834

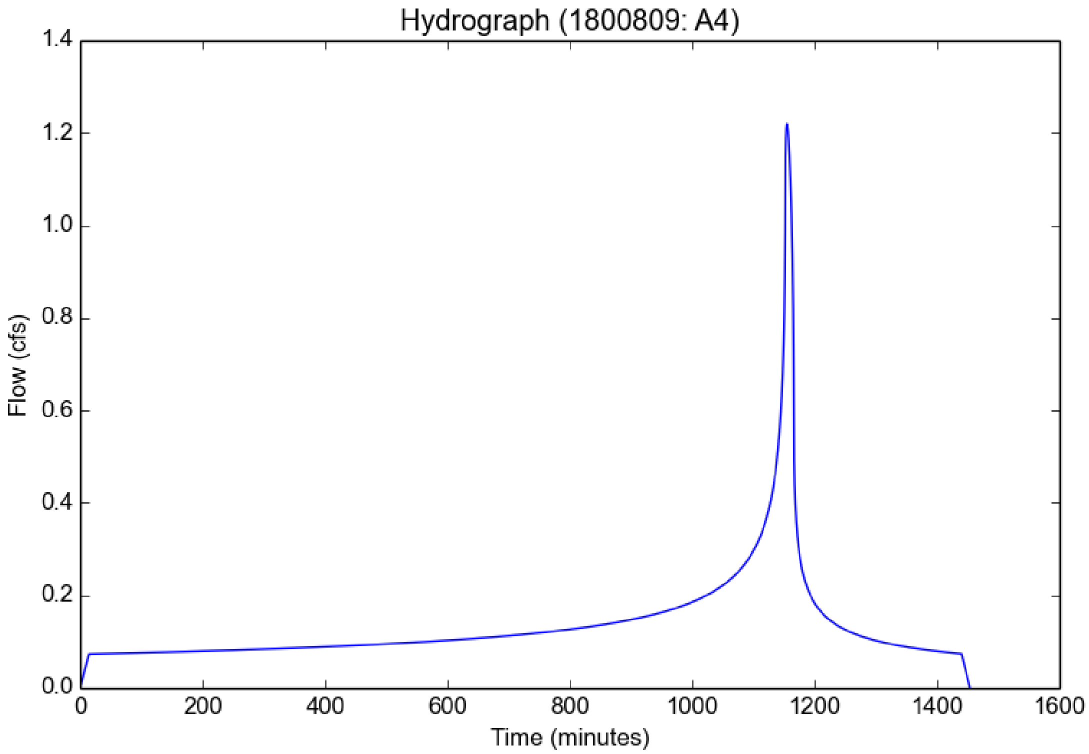


FIGURE 5 - A4 50yr
Peak Flow Hydrology Analysis
(Pre-Project)

Peak Flow

File location: P:/2018/1800807 Harvard-Westlake Weddington Complex/ENGR/EIR/Technical Reports/Hydrology and Water Quality/Appendices/HydroCa
Version: HydroCalc 1.0.3

Input Parameters

Project Name	Harvard Westlake
Subarea ID	A4
Area (ac)	3.35
Flow Path Length (ft)	250.0
Flow Path Slope (vft/hft)	0.01
50-yr Rainfall Depth (in)	7.2
Percent Impervious	0.95
Soil Type	16
Design Storm Frequency	50-yr
Fire Factor	0
LID	False

Output Results

Modeled (50-yr) Rainfall Depth (in)	7.2
Peak Intensity (in/hr)	4.2957
Undeveloped Runoff Coefficient (Cu)	0.9
Developed Runoff Coefficient (Cd)	0.9
Time of Concentration (min)	5.0
Clear Peak Flow Rate (cfs)	12.9516
Burned Peak Flow Rate (cfs)	12.9516
24-Hr Clear Runoff Volume (ac-ft)	1.7265
24-Hr Clear Runoff Volume (cu-ft)	75204.7101

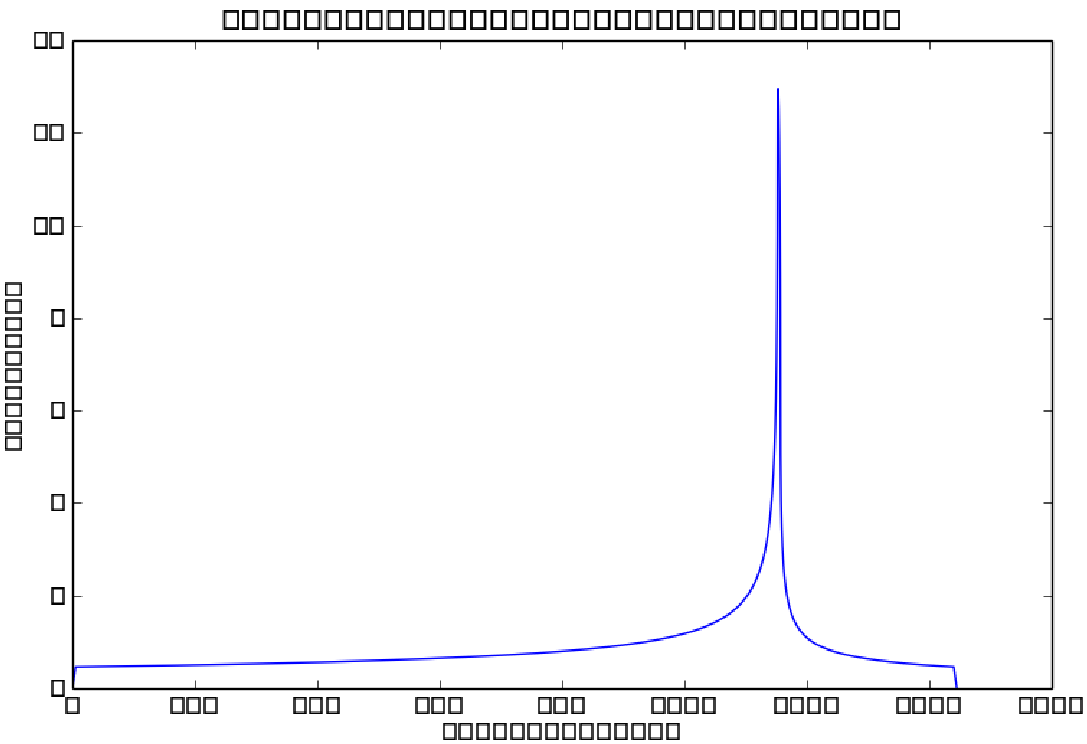


FIGURE 5 - A5 85th
Peak Flow Hydrology Analysis
(Pre-Project)

Peak Flow

File location: P:/2018/1800807 Harvard-Westlake Weddington Complex/ENGR/WATER/Reports/CEQA Hydrology Report/Appendices/EJ HYDROCALCS
Version: HydroCalc 1.0.3

Input Parameters

Project Name	1800809
Subarea ID	A5
Area (ac)	1.39
Flow Path Length (ft)	150.0
Flow Path Slope (vft/hft)	0.01
85th Percentile Rainfall Depth (in)	1.15
Percent Impervious	0.95
Soil Type	16
Design Storm Frequency	85th percentile storm
Fire Factor	0
LID	True

Output Results

Modeled (85th percentile storm) Rainfall Depth (in)	1.15
Peak Intensity (in/hr)	0.4954
Undeveloped Runoff Coefficient (Cu)	0.1867
Developed Runoff Coefficient (Cd)	0.8643
Time of Concentration (min)	10.0
Clear Peak Flow Rate (cfs)	0.5951
Burned Peak Flow Rate (cfs)	0.5951
24-Hr Clear Runoff Volume (ac-ft)	0.1136
24-Hr Clear Runoff Volume (cu-ft)	4949.9626

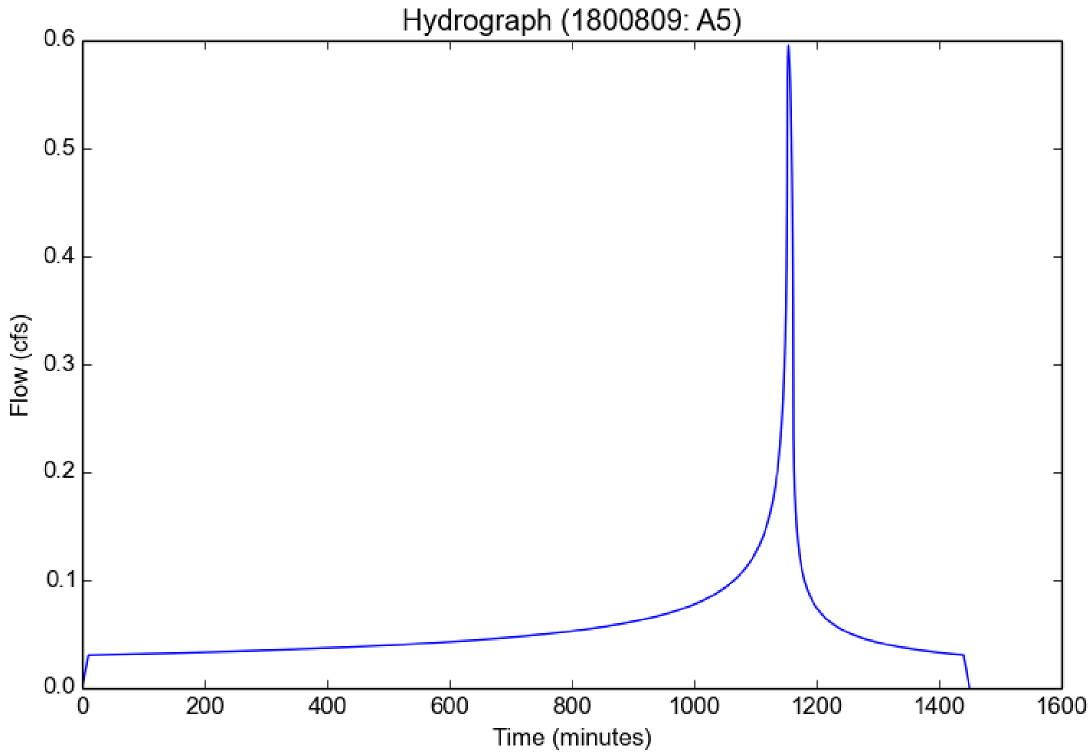


FIGURE 5 - A5 50yr
Peak Flow Hydrology Analysis
(Pre-Project)

Peak Flow H

File location: P:/2018/1800807 Harvard-Westlake Weddington Complex/ENGR/EIR/Technical Reports/Hydrology and Water Quality/Appendices/HydroCa
Version: HydroCalc 1.0.3

Input Parameters

Project Name	Harvard Westlake
Subarea ID	A5
Area (ac)	1.39
Flow Path Length (ft)	150.0
Flow Path Slope (vft/hft)	0.01
50-yr Rainfall Depth (in)	7.2
Percent Impervious	0.95
Soil Type	16
Design Storm Frequency	50-yr
Fire Factor	0
LID	False

Output Results

Modeled (50-yr) Rainfall Depth (in)	7.2
Peak Intensity (in/hr)	4.2957
Undeveloped Runoff Coefficient (Cu)	0.9
Developed Runoff Coefficient (Cd)	0.9
Time of Concentration (min)	5.0
Clear Peak Flow Rate (cfs)	5.3739
Burned Peak Flow Rate (cfs)	5.3739
24-Hr Clear Runoff Volume (ac-ft)	0.7164
24-Hr Clear Runoff Volume (cu-ft)	31204.3424

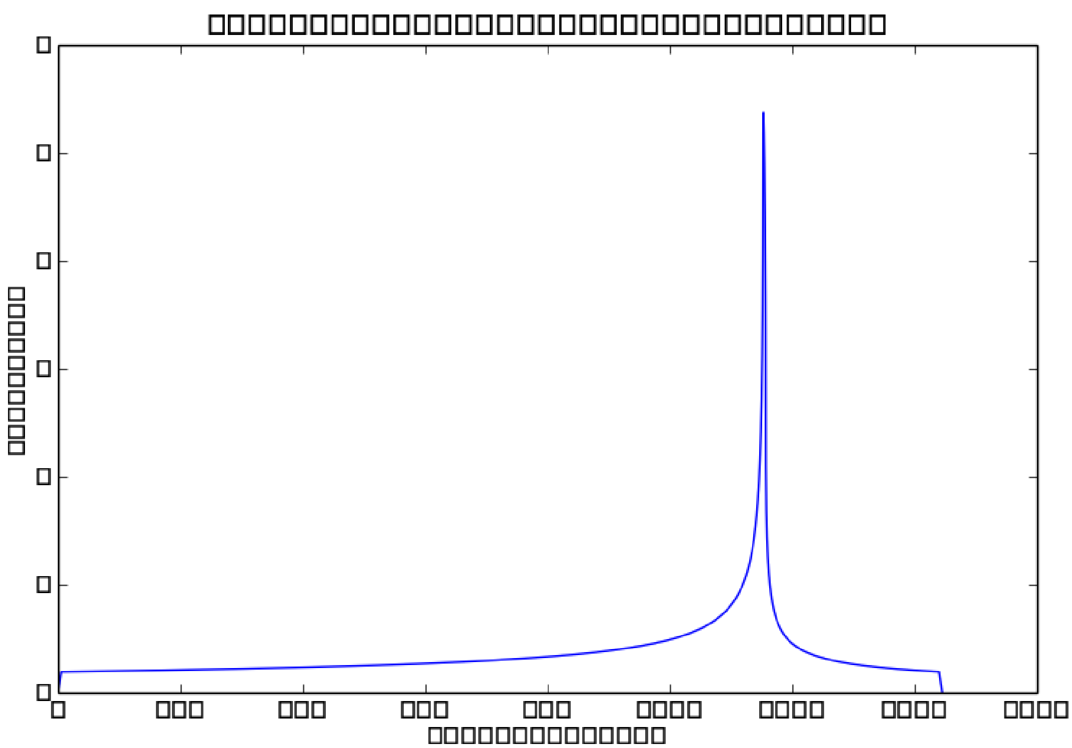


FIGURE 6

Pre and Post Project Peak Flow, 50-yr Storm Event, Hydrology Analysis

FIGURE 6
Peak Flow Hydrology Analysis
(Pre-Project)

Peak Flow I

File location: P:/2018/1800807 Harvard-Westlake Weddington Complex/ENGR/STORM/Hydrology/Hydrocalc/Harvard Westlake - River Park - Existing-50
Version: HydroCalc 1.0.3

Input Parameters

Project Name	Harvard Westlake
Subarea ID	River Park - Pre Project
Area (ac)	17.2
Flow Path Length (ft)	500.0
Flow Path Slope (vft/hft)	0.01
50-yr Rainfall Depth (in)	7.2
Percent Impervious	0.3
Soil Type	16
Design Storm Frequency	50-yr
Fire Factor	0
LID	False

Output Results

Modeled (50-yr) Rainfall Depth (in)	7.2
Peak Intensity (in/hr)	3.9429
Undeveloped Runoff Coefficient (Cu)	0.8977
Developed Runoff Coefficient (Cd)	0.8984
Time of Concentration (min)	6.0
Clear Peak Flow Rate (cfs)	60.9294
Burned Peak Flow Rate (cfs)	60.9294
24-Hr Clear Runoff Volume (ac-ft)	4.3534
24-Hr Clear Runoff Volume (cu-ft)	189635.5722

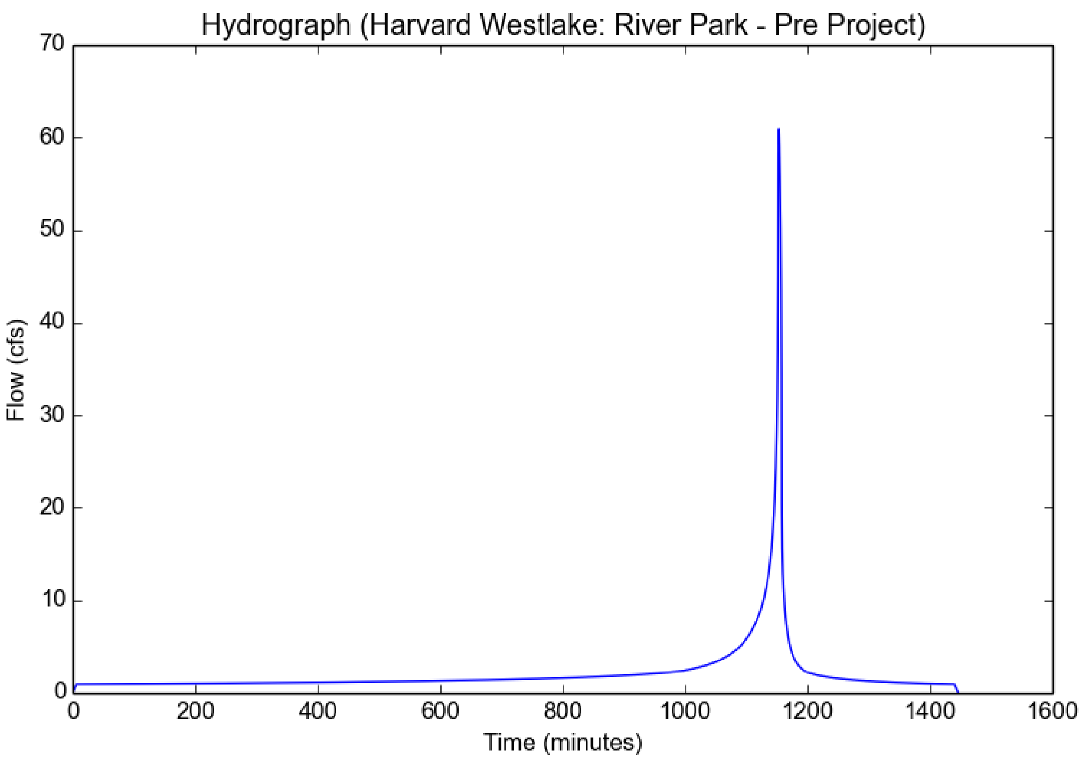


FIGURE 6
Peak Flow Hydrology Analysis
(Post-Project)

Peak Flow

File location: C:/Users/charrop/Desktop/Harvard Westlake - River Park - Proposed 50yr.pdf
Version: HydroCalc 1.0.3

Input Parameters

Project Name	Harvard Westlake
Subarea ID	River Park - Proposed
Area (ac)	17.19
Flow Path Length (ft)	500.0
Flow Path Slope (vft/hft)	0.01
50-yr Rainfall Depth (in)	7.2
Percent Impervious	0.59
Soil Type	16
Design Storm Frequency	50-yr
Fire Factor	0
LID	False

Output Results

Modeled (50-yr) Rainfall Depth (in)	7.2
Peak Intensity (in/hr)	3.9429
Undeveloped Runoff Coefficient (Cu)	0.8977
Developed Runoff Coefficient (Cd)	0.8991
Time of Concentration (min)	6.0
Clear Peak Flow Rate (cfs)	60.9384
Burned Peak Flow Rate (cfs)	60.9384
24-Hr Clear Runoff Volume (ac-ft)	6.3623
24-Hr Clear Runoff Volume (cu-ft)	277139.7802

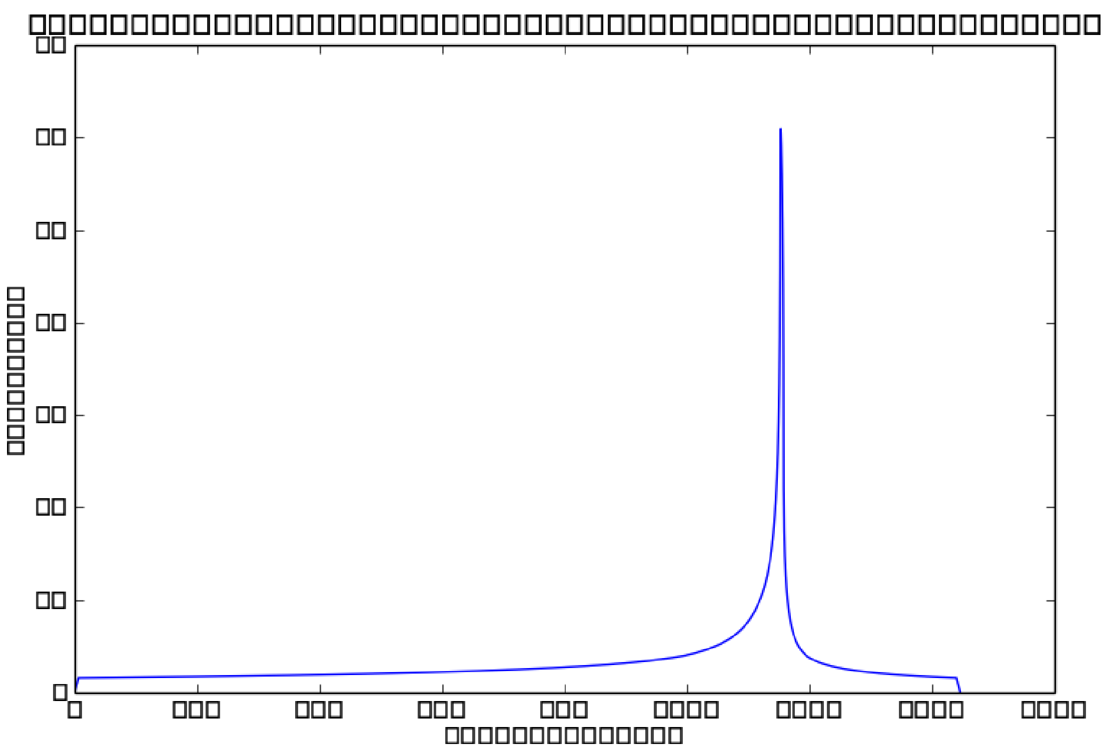


FIGURE 7

Project Site and Offsite, 85th Percentile Storm Event, LID Design Volume

FIGURE 7
85th Percentile Storm Event, LID Design Volume (Project Site)

Peak Flow

File location: X:/char

Version: HydroCalc 1

Input Parameters

Project Name	Harvard Westlake
Subarea ID	River Park - Proposed
Area (ac)	17.19
Flow Path Length (ft)	500.0
Flow Path Slope (vft/hft)	0.01
85th Percentile Rainfall Depth (in)	1.15
Percent Impervious	0.59
Soil Type	16
Design Storm Frequency	85th percentile storm
Fire Factor	0
LID	True

Output Results

Modeled (85th percentile storm) Rainfall Depth (in)	1.15
Peak Intensity (in/hr)	0.3003
Undeveloped Runoff Coefficient (Cu)	0.1
Developed Runoff Coefficient (Cd)	0.572
Time of Concentration (min)	29.0
Clear Peak Flow Rate (cfs)	2.953
Burned Peak Flow Rate (cfs)	2.953
24-Hr Clear Runoff Volume (ac-ft)	0.9345
24-Hr Clear Runoff Volume (cu-ft)	40707.7317

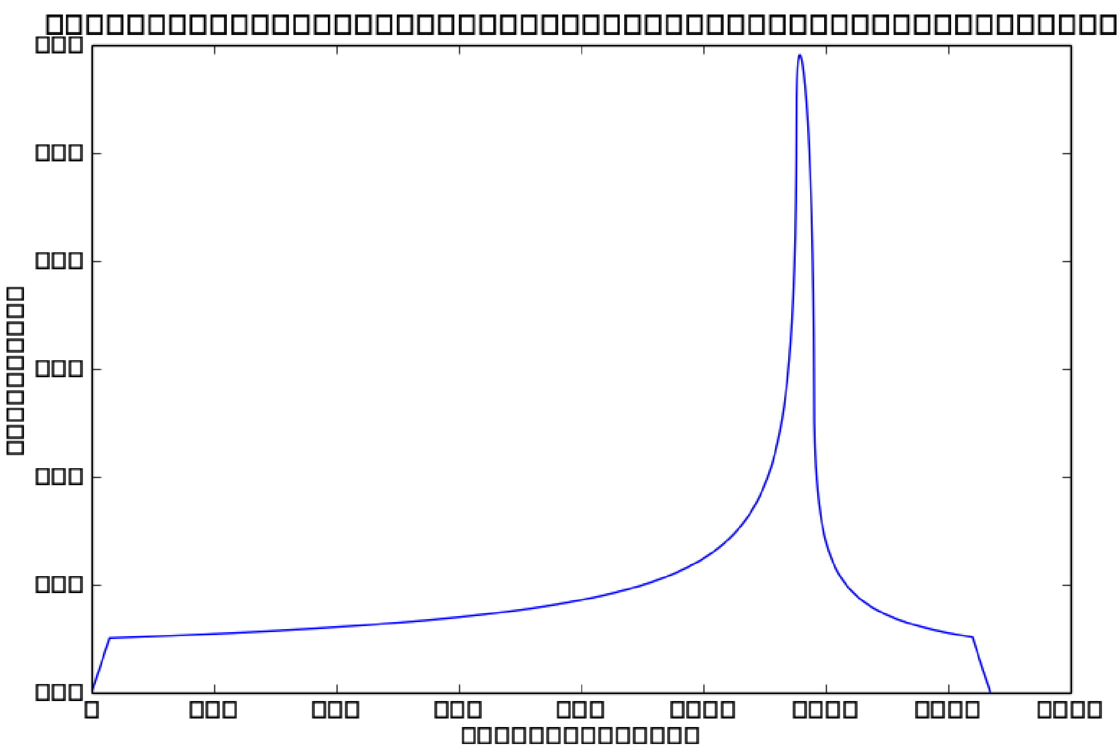


FIGURE 7
85th Percentile Storm Event, LID Design Volume (Offsite)

