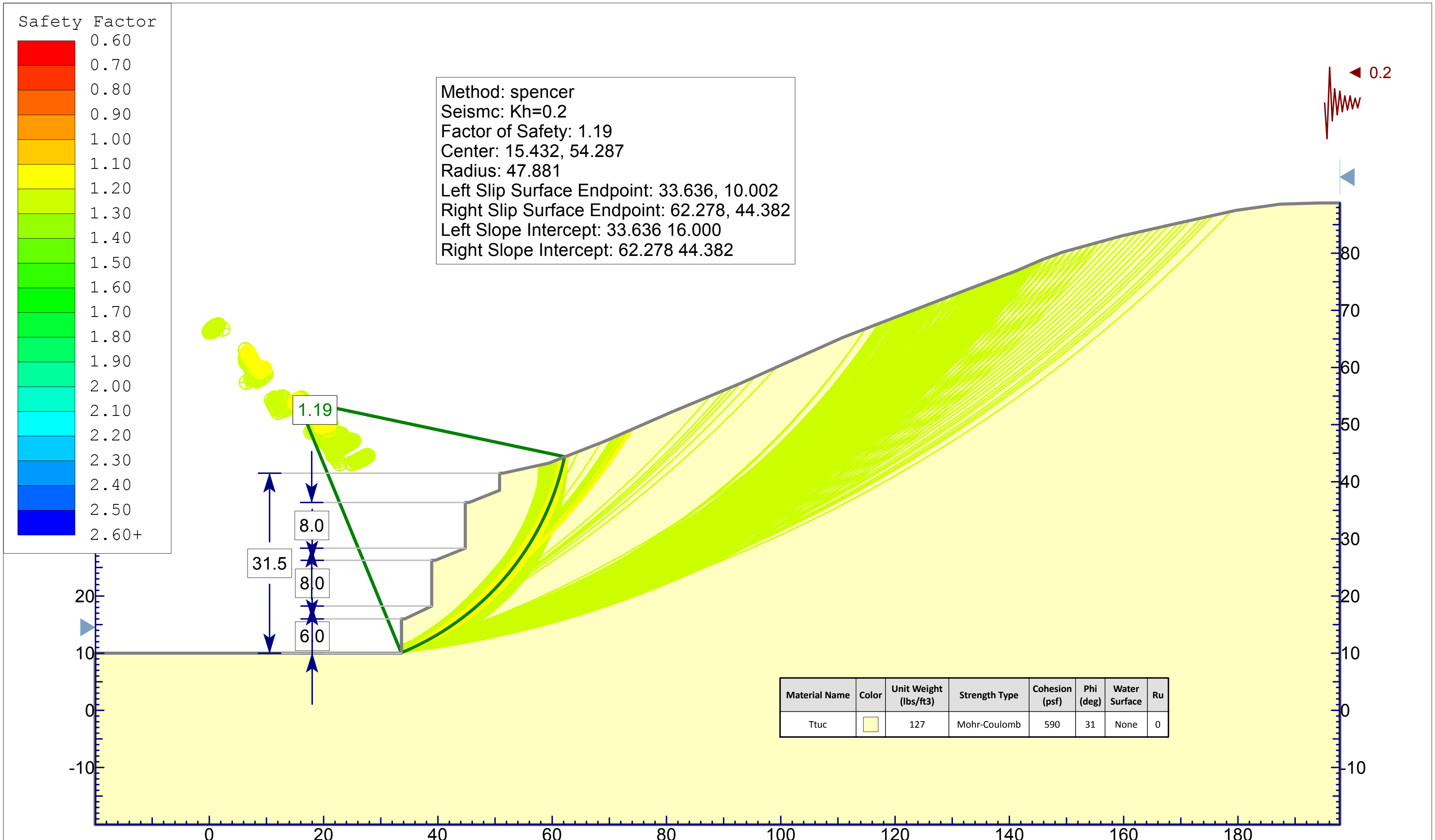


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Ru
Ttuc		127	Mohr-Coulomb	590	31	None	0

Project	SLIDE - An Interactive Slope Stability Program				
Analysis Description					
Drawn By	CHJ Consultants	*Author*	F.Yi	*Scale*	1:300
File Name	15473-3A Section D-D.slim	*Date*	6/22/2016	*Enclosure*	C-2

SLIDEINTERPRET 6.039

CHJ Consultants
1355 E. Cooley Drive, Suite C, Colton, CA 92324 ◆ Phone (909) 824-7311 ◆ Fax (909) 503-1136
15345 Anacapa Road, Suite D, Victorville, CA 92392 ◆ Phone (760) 243-0506 ◆ Fax (760) 243-1225
77-564A Country Club Drive, Suite 122, Palm Desert, CA 92211 ◆ Phone (760) 772-8234 ◆ Fax (909) 503-1136

July 26, 2016

Oakmont Senior Living
9249 Old Redwood Highway, Suite 200
Windsor, California 95492
Attention: Mr. Wayne Sant, Vice President, Development

Job No. 15473-3A

Subject: Addendum to Geotechnical Investigation Report
Response to Geotechnical Review Sheet Dated July 11, 2016
Proposed Oakmont of Agoura Hills Senior Facility
29353 Canwood Street
Agoura Hills, California

References: See Attached References Sheet

Dear Mr. Sant:

As requested, we have examined the review comments by GeoDynamics, Incorporated, prepared on behalf of the City of Agoura Hills and dated July 11, 2016. We provide our responses below. This letter addresses only the Report Review Comments. The reviewer's comments appear below in italics, followed by our response.