Peak Flow H

File location: P:/2018/1 Report/Appendices/1800807 (P) Drain
Version: HydroCalc 1.0.2

Input Parameters

Project Name	1800809
Subarea ID	Offsite
Area (ac)	38.64
Flow Path Length (ft)	1200.0
Flow Path Slope (vft/hft)	0.01
85th Percentile Rainfall Depth (in)	1.15
Percent Impervious	0.8
Soil Type	16
Design Storm Frequency	85th percentile storm
Fire Factor	0
LID	True

Output Results

Modeled (85th percentile storm) Rainfall Depth (in)	1.15
Peak Intensity (in/hr)	0.2523
Undeveloped Runoff Coefficient (Cu)	0.1
Developed Runoff Coefficient (Cd)	0.74
Time of Concentration (min)	42.0
Clear Peak Flow Rate (cfs)	7.2154
Burned Peak Flow Rate (cfs)	7.2154
24-Hr Clear Runoff Volume (ac-ft)	2.7176
24-Hr Clear Runoff Volume (cu-ft)	118380.1889

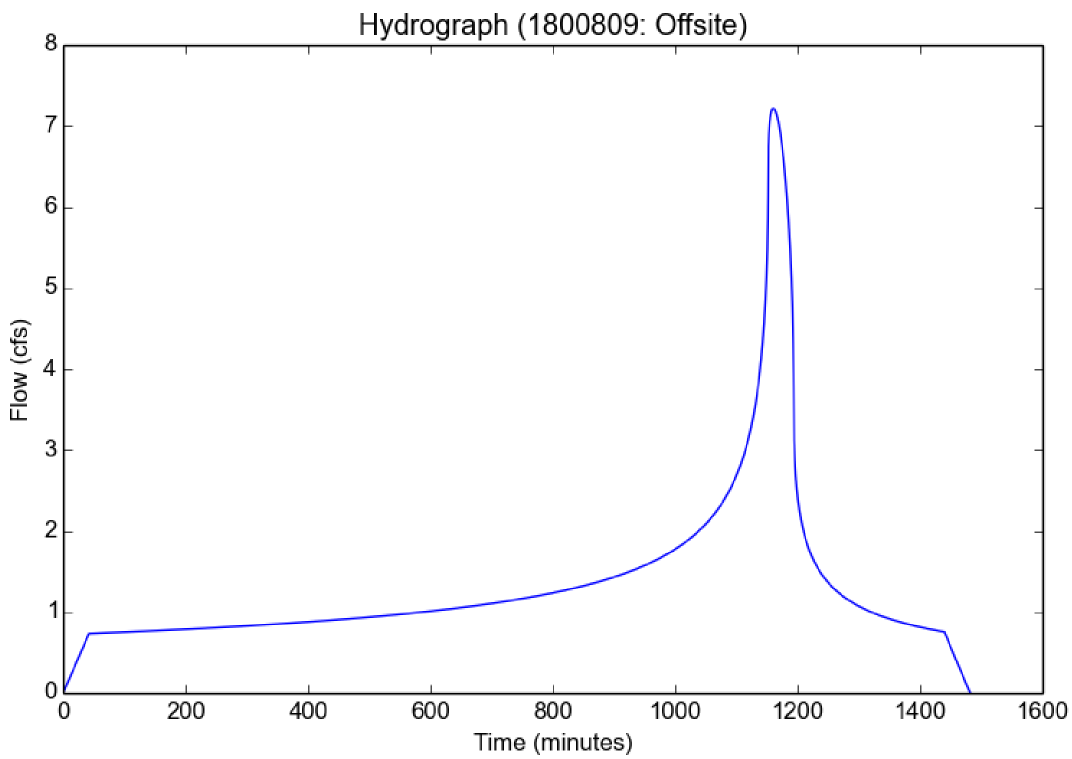
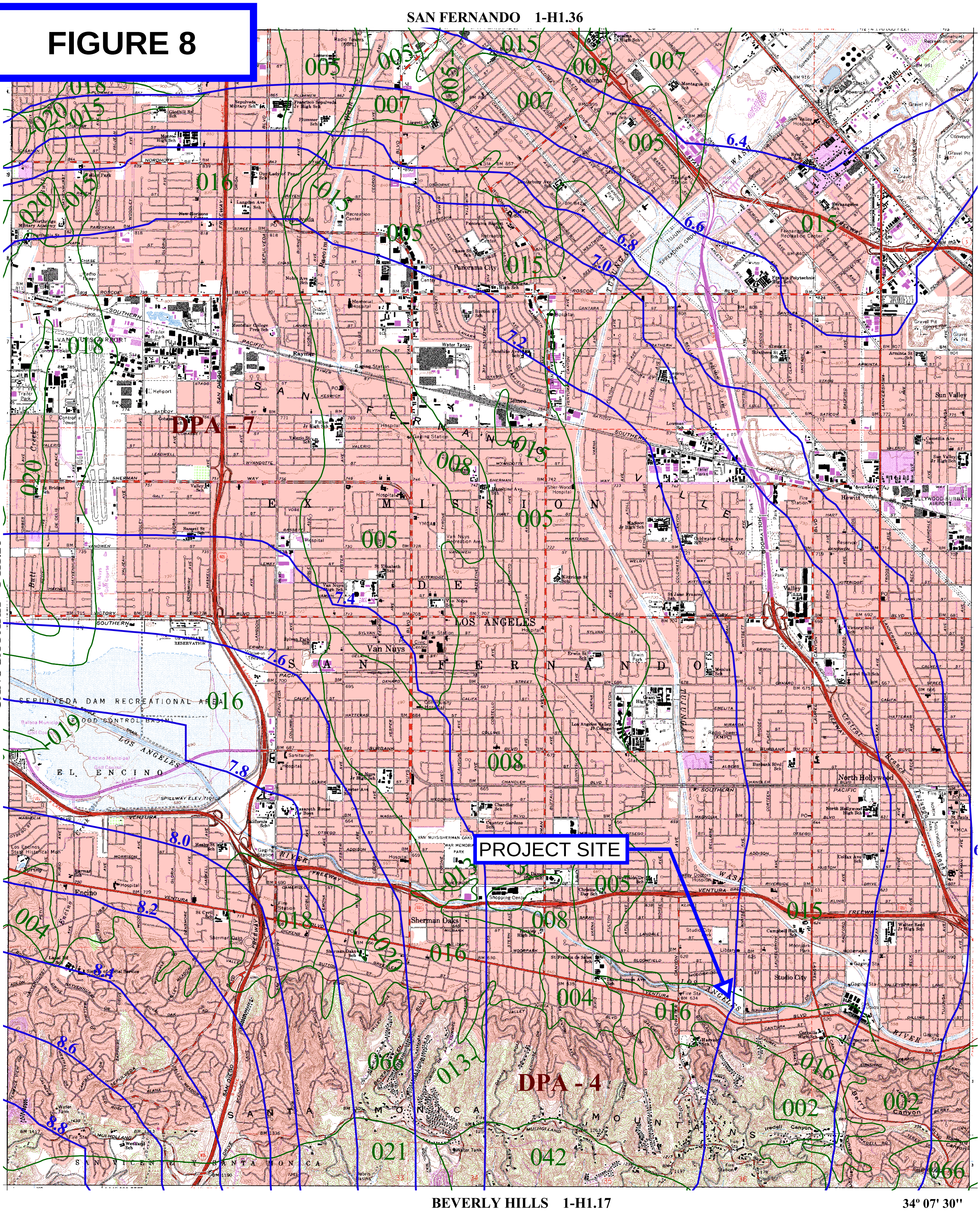


FIGURE 8



016 SOIL CLASSIFICATION AREA

7.2 INCHES OF RAINFALL

DPA - 6 DEBRIS POTENTIAL AREA

1 0 1 2 Miles

N E S W

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

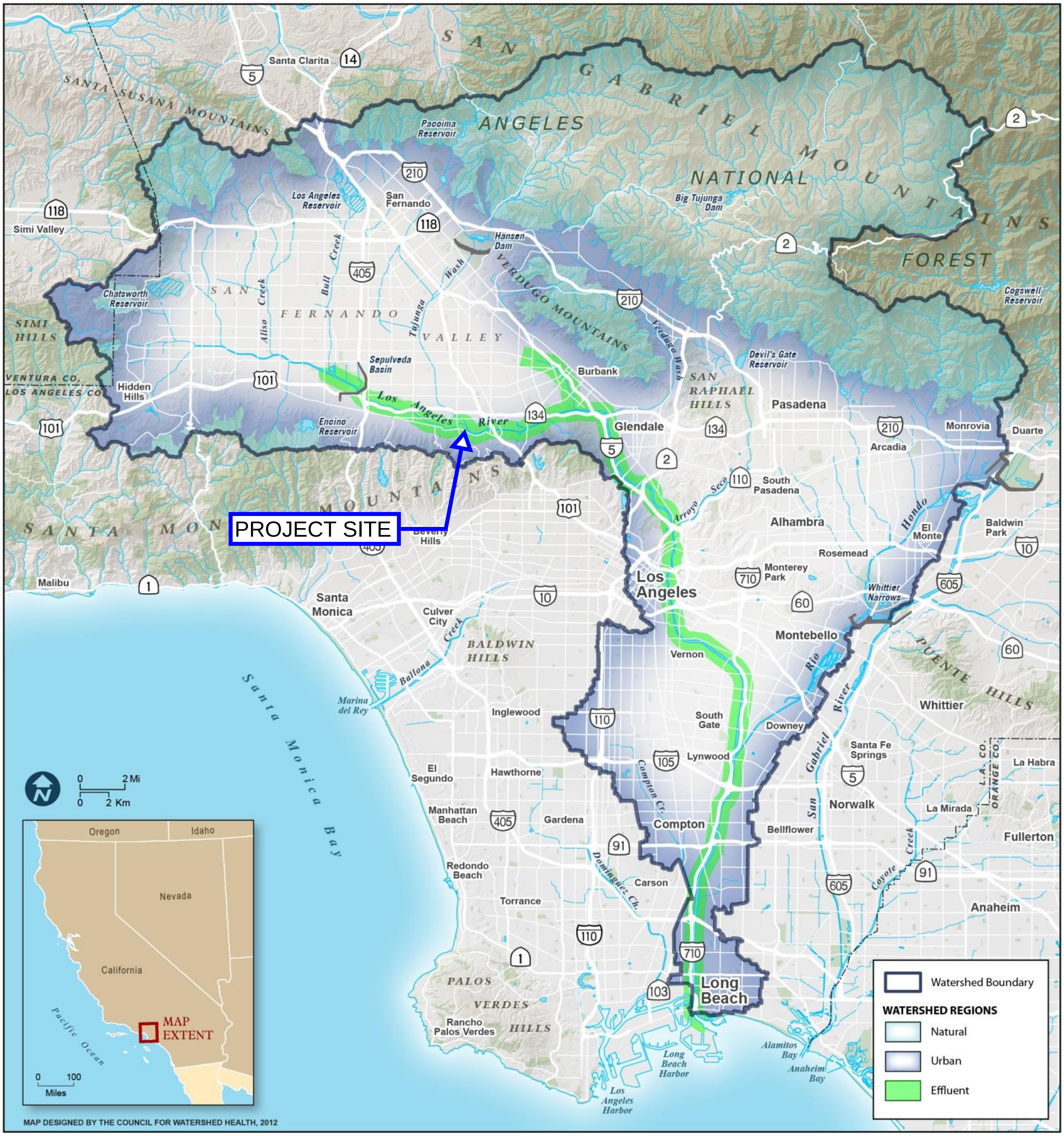
VAN NUYS
50-YEAR 24-HOUR ISOHYET

1-H1.27

LOS ANGELES COUNTY PUBLIC WORKS Public Service That Works

FIGURE 9

Map: Los Angeles River Watershed



Credit: Council for Watershed Health

National Flood Hazard Layer FIRMette

FEMA

FIGURE 10

Legend

SEE FIS REPORT FOR DETAILED LEGEND AND INDEX MAP FOR FIRM PANEL LAYOUT

SPECIAL FLOOD HAZARD AREAS		Without Base Flood Elevation (BFE) *Zone A, V, A99*
		With BFE or Depth *Zone AE, AO, AH, VE, AR*
		Regulatory Floodway
OTHER AREAS OF FLOOD HAZARD		0.2% Annual Chance Flood Hazard, Areas of 1% annual chance flood with average depth less than one foot or with drainage areas of less than one square mile *Zone X*
		Future Conditions 1% Annual Chance Flood Hazard *Zone X*
		Area with Reduced Flood Risk due to Levee. See Notes. *Zone X*
		Area with Flood Risk due to Levee *Zone D*
OTHER AREAS	NO SCREEN	Area of Minimal Flood Hazard *Zone X*
		Effective LOMRs
		Area of Undetermined Flood Hazard *Zone D*
GENERAL STRUCTURES		Channel, Culvert, or Storm Sewer
		Levee, Dike, or Floodwall
OTHER FEATURES	B 20.2 / 17.5	Cross Sections with 1% Annual Chance Water Surface Elevation
	8	Coastal Transect
	513	Base Flood Elevation Line (BFE)
		Limit of Study
		Jurisdiction Boundary
		Coastal Transect Baseline
		Profile Baseline
		Hydrographic Feature
MAP PANELS		Digital Data Available
		No Digital Data Available
		Unmapped

N

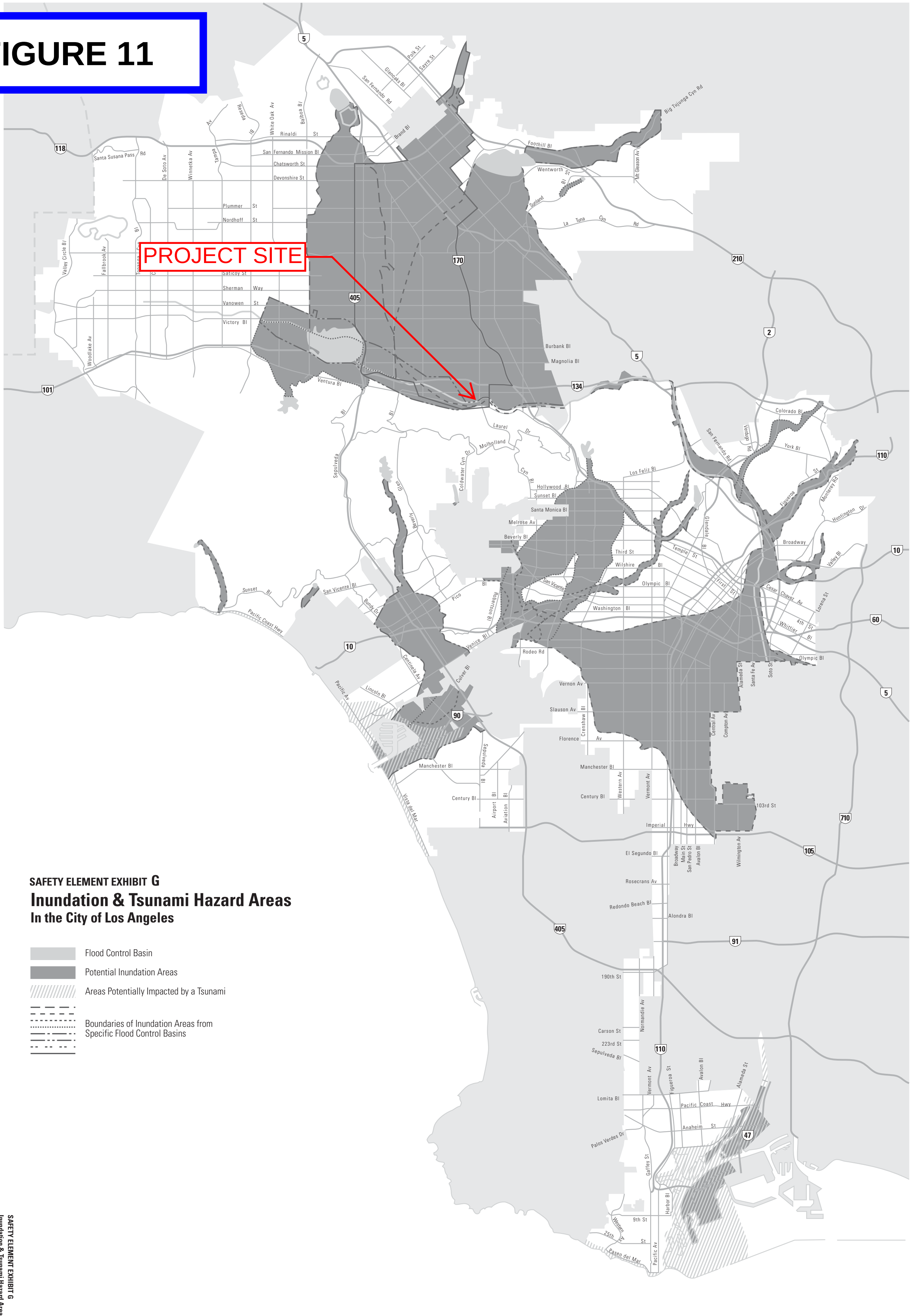
The pin displayed on the map is an approximate point selected by the user and does not represent an authoritative property location.

This map complies with FEMA's standards for the use of digital flood maps if it is not void as described below. The basemap shown complies with FEMA's basemap accuracy standards

The flood hazard information is derived directly from the authoritative NFHL web services provided by FEMA. This map was exported on 5/8/2020 at 2:54:31 PM and does not reflect changes or amendments subsequent to this date and time. The NFHL and effective information may change or become superseded by new data over time.

This map image is void if the one or more of the following map elements do not appear: basemap imagery, flood zone labels, legend, scale bar, map creation date, community identifiers, FIRM panel number, and FIRM effective date. Map images for unmapped and unmodernized areas cannot be used for regulatory purposes.

FIGURE 11



SAFETY ELEMENT EXHIBIT G

Inundation & Tsunami Hazard Areas

In the City of Los Angeles

Flood Control Basin

Potential Inundation Areas

Areas Potentially Impacted by a Tsunami

Boundaries of Inundation Areas from Specific Flood Control Basins

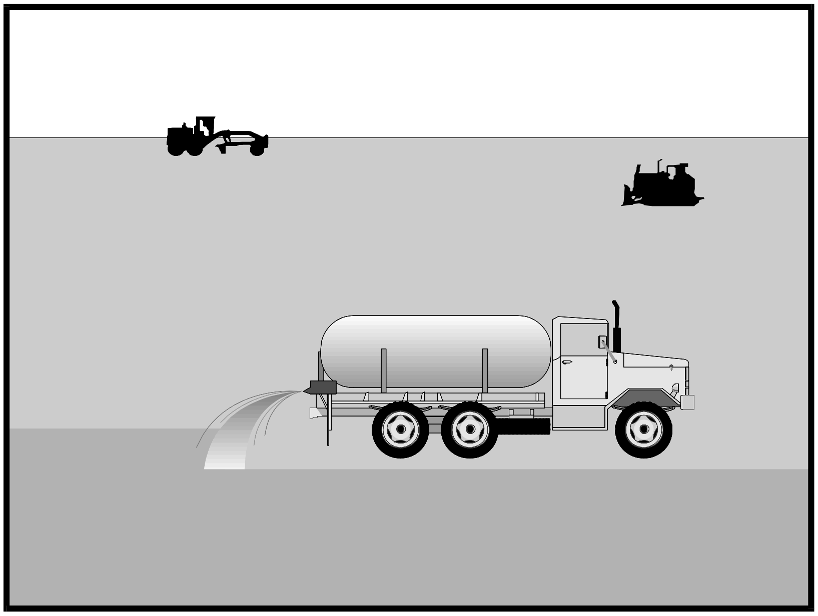
SAFETY ELEMENT EXHIBIT G
Inundation & Tsunami Hazard Areas

Sources: Environmental Impact Report, Framework Element, Los Angeles City General Plan, May 1995; Technical Appendix to the Safety Element of the Los Angeles County General Plan Hazard Reducation in Los Angeles County, Volume 2, Plate 6, "Flood and Inundation Hazards" January 1990; California Environmental Quality Act of 1970 (CEQA), Public Resources Code Section 21000 *et. seq.* with guidelines as amended, 1992; California Government Code Title 7 chapter 3, article 5 section 65302(g), as amended 1993.

Prepared by the General Plan Framework Section • City of Los Angeles Planning Department • Citywide Graphics • March, 1994 • Council File No. 89-2104

EXHIBIT 1
TYPICAL SWPPP BMPs

Soil Binde EC-5



Description and Purpose

Soil binding consists of application and maintenance of a soil stabilizer to exposed soil surfaces. Soil binders are materials applied to the soil surface to temporarily prevent water and wind induced erosion of exposed soils on construction sites.

Suitable Applications

Soil binders are typically applied to disturbed areas requiring temporary protection. Because soil binders, when used as a stand-alone practice, can often be incorporated into the soil, they are a good alternative to mulches in areas where grading activities will soon resume. Soil binders are commonly used in the following areas:

- Rough graded soils that will be inactive for a short period of time
- Soil stockpiles
- Temporary haul roads prior to placement of crushed rock
- Compacted soil road base
- Construction staging, materials storage, and layout areas

Limitations

- Soil binders are temporary in nature and may need reapplication.

Categories

EC	Erosion Control	☑
SE	Sediment Control	
TC	Tracking Control	
WE	Wind Erosion Control	☒
NS	Non-Stormwater Management Control	
WM	Waste Management and Materials Pollution Control	

Legend:

☑ **Primary Category**

☒ **Secondary Category**

Targeted Constituents

Sediment	☑
Nutrients	
Trash	
Metals	
Bacteria	
Oil and Grease	
Organics	

Potential Alternatives

EC-3 Hydraulic Mulch

EC-4 Hydroseeding

EC-6 Straw Mulch

EC-7 Geotextiles and Mats

EC-8 Wood Mulching

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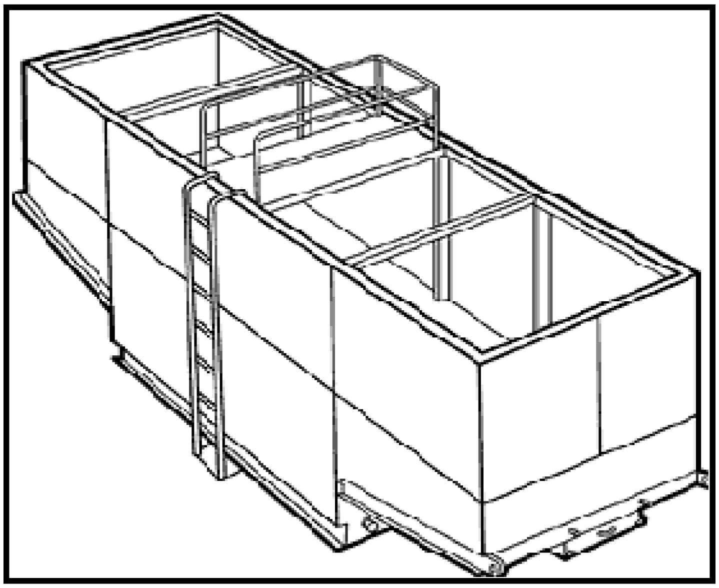
Dewatering Operations NS-2



Categories

EC	Erosion Control	
SE	Sediment Control	☒
TC	Tracking Control	
WE	Wind Erosion Control	
NS	Non-Stormwater Management Control	☑
WM	Waste Management and Materials Pollution Control	

Legend:

☑ **Primary Category**

☒ **Secondary Category**

Targeted Constituents

Sediment	☑
Nutrients	
Trash	
Metals	
Bacteria	
Oil and Grease	☑
Organics	

Potential Alternatives

SE-5: Fiber Roll

SE-6: Gravel Bag Berm

Description and Purpose

Dewatering operations are practices that manage the discharge of pollutants when non-stormwater and accumulated precipitation (stormwater) must be removed from a work location to proceed with construction work or to provide vector control.

The General Permit incorporates Numeric Effluent Limits (NEL) and Numeric Action Levels (NAL) for turbidity (see Section 2 of this handbook to determine your project's risk level and if you are subject to these requirements).

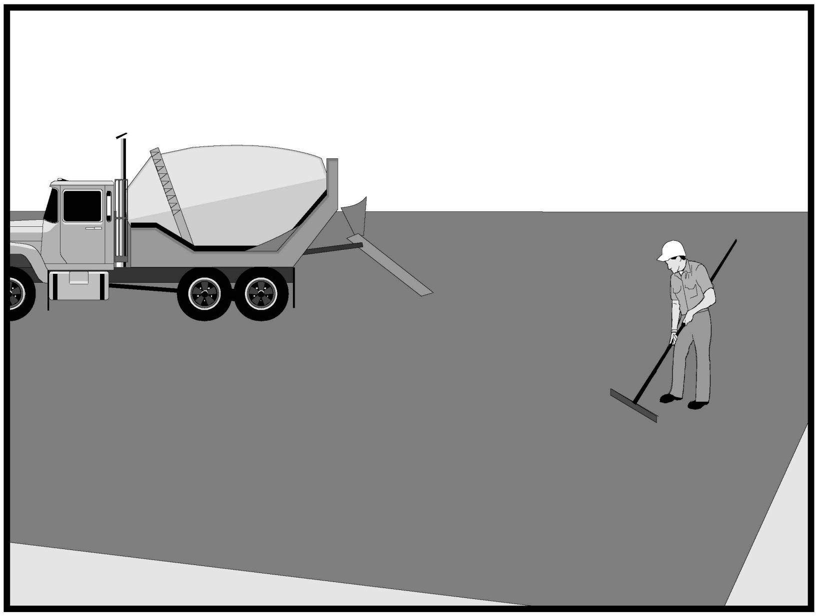
Discharges from dewatering operations can contain high levels of fine sediment that, if not properly treated, could lead to exceedences of the General Permit requirements.

Suitable Applications

These practices are implemented for discharges of non-stormwater from construction sites. Non-stormwaters include, but are not limited to, groundwater, water from cofferdams, water diversions, and waters used during construction activities that must be removed from a work area to facilitate construction.

Practices identified in this section are also appropriate for implementation when managing the removal of accumulated precipitation (stormwater) from depressed areas at a construction site.

Stormwater mixed with non-stormwater should be managed as non-stormwater.



Categories

EC	Erosion Control	
SE	Sediment Control	
TC	Tracking Control	
WE	Wind Erosion Control	
NS	Non-Stormwater Management Control	☑
WM	Waste Management and Materials Pollution Control	☒

Legend:

☑ **Primary Category**

☒ **Secondary Category**

Targeted Constituents

Sediment	☑
Nutrients	
Trash	
Metals	
Bacteria	
Oil and Grease	☑
Organics	

Potential Alternatives

None

Description and Purpose

Prevent or reduce the discharge of pollutants from paving operations, using measures to prevent runon and runoff pollution, properly disposing of wastes, and training employees and subcontractors.

The General Permit incorporates Numeric Effluent Limits (NEL) and Numeric Action Levels (NAL) for pH and turbidity (see Section 2 of this handbook to determine your project's risk level and if you are subject to these requirements).

Many types of construction materials associated with paving and grinding operations, including mortar, concrete, and cement and their associated wastes have basic chemical properties that can raise pH levels outside of the permitted range. Additional care should be taken when managing these materials to prevent them from coming into contact with stormwater flows, which could lead to exceedances of the General Permit requirements.

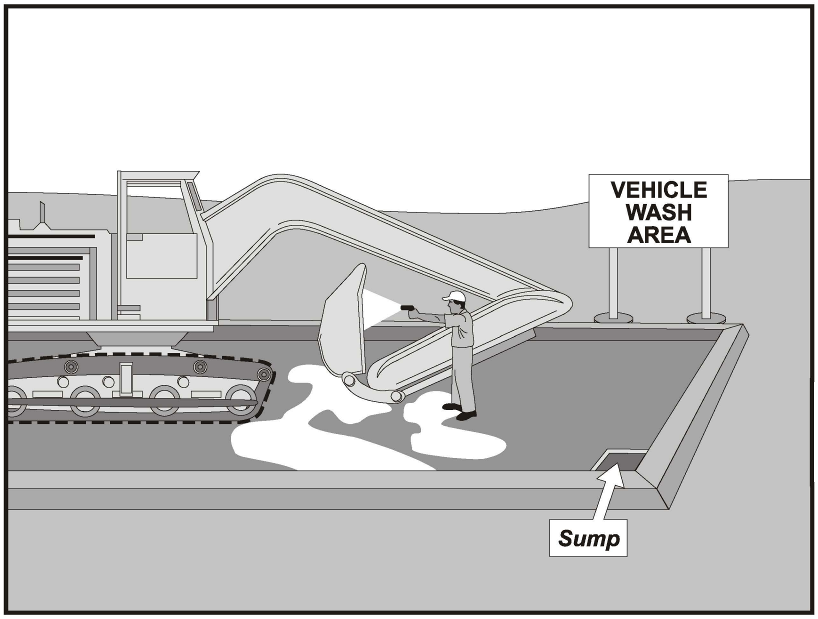
Suitable Applications

These procedures are implemented where paving, surfacing, resurfacing, or sawcutting, may pollute stormwater runoff or discharge to the storm drain system or watercourses.

Limitations

- Paving opportunities may be limited during wet weather.
- Discharges of freshly paved surfaces may raise pH to environmentally harmful levels and trigger permit violations.

Vehicle and Equipment Cleaning NS-8



Categories

EC	Erosion Control	
SE	Sediment Control	
TC	Tracking Control	
WE	Wind Erosion Control	
NS	Non-Stormwater Management Control	☑
WM	Waste Management and Materials Pollution Control	

Legend:

☑ **Primary Objective**

☒ **Secondary Objective**

Targeted Constituents

Sediment	☑
Nutrients	☑
Trash	
Metals	
Bacteria	
Oil and Grease	☑
Organics	☑

Potential Alternatives

None

Description and Purpose

Vehicle and equipment cleaning procedures and practices eliminate or reduce the discharge of pollutants to stormwater from vehicle and equipment cleaning operations. Procedures and practices include but are not limited to: using offsite facilities; washing in designated, contained areas only; eliminating discharges to the storm drain by infiltrating the wash water; and training employees and subcontractors in proper cleaning procedures.

Suitable Applications

These procedures are suitable on all construction sites where vehicle and equipment cleaning is performed.

Limitations

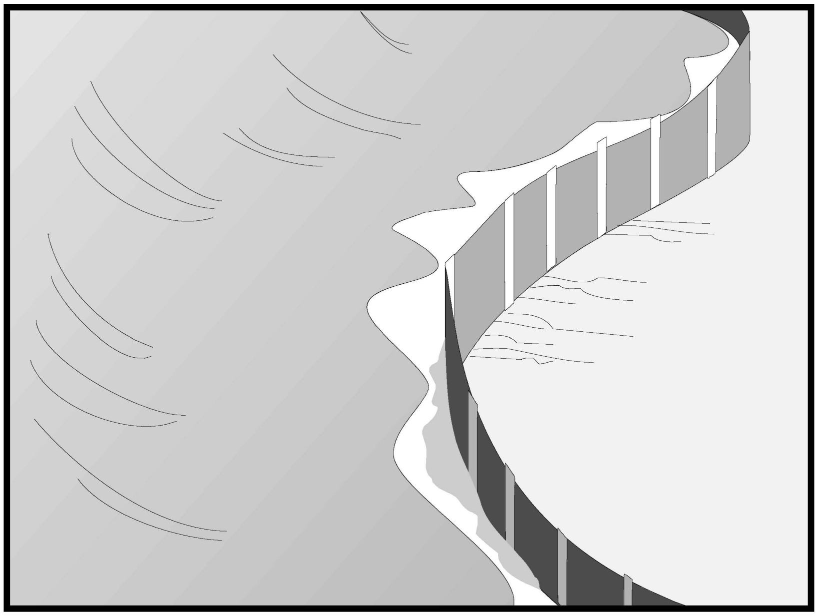
Even phosphate-free, biodegradable soaps have been shown to be toxic to fish before the soap degrades. Sending vehicles/equipment offsite should be done in conjunction with TC-1, Stabilized Construction Entrance/Exit.

Implementation

Other options to washing equipment onsite include contracting with either an offsite or mobile commercial washing business. These businesses may be better equipped to handle and dispose of the wash waters properly. Performing this work offsite can also be economical by eliminating the need for a separate washing operation onsite.

If washing operations are to take place onsite, then:

Silt Fence SE-1



Categories

EC	Erosion Control	
SE	Sediment Control	☑
TC	Tracking Control	
WE	Wind Erosion Control	
NS	Non-Stormwater Management Control	
WM	Waste Management and Materials Pollution Control	

Legend:

☑ **Primary Category**

☒ **Secondary Category**

Targeted Constituents

Sediment	☑
Nutrients	
Trash	
Metals	
Bacteria	
Oil and Grease	
Organics	

Potential Alternatives

SE-5 Fiber Rolls

SE-6 Gravel Bag Berm

SE-8 Sandbag Barrier

SE-10 Storm Drain Inlet Protection

SE-14 Biofilter Bags

Description and Purpose

A silt fence is made of a woven geotextile that has been entrenched, attached to supporting poles, and sometimes backed by a plastic or wire mesh for support. The silt fence detains sediment-laden water, promoting sedimentation behind the fence.

Suitable Applications

Silt fences are suitable for perimeter control, placed below areas where sheet flows discharge from the site. They could also be used as interior controls below disturbed areas where runoff may occur in the form of sheet and rill erosion and around inlets within disturbed areas (SE-10). Silt fences are generally ineffective in locations where the flow is concentrated and are only applicable for sheet or overland flows. Silt fences are most effective when used in combination with erosion controls. Suitable applications include:

- Along the perimeter of a project.
- Below the toe or down slope of exposed and erodible slopes.
- Along streams and channels.
- Around temporary spoil areas and stockpiles.
- Around inlets.
- Below other small cleared areas.

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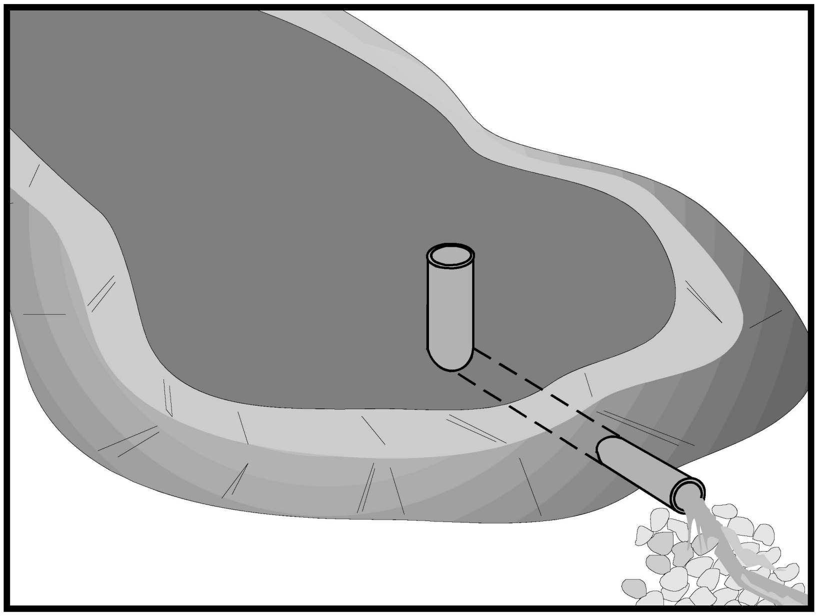

Categories

EC	Erosion Control	
SE	Sediment Control	☑
TC	Tracking Control	
WE	Wind Erosion Control	
NS	Non-Stormwater Management Control	
WM	Waste Management and Materials Pollution Control	

Legend:

☑ **Primary Category**

☒ **Secondary Category**

Targeted Constituents

Sediment	☑
Nutrients	
Trash	☑
Metals	
Bacteria	
Oil and Grease	
Organics	

Potential Alternatives

SE-3 Sediment Trap (for smaller areas)

Description and Purpose

A sediment basin is a temporary basin formed by excavation or by constructing an embankment so that sediment-laden runoff is temporarily detained under quiescent conditions, allowing sediment to settle out before the runoff is discharged.

Sediment basin design guidance presented in this fact sheet is intended to provide options, methods, and techniques to optimize temporary sediment basin performance and basin sediment removal. Basin design guidance provided in this fact sheet is not intended to guarantee basin effluent compliance with numeric discharge limits (numeric action levels or numeric effluent limits for turbidity). Compliance with discharge limits requires a thoughtful approach to comprehensive BMP planning, implementation, and maintenance. Therefore, optimally designed and maintained sediment basins should be used in conjunction with a comprehensive system of BMPs that includes:

- Diverting runoff from undisturbed areas away from the basin

- Erosion control practices to minimize disturbed areas on-site
and to provide temporary stabilization and interim sediment controls (e.g., stockpile perimeter control, check dams, perimeter controls around individual lots) to reduce the basin's influent sediment concentration.

At some sites, sediment basin design enhancements may be required to adequately remove sediment. Traditional

Street Sweeping and Vacuuming SE-7

Categories

EC	Erosion Control	
SE	Sediment Control	☒
TC	Tracking Control	☑
WE	Wind Erosion Control	
NS	Non-Stormwater Management Control	
WM	Waste Management and Materials Pollution Control	

Legend:

☑ **Primary Objective**

☒ **Secondary Objective**

Targeted Constituents

Sediment	☑
Nutrients	
Trash	☑
Metals	
Bacteria	
Oil and Grease	☑
Organics	

Potential Alternatives

None

Description and Purpose

Street sweeping and vacuuming includes use of self-propelled and walk-behind equipment to remove sediment from streets and roadways, and to clean paved surfaces in preparation for final paving. Sweeping and vacuuming prevents sediment from the project site from entering storm drains or receiving waters.

Suitable Applications

Sweeping and vacuuming are suitable anywhere sediment is tracked from the project site onto public or private paved streets and roads, typically at points of egress. Sweeping and vacuuming are also applicable during preparation of paved surfaces for final paving.

Limitations

Sweeping and vacuuming may not be effective when sediment is wet or when tracked soil is caked (caked soil may need to be scraped loose).

Implementation

- Controlling the number of points where vehicles can leave the site will allow sweeping and vacuuming efforts to be focused, and perhaps save money.
- Inspect potential sediment tracking locations daily.
- Visible sediment tracking should be swept or vacuumed on a daily basis.
- Do not use kick brooms or sweeper attachments. These tend to spread the dirt rather than remove it.

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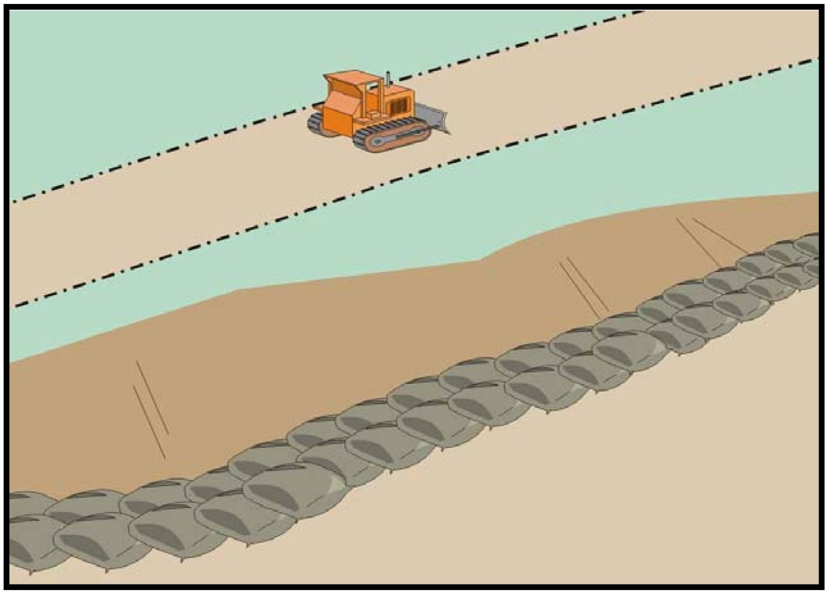
Sandbag Barrier SE-8



Categories

EC	Erosion Control	☒
SE	Sediment Control	☑
TC	Tracking Control	
WE	Wind Erosion Control	
NS	Non-Stormwater Management Control	
WM	Waste Management and Materials Pollution Control	

Legend:

☑ **Primary Category**

☒ **Secondary Category**

Targeted Constituents

Sediment	☑
Nutrients	
Trash	
Metals	
Bacteria	
Oil and Grease	
Organics	

Potential Alternatives

SE-1 Silt Fence

SE-5 Fiber Rolls

SE-6 Gravel Bag Berm

SE-14 Biofilter Bags

Description and Purpose

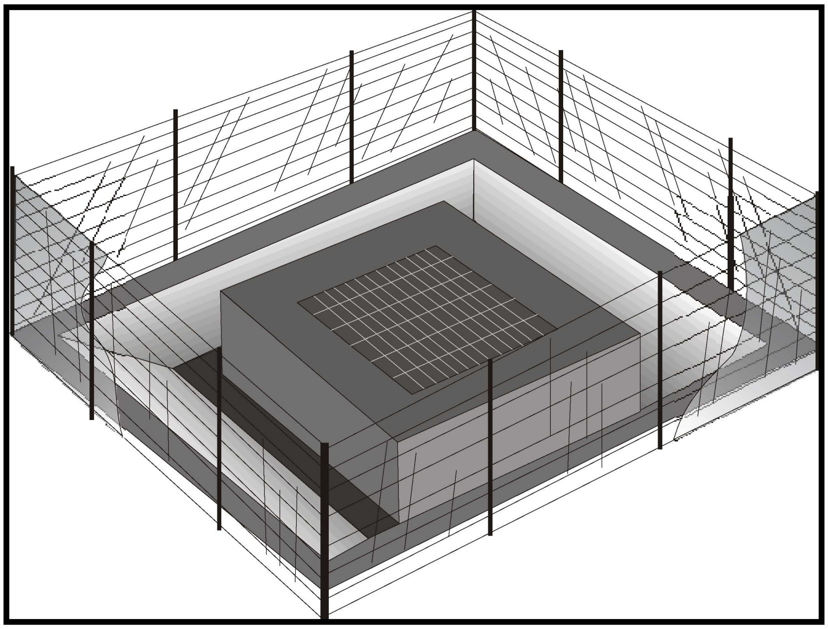
A sandbag barrier is a series of sand-filled bags placed on a level contour to intercept or to divert sheet flows. Sandbag barriers placed on a level contour pond sheet flow runoff, allowing sediment to settle out.

Suitable Applications

Sandbag barriers may be suitable:

- As a linear sediment control measure:
  - Below the toe of slopes and erodible slopes.
  - As sediment traps at culvert/pipe outlets.
  - Below other small cleared areas.
  - Along the perimeter of a site.
  - Down slope of exposed soil areas.
  - Around temporary stockpiles and spoil areas.
  - Parallel to a roadway to keep sediment off paved areas.
  - Along streams and channels.
- As linear erosion control measure:
  - Along the face and at grade breaks of exposed and erodible slopes to shorten slope length and spread runoff as sheet flow.

CASQA
CALIFORNIA STORMWATER QUALITY ASSOCIATION®



Categories

EC	Erosion Control	
SE	Sediment Control	☑
TC	Tracking Control	
WE	Wind Erosion Control	
NS	Non-Stormwater Management Control	
WM	Waste Management and Materials Pollution Control	

Legend:

☑ **Primary Category**

☒ **Secondary Category**

Targeted Constituents

Sediment	☑
Nutrients	
Trash	☒
Metals	
Bacteria	
Oil and Grease	
Organics	

Potential Alternatives

SE-1 Silt Fence

SE-5 Fiber Rolls

SE-6 Gravel Bag Berm

SE-8 Sandbag Barrier

SE-14 Biofilter Bags

Description and Purpose

Storm drain inlet protection consists of a sediment filter or an impounding area in, around or upstream of a storm drain, drop inlet, or curb inlet. Storm drain inlet protection measures temporarily pond runoff before it enters the storm drain, allowing sediment to settle. Some filter configurations also remove sediment by filtering, but usually the ponding action results in the greatest sediment reduction. Temporary geotextile storm drain inserts attach underneath storm drain grates to capture and filter storm water.

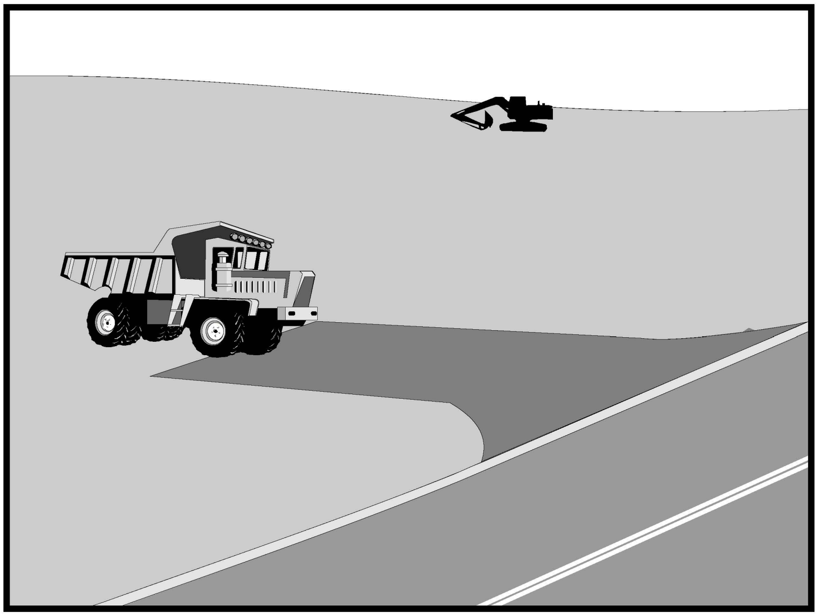
Suitable Applications

Every storm drain inlet receiving runoff from unstabilized or otherwise active work areas should be protected. Inlet protection should be used in conjunction with other erosion and sediment controls to prevent sediment-laden stormwater and non-stormwater discharges from entering the storm drain system.

Limitations

- Drainage area should not exceed 1 acre.
- In general straw bales should not be used as inlet protection.
- Requires an adequate area for water to pond without encroaching into portions of the roadway subject to traffic.

Stabilized Construction Entrance/Exit TC-1



Categories

EC	Erosion Control	☒
SE	Sediment Control	☒
TC	Tracking Control	☑
WE	Wind Erosion Control	
NS	Non-Stormwater Management Control	
WM	Waste Management and Materials Pollution Control	

Legend:

☑ **Primary Objective**

☒ **Secondary Objective**

Targeted Constituents

Sediment	☑
Nutrients	
Trash	
Metals	
Bacteria	
Oil and Grease	
Organics	

Potential Alternatives

None

Description and Purpose

A stabilized construction access is defined by a point of entrance/exit to a construction site that is stabilized to reduce the tracking of mud and dirt onto public roads by construction vehicles.

Suitable Applications

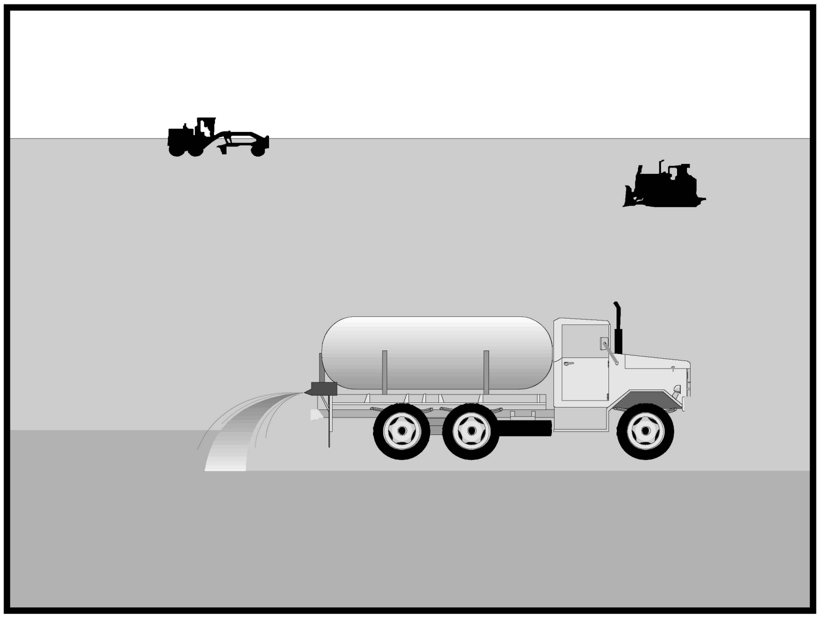
Use at construction sites:

- Where dirt or mud can be tracked onto public roads.
- Adjacent to water bodies.
- Where poor soils are encountered.
- Where dust is a problem during dry weather conditions.

Limitations

- Entrances and exits require periodic top dressing with additional stones.
- This BMP should be used in conjunction with street sweeping on adjacent public right of way.
- Entrances and exits should be constructed on level ground only.
- Stabilized construction entrances are rather expensive to construct and when a wash rack is included, a sediment trap of some kind must also be provided to collect wash water

Wind Erosion Control WE-1



Categories

EC	Erosion Control	
SE	Sediment Control	☒
TC	Tracking Control	
WE	Wind Erosion Control	☑
NS	Non-Stormwater Management Control	
WM	Waste Management and Materials Pollution Control	

Legend:

☑ **Primary Category**

☒ **Secondary Category**

Targeted Constituents

Sediment	☑
Nutrients	
Trash	
Metals	
Bacteria	
Oil and Grease	
Organics	

Potential Alternatives

EC-5 Soil Binders

Description and Purpose

Wind erosion or dust control consists of applying water or other chemical dust suppressants as necessary to prevent or alleviate dust nuisance generated by construction activities. Covering small stockpiles or areas is an alternative to applying water or other dust palliatives.

California's Mediterranean climate, with a short "wet" season and a typically long, hot "dry" season, allows the soils to thoroughly dry out. During the dry season, construction activities are at their peak, and disturbed and exposed areas are increasingly subject to wind erosion, sediment tracking and dust generated by construction equipment. Site conditions and climate can make dust control more of an erosion problem than water based erosion. Additionally, many local agencies, including Air Quality Management Districts, require dust control and/or dust control permits in order to comply with local nuisance laws, opacity laws (visibility impairment) and the requirements of the Clean Air Act. Wind erosion control is required to be implemented at all construction sites greater than 1 acre by the General Permit.

Suitable Applications

Most BMPs that provide protection against water-based erosion will also protect against wind-based erosion and dust control requirements required by other agencies will generally meet wind erosion control requirements for water quality protection. Wind erosion control BMPs are suitable during the following construction activities:

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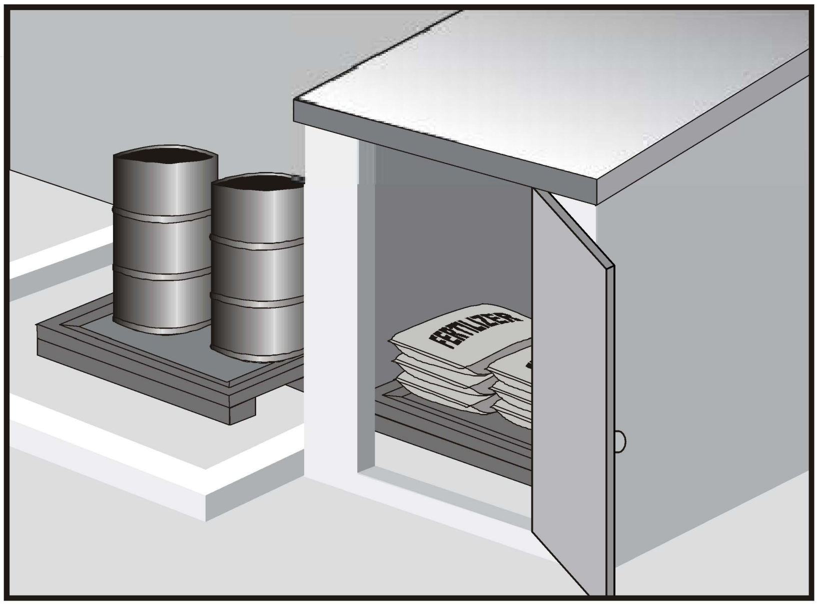
Material Delivery and Storage WM-1



Categories

EC	Erosion Control	
SE	Sediment Control	
TC	Tracking Control	
WE	Wind Erosion Control	
NS	Non-Stormwater Management Control	
WM	Waste Management and Materials Pollution Control	☑

Legend:

☑ **Primary Category**

☒ **Secondary Category**

Targeted Constituents

Sediment	☑
Nutrients	☑
Trash	☑
Metals	☑
Bacteria	
Oil and Grease	☑
Organics	☑

Potential Alternatives

None

Description and Purpose

Prevent, reduce, or eliminate the discharge of pollutants from material delivery and storage to the stormwater system or watercourses by minimizing the storage of hazardous materials onsite, storing materials in watertight containers and/or a completely enclosed designated area, installing secondary containment, conducting regular inspections, and training employees and subcontractors.

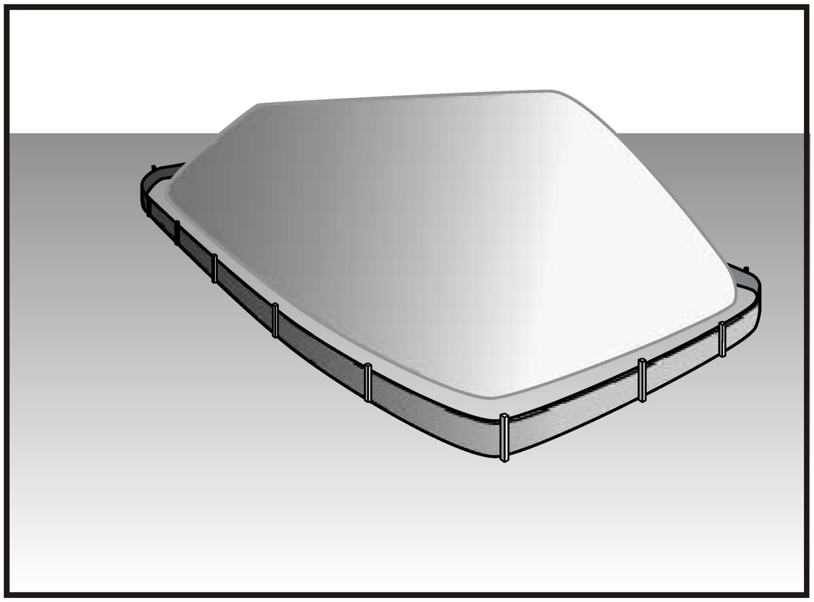
This best management practice covers only material delivery and storage. For other information on materials, see WM-2, Material Use, or WM-4, Spill Prevention and Control. For information on wastes, see the waste management BMPs in this section.

Suitable Applications

These procedures are suitable for use at all construction sites with delivery and storage of the following materials:

- Soil stabilizers and binders
- Pesticides and herbicides
- Fertilizers
- Detergents
- Plaster
- Petroleum products such as fuel, oil, and grease

Stockpile Management WM-3



Categories

EC	Erosion Control	
SE	Sediment Control	☒
TC	Tracking Control	
WE	Wind Erosion Control	
NS	Non-Stormwater Management Control	☒
WM	Waste Management and Materials Pollution Control	☑

Legend:

☑ **Primary Category**

☒ **Secondary Category**

Targeted Constituents

Sediment	☑
Nutrients	☑
Trash	☑
Metals	☑
Bacteria	
Oil and Grease	☑
Organics	☑

Potential Alternatives

None

Description and Purpose

Stockpile management procedures and practices are designed to reduce or eliminate air and stormwater pollution from stockpiles of soil, soil amendments, sand, paving materials such as portland cement concrete (PCC) rubble, asphalt concrete (AC), asphalt concrete rubble, aggregate base, aggregate sub base or pre-mixed aggregate, asphalt minder (so called "cold mix" asphalt), and pressure treated wood.

Suitable Applications

Implement in all projects that stockpile soil and other loose materials.

Limitations

- Plastic sheeting as a stockpile protection is temporary and hard to manage in windy conditions. Where plastic is used, consider use of plastic tarps with nylon reinforcement which may be more durable than standard sheeting.

- Plastic sheeting can increase runoff volume due to lack of infiltration and potentially cause perimeter control failure.

- Plastic sheeting breaks down faster in sunlight.

- The use of Plastic materials and photodegradable plastics should be avoided.

Implementation

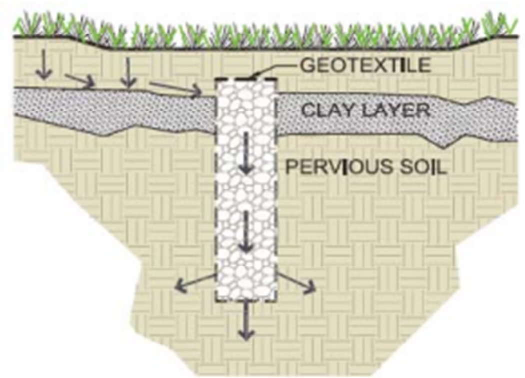
Protection of stockpiles is a year-round requirement. To properly manage stockpiles:

CASQA
CALIFORNIA STORMWATER QUALITY ASSOCIATION

EXHIBIT 2
TYPICAL LID BMPs

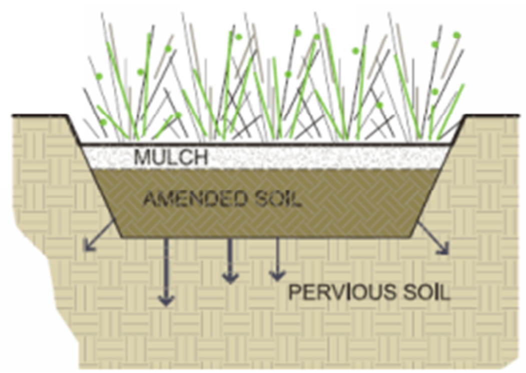
Dry Wells

A dry well is defined as an excavated, bored, drilled, or driven shaft or hole whose depth is greater than its width. Drywells are similar to infiltration trenches in their design and function, as they are designed to temporarily store and infiltrate runoff, primarily from rooftops or other impervious areas with low pollutant loading. A dry well may be either a drilled borehole filled with aggregate or a prefabricated storage chamber or pipe segment.



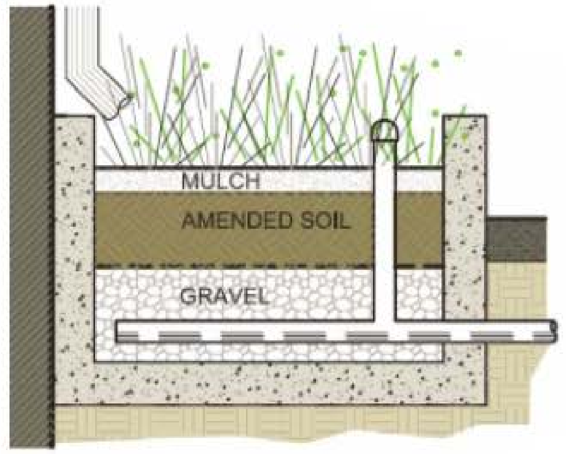
Bioretention

Bioretention stormwater treatment facilities are landscaped shallow depressions that capture and filter stormwater runoff. These facilities function as a soil and plant-based filtration device that removes pollutants through a variety of physical, biological, and chemical treatment processes. The facilities normally consist of a ponding area, mulch layer, planting soils, plantings, and, optionally, a subsurface gravel reservoir layer.

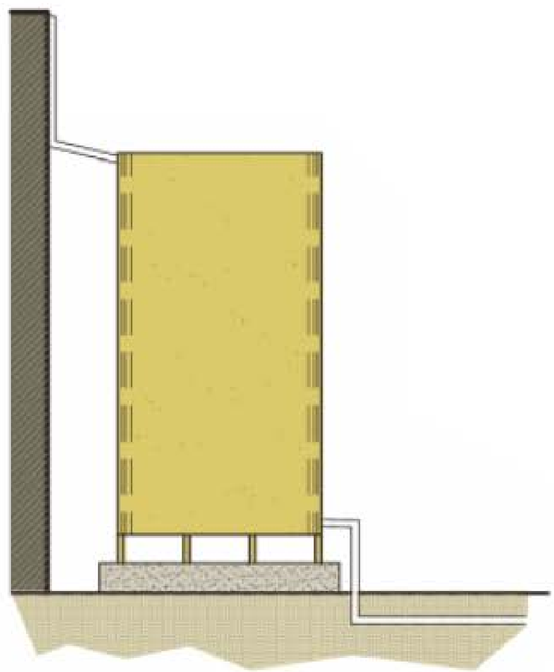


Planter Boxes

Planter boxes are bioretention treatment control measures that are completely contained within an impermeable structure with an underdrain (they do not infiltrate). They are similar to bioretention facilities with underdrains except they are situated at or above ground and are bound by impermeable walls. Planter boxes may be placed adjacent to or near buildings, other structures, or sidewalks.



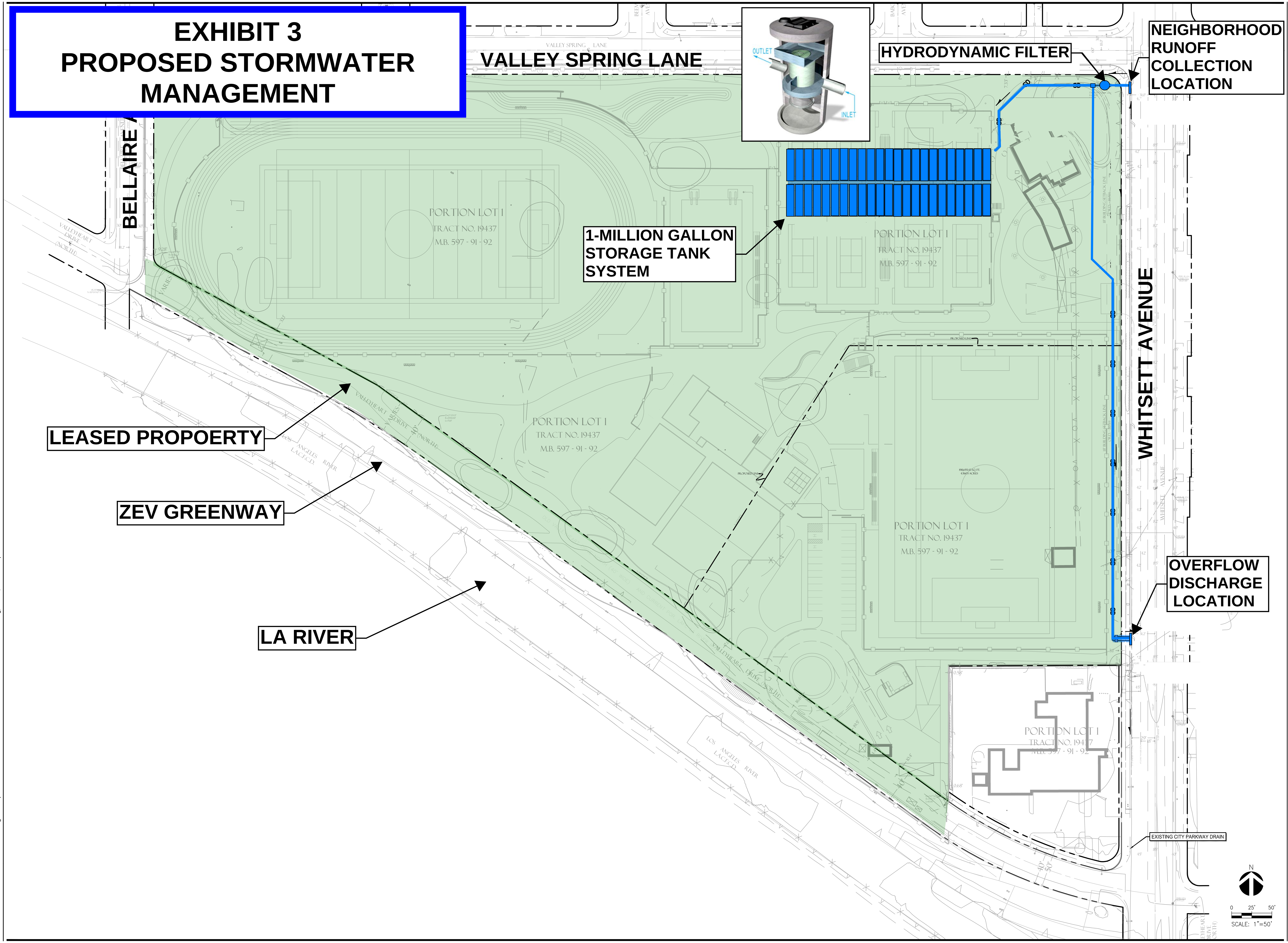
4.5 CAPTURE AND USE BMPS



Cistern Example

Capture and Use refers to a specific type of BMP that operates by capturing stormwater runoff and holding it for efficient use at a later time. On a commercial or industrial scale, capture and use BMPs are typically synonomous with cisterns, which can be implemented both above and below ground. Cisterns are sized to store a specified volume of water with no surface discharge until this volume is exceeded. The primary use of captured runoff is for subsurface drip irrigation purposes. The temporary storage of roof runoff reduces the runoff volume from a property and may reduce the peak runoff velocity for small, frequently occurring storms. In addition, by reducing the amount of stormwater runoff that flows overland into a stormwater conveyance system, less pollutants are transported through the conveyance system into local streams and the ocean. The onsite use of the harvested water for non-potable domestic purposes conserves City-supplied potable water and, where directed to unpaved surfaces, can recharge groundwater in local aquifers.

EXHIBIT 3
PROPOSED STORMWATER MANAGEMENT



4141 Whitsett Ave
Studio City, CA 91604

Date Description

Seal / Signature

NOT FOR CONSTRUCTION

Project Name
Project Name

Project Number
000.0000.000

Description
STORM DRAIN SYSTEM SCHEMATIC

Scale
PER PLAN

EX-4

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EXHIBIT 4
Rainwater Harvesting Standards

CONTECH®
ENGINEERED SOLUTIONS

DuroMaxx®
Rainwater Harvesting Cisterns

The experts you need to solve your stormwater challenges

Contech is the leader in stormwater solutions, helping engineers, contractors and owners with infrastructure and land development projects throughout North America.

With our responsive team of stormwater experts, local regulatory expertise and flexible solutions, Contech is the trusted partner you can count on for stormwater management solutions.

Your Contech Team

STORMWATER CONSULTANT

It's my job to recommend the best solution to meet permitting requirements.

STORMWATER DESIGN ENGINEER

I work with consultants to design the best approved solution to meet your project's needs.

REGULATORY MANAGER

I understand the local stormwater regulations and what solutions will be approved.

SALES ENGINEER

I make sure our solutions meet the needs of the contractor during construction.

Contech is your partner in stormwater management solutions

Cisterns for Stormwater Reuse and Runoff Reduction

We provided Yakult Manufacturing in Fountain Valley, California with two DuroMaxx® rainwater harvesting cisterns to capture and reuse runoff from rooftops, parking lots, and other impervious surfaces.

Low Impact Development strives to eliminate runoff by promoting infiltration wherever practical. If your site has high groundwater, soils with low permeability, bedrock, or other limiting conditions, infiltration alone may not provide enough runoff reduction to meet regulations. That's why rainwater harvesting is an important tool to help meet runoff reduction requirements.

The DuroMaxx® Rainwater Harvesting Cistern helps achieve stormwater management goals by reducing stormwater runoff while providing cost savings through the reduction of potable water use.

DuroMaxx®
STEEL REINFORCED PE TECHNOLOGY

DuroMaxx® Rainwater Harvesting Cisterns

Strength of steel and the durability of plastic ...

Our Rainwater Harvesting Cisterns are made from DuroMaxx Steel Reinforced Polyethylene (SRPE). The eighty (80) ksi steel reinforcing ribs provide the strength and pressure rated polyethylene (PE) resin provides the durability. The combination of materials results in an extraordinarily strong and durable below ground cistern.

- Available up to 120” diameter
- Includes prefabricated access points
- Lightweight - easily handled and quickly installed, often without the use of heavy construction equipment
- H-25 traffic rated design

DuroMaxx Rainwater Harvesting Cisterns have been certified to be in compliance with the Uniform Plumbing Code (UPC®) by The International Association of Plumbing and Mechanical Officials (IAPMO) Research and Testing. The DuroMaxx Rainwater Harvesting Cistern is also approved by Los Angeles City and has a research report number (RR 5726).

Engineers can now write specifications for rainwater harvesting cisterns based on a nationally recognized standard that address issues such as structural design, leakage, and repeatable manufacturing processes. Contech is one of the few companies that have received IAPMO/UPC certification for rainwater harvesting cisterns.

A 182,000 gallon DuroMaxx rainwater harvesting cistern was used at the Oceano Apartments in Woodland Hills, California to meet runoff reduction goals at this 3.57 acre site.

Learn More:
www.ContechES.com/rwh

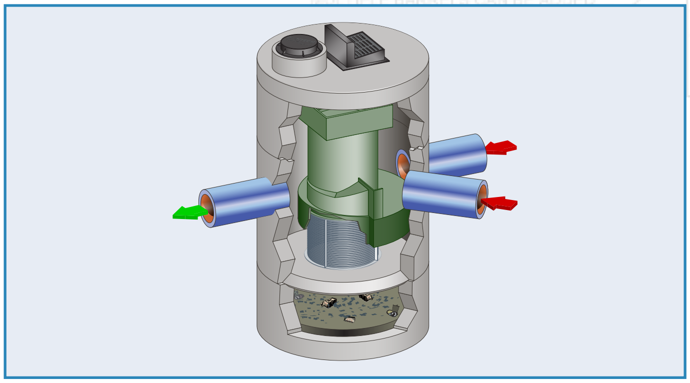
Contech is one of the few companies that have received IAPMO/UPC certification for rainwater harvesting cisterns.

DuroMaxx Rainwater Harvesting Cisterns are UPC Compliant

Pretreating harvested water protects pumps, filters, & fixtures from damage

APPLICATION TIPS

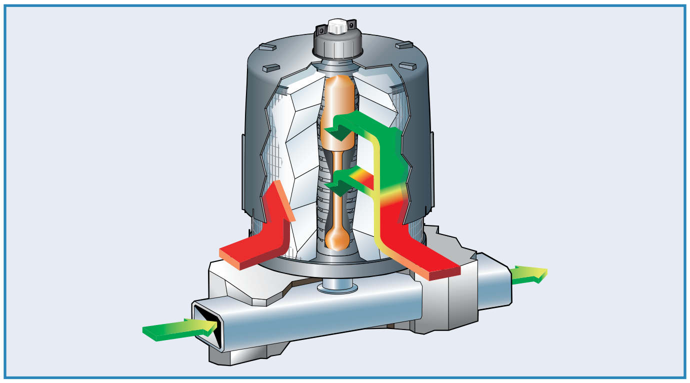
- Pretreating rainwater harvesting cisterns protects downstream pumps, filters, and fixtures from damage or clogging, and lowers cleaning and maintenance costs by keeping pollutants out of the cistern and mechanical system. Contech offers a number of pretreatment devices including CDS, StormFilter, and Jellyfish.
- For best performance, all rainwater harvesting cisterns should be leak tested and results documented using a positive pressure air test.
- All rainwater harvesting cisterns should include an inlet calming device that will introduce water to the cistern with little to no turbulence.



CDS

The CDS® hydrodynamic separator is the preferred rainwater harvesting pretreatment device. CDS is an underground stormwater treatment device that uses swirl concentration and continuous deflective separation to screen, separate and trap trash, debris, sediment, and hydrocarbons from runoff.

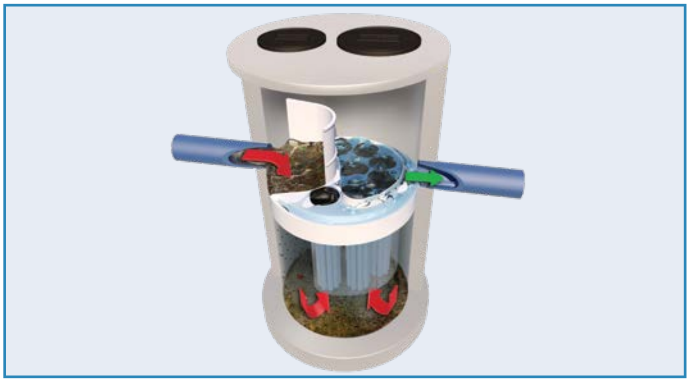
Learn More:
www.ContechES.com/cds



The Stormwater Management StormFilter®

The Stormwater Management StormFilter® uses rechargeable, media-filled cartridges that absorb and retain the most challenging target pollutants including dissolved metals, hydrocarbons, nutrients, metals and other common pollutants found in stormwater runoff.

Learn More:
www.ContechES.com/stormfilter

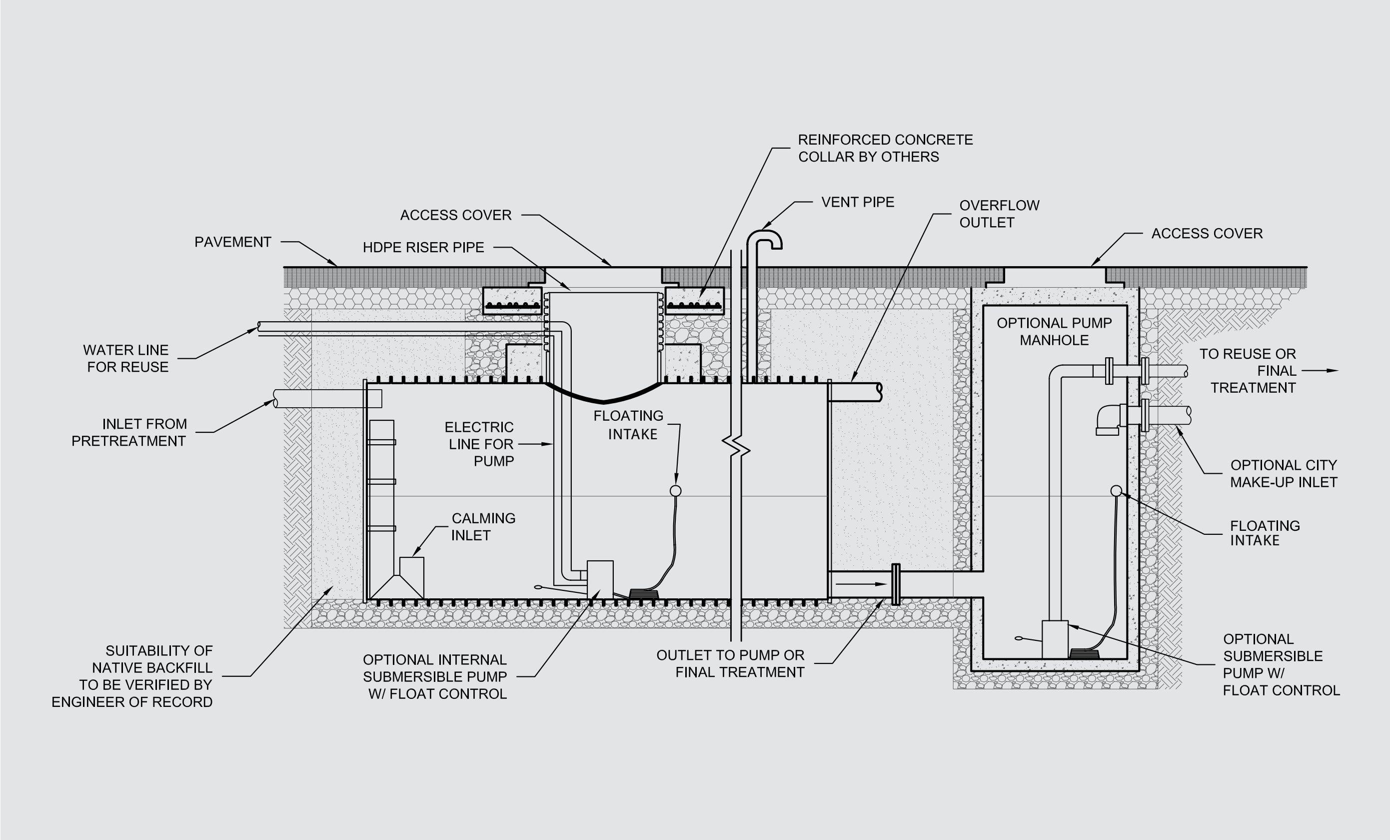


Jellyfish®

The Jellyfish® Filter is an engineered stormwater quality treatment technology featuring pretreatment and membrane filtration in a compact stand-alone treatment system, which removes a high level and a wide variety of stormwater pollutants.

Learn More:
www.ContechES.com/jellyfish

Typical Underground Cistern Components



DuroMaxx® Rainwater Harvesting Cistern Certifications

Multiple cistern layouts are available. All cisterns are tested for watertightness prior to shipment.

- IAPMO IGC 329 Certified
- Uniform Plumbing Code (UPC®)
- City of Los Angeles RR Approval RR 5726

Each DuroMaxx Rainwater Harvesting Cistern is custom built per the site requirements.

From inlet and outlet stub placement and size to access riser height, each cistern is designed to fit the site and provide the most economical storage solution.

Each cistern is ready to accept internal components such as pumps and level sensors or these components can be placed in a downstream wet well. Contech Design Engineers can also assist in designing each cistern to help you meet local requirements.

Rainwater Harvesting helps meet runoff reduction requirements

Rainwater Harvesting Cistern Options

Cistern sizes for every site ...

Contech offers a variety of standard cistern sizes designed to accommodate a variety of storage requirements. Cistern storage volumes range from 2,000 – 22,500 gallons, and multiple cisterns can be connected using a small diameter manifold. Custom cistern sizes are also available.

NOMINAL VOLUME (GAL)	NOMINAL DIAMETER (IN)	LENGTH (FT)	PICK WEIGHT (LB)	TOTAL VOLUME (GAL)
2,000	60	16	1,250	2,280
3,000		24	1,750	3,420
4,500		32	2,000	4,560
6,500		48	2,750	6,840
3,000	72	16	1,750	3,281
4,500		24	2,250	4,922
6,500		32	2,750	6,563
9,500		48	4,000	9,844
4,000	84	16	2,250	4,465
6,500		24	2,750	6,697
8,500		32	3,250	8,929
13,000		48	4,500	13,394
5,500	96	16	2,500	5,830
8,500		24	3,250	8,744
11,500		32	4,000	11,659
17,000		48	5,250	17,489
14,000	108	30	4,250	14,277
19,000		40	5,250	19,036
16,500	120	29	4,750	16,503
20,000		36	5,500	20,486
22,500		40	6,000	22,762

* Custom cistern sizes available. Please contact Contech at 800-338-1122.

A partner you can rely on

STORMWATER SOLUTIONS

PIPE SOLUTIONS

STRUCTURES SOLUTIONS

Few companies offer the wide range of high-quality stormwater resources you can find with us — state-of-the-art products, decades of expertise, and all the maintenance support you need to operate your system cost-effectively.

THE CONTECH WAY

Contech® Engineered Solutions provides innovative, cost-effective site solutions to engineers, contractors, and developers on projects across North America. Our portfolio includes bridges, drainage, erosion control, retaining wall, sanitary sewer and stormwater management products.

TAKE THE NEXT STEP

For more information: www.ContechES.com

CONTECH® ENGINEERED SOLUTIONS

Get social with us:

800-338-1122 | www.ContechES.com

EXHIBIT 5
Proposed Irrigation Study by Mia Lehrer and Associates (MLA) (Landscape Architect)

Ridge 1 flat area drip zones
Water Use Calculations

WATER EFFICIENT LANDSCAPE WORKSHEET

Landscape Area (LA)	
Regular	267,585
Special	0
TOTAL	**267,585**

Reference Evapotranspiration (Eto)	
Reference Site	Eto
Los Angeles	50.1

Conversion Factor (to Gallons/SF)
0.62

Evapotranspiration Adjustment Factor (ETAF)
0.45

Estimated Applied Water Use (EAWU):

REGULAR LANDSCAPE AREAS

Hydrozone No.	Hydrozone Description	Hydrozone Area (FT^2)	Plant Factor (PF)	Irrigation Method	Irrigation Efficiency (IE)	ETAF (PF/IE)	(ETAF x Area)	Estimated Total Water Use (Gallons) (ETWU)
H-1	Low Water Usage	18,579	0.2	Drip	0.81	0.25	4,587.41	142,494.05
H-2	Low Water Usage	162,874	0.2	Spray	0.75	0.27	43,433.07	1,349,117.92
H-3	Medium Water Usage	25,459	0.5	Drip	0.81	0.62	15,715.43	488,152.75
H-4	Medium Water Usage	51,913	0.5	Spray	0.75	0.67	34,608.67	1,075,014.40
H-WF-1	Water Feature	8,760	1	Recirculating	1	1.00	8,760.00	272,103.12
	Total Area	**267,585**			**TOTALS**	**267585**	**107,104.57**	**3,326,882.24**

SPECIAL LANDSCAPE AREAS - IRRIGATED BY <*source*>

Hydrozone No.	Hydrozone Description	Hydrozone Area (FT^2)	Plant Factor (PF)	Irrigation Method	Irrigation Efficiency (IE)	ETAF (PF/IE)	(ETAF x Area)	Estimated Total Water Use (ETWU)
H-4	Native Planting (L)	0	0.2	DRIP	0.81	0.25	0.00	0.00
H-5	Bioswale (L)	0	0.2	DRIP	0.81	0.25	0.00	0.00
H-6	Demo Garden (M)	0	0.5	DRIP	0.81	0.62	0.00	0.00
H-7	Roof Garden (L)	0	0.2	DRIP	0.81	0.25	0.00	0.00
	Total Area	**0**			**TOTALS**	**0**	**0.00**	**0.00**

ETAF Calculations	
Regular Landscape Areas	
Average ETAF	**0.40**
All Landscape Areas	
Sitewide ETAF	**0.40**

ESTIMATED TOTAL WATER USE (ETWU)	**3,326,882**	**GALLONS**
MAXIMUM ALLOW WATER ALLOWANCE (MAWA)	**3,740,276**	**GALLONS**

Ridge 1 flat area drip zones
Water Use Calculations

Monthly Estimated Total Water Use (ETWU):

JANUARY	Reference Eto	2.2					
Hydrozone No.	Hydrozone Description	Hydrozone Area (FT^2)	Plant Factor (PF)	Irrigation Method	Irrigation Efficiency (IE)	Hydrozone Water Use	
H-1	Low Water Usage	18579	0.2	Drip	0.81	6,257.22	
H-2	Low Water Usage	162874	0.2	Spray	0.75	59,242.70	
H-3	Medium Water Usage	25459	0.5	Drip	0.81	21,435.85	
H-WF-1	Water Feature	8760	1	Recirculating	1	11,948.64	
SLA							
H-4	Native Planting (L)	0	0.2	DRIP	0.81	-	
H-5	Bioswale (L)	0	0.2	DRIP	0.81	-	
H-6	Demo Garden (M)	0	0.5	DRIP	0.81	-	
H-7	Roof Garden (L)	0	0.2	DRIP	0.81	-	
					TOTAL	**98,884.42**	GALLONS

FEBRUARY	Reference Eto	2.7					
Hydrozone	Hydrozone	Hydrozone Area	Plant Factor	Irrigation Method	Irrigation Efficiency	Hydrozone	
H-1	Low Water Usage	18579	0.2	Drip	0.81	7,679.32	
H-2	Low Water Usage	162874	0.2	Spray	0.75	72,706.95	
H-3	Medium Water Usage	25459	0.5	Drip	0.81	26,307.63	
H-WF-1	Water Feature	8760	1	Recirculating	1	14,664.24	
SLA							
H-4	Native Planting (L)	0	0.2	DRIP	0.81	-	
H-5	Bioswale (L)	0	0.2	DRIP	0.81	-	
H-6	Demo Garden (M)	0	0.5	DRIP	0.81	-	
H-7	Roof Garden (L)	0	0.2	DRIP	0.81	-	
					TOTAL	**121,358.15**	GALLONS

Ridge 1 flat area drip zones
Water Use Calculations

MARCH	Reference Eto	3.7					
Hydrozone	Hydrozone	Hydrozone Area	Plant Factor	Irrigation Method	Irrigation Efficiency	Hydrozone	
H-1	Low Water Usage	18579	0.2	Drip	0.81	10,523.51	
H-2	Low Water Usage	162874	0.2	Spray	0.75	99,635.45	
H-3	Medium Water Usage	25459	0.5	Drip	0.81	36,051.20	
H-WF-1	Water Feature	8760	1	Recirculating	1	20,095.44	
SLA							
H-4	Native Planting (L)	0	0.2	DRIP	0.81	-	
H-5	Bioswale (L)	0	0.2	DRIP	0.81	-	
H-6	Demo Garden (M)	0	0.5	DRIP	0.81	-	
H-7	Roof Garden (L)	0	0.2	DRIP	0.81	-	
					TOTAL	**166,305.61**	GALLONS

APRIL	Reference Eto	4.7					
Hydrozone	Hydrozone	Hydrozone Area	Plant Factor	Irrigation Method	Irrigation Efficiency	Hydrozone	
H-1	Low Water Usage	18579	0.2	Drip	0.81	13,367.71	
H-2	Low Water Usage	162874	0.2	Spray	0.75	126,563.96	
H-3	Medium Water Usage	25459	0.5	Drip	0.81	45,794.77	
H-WF-1	Water Feature	8760	1	Recirculating	1	25,526.64	
SLA							
H-4	Native Planting (L)	0	0.2	DRIP	0.81	-	
H-5	Bioswale (L)	0	0.2	DRIP	0.81	-	
H-6	Demo Garden (M)	0	0.5	DRIP	0.81	-	
H-7	Roof Garden (L)	0	0.2	DRIP	0.81	-	
					TOTAL	**211,253.07**	GALLONS

Ridge 1 flat area drip zones
Water Use Calculations

MAY	Reference Eto	5.5				
Hydrozone	Hydrozone	Hydrozone Area	Plant Factor	Irrigation Method	Irrigation Efficiency	Hydrozone
H-1	Low Water Usage	18579	0.2	Drip	0.81	15,643.06
H-2	Low Water Usage	162874	0.2	Spray	0.75	148,106.76
H-3	Medium Water Usage	25459	0.5	Drip	0.81	53,589.62
H-WF-1	Water Feature	8760	1	Recirculating	1	29,871.60
SLA						
H-4	Native Planting (L)	0	0.2	DRIP	0.81	-
H-5	Bioswale (L)	0	0.2	DRIP	0.81	-
H-6	Demo Garden (M)	0	0.5	DRIP	0.81	-
H-7	Roof Garden (L)	0	0.2	DRIP	0.81	-
					TOTAL	**247,211.04** GALLONS

JUNE	Reference Eto	5.8				
Hydrozone	Hydrozone	Hydrozone Area	Plant Factor	Irrigation Method	Irrigation Efficiency	Hydrozone
H-1	Low Water Usage	18579	0.2	Drip	0.81	16,496.32
H-2	Low Water Usage	162874	0.2	Spray	0.75	72,706.95
H-3	Medium Water Usage	25459	0.5	Drip	0.81	26,307.63
H-WF-1	Water Feature	8760	1	Recirculating	1	14,664.24
SLA						
H-4	Native Planting (L)	0	0.2	DRIP	0.81	-
H-5	Bioswale (L)	0	0.2	DRIP	0.81	-
H-6	Demo Garden (M)	0	0.5	DRIP	0.81	-
H-7	Roof Garden (L)	0	0.2	DRIP	0.81	-
					TOTAL	**130,175.14** GALLONS

JULY	Reference Eto	6.2				
Hydrozone	Hydrozone	Hydrozone Area	Plant Factor	Irrigation Method	Irrigation Efficiency	Hydrozone
H-1	Low Water Usage	18579	0.2	Drip	0.81	17,633.99
H-2	Low Water Usage	162874	0.2	Spray	0.75	166,956.71
H-3	Medium Water Usage	25459	0.5	Drip	0.81	60,410.12
H-WF-1	Water Feature	8760	1	Recirculating	1	33,673.44
SLA						
H-4	Native Planting (L)	0	0.2	DRIP	0.81	-
H-5	Bioswale (L)	0	0.2	DRIP	0.81	-
H-6	Demo Garden (M)	0	0.5	DRIP	0.81	-
H-7	Roof Garden (L)	0	0.2	DRIP	0.81	-
					TOTAL	**278,674.26** GALLONS

Ridge 1 flat area drip zones
Water Use Calculations

AUGUST	Reference Eto	5.9					
Hydrozone	Hydrozone	Hydrozone Area	Plant Factor	Irrigation Method	Irrigation Efficiency	Hydrozone	
H-1	Low Water Usage	18579	0.2	Drip	0.81	16,780.74	
H-2	Low Water Usage	162874	0.2	Spray	0.75	158,878.16	
H-3	Medium Water Usage	25459	0.5	Drip	0.81	57,487.05	
H-WF-1	Water Feature	8760	1	Recirculating	1	32,044.08	
SLA							
H-4	Native Planting (L)	0	0.2	DRIP	0.81	-	
H-5	Bioswale (L)	0	0.2	DRIP	0.81	-	
H-6	Demo Garden (M)	0	0.5	DRIP	0.81	-	
H-7	Roof Garden (L)	0	0.2	DRIP	0.81	-	
					TOTAL	**265,190.02**	GALLONS
SEPTEMBER	Reference Eto	5					
Hydrozone	Hydrozone	Hydrozone Area	Plant Factor	Irrigation Method	Irrigation Efficiency	Hydrozone	
H-1	Low Water Usage	18579	0.2	Drip	0.81	14,220.96	
H-2	Low Water Usage	162874	0.2	Spray	0.75	134,642.51	
H-3	Medium Water Usage	25459	0.5	Drip	0.81	48,717.84	
H-WF-1	Water Feature	8760	1	Recirculating	1	27,156.00	
SLA							
H-4	Native Planting (L)	0	0.2	DRIP	0.81	-	
H-5	Bioswale (L)	0	0.2	DRIP	0.81	-	
H-6	Demo Garden (M)	0	0.5	DRIP	0.81	-	
H-7	Roof Garden (L)	0	0.2	DRIP	0.81	-	
					TOTAL	**224,737.31**	GALLONS
OCTOBER	Reference Eto	3.9					
Hydrozone No.	Hydrozone Description	Hydrozone Area (FT^2)	Plant Factor (PF)	Irrigation Method	Irrigation Efficiency (IE)	Hydrozone Water Use	
H-1	Low Water Usage	18579	0.2	Drip	0.81	11,092.35	
H-2	Low Water Usage	162874	0.2	Spray	0.75	105,021.16	
H-3	Medium Water Usage	25459	0.5	Drip	0.81	37,999.91	
H-WF-1	Water Feature	8760	1	Recirculating	1	21,181.68	
SLA							
H-4	Native Planting (L)	0	0.2	DRIP	0.81	-	
H-5	Bioswale (L)	0	0.2	DRIP	0.81	-	
H-6	Demo Garden (M)	0	0.5	DRIP	0.81	-	
H-7	Roof Garden (L)	0	0.2	DRIP	0.81	-	
					TOTAL	**175,295.10**	GALLONS

Ridge 1 flat area drip zones
Water Use Calculations

NOVEMBER	Reference Eto	2.6				
Hydrozone No.	Hydrozone Description	Hydrozone Area (FT^2)	Plant Factor (PF)	Irrigation Method	Irrigation Efficiency (IE)	Hydrozone Water Use
H-1	Low Water Usage	18579	0.2	Drip	0.81	7,394.90
H-2	Low Water Usage	162874	0.2	Spray	0.75	70,014.10
H-3	Medium Water Usage	25459	0.5	Drip	0.81	25,333.28
H-WF-1	Water Feature	8760	1	Recirculating	1	14,121.12
SLA						
H-4	Native Planting (L)	0	0.2	DRIP	0.81	-
H-5	Bioswale (L)	0	0.2	DRIP	0.81	-
H-6	Demo Garden (M)	0	0.5	DRIP	0.81	-
H-7	Roof Garden (L)	0	0.2	DRIP	0.81	-
					TOTAL	**116,863.40** GALLONS

DECEMBER	Reference Eto	1.9				
Hydrozone No.	Hydrozone Description	Hydrozone Area (FT^2)	Plant Factor (PF)	Irrigation Method	Irrigation Efficiency (IE)	Hydrozone Water Use
H-1	Low Water Usage	18579	0.2	Drip	0.81	5,403.97
H-2	Low Water Usage	162874	0.2	Spray	0.75	51,164.15
H-3	Medium Water Usage	25459	0.5	Drip	0.81	18,512.78
H-WF-1	Water Feature	8760	1	Recirculating	1	10,319.28
SLA						
H-4	Native Planting (L)	0	0.2	DRIP	0.81	-
H-5	Bioswale (L)	0	0.2	DRIP	0.81	-
H-6	Demo Garden (M)	0	0.5	DRIP	0.81	-
H-7	Roof Garden (L)	0	0.2	DRIP	0.81	-
					TOTAL	**85,400.18** GALLONS

ATTACHMENT 1
Geotechnical Engineer Investigation Proposed Academic and Athletic Development by Geotechnologies, Inc dated July 2,2019 and revised June 20, 2020

July 2, 2019
Revised April 28, 2020
File Number 21796

Harvard-Westlake School
3700 Coldwater Canyon Avenue
Studio City, California 91604

Attention: David Weil

Subject: Geotechnical Engineering Investigation
Proposed Academic and Athletic Development
4141 Whitsett Avenue, Studio City, California

Dear Mr. Weil:

This letter transmits the Geotechnical Engineering Investigation for the subject site prepared by Geotechnologies, Inc. This report provides geotechnical recommendations for the development of the site, including earthwork, seismic design, retaining walls, excavations, shoring and foundation design. Engineering for the proposed project should not begin until approval of the geotechnical investigation is granted by the local building official. Significant changes in the geotechnical recommendations may result due to the building department review process.

The validity of the recommendations presented herein is dependent upon review of the geotechnical aspects of the project during construction by this firm. The subsurface conditions described herein have been projected from limited subsurface exploration and laboratory testing. The exploration and testing presented in this report should in no way be construed to reflect any variations which may occur between the exploration locations or which may result from changes in subsurface conditions.

Should you have any questions please contact this office.

Respectfully submitted,
GEOTECHNOLOGIES, INC.

GREGORIO VARELA
R.C.E. 81201

GV:km

Distribution: (4) Addressee

Email to: [DWeil@hw.com]

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GEOTECHNICAL ENGINEERING INVESTIGATION
PROPOSED ACADEMIC AND ATHLETIC DEVELOPMENT
4141 WHITSETT AVENUE
STUDIO CITY, CALIFORNIA

INTRODUCTION

This report presents the results of the geotechnical engineering investigation performed on the subject site. The purpose of this investigation was to identify the distribution and engineering properties of the earth materials underlying the site, and to provide geotechnical recommendations for the design of the proposed development.

This investigation included excavation of five exploratory borings and four Cone Penetration Test soundings (CPT's), collection of representative samples, laboratory testing, engineering analysis, review of published geologic data, review of available geotechnical engineering information and the preparation of this report. The exploratory excavation locations are shown on the enclosed Site Plan. The results of the exploration and the laboratory testing are presented in the Appendix of this report.

This office had previously performed geotechnical investigations at the subject site, as part of previously proposed developments. A total of twenty-two exploratory excavations were performed in 2000, 2007 and 2016, as part of these previous investigations. Information obtained from these previous exploratory excavations has been considered in the preparation of this report. The location of these previous exploratory excavations is shown in the enclosed Site Plan; logs of the previous excavations may be found in the Appendix of this report.

PROPOSED DEVELOPMENT

Information concerning the proposed development was furnished by the client. In addition, the Entitlement Application Project Design Package, dated March 5, 2019, was reviewed for the preparation of this investigation.

The proposed development consists of the construction of a gymnasium, an underground parking garage to be overlain by athletic fields, an underground water storage tank to be overlain by tennis courts, and a swimming pool complex. In addition to the proposed structures, miscellaneous spectator bleachers, walkways and athletic fields, are also being proposed.

The majority of the proposed gymnasium structure will be serviced by a subterranean basement, while the rest of this structure will be built at-grade. The finished floor elevation of the proposed basement will be elevation 609 feet. Similarly, the finished floor elevation of the subterranean parking garage will also be elevation 609 feet. The bottom of the proposed underground water storage tank is expected to extend to elevation 610 feet. The bottom of the proposed pool is expected to extend to elevation 614 feet. The enclosed Site Plan and Cross Section A-A' and B-B' show the anticipated location, alignment, and depth of the proposed development.

It is anticipated that grading will consist of excavations to depths ranging between 15 and 24 feet for construction of the proposed subterranean basement and parking level, underground water storage tank, foundation elements, and for the recommended removal and recompaction.

Any changes in the design of the project or location of any structure, as outlined in this report, should be reviewed by this office. The recommendations contained in this report should not be considered valid until reviewed and modified or reaffirmed, in writing, subsequent to such review.

SITE CONDITIONS

The subject site consists of the Weddington Golf and Tennis complex, located at 4141 Whitsett Avenue, in the Studio City area of the City of Los Angeles, California. The subject site is bounded by Valley Spring Lane to the north, Whitsett Avenue to the east, and the Los Angeles River flood control channel to the south and west. The subject site is shown relative to nearby topographic features in the enclosed Vicinity Map.

The majority of the subject site is roughly level, with a total relief of approximately 6 feet. South of the site, a 10 to 15 foot high, 2:1 slope descends towards the Los Angeles River channel. There is an existing level area approximately 25 feet wide adjacent to the vertical channel walls. The site's topography is illustrated in the enclosed Cross Sections.

Vegetation on the site consists of grasses, shrubs and trees in landscaped areas. Drainage is by sheetflow along the existing contours generally southward, or towards area drains.

GEOTECHNICAL EXPLORATION

FIELD EXPLORATION

The site was explored on May 8 and 9, and June 3, 2019 by excavating five exploratory borings and performing four Cone Penetration Test sounding (CPT). The borings were excavated to depths ranging between 55 and 65 feet below grade with the aid of a drilling machine using 8-inch diameter hollowstem augers. The CPT's were conducted to depths between 56.76 and 64.94 feet below grade. The borings and CPT's locations are shown on the enclosed Site Plan, and interpretation of the geologic materials encountered is provided in the enclosed Boring Logs, A-Plates, and CPT Data Logs in the Appendix. For continuity purposes, the borings were labeled Borings B7 through B8.

This firm had previously conducted three geotechnical explorations at the site, on March 30 and 31, 2000, on June 4, 6, and 12, 2007 (our File No. 20255), and on September 29 and 30, 2016 (our File No. 21311). A total of twenty-one exploratory borings, one test pit, and two CPT's were excavated as part of the previous explorations. The borings varied in depth from 30 to 65 feet below the existing site grade, and the test pit was excavated to a depth of 6 feet. The CPT's were conducted to a depth of 57.41 and 50.20 feet below grade. These previous excavation locations are also shown in the enclosed Site Plan. The logs of these previous excavations are also included in the Appendix for reference.

The location of exploratory excavations was determined from hardscape features shown on the attached Site Plan. The location of the exploratory excavations should be considered accurate only to the degree implied by the method used.

GEOLOGIC MATERIALS

Fill Material

Fill materials were encountered during exploration to depths between zero and 7 feet below the existing ground surface. The fill consists of sandy silt, silty sand, sandy clay and clayey sand, which range from light brown to dark brown in color, and are slightly moist to moist, medium dense to dense, and fine to coarse grained.

Native Soils

The native soils underlying the site consist of silty sand, clayey silt, silty clay, clayey sand, sandy silt and sand, which range from light brown to grey to dark brown, and are slightly moist to wet, medium to very dense, or medium firm to stiff, and fine to coarse grained. The native earth materials consist of alluvial sediments deposited by river and stream action typical to this area of the San Fernando Valley.

Bedrock

Bedrock was encountered below the native soils in some of the exploratory borings, at depths ranging from approximately 42½ to 56½ feet below the existing site grade. The bedrock consists of shale, siltstone, sandstone and mudstone of the Miocene Monterey formation. The bedrock is light brown to gray to grayish-green to black, moist to very moist, and moderately hard to very hard. More detailed profiles of the earth materials may be obtained from the individual boring logs.

Groundwater

Groundwater was encountered during exploration, to depths ranging between 24½ and 49½ feet below grade. The historically highest groundwater level for the site was established by review of California Geological Survey Seismic Hazard Zone Report of the Van Nuys Quadrangle, Plate 1.2 entitled "Historically Highest Ground Water Contours" (2005). Review of this plate indicates that the historically highest groundwater level at the site is at the ground surface. A copy of this plate has been enclosed herein.

Fluctuations in the level of groundwater may occur due to variations in rainfall, temperature, and other factors not evident at the time of the measurements reported herein. Fluctuations also may occur across the site. High groundwater levels can result in changed conditions.

Caving

Caving could not be directly observed during excavation of the borings due to the type of excavation equipment utilized. Caving was not experienced during excavation of the test pit. Based on the experience of this firm, large diameter excavations below the groundwater table may experience caving.

SEISMIC EVALUATION

REGIONAL GEOLOGIC SETTING

The subject property is located in the Transverse Ranges Geomorphic Province. The Transverse Ranges are characterized by roughly east-west trending mountains and the northern and southern boundaries are formed by reverse fault scarps. The convergent deformational features of the Transverse Ranges are a result of north-south shortening due to plate tectonics. This has resulted in local folding and uplift of the mountains along with the propagation of thrust faults (including blind thrusts). The intervening valleys have been filled with sediments derived from the bordering mountains.

REGIONAL FAULTING

Based on criteria established by the California Division of Mines and Geology (CDMG) now called California Geologic Survey (CGS), Faults may be categorized as Holocene-active, Pre-Holocene faults, and Age-undetermined faults. Holocene-active faults are those which show evidence of surface displacement within the last 11,700 years. Pre-Holocene faults are those that have not moved in the past 11,700 years. Age-undetermined faults are faults where the recency of fault movement has not been determined.

Buried thrust faults are faults without a surface expression but are a significant source of seismic activity. They are typically broadly defined based on the analysis of seismic wave recordings of hundreds of small and large earthquakes in the southern California area. Due to the buried nature of these thrust faults, their existence is usually not known until they produce an earthquake. The risk for surface rupture potential of these buried thrust faults is inferred to be low (Leighton, 1990). However, the seismic risk of these buried structures in terms of recurrence and maximum potential magnitude is not well established. Therefore, the potential for surface rupture on these surface-verging splays at magnitudes higher than 6.0 cannot be precluded.

SEISMIC HAZARDS AND DESIGN CONSIDERATIONS

The primary geologic hazard at the site is moderate to strong ground motion (acceleration) caused by an earthquake on any of the local or regional faults. The potential for other earthquake-induced hazards was also evaluated including surface rupture, liquefaction, dynamic settlement, inundation and landsliding.

Surface Rupture

In 1972, the Alquist-Priolo Special Studies Zones Act (now known as the Alquist-Priolo Earthquake Fault Zoning Act) was passed into law. As revised in 2018, the Act defines “Holocene-active” Faults utilizing the same aging criteria as that used by California Geological Survey (CGS). However, established state policy has been to zone only those faults which have direct evidence of movement within the last 11,700 years. It is this recency of fault movement that the CGS considers as a characteristic for faults that have a relatively high potential for ground rupture in the future.

CGS policy is to delineate a boundary from 200 to 500 feet wide on each side of the Holocene-Active fault trace based on the location precision, the complexity, or the regional significance of the fault. If a site lies within an Earthquake Fault Zone, a geologic fault rupture investigation must be performed that demonstrates that the proposed building site is not threatened by surface displacement from the fault before development permits may be issued.

Ground rupture is defined as surface displacement which occurs along the surface trace of the causative fault during an earthquake. Based on research of available literature and results of site reconnaissance, no known Holocene-active or Pre-Holocene faults underlie the subject site. In addition, the subject site is not located within an Alquist-Priolo Earthquake Fault Zone. Based on these considerations, the potential for surface ground rupture at the subject site is considered low.

Liquefaction

Liquefaction is a phenomenon in which saturated silty to cohesionless soils below the groundwater table are subject to a temporary loss of strength due to the buildup of excess pore pressure during cyclic loading conditions such as those induced by an earthquake. Liquefaction-related effects include loss of bearing strength, amplified ground oscillations, lateral spreading, and flow failures.

Liquefaction typically occurs in areas where groundwater is less than 50 feet from the surface, and where the soils are composed of poorly consolidated, fine to medium-grained sand. In addition to the necessary soil conditions, the ground acceleration and duration of the earthquake must also be of a sufficient level to initiate liquefaction.

The Seismic Hazards Zone Map of the Van Nuys Quadrangle by the State of California (CDMG, 1998), indicates that the subject site is located within an area designated as "Liquefiable." This determination is based on groundwater depth records, soil type and distance to a fault capable of producing a substantial earthquake. A copy of this map is provided in the Appendix.

Liquefaction analyses were performed utilizing Standard Penetration Test (SPT) data collected in Borings B7, B8, B9, B10 and B11, laboratory test data, and CPT's 1 through 6. The CPT's were performed adjacent to the borings, for the purpose of comparison and correlation of soil data.

Groundwater was encountered in the borings at depths between 24½ and 49½ feet below the ground surface. According to the Seismic Hazard Zone Report of Van Nuys 7½-Minute Quadrangle (CDMG, 2005), the historic high groundwater level for the subject site was at the surface. The enclosed liquefaction analyses take into consideration the historically highest groundwater level at the ground surface (depth = 0), as well as a current groundwater level of 24½ feet.

Section 11.8.3 of ASCE 7-16 indicates that the potential for liquefaction shall be evaluated utilizing an acceleration consistent with the MCE_G PGA. Utilizing the OSHPD seismic utility program, this corresponds to a PGA_M of 0.95g. The USGS Probabilistic Seismic Hazard Deaggregation program (USGS, 2014) indicates a PGA of 0.87g (2 percent in 50 years ground motion) and a modal magnitude of 6.9 for the site. The liquefaction potential evaluations were performed by utilizing a magnitude 6.9 earthquake, and a peak horizontal acceleration of 0.95g.

Standard Penetration Test (SPT) – Liquefaction Analysis

Site-specific liquefaction analyses were performed following the Recommended Procedures for Implementation of CDMG Special Publication 117, Guidelines for Analyzing and Mitigating Liquefaction in California (Martin and Lew, 1999). Recommendations provided in CGS Special Publication 117A were also incorporated in to the analysis (CDMG, 2008), as were recommendations from EERI Monograph (MNO-12) by (Idriss and Boulanger, 2008).

The enclosed “Liquefaction Evaluation” analyses are based on Borings B7, B8, B9, B10 and B11. Standard Penetration Test (SPT) data were collected at 5-foot intervals. Samples of the collected materials were conveyed to the laboratory for testing and analysis. The percent passing a Number 200 sieve, Atterberg Limits, and the plasticity index (PI) of representative samples of the soils encountered in the exploratory borings are presented on the enclosed E-Plates and F-Plates.

Based on CGS Special Publication 117A (CDMG, 2008) and (Bray and Sancio, 2006), the vast majority of liquefaction hazards are associated with sandy soils and silty soils of low plasticity. Furthermore, soils having a PI greater than 18 exhibit clay-like behavior, and the liquefaction potential of these soils are considered to be low. The results of Atterberg Limits testing (shown on Plate F) indicate that some of soil layers below the subject site have PI greater than 18. Therefore, these soils are not considered prone to liquefaction, and the analysis of these soil layers was turned off in the liquefaction susceptibility columns.

The liquefaction analyses indicate that factors of safety against liquefaction are below 1.3 for some of the soil layers and/or lenses encountered in borings B7, B8, B9, B10 and B11. These potentially liquefiable layers occur from 0 to 20 feet, and 27½ to 50 feet. The factor of safety against liquefaction is defined as the ratio of the cyclic stress ratio to cause liquefaction to the earthquake-induced cyclic stress ratio. Therefore, the liquefaction analyses indicate these soil layers and/or lenses may liquefy in the event of an earthquake on a local or regional fault.

The liquefaction analyses are based on SPT data and in-situ samples collected every 5 feet. Therefore, the liquefaction potential of soils between sample points is not well defined. Cone Penetration Testing (CPT) provides a continuous profiling of the underlying earth materials based on correlations between cone tip resistance and friction ratio. Liquefaction analyses based on the three CPTs are discussed in the following section.

Cone Penetration Test (CPT) – Liquefaction Analysis

CPT data were analyzed utilizing the liquefaction assessment software CLiq v.2.0.0.6.92 (Geologismiki, 2006). The analyses are based on published articles by (Robertson and Wride, 1998) and (Youd et al. 2001). The program estimates the grain characteristics directly from the CPT data and incorporates the interpreted results into an evaluation of their resistance to cyclic loading.

The liquefaction analyses of the CPTs indicate some of the soil layers and/or lenses at varying depths below the ground surface would be susceptible to liquefaction. Based on the analyses, the potentially liquefiable soils occur throughout the soil column. The shallowest potentially liquefiable soils would occur at a depth just below the ground surface, while the deepest encountered at a depth of approximately 65 feet. The potentially liquefiable layers and/or lenses are between approximately a few inches and 3 feet in thickness. It is noted the basement excavation is considered in the CPT liquefaction analyses. Therefore, shallower liquefiable layers and/or lenses may exist.

Dynamic Settlement

Liquefaction settlement analyses have been performed utilizing the results of the liquefaction analysis based on SPT blow count data and the CPT sounding. The settlement analyses take into account the grading recommendations provided in following sections.

Based on the enclosed SPT liquefaction settlement analyses, total settlement at the existing ground surface due to liquefaction could be expected to range between 1.60 and 2.77 inches. Utilizing the CPT data, total settlement at the existing ground surface due to liquefaction could be expected to range between 0.54 to 2.71 inches.

According to (Martin and Lew, 1999), the differential settlement used in foundation design should be up to two-thirds of the total settlement. However, where at least two site-specific liquefaction analyses are conducted, the City of Los Angeles permits that the differential seismically induced settlement be taken as no less than one half of the maximum total calculated settlement, or 1.39 inches. The differential settlement would be expected to occur over a distance of 30 feet.

Surface Manifestation

It has been shown in studies by O'Rourke and Pease (1997) and Youd and Garris (1995), building upon work by Ishihara (1985), that the visible effects of liquefaction on the ground surface are only manifested if the relative and absolute thicknesses of liquefiable soils to overlying non-liquefiable surface material fall within a certain range. Surface manifestations of liquefaction include phenomena such as sand boils.

The liquefaction analyses indicate relative thicknesses of liquefiable to non-liquefiable soils that are within the bounds where surface manifestations have been observed during past earthquakes. According to (Boulanger and Idriss, 2008), “damage from liquefaction is seldom, however, due to sand boils themselves, but rather due to the loss of strength and stiffness in the soils that have liquefied and the associated ground deformations that ensue.”

The potentially liquefiable soils below the site occur in layers and/or lenses that are not laterally extensive throughout the site. Provided the recommendations presented herein are implemented during design and construction of the proposed structure, the potential for surface manifestations of liquefaction affecting the proposed structure is considered to be low.

Lateral Spreading

Lateral spreading is the most pervasive type of liquefaction-induced ground failure. During lateral spread, blocks of mostly intact, surficial soil displace downslope or towards a free face along a shear zone that has formed within the liquefied sediment. According to the procedure provided by Bartlett, Hansen, and Youd, “Revised Multilinear Regression Equations for Prediction of Lateral Spread Displacement”, ASCE, Journal of Geotechnical Engineering, Vol. 128, No. 12, December 2002, when the saturated cohesionless sediments with $(N_1)_{60} > 15$, significant displacement is not likely for $M < 8$ earthquakes.

The saturated cohesionless sediments underlying the site have corrected $(N_1)_{60}$ value greater than 15. According to the USGS Probabilistic Seismic Hazard Deaggregation program (USGS, 2008), the modal predominant earthquake magnitude (M_W) for the site is 6.9. In addition, the potentially liquefiable layer consists of a stratified layer, which is not expected to be continuous throughout the site. Therefore, the potential for lateral spread is considered to be remote for the subject site.

Tsunamis, Seiches and Flooding

Tsunamis are large ocean waves generated by sudden water displacement caused by a submarine earthquake, landslide, or volcanic eruption. The site is high enough and far enough from the ocean to preclude being prone to hazards of a tsunami.

Seiches are oscillations generated in enclosed bodies of water which can be caused by ground shaking associated with an earthquake. Review of the County of Los Angeles Flood and Inundation Hazards Map, Leighton (1990), indicates the site lies within mapped inundation boundaries due to a seiche or a breached upgradient reservoir. A determination of whether a higher site elevation would remove the site from the potential inundation zones is beyond the scope of this investigation.

Landsliding

The probability of seismically-induced landslides occurring on the site is considered to be low due to the general lack of elevation difference across or adjacent to the site.

CONCLUSIONS AND RECOMMENDATIONS

Based upon the exploration, laboratory testing, and research, it is the finding of Geotechnologies, Inc. that construction of the proposed structures is considered feasible from a geotechnical engineering standpoint provided the advice and recommendations presented herein are followed and implemented during construction.

During exploration, fill materials were observed to extend up to a depth of 7 feet. The existing fill materials are unsuitable for support of new foundations, but may be reused for the preparation of a compacted fill pad. Groundwater was observed during exploration to depths ranging between 24½ and 49½ feet below the existing grade. The historically highest groundwater level for the site is reported to be at the ground surface.

The proposed structures will be subject to static and seismically induced settlement. Based on the enclosed SPT and CPT liquefaction analyses, seismically induced settlement is anticipated from the ground surface. Removal and recompaction of the existing upper soils layer will be required to reduce the anticipated settlement to a level which will be tolerable for a mat foundation system. For the at-grade portion of the proposed gymnasium, and also for the pool structure, the soils located within the building area shall be removed and recompacted to a minimum depth of 15 feet below the existing grade. In addition, the compacted fill should extend horizontally beyond the edge of the foundation, for a distance equal to the thickness of compacted fill installed below the bottom of the foundations.

For the proposed subterranean garage, the subterranean basement below the gymnasium, and for the underground stormwater retention tank, the soils located within their building area shall be removed and recompacted to a depth of 5 feet below the bottom of the foundations. For these subterranean structures, a horizontal over-excavation beyond the edge of the proposed foundations is not necessary.

If the soils removal and recompaction recommended above are performed, it is anticipated that seismically induced settlement between 0.54 and 2.77 inches could potentially occur as a result of liquefaction. Such settlements are typically most damaging when the settlements are differential in nature across the length of structures. Seismically induced differential settlement is anticipated to be on the order of 1.39 inches. Additionally, the structures will be subject to static settlement. The total static settlement is not expected to exceed ½-inch, while the static differential settlement is not expected to exceed ¼-inch. The static and seismically-induced settlements are additive. Based on the anticipated settlement, it is recommended that the proposed structures be supported on a mat foundation system, bearing in a newly placed compacted fill pad.

In accordance with the City of Los Angeles requirements, where elements of a proposed development extend below the historically highest groundwater level, these structural elements should either be designed to resist potential hydrostatic forces, or a permanent dewatering system should be installed so that external water pressure does not develop against the proposed retaining walls and mat footing.

At the site, the historically highest groundwater level has been determined to be at the ground surface. This firm recommends that the elements of the proposed structure which will extend below the existing ground surface, such as retaining walls and foundation systems, be designed to resist the potential hydrostatic forces. This will require that the proposed subterranean retaining walls are designed for hydrostatic pressure, and the mat foundations are designed to resist hydrostatic uplift. The hydrostatic pressure and uplift shall be based on the historically highest groundwater level, which is at the ground surface.

Foundations for small outlying structures that are not intended for human occupancy, such as privacy walls, bleachers, canopies and trash enclosures, which will not be tied-in to the proposed structures may be supported on conventional foundations bearing in native alluvial soils and/or properly placed compacted fill.

The validity of the conclusions and design recommendations presented herein is dependent upon review of the geotechnical aspects of the proposed construction by this firm. The subsurface conditions described herein have been projected from excavations on the site as indicated and should in no way be construed to reflect any variations which may occur between these excavations or which may result from changes in subsurface conditions. Any changes in the design, as outlined in this report, should be reviewed by this office. The recommendations contained herein should not be considered valid until reviewed and modified or reaffirmed subsequent to such review.

2019 CALIFORNIA BUILDING CODE SEISMIC PARAMETERS

According to Table 20.3-1 presented in ASCE 7-16, the subject site is classified as Site Class F due to the liquefiable nature of the underlying soils. For Site Class F soils, ASCE 7-16 requires that a site-specific response spectrum evaluation be conducted. However, according to Section 20.3.1 of ASCE 7-16 (site class definition for Site Class F) the following exception is provided under Site Classification F:

EXCEPTION: *For structures having fundamental periods of vibration equal to or less than 0.5 s, site-response analysis is not required to determine spectral accelerations for liquefiable soils. Rather, a site class is permitted to be determined in accordance with Section 20.3 and the corresponding values of* F_a *and* F_v *determined from Tables 11.4-1 and 11.4-2.*

The soils underlying the subject site do not fall under any other characteristics of Site Class F, but fall within the characteristics of Site Class D. In addition, it is anticipated that the proposed structures will have a fundamental period of vibration of less than 0.5 second. Therefore, the subject site may be classified as Site Class D, which corresponds to a "Stiff Soil" Profile in accordance with the ASCE 7 standard.

2019 CALIFORNIA BUILDING CODE SEISMIC PARAMETERS	
Site Class	D **(Limited to structures with a fundamental period of vibration equal or less than 0.5 second)**
Mapped Spectral Acceleration at Short Periods (S_S)	2.059g
Site Coefficient (F_a)	1.0
Maximum Considered Earthquake Spectral Response for Short Periods (S_{MS})	2.059g
Five-Percent Damped Design Spectral Response Acceleration at Short Periods (S_{DS})	1.373g
Mapped Spectral Acceleration at One-Second Period (S_1)	0.737g
Site Coefficient (F_v)	1.7*
Maximum Considered Earthquake Spectral Response for One-Second Period (S_{M1})	1.253g*
Five-Percent Damped Design Spectral Response Acceleration for One-Second Period (S_{D1})	0.835g*

* *According to ASCE 7-16, a Long Period Site Coefficient (F_v) of 1.7 may be utilized provided that the value of the Seismic Response Coefficient (C_s) is determined by Equation 12.8-2 for values of $T \leq 1.5T_s$ and taken as equal to 1.5 times the value computed in accordance with either Equation 12.8-3 for $T_L \geq T > 1.5T_s$ or equation 12.8-4 for $T > T_L$. Alternatively, a site-specific ground motion hazard analysis may be performed in accordance with ASCE 7-16 Section 21.1 and/or a ground motion hazard analysis in accordance with ASCE 7-16 Section 21.2 to determine ground motions for any structure.*

EXPANSIVE SOILS

The onsite geologic materials are in the low expansion range. The Expansion Index was found to be between 17 and 35 for representative bulk samples.

WATER-SOLUBLE SULFATES

The Portland cement portion of concrete is subject to attack when exposed to water-soluble sulfates. Usually the two most common sources of exposure are from soil and marine environments.

The sources of natural sulfate minerals in soils include the sulfates of calcium, magnesium, sodium, and potassium. When these minerals interact and dissolve in subsurface water, a sulfate concentration is created, which will react with exposed concrete. Over time sulfate attack will destroy improperly proportioned concrete well before the end of its intended service life.

The water-soluble sulfate content of the onsite geologic materials was tested by California Test 417. The water-soluble sulfate content was determined to be less than 0.1% percentage by weight for the soils tested. Based on American Concrete Institute (ACI) Standard 318-08, the sulfate exposure is considered to be negligible for geologic materials with less than 0.1% and Type I cement may be utilized for concrete foundations in contact with the site soils.

DEWATERING

Groundwater was observed during exploration to depths ranging between 24½ and 49½ feet below the existing grade. The bottom of the proposed structures, including their foundations elements, is not anticipated to extend below a depth of 19 feet below grade, with soils removals extending 5 feet below this depth. Based on this consideration, implementation of a dewatering program is not anticipated to be needed during construction.

The historically highest groundwater level for the site is reported to be at the ground surface. The City of Los Angeles, Department of Building and Safety, requires that this historically highest groundwater level be considered when designing the underground portion of the proposed structures. In lieu of installing a permanent dewatering system, this firm recommends that the subterranean elements of the proposed structure are designed for an undrained condition with full hydrostatic pressure.

METHANE ZONES

Based on review of the NavigateLA Website, developed by the City of Los Angeles, Bureau of Engineering, Department of Public Works, the subject site is not located within the limits of a City of Los Angeles Methane Zone or Methane Buffer Zone.

GRADING GUIDELINES

Site Preparation

- A thorough search should be made for possible underground utilities and/or structures. Any existing or abandoned utilities or structures located within the footprint of the proposed grading should be removed or relocated as appropriate.

- All vegetation, existing fill, and soft or disturbed geologic materials should be removed from the areas to receive controlled fill. All existing fill materials and any disturbed geologic materials resulting from grading operations shall be completely removed and properly recompacted prior to foundation excavation.

- Any vegetation or associated root system located within the footprint of the proposed structures should be removed during grading.

- Subsequent to the indicated removals, the exposed grade shall be scarified to a depth of six inches, moistened to optimum moisture content, and recompacted in excess of the minimum required comparative density.

- The excavated areas shall be observed by the geotechnical engineer prior to placing compacted fill.

Recommended Overexcavation

In order to reduce the potential seismically-induced settlement, it is recommended that the existing upper soils be removed and recompacted for support of the proposed structures. For the at-grade portion of the proposed gymnasium, and also for the pool structure, the soils located within these building areas shall be removed to a minimum depth of 15 feet below the existing grade. In addition, the removal should extend horizontally beyond the edge of the foundation, for a distance equal to the thickness of compacted fill to be installed below the bottom of the foundations.

For the proposed subterranean garage, the subterranean basement below the gymnasium, and for the underground stormwater retention tank, the soils located within their building area shall be removed to a depth of 5 feet below the bottom of the foundations. A horizontal over-excavation beyond the edge of the proposed foundations is not necessary.

It is very important that the position of the proposed structures is accurately located so that the limits of the graded area are accurate and the grading operation proceeds efficiently. Since the site grading will result in a net export, it is recommended that the dryer, sandier or more granular materials be segregated and utilized for the preparation of the recommended compacted fill pad. The more clayey, wetter and/or expansive materials should be exported. It is recommended that the soils to be utilized for the preparation of a compacted fill pad are well blended to reduce their overall expansion index and moisture.

Compaction

All fill should be mechanically compacted in layers not more than 8 inches thick. The City of Los Angeles Department of Building and Safety requires a minimum comparative compaction of 95 percent of the laboratory maximum density where the soils to be utilized in the fill have less than 15 percent finer than 0.005 millimeters. Fill materials having more than 15 percent finer than 0.005 millimeters may be compacted to a minimum of 90 percent of the maximum density. Comparative compaction is defined, for purposes of these guidelines, as the ratio of the in-place density to the maximum density as determined by applicable ASTM testing.

Field observation and testing shall be performed by a representative of the geotechnical engineer during grading to assist the contractor in obtaining the required degree of compaction and the proper moisture content. Where compaction is less than required, additional compactive effort shall be made with adjustment of the moisture content, as necessary, until a minimum of 90 percent (or 95 percent for cohesionless soils having less than 15 percent finer than 0.005 millimeters) compaction is obtained.

Acceptable Materials

The excavated onsite materials are considered satisfactory for reuse in the controlled fills as long as any debris and/or organic matter is removed. Any imported materials shall be observed and tested by the representative of the geotechnical engineer prior to use in fill areas. Imported materials should contain sufficient fines so as to be relatively impermeable and result in a stable subgrade when compacted. Any required import materials should consist of geologic materials with an expansion index of less than 50. The water-soluble sulfate content of the import materials should be less than 0.1% percentage by weight.

Imported materials should be free from chemical or organic substances which could affect the proposed development. A competent professional should be retained in order to test imported materials and address environmental issues and organic substances which might affect the proposed development.

Utility Trench Backfill

Utility trenches should be backfilled with controlled fill. The utility should be bedded with clean sands at least one foot over the crown. The remainder of the backfill may be onsite soil compacted to 90 percent (or 95 percent for cohesionless soils having less than 15 percent finer than 0.005 millimeters) of the laboratory maximum density. Utility trench backfill should be tested by representatives of this firm in accordance with the most recent revision of ASTM D-1557.

Wet Soils

At the time of exploration, the soils which will be exposed at the bottom of the excavations, and some of the soils to be used for the creation of a compacted fill pad, were well above optimum moisture content. It is anticipated that the excavated material to be placed as compacted fill, and the materials exposed at the bottom of excavated plane will require drying and aeration prior to recompaction.

Pumping (yielding or vertical deflection) of the high-moisture content soils at the bottom of the excavation planes may occur during operation of heavy equipment. Where pumping is encountered, angular minimum ¾-inch gravel should be placed and worked into the subgrade. The exact thickness of the gravel would be a trial and error procedure, and would be determined in the field. It would likely be on the order of 1 to 2 feet thick.

The gravel will help to densify the subgrade as well as function as a stabilization material upon which heavy equipment may operate. It is not recommended that rubber tire construction equipment attempt to operate directly on the pumping subgrade soils prior to placing the gravel. Direct operation of rubber tire equipment on the soft subgrade soils will likely result in excessive disturbance to the soils, which in turn will result in a delay to the construction schedule since those disturbed soils would then have to be removed and properly recompacted. Extreme care should be utilized to place gravel as the subgrade becomes exposed.

The simplest method to reduce the moisture content of the on-site soils would involve spreading out the soils in order to dry them naturally while the weather is warm and sunny. As an alternative, dry soils could be imported and used for one of two purposes. The existing saturated soils could be replaced by the dry soils, or the dry soils could be blended with the onsite soils in order to reduce the overall moisture content.

The use of lime or cement is also an acceptable method of reducing moisture content in soils. Lime or cement should be added to the soils at a minimum rate of 5 percent by weight. The lime or cement shall be thoroughly mixed and blended with the soils to be treated. A uniform distribution of the lime or cement within the treated soil is critical. If lime or cement will be utilized for the drying of soils near the subgrade of the structure, it is recommended that the entire building subgrade is treated in order to achieve a uniform and stable subgrade. This recommendation is intended to prevent the effects of possible hard versus soft areas.

The entire mixing operation should be completed within 72 hours of the initial use of lime or cement. The treated soil should be compacted to a minimum relative compaction of 90 percent (or 95 percent for cohesionless soils having less than 15 percent finer than 0.005 millimeters) of the laboratory maximum density for the mixed material. Final compaction should be completed within 36 hours of final mixing.

Shrinkage

Shrinkage results when a volume of soil removed at one density is compacted to a higher density. A shrinkage factor between 10 and 20 percent should be anticipated when excavating and recompacting the existing fill and underlying native geologic materials on the site to an average comparative compaction of 92 percent.

Weather Related Grading Considerations

When rain is forecast all fill that has been spread and awaits compaction shall be properly compacted prior to stopping work for the day or prior to stopping due to inclement weather. These fills, once compacted, shall have the surface sloped to drain to an area where water can be removed.

Temporary drainage devices should be installed to collect and transfer excess water to the street 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. Drainage should not be allowed to flow uncontrolled over any descending slope.

Work may start again, after a period of rainfall, once the site has been reviewed by a representative of this office. Any soils saturated by the rain shall be removed and aerated so that the moisture content will fall within three percent of the optimum moisture content.

Surface materials previously compacted before the rain shall be scarified, brought to the proper moisture content and recompacted prior to placing additional fill, if considered necessary by a representative of this firm.

Geotechnical Observations and Testing During Grading

Geotechnical observations and testing during grading are considered to be a continuation of the geotechnical investigation. It is critical that the geotechnical aspects of the project be reviewed by representatives of Geotechnologies, Inc. during the construction process. Compliance with the design concepts, specifications or recommendations during construction requires review by this firm during the course of construction. Any fill which is placed should be observed, tested, and verified if used for engineered purposes. Please advise this office at least twenty-four hours prior to any required site visit.

FOUNDATION DESIGN

Mat Foundation

Based on the anticipated static and seismically-induced settlement, it is the recommendation of this firm that the proposed structures are supported on a mat foundation system, bearing in a newly placed compacted fill pad.

It is anticipated that the proposed structures will have an average bearing pressure of less than 1,000 pounds per square foot. Foundation bearing pressure will vary across the mat footings, with the highest concentrated loads located at the central cores of the mat foundations.

Given the size of the proposed mat foundation, an average bearing pressure of 1,000 pounds per square foot is well below the allowable bearing pressures, with factor of safety well exceeding 3. For design purposes, an allowable bearing pressure of 2,000 pounds per square foot, with locally higher pressures up to 5,000 pounds per square foot may be utilized in the mat foundation design. The mat foundation may be designed utilizing a modulus of subgrade reaction of 250 pounds per cubic inch. This value is a unit value for use with a one-foot square footing. The modulus should be reduced in accordance with the following equation when used with larger foundations.

$$K = K_1 * [(B + 1) / (2 * B)]^2$$

where K = Reduced Subgrade Modulus
K1 = Unit Subgrade Modulus
B = Foundation Width (feet)

The bearing values indicated above are for the total of dead and frequently applied live loads, and may be increased by one third for short duration loading, which includes the effects of wind or seismic forces. Since the recommended bearing value is a net value, the weight of concrete in the foundations may be taken as 50 pounds per cubic foot and the weight of the soil backfill may be neglected when determining the downward load on the foundations.

Hydrostatic Considerations for Mat Foundations

The proposed mat foundation shall be waterproofed and designed to withstand the hydrostatic uplift pressure based on the historically highest water level, which is considered to be at the ground surface. The uplift pressure to be used in design should be 62.4(H) pounds per square foot. Since the historically highest groundwater level at the site is considered to be at the ground surface, "H" is the vertical distance between the bottom of the mat and the ground surface. The installation of uplift anchors may be necessary to provide resistance against the anticipated hydrostatic uplift pressures acting on the recommended mat foundations.

If necessary, uplift anchors may be designed to provide resistance against the anticipated hydrostatic uplift pressures acting on the recommended mat foundations. Uplift anchors should be a minimum of 12 inches in diameter, and should be embedded a minimum of 20 feet into the underlying native soils and/or bedrock. Preliminarily, it is assumed that pressure grouted anchors will be utilized. Uplift anchors may be designed using a frictional capacity of 2 kips per lineal foot.

Miscellaneous Conventional Foundations

The use of conventional foundations is limited to miscellaneous structures not intended for human occupancy, such as privacy walls, trash enclosures, canopies and bleachers, which will not be rigidly connected to the proposed apartment structure. Miscellaneous conventional foundations may bear in properly compacted fill, or may be deepened through any existing fill to bear in undisturbed native alluvial soils. Miscellaneous continuous foundations may be designed for a bearing capacity of 1,500 pounds per square foot, 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 material. No bearing capacity increases are recommended.

Since the recommended bearing capacity is a net value, the weight of concrete in the foundations may be taken as 50 pounds per cubic foot and the weight of the soil backfill may be neglected when determining the downward load on the foundations.

All continuous foundations should be reinforced with a minimum of four #4 steel bars. Two should be placed near the top of the foundation, and two should be placed near the bottom.

The recommendations provided herein for these miscellaneous structures are not intended to mitigate the effects of liquefaction. The client should be aware that a liquefaction event on the subject site could result in damage to these miscellaneous structures.

Lateral Design for Mat Foundation and Miscellaneous Conventional Foundations

Resistance to lateral loading may be provided by friction acting at the base of foundations and by passive earth pressure. An allowable coefficient of friction of 0.3 may be used with the dead load forces.

Passive geologic pressure for the sides of foundations poured against undisturbed or recompacted soil may be computed as an equivalent fluid having a density of 200 pounds per cubic foot with a maximum earth pressure of 2,000 pounds per square foot.

The passive and friction components may be combined for lateral resistance without reduction. A one-third increase in the passive value may be used for short duration loading such as wind or seismic forces.

Mat Foundation Settlement

Settlement of a mat foundation is expected to occur on application of loading. The maximum settlement is expected to occur below the central portion of the mat, and would not be expected to exceed ½-inch. The settlement along the edges of the mat is expected to be on the order of ¼-inch. Therefore, the differential settlement anticipated across the mat is not expected to exceed ¼-inch.

In addition to static settlement, the maximum total seismic settlement due to a major seismic event is expected to be on the order of 2.77 inches, and the anticipated seismically induced differential settlement is anticipated to be on the order of 1.39 inches. The static and seismic settlement reported herein are additive.

RETAINING WALL DESIGN

It is anticipated that retaining walls up to 15½ feet in height may be required for the proposed subterranean parking garage, subterranean basement, swimming pool and underground stormwater storage tank. As a precautionary measure, recommendations to aid in the design of retaining walls up to a height of 17 feet are provided herein. Retaining walls may be designed as indicated below, depending on whether the walls will be restrained or cantilevered. Retaining wall foundations may be designed in accordance with the provisions of the "Foundation Design" section of this report.

The historically highest groundwater level for the site is reported to be at the ground surface. Therefore, retaining wall extending below the ground surface shall be designed for an undrained condition with full hydrostatic pressure.

Additional pressure should be added for a surcharge condition due to vehicular traffic or adjacent structures. Based on review of the enclosed Site Plan, it is not anticipated that the proposed retaining walls will be surcharged by existing structures. However, vehicular traffic is expected in the vicinity of the proposed structure. For traffic surcharge, the upper 10 feet of any retaining wall adjacent to streets, driveways or parking areas should be designed to resist a uniform lateral pressure of 100 pounds per square foot, acting as a result of an assumed 300 pounds per square foot traffic surcharge. If the traffic is more than 10 feet from the retaining walls, the traffic surcharge may be neglected.

Cantilever Retaining Walls

Cantilever retaining walls supporting a level backslope may be designed utilizing a triangular distribution of active earth pressure. In addition, cantilever retaining walls extending below the ground surface shall be designed for an undrained condition with full hydrostatic pressure. Miscellaneous cantilever retaining walls to be built above grade, such as planter walls, do not require to be designed for an undrained condition, provided that a subdrain system is installed at their base. Cantilever retaining walls may be designed utilizing the following tables:

Height of Retaining Wall	Cantilever Retaining Wall **Below** the Ground Surface **without** Wall Subdrain System Triangular Distribution of At-Rest Earth Pressure
Up to 17 feet	93 pcf (including hydrostatic pressure)

Height of Retaining Wall	Miscellaneous Cantilever Retaining Wall **Above** the Ground Surface Level **with** Wall Subdrain System Triangular Distribution of At-Rest Earth Pressure
Up to 5 feet	30 pcf

For these equivalent fluid pressures to be valid, walls which are to be restrained at the top should be backfilled prior to the upper connection being made. Additional active pressure should be added for a surcharge condition due to sloping ground, vehicular traffic or adjacent structures.

Restrained Retaining Walls

Restrained subterranean retaining walls supporting a level back slope may be designed to resist a triangular distribution of earth pressure. It is recommended the walls be designed to resist the greater of the at-rest pressure, or the active pressure plus the seismic pressure, as discussed in the "Dynamic (Seismic) Earth Pressure" section below. Wall pressures are provided in the following table for hydrostatic design. Pressures for drained conditions are also provided for designs that incorporate a subdrain above the historic high water level.

RESTRAINED BASEMENT WALLS *(HYDROSTATIC DESIGN)*		
Height of Retaining Wall (feet)	**AT-REST EARTH PRESSURE (Pounds per Cubic Foot) Includes Hydrostatic Pressure of 62.4 pcf**	**ACTIVE EARTH PRESSURE *(To be Combined with Dynamic Seismic Earth Pressure) Includes Hydrostatic Pressure of 62.4 pcf**
Up to 17 feet	94	93

Additional active pressure should be added for a surcharge condition due to vehicular traffic, slopes, or adjacent structures.

Dynamic (Seismic) Earth Pressure

Retaining walls exceeding 6 feet in height shall be designed to resist the additional earth pressure caused by seismic ground shaking. A triangular pressure distribution should be utilized for the additional seismic loads, with an equivalent fluid pressure of 25 pounds per cubic foot. When using the load combination equations from the building code, the seismic earth pressure should be combined with the lateral active earth pressure for analyses of restrained basement walls under seismic loading condition. The dynamic earth pressure may be omitted where the retaining wall is 6 feet in height or less.

Surcharge from Adjacent Structures

The following surcharge equation provided in the LADBS Information Bulletin Document No. P/BC 2014-83, may be utilized to determine the surcharge loads on basement walls and shoring system from existing and proposed structures located within the 1:1 (h:v) surcharge influence zone of the excavation and basement.

Resultant lateral force: $R = (0.3*P*h^2)/(x^2+h^2)$

Location of lateral resultant: $d = x*[(x^2/h^2+1)*\tan^{-1}(h/x)-(x/h)]$

where:

R	=	resultant lateral force measured in pounds per foot of wall width.
P	=	resultant surcharge loads of continuous or isolated footings measured in pounds per foot of length parallel to the wall.
x	=	distance of resultant load from back face of wall measured in feet.
h	=	depth below point of application of surcharge loading to top of wall footing measured in feet.
d	=	depth of lateral resultant below point of application of surcharge loading measure in feet.
$\tan^{-1}(h/x)$	=	the angle in radians whose tangent is equal to h/x.

The structural engineer and shoring engineer may use this equation to determine the surcharge loads based on the loading of the adjacent structures located within the surcharge influence zone.

Retaining Wall Drainage

The installation of a retaining wall drainage system will only be necessary for miscellaneous cantilever retaining walls to be built above grade, such as planter walls. A drainage system is not required for retaining walls extending below grade, because they will be designed to resist hydrostatic pressure.

Retaining wall subdrains may consist of four-inch diameter perforated pipes, placed with perforations facing down. The pipe shall be encased in at least one-foot of gravel around the pipe, wrapped in filter fabric. The gravel may consist of three-quarter inch to one inch crushed rocks. As an alternative to the standard perforated subdrain pipe and gravel drainage system, the use of gravel pockets and weepholes is an acceptable drainage method. Weepholes shall be a minimum of 2 inches in diameter, placed at 8 feet on center along the base of the wall. Gravel pockets shall be a minimum of 1 cubic foot in dimension, and may consist of three-quarter inch to one inch crushed rocks, wrapped in filter fabric. A collector pipe shall be installed to direct collected waters to a suitable location.

Certain types of subdrain pipe are not acceptable to the various municipal agencies, it is recommended that prior to purchasing subdrainage pipe, the type and brand is cleared with the proper municipal agencies. Subdrainage pipes should outlet to an acceptable location. Some municipalities do not allow the use of flat-drainage products, such as Miradrain. The use of such a product should be researched with the building official. The City of Los Angeles only allows the use of flat drainage products when in conjunction with a conventional perforated subdrain pipe and gravel, or gravel pockets and weepholes.

Waterproofing

Moisture effecting retaining walls is one of the most common post construction complaints. Poorly applied or omitted waterproofing can lead to efflorescence or standing water inside the building. Efflorescence is a process in which a powdery substance is produced on the surface of the concrete by the evaporation of water. The white powder usually consists of soluble salts such as gypsum, calcite, or common salt. Efflorescence is common to retaining walls and does not affect their strength or integrity.

It is recommended that retaining walls be waterproofed. Waterproofing design and inspection of its installation is not the responsibility of the geotechnical engineer. A qualified waterproofing consultant should be retained in order to recommend a product or method which would provide protection to below grade walls.

Retaining Wall Backfill

Any required backfill should be mechanically compacted in layers not more than 8 inches thick, to at least 90 percent (or 95 percent for cohesionless soils having less than 15 percent finer than 0.005 millimeters) relative compaction, obtainable by the most recent revision of ASTM D 1557 method of compaction. Flooding should not be permitted. Compaction within 5 feet, measured horizontally, behind a retaining structure should be achieved by use of light weight, hand operated compaction equipment.

Proper compaction of the backfill will be necessary to reduce settlement of overlying walks and paving. Some settlement of required backfill should be anticipated, and any utilities supported therein should be designed to accept differential settlement, particularly at the points of entry to the structure.

TEMPORARY EXCAVATIONS

Excavations on the order of 15 to 24 feet in height may be anticipated for construction of the proposed subterranean retaining walls, mat foundations, and recommended grading. The excavations are expected to expose fill and dense native soils, which are suitable for vertical excavations up to 5 feet where not surcharged by adjacent traffic or structures. Vertical excavations exceeding 5 feet, or excavations which will be surcharged by adjacent traffic or structures should be shored.

Where sufficient space is available, temporary unsurcharged embankments could be cut at a uniform 1:1 (horizontal:vertical) slope gradient to a maximum depth of 25 feet. A uniform sloped excavation is sloped from bottom to top and does not have a vertical component.

Where sloped embankments are utilized, the tops of the slopes should be barricaded to prevent vehicles and storage loads near the top of slope within a horizontal distance equal to the depth of the excavation. If the temporary construction embankments are to be maintained during the rainy season, berms are strongly recommended along the tops of the slopes to prevent runoff water from entering the excavation and eroding the slope faces. Water should not be allowed to pond on top of the excavation nor to flow towards it.

Excavation Observations

It is critical that the soils exposed in the cut slopes are observed by a representative of Geotechnologies, Inc. during excavation so that modifications of the slopes can be made if variations in the geologic material conditions occur. Many building officials require that temporary excavations should be made during the continuous observations of the geotechnical engineer. All excavations should be stabilized within 30 days of initial excavation.

SHORING DESIGN

The following information on the design and installation of the shoring is as complete as possible at this time. It is suggested that a review of the final shoring plans and specifications be made by this office prior to bidding or negotiating with a shoring contractor be made.

The recommended method of shoring consists of steel soldier piles, placed in drilled holes and backfilled with concrete. As discussed below vibrating methods may also be utilized. The soldier piles may be designed as cantilevers or laterally braced utilizing drilled tie-back anchors or raker braces.

Soldier Piles – Drilled

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 earth materials. For design purposes, an allowable passive value for the earth materials below the bottom plane of excavation may be assumed to be 500 pounds per square foot per foot. To develop the full lateral value, provisions should be implemented to assure firm contact between the soldier piles and the undisturbed earth materials.

The frictional resistance between the soldier piles and retained earth material may be used to resist the vertical component of the anchor load. The coefficient of friction may be taken as 0.3 based on uniform contact between the steel beam and lean-mix concrete and retained earth. 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 500 pounds per square foot. The minimum depth of embedment for shoring piles is 5 feet below the bottom of the footing excavation, or 7 feet below the bottom of excavated plane, whichever is deeper.

Groundwater was encountered during exploration at depths ranging between 24½ and 49½ feet below the existing site grade. If the piles will extend below the groundwater level, caving of the saturated earth materials below the groundwater level may occur during drilling of piles. Casing or polymer drilling fluid may be required during drilling in order to maintain open shafts. If 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.

Piles placed below the water level will require the use of a tremie to place the concrete into the bottom of the hole. A tremie shall consist of a water-tight tube having a diameter of not less than 10 inches with a hopper at the top. The tube shall 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 shall 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 shall be closed at the start of the work to prevent water entering the tube and shall be entirely sealed at all times, except when the concrete is being placed. The tremie tube shall be kept full of concrete. The flow shall be continuous until the work is completed and the resulting concrete seal shall be monolithic and homogeneous. The tip of the tremie tube shall always be kept about five 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.

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

Soldier Piles – Vibration Method of Installation

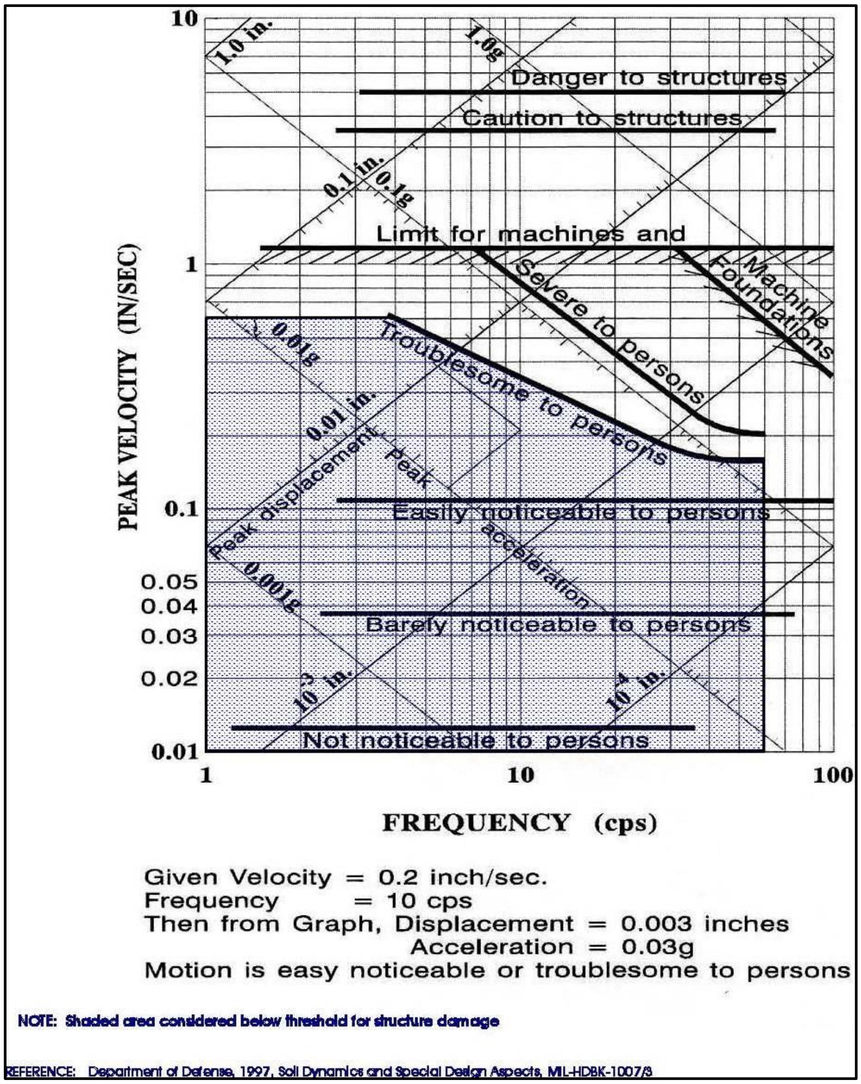
The vibration method of shoring pile installation is acceptable to this firm from a geotechnical standpoint provided the recommendations presented herein are implemented. When using the vibration method of installing the soldier beams, the minimum embedment depth shall be 10 feet below the lowest excavated plane. The available passive resistance of the pile may be determined using the diagonal length from the outer edges of opposite flange sections.

Predrilling may be utilized by the shoring contractor in order to vibrate and install the shoring beams to the design depths. However, the depth of the predrilled holes should not exceed the planned excavation depth, which for the project is expected to be a maximum of 20 feet below grade. In addition, it is recommended that the diameter of the predrilled holes does not exceed 75 percent of the depth of the web of the I-beam. When predrilling, the auger shall be backspun out of the pilot holes, leaving the soils in place. All shoring (predrilling, installation of shoring piles, and lagging) shall be performed under the continuous inspections by a deputy grading inspector of this firm.

It should be noted that Bedrock of the Monterey Formation was encountered at depths ranging between 42½ to 56½ feet below the existing grade. The shoring designer and contractor should be aware that, where the soldier beams will extend into this moderately hard to hard layer, predrilling will not be permitted at this depth. Vibrating soldier beams into the undisturbed Modelo Formation bedrock may be challenging.

The allowable level of vibration that results from the installation of the piles should not exceed a threshold where occupants of the nearby structures are disturbed, despite higher vibration tolerances that a building may endure without deformation. There is a relationship between particle velocity and vibration frequency that will occur due to the installation. A range of tolerable particle peak velocity and frequency of vibration is shown in the graph below. The shaded area on the graph is considered within acceptable limits to avoid damage to nearby structures. The acceptable limits should be measured at the neighboring structures.

The vibrations should be monitored with a seismograph during pile installation to detect the magnitude of vibration and oscillation experienced by the adjacent structure. The results should be recorded and provided to the owner. If, during installation, the vibrations exceed the range shown on the graph below, the shoring contractor should modify the installation procedure to reduce the values to the acceptable range.



Lagging

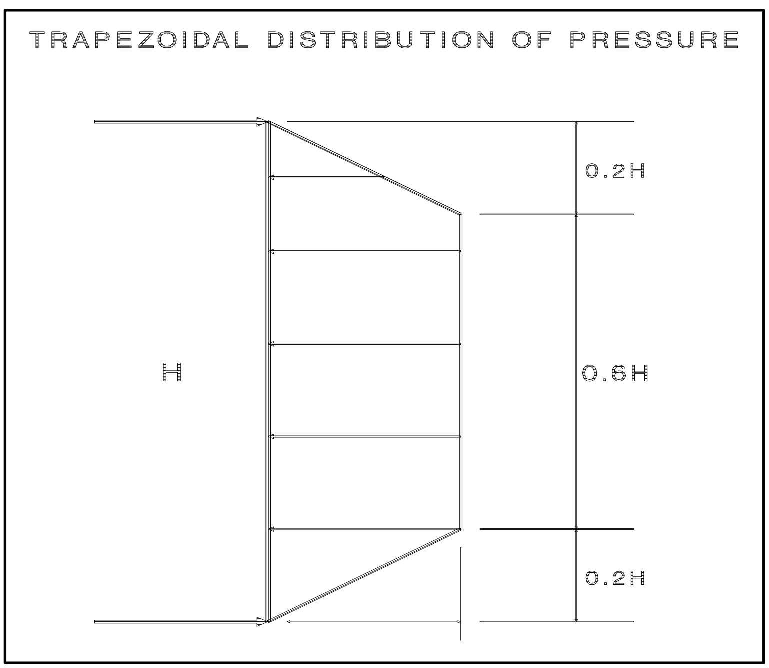
Soldier piles and anchors should be designed for the full anticipated pressures. Lagging will be required throughout the entire depth of the excavation. Due to arching in the geologic materials, the pressure on the lagging will be less. It is recommended that the lagging should be designed for the full design pressure but be limited to a maximum of 400 pounds per square foot. It is recommended that a representative of this firm observe the installation of lagging to insure uniform support of the excavated embankment.

Lateral Pressures

A triangular distribution of lateral earth pressure should be utilized for the design of cantilevered shoring system. A trapezoidal distribution of lateral earth pressure would be appropriate where shoring is to be restrained at the top by bracing or tie backs. The design of trapezoidal distribution of pressure is shown in the diagram below. Equivalent fluid pressures for the design of cantilevered and restrained shoring are presented in the following table:

Height of Shoring (feet)	Cantilever Shoring System Equivalent Fluid Pressure (pcf) Triangular Distribution of Pressure	Restrained Shoring System Lateral Earth Pressure (psf)* Trapezoidal Distribution of Pressure
Up to 20	30 pcf	19H psf
20 to 25	35 pcf	22H psf

*Where H is the height of the shoring in feet.



Where a combination of sloped embankment and shoring is utilized, the pressure will be greater and must be determined for each combination. Additional active pressures should be applied where the shoring will be surcharged by adjacent traffic or structures.

The upper ten feet of the retaining wall adjacent to streets, driveways or parking areas should be designed to resist a uniform lateral pressure of 100 pounds per square foot, acting as a result of an assumed 300 pounds per square foot surcharge behind the walls due to normal street traffic. If the traffic is kept back at least ten feet from the retaining walls, the traffic surcharge may be neglected.

Tied-Back Anchors

Tied-back anchors may be used to resist lateral loads. 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.

Drilled friction anchors may be designed for a skin friction of 350 pounds per square foot. Pressure grouted anchor may be designed for a skin friction of 2,000 pounds per square foot. Where belled anchors are utilized, the capacity of belled anchors may be designed by assuming the diameter of the bonded zone is equivalent to the diameter of the bell. Only the frictional resistance developed beyond the active wedge would be effective in resisting lateral loads.

It is recommended that at least 3 of the initial anchors have their capacities tested to 200 percent of their design capacities for a 24-hour period to verify their design capacity. The total deflection during this test should not exceed 12 inches. The anchor deflection should not exceed 0.75 inches during the 24 hour period, measured after the 200 percent load has been applied.

All 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.

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. Where satisfactory tests are not attained, the anchor diameter and/or length should be increased or additional anchors installed until satisfactory test results are obtained. The installation and testing of the anchors should be observed by the geotechnical engineer. Minor caving during drilling of the anchors should be anticipated.

Anchor Installation

Tied-back anchors may be installed between 20 and 45 degrees below the horizontal. Caving of the anchor shafts, particularly within sand deposits, should be anticipated and the following provisions should be implemented in order to minimize such caving. 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.

Deflection

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 estimated that the deflection could be on the order of one inch at the top of the shored embankment. If greater deflection occurs during construction, additional bracing may be necessary to minimize settlement of adjacent buildings and utilities in adjacent street and alleys. If desired to reduce the deflection, a greater active pressure could be used in the shoring design. Where internal bracing is used, the rakers should be tightly wedged to minimize deflection. The proper installation of the raker braces and the wedging will be critical to the performance of the shoring.

The City of Los Angeles Department of Building and Safety requires limiting shoring deflection to ½ inch at the top of the shored embankment where a structure is within a 1:1 (h:v) plane projected up from the base of the excavation. A maximum deflection of 1-inch has been allowed provided there are no structures within a 1:1 (h:v) plane drawn upward from the base of the excavation.

Monitoring

Because of the depth of the excavation, some mean 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. Also, some means of periodically checking the load on selected anchors will be necessary, where applicable.

Some movement of the shored embankments should be anticipated as a result of the relatively deep excavation. It is recommended that photographs of the existing buildings on the adjacent properties be made during construction to record any movements for use in the event of a dispute.

Shoring Observations

It is critical that the installation of shoring is observed by a representative of Geotechnologies, Inc. Many building officials require that shoring installation should be performed during continuous observation of a representative of the geotechnical engineer. The observations insure that the recommendations of the geotechnical report are implemented and so that modifications of the recommendations can be made if variations in the geologic material or groundwater conditions warrant. The observations will allow for a report to be prepared on the installation of shoring for the use of the local building official, where necessary.

SLABS ON GRADE

Outdoor Concrete Slabs

Outdoor concrete flatwork should be a minimum of 4 inches in thickness. Outdoor concrete flatwork should be cast over undisturbed alluvial soils or properly controlled fill materials. Any geologic materials loosened or over-excavated should be wasted from the site or properly compacted to 90 percent (or 95 percent for cohesionless soils having less than 15 percent finer than 0.005 millimeters) of the maximum dry density.

Outdoor flatwork should be reinforced with a minimum of #3 steel bars on 24-inch centers each way.

Design of Slabs That Receive Moisture-Sensitive Floor Coverings

Geotechnologies, Inc. does not practice in the field of moisture vapor transmission evaluation and mitigation. Therefore, it is recommended that a qualified consultant be engaged to evaluate the general and specific moisture vapor transmission paths and any impact on the proposed construction. The qualified consultant should provide recommendations for mitigation of potential adverse impacts of moisture vapor transmission on various components of the structure.

Where dampness would be objectionable, it is recommended that the floor slabs should be waterproofed. A qualified waterproofing consultant should be retained in order to recommend a product or method which would provide protection for concrete slabs-on-grade.

Concrete Crack Control

The recommendations presented in this report are intended to reduce the potential for cracking of concrete slabs-on-grade due to settlement. However, even where these recommendations have been implemented, foundations, stucco walls and concrete slabs-on-grade may display some cracking due to minor soil movement and/or concrete shrinkage. The occurrence of concrete cracking may be reduced and/or controlled by limiting the slump of the concrete used, proper concrete placement and curing, and by placement of crack control joints at reasonable intervals, in particular, where re-entrant slab corners occur.

For standard control of concrete cracking, a maximum crack control joint spacing of 12 feet should not be exceeded. Lesser spacings would provide greater crack control. Joints at curves and angle points are recommended. The crack control joints should be installed as soon as practical following concrete placement. Crack control joints should extend a minimum depth of one-fourth the slab thickness. Construction joints should be designed by a structural engineer.

Complete removal of the existing fill soils beneath outdoor flatwork such as walkways or patio areas, is not required. However, due to the rigid nature of concrete, some cracking, a shorter design life and increased maintenance costs should be anticipated. In order to provide uniform support beneath the flatwork it is recommended that a minimum of 12 inches of the exposed subgrade beneath the flatwork be scarified and recompacted to 90 percent (or 95 percent for cohesionless soils having less than 15 percent finer than 0.005 millimeters) relative compaction.

PAVEMENTS

Prior to placing paving, the existing grade should be scarified to a depth of 12 inches, moistened as required to obtain optimum moisture content, and recompacted to 90 percent(or 95 percent for cohesionless soils having less than 15 percent finer than 0.005 millimeters) of the maximum dry density as determined by the most recent revision of ASTM D 1557. The client should be aware that removal of all existing fill in the area of new paving is not required. However, pavement constructed in this manner will most likely have a shorter design life and increased maintenance costs. The following pavement sections are recommended:

Asphalt Paving Sections:

Service	Asphalt Pavement Thickness (Inches)	Base Course (Inches)
Passenger Vehicles	3	5
Light to Medium Trucks	4	7
Heavy Trucks and Fire Trucks	6	9

Concrete Paving Sections:

Service	Concrete Pavement Thickness (Inches)	Base Course (Inches)
Passenger Vehicles and Light to Medium Trucks	6	4
Heavy Trucks and Fire Trucks	7½	6

Aggregate base should be compacted to a minimum of 95 percent of the most recent revision of ASTM D 1557 laboratory maximum dry density. Base materials should conform to Sections 200-2.2 or 200-2.4 of the "Standard Specifications for Public Works Construction", (Green Book), latest edition.

For standard crack control maximum expansion joint spacing of 12 feet should not be exceeded. Lesser spacings would provide greater crack control. Joints at curves and angle points are recommended. Concrete paving should be reinforced with a minimum of #3 steel bars on 24-inch centers each way.

The performance of pavement is highly dependent upon providing positive surface drainage away from the edges. Ponding of water on or adjacent to pavement can result in saturation of the subgrade materials and subsequent pavement distress. If planter islands are planned, the perimeter curb should extend a minimum of 12 inches below the bottom of the aggregate base.

SITE DRAINAGE

Proper surface drainage is critical to the future performance of the project. Saturation of a soil can cause it to lose internal shear strength and increase its compressibility, resulting in a change in the designed engineering properties. Proper site drainage should be maintained at all times.

All site drainage, with the exception of any required to disposed of onsite by stormwater regulations, should be collected and transferred to the street in non-erosive drainage devices. The proposed structure should be provided with roof drainage. Discharge from downspouts, roof drains and scuppers should not be permitted on unprotected soils within five feet of the building perimeter. Drainage should not be allowed to pond anywhere on the site, and especially not against any foundation or retaining wall. Drainage should not be allowed to flow uncontrolled over any descending slope. Planters which are located within a distance equal to the depth of a retaining wall should be sealed to prevent moisture adversely affecting the wall. Planters which are located within five feet of a foundation should be sealed to prevent moisture affecting the earth materials supporting the foundation.

STORMWATER DISPOSAL

Recently regulatory agencies have been requiring the disposal of a certain amount of stormwater generated on a site by infiltration into the site soils. Increasing the moisture content of a soil can cause it to lose internal shear strength and increase its compressibility, resulting in a change in the designed engineering properties. This means that any overlying structure, including buildings, pavements and concrete flatwork, could sustain damage due to saturation of the subgrade soils. Structures serviced by subterranean levels could be adversely impacted by stormwater disposal by increasing the design fluid pressures on retaining walls and causing leaks in the walls. Proper site drainage is critical to the performance of any structure in the built environment.

Groundwater was encountered below the subject site at depths between 24½ and 49½ feet below grade. It is the opinion of this firm that this is water is perched on top of the underlying clay soils and bedrock, which are relatively impervious layers. On-site filtration of stormwater would acute the existing perched water condition. In addition, the native alluvial site soils are prone to liquefaction when saturated. Based on these considerations, on-site stormwater infiltration is not recommended for the subject site.

Where infiltration of stormwater into the subgrade soils is not advisable, most Building Officials have allowed the stormwater to be filtered through soils in planter areas. Once the water has been filtered through a planter it may be released into the storm drain system. It is recommended that overflow pipes are incorporated into the design of the discharge system in the planters to prevent flooding. In addition, the planters shall be sealed and waterproofed to prevent leakage. Please be advised that adverse impact to landscaping and periodic maintenance may result due to excessive water and contaminants discharged into the planters.

It is recommended that the design team (including the structural engineer, waterproofing consultant, plumbing engineer, and landscape architect) be consulted in regards to the design and construction of filtration systems.

DESIGN REVIEW

Engineering of the proposed project should not begin until approval of the geotechnical report by the Building Official is obtained in writing. Significant changes in the geotechnical recommendations may result during the building department review process.

It is recommended that the geotechnical aspects of the project be reviewed by this firm during the design process. This review provides assistance to the design team by providing specific recommendations for particular cases, as well as review of the proposed construction to evaluate whether the intent of the recommendations presented herein are satisfied.

CONSTRUCTION MONITORING

Geotechnical observations and testing during construction are considered to be a continuation of the geotechnical investigation. It is critical that this firm review the geotechnical aspects of the project during the construction process. Compliance with the design concepts, specifications or recommendations during construction requires review by this firm during the course of construction. All foundations should be observed by a representative of this firm prior to placing concrete or steel. Any fill which is placed should be observed, tested, and verified if used for engineered purposes. Please advise Geotechnologies, Inc. at least twenty-four hours prior to any required site visit.

If conditions encountered during construction appear to differ from those disclosed herein, notify Geotechnologies, Inc. immediately so the need for modifications may be considered in a timely manner.

It is the responsibility of the contractor to ensure that all excavations and trenches are properly sloped or shored. All temporary excavations should be cut and maintained in accordance with applicable OSHA rules and regulations.

EXCAVATION CHARACTERISTICS

The exploration performed for this investigation is limited to the geotechnical excavations described. Direct exploration of the entire site would not be economically feasible. The owner, design team and contractor must understand that differing excavation and drilling conditions may be encountered based on boulders, gravel, oversize materials, groundwater and many other conditions. Fill materials, especially when they were placed without benefit of modern grading codes, regularly contain materials which could impede efficient grading and drilling. Southern California sedimentary bedrock is known to contain variable layers which reflect differences in depositional environment. Such layers may include abundant gravel, cobbles and boulders. Similarly bedrock can contain concretions. Concretions are typically lenticular and follow the bedding. They are formed by mineral deposits. Concretions can be very hard. Excavation and drilling in these areas may require full size equipment and coring capability. The contractor should be familiar with the site and the geologic materials in the vicinity.

CLOSURE AND LIMITATIONS

The purpose of this report is to aid in the design and completion of the described project. Implementation of the advice presented in this report is intended to reduce certain risks associated with construction projects. The professional opinions and geotechnical advice contained in this report are sought because of special skill in engineering and geology and were prepared in accordance with generally accepted geotechnical engineering practice. Geotechnologies, Inc. has a duty to exercise the ordinary skill and competence of members of the engineering profession. Those who hire Geotechnologies, Inc. are not justified in expecting infallibility, but can expect reasonable professional care and competence.

The scope of the geotechnical services provided did not include any environmental site assessment for the presence or absence of organic substances, hazardous/toxic materials in the soil, surface water, groundwater, or atmosphere, or the presence of wetlands.

Proper compaction is necessary to reduce settlement of overlying improvements. Some settlement of compacted fill should be anticipated. Any utilities supported therein should be designed to accept differential settlement. Differential settlement should also be considered at the points of entry to the structure.

If corrosion sensitive improvements are planned, it is recommended that a comprehensive corrosion study should be commissioned. The study will develop recommendations to avoid premature corrosion of buried pipes and concrete structures in direct contact with the soils.

GEOTECHNICAL TESTING

Classification and Sampling

The soil is continuously logged by a representative of this firm and classified by visual examination in accordance with the Unified Soil Classification system. The field classification is verified in the laboratory, also in accordance with the Unified Soil Classification System. Laboratory classification may include visual examination, Atterberg Limit Tests and grain size distribution. The final classification is shown on the excavation logs.

Samples of the geologic materials encountered in the exploratory excavations were collected and transported to the laboratory. Undisturbed samples of soil are obtained at frequent intervals. Unless noted on the excavation logs as an SPT sample, samples acquired while utilizing a hollow-stem auger drill rig are obtained by driving a thin-walled, California Modified Sampler with successive 30-inch drops of a 140-pound hammer. The soil is retained in brass rings of 2.50 inches outside diameter and 1.00 inch in height. The central portion of the samples are stored in close fitting, waterproof containers for transportation to the laboratory. Samples noted on the excavation logs as SPT samples are obtained in accordance with the most recent revision of ASTM D 1586. Samples are retained for 30 days after the date of the geotechnical report.

Moisture and Density Relationships

The field moisture content and dry unit weight are determined for each of the undisturbed soil samples, and the moisture content is determined for SPT samples by the most recent revision of ASTM D 4959 or ASTM D 4643. This information is useful in providing a gross picture of the soil consistency between exploration locations and any local variations. The dry unit weight is determined in pounds per cubic foot and shown on the "Excavation Logs", A-Plates. The field moisture content is determined as a percentage of the dry unit weight.

Direct Shear Testing

Shear tests are performed by the most recent revision of ASTM D 3080 with a strain controlled, direct shear machine manufactured by Soil Test, Inc. or a Direct Shear Apparatus manufactured by GeoMatic, Inc. The rate of deformation is approximately 0.025 inches per minute. Each sample is sheared under varying confining pressures in order to determine the Mohr-Coulomb shear strength parameters of the cohesion intercept and the angle of internal friction. Samples are generally tested in an artificially saturated condition. Depending upon the sample location and future site conditions, samples may be tested at field moisture content. The results are plotted on the "Shear Test Diagram," B-Plates.

The most recent revision of ASTM 3080 limits the particle size to 10 percent of the diameter of the direct shear test specimen. The sheared sample is inspected by the laboratory technician running the test. The inspection is performed by splitting the sample along the sheared plane and observing the soils exposed on both sides. Where oversize particles are observed in the shear plane, the results are discarded and the test run again with a fresh sample.

Consolidation Testing

Settlement predictions of the soil's behavior under load are made on the basis of the consolidation tests using the most recent revision of ASTM D 2435. The consolidation apparatus is designed to receive a single one-inch high ring. Loads are applied in several increments in a geometric progression, and the resulting deformations are recorded at selected time intervals. Porous stones are placed in contact with the top and bottom of each specimen to permit addition and release of pore fluid. Samples are generally tested at increased moisture content to determine the effects of water on the bearing soil. The normal pressure at which the water is added is noted on the drawing. Results are plotted on the "Consolidation Test," C-Plates.

Expansion Index Testing

The expansion tests performed on the remolded samples are in accordance with the Expansion Index testing procedures, as described in the most recent revision of ASTM D 4829. The soil sample is compacted into a metal ring at a saturation degree of 50 percent. The ring sample is then placed in a consolidometer, under a vertical confining pressure of 1 lbf/square inch and inundated with distilled water. The deformation of the specimen is recorded for a period of 24 hour or until the rate of deformation becomes less than 0.0002 inches/hour, whichever occurs first. The expansion index, EI, is determined by dividing the difference between final and initial height of the ring sample by the initial height, and multiplied by 1,000. Results are presented on Plate D of this report.

Laboratory Compaction Characteristics

The maximum dry unit weight and optimum moisture content of a soil are determined by use of the most recent revision of ASTM D 1557. A soil at a selected moisture content is placed in five layers into a mold of given dimensions, with each layer compacted by 25 blows of a 10 pound hammer dropped from a distance of 18 inches subjecting the soil to a total compactive effort of about 56,000 pounds per cubic foot. The resulting dry unit weight is determined. The procedure is repeated for a sufficient number of moisture contents to establish a relationship between the dry unit weight and the water content of the soil. The data when plotted represent a curvilinear relationship known as the compaction curve. The values of optimum moisture content and modified maximum dry unit weight are determined from the compaction curve. Results are presented on Plate D of this report.

Grain Size Distribution

These tests cover the quantitative determination of the distribution of particle sizes in soils. Sieve analysis is used to determine the grain size distribution of the soil larger than the Number 200 sieve. The most recent revision of ASTM D 422 is used to determine particle sizes smaller than the Number 200 sieve. A hydrometer is used to determine the distribution of particle sizes by a sedimentation process. The grain size distributions are plotted on the E-Plates presented in the Appendix of this report.

Atterberg Limits

Depending on their moisture content, cohesive soils can be solid, plastic, or liquid. The water contents corresponding to the transitions from solid to plastic or plastic to liquid are known as the Atterberg Limits. The transitions are called the plastic limit and liquid limit. The difference between the liquid and plastic limits is known as the plasticity index. ASTM D 4318 is utilized to determine the Atterberg Limits. The results are shown on the enclosed F-Plates.

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