Report Review Comments

1. *The consultant should review development plans as they become available to verify compliance with recommendations in the above-referenced reports. A geotechnical map using the proposed grading plan as base map should be included. Cross-sections should be updated as necessary to reflect changes in the proposed grading relative to the current grading concept. Additional geotechnical recommendations should be provided as necessary.*

__Note:__ The reviewers appreciate that the consultant addressed this comment, but this comment should be addressed during the design stage of the project, when final development plans become available. Note that ALL geologic data - bedding attitudes in particular - should be plotted on the geologic map..

An updated geological and geotechnical map will be provided during the design stage of the project when final development plans become available.

2. *The consultant should discuss and evaluate the potential for interaction between closely located retaining walls (example: stacked retaining walls) using an appropriate method of analyses. Please note that the 1 :1 criterion is not acceptable for lateral surcharge unless substantiated with analyses and/or references.*
   __Note:__ Comment #6 of the Planning/Feasibility Comments does not address this comment. This comment is about the potential for lateral surcharge on the lower retaining wall due to the foundation load of the upper retaining wall.

As mentioned in our previous response letter, the *cut slope* is self-stable and satisfies required minimum factor of safety values for both static and seismic conditions. Because of that, it is the opinion of this firm that it is not necessary in the design of the lower wall to consider the lateral surcharge from the upper wall. As mentioned in our previous response letter, "The design engineer should ensure the stability of walls."

If the wall will be built such that compacted fill will be used behind the wall, this firm should be contacted to provide further recommendations at the design stage when the wall type and a detailed cross section are available.

3. *The consultant should provide recommendations for the foundation to slope setback in accordance with the City of Agoura Hills building ordinance.*
   __Note:__ The consultant provided setback recommendations based on the California Building Code (CBC). But the City of Agoura Hills has more stringent recommendations for foundation to slope

setback. As requested in the above comment, the consultant should provide recommendations for the foundation to slope setback in accordance with the City of Agoura Hills building ordinance.

Foundations on or adjacent to slope surfaces shall be designed in accordance with Section 1808.7.1 for building clearance from an ascending slope and Section 1808.7.2 for footing setback from a descending slope surface, in accordance with the City of Agoura Hills, Title 24 Adoption – Ordinance 10-381.

4. *The consultant should provide recommendations for the minimum depth of embedment of footings below lowest adjacent grade, with due considerations to the highly expansive nature of on-site soils.*
   Note: *the consultant responded to this comment by stating that "Due to the high expansive nature of the on-site soils and the volume of expansive soil to be replaced, conventional spread foundation is not considered to be suitable footing type." Thereupon, the consultant should provide recommendations for alternative foundation system..*

As recommended in the "Foundation Design" section of our report, "Structural design measures including design of slab-on-grade foundations in accordance with 'WRI/CRSI Design of Slab-On-Ground Foundations' or 'PTI Standard Requirements for Design of Shallow Post-Tensioned Concrete Foundations of Expansive Soils' would be necessary." Either way, the slab should be designed as a mat foundation.

This letter should be included with and considered part of the Geotechnical Investigation report for the project.

We appreciate this opportunity to provide geotechnical services for this project. If you should have any questions or comments concerning this letter, please do not hesitate to contact this firm at your convenience.

REGISTERED PROFESSIONAL ENGINEER
FENG YI
No. 2967
Exp. 06-30-17
GEOTECHNICAL
STATE OF CALIFORNIA

Respectfully submitted,
CHJ CONSULTANTS

Fred Yi, Ph.D., G.E. 2967
Chief Engineer

Robert J. Johnson, G.E. 443
President

REGISTERED PROFESSIONAL ENGINEER
ROBERT J. JOHNSON
No. 443
Exp. 03-31-17
GEOTECHNICAL
STATE OF CALIFORNIA

FY/RJJ:fy/lb

Enclosures: City of Agoura Hills - Geotechnical Review Sheet Dated July 11, 2016

REFERENCES

CHJ Consultants, 2015, Geotechnical Investigation, Oakmont of Agoura Hills, 29353 Canwood Street, Agoura Hills, California, Report Prepared For Oakmont Senior Living, Job No. 15473-3, Dated October 21, 2015

CHJ Consultants, 2016, Addendum to Geotechnical Investigation Report, Response to Geotechnical Review Sheet, Proposed Oakmont of Agoura Hills Senior Facility, 29353 Canwood Street, Agoura Hills, California, Report Prepared for Oakmont Senior Living, Job No. 15473-3A, Dated June 14, 2016

City of Agoura Hills, Title 24 Adoption – Ordinance 10-381

GeoDynamics, Incorporated, City of Agoura Hills – Geotechnical Review Sheet, CUP-001231-2016, 29353 Canwood Street, Agoura Hills, California, Dated July 11, 2016

GeoDynamics, Inc.

Applied Earth Sciences
Geotechnical Engineering & Engineering Geology Consultants

Date: July 11, 2016
GDI #: 16.00103.0211

CITY OF AGOURA HILLS - GEOTECHNICAL REVIEW SHEET

To:	Allison Cook
Project Location:	29353 Canwood Street, Agoura Hills, California.
Planning Case #:	CUP-001231-2016, SIGN-01232-2016, OAK-01233-2016
Building & Safety #:	None
Geotechnical Report:	CHJ Consultants (2016), "*Addendum to Geotechnical Investigation Report, Response to Geotechnical Review Sheet, Proposed Oakmont of Agoura Hills Senior Facility, 29353 Canwood Street, Agoura Hills, California*" J. N. 15473-3A, dated June 14, 2016. CHJ Consultants (2015), "*Geotechnical Investigation, Oakmont of Agoura Hills, 29353 Canwood Street, Agoura Hills, California*" J. N. 15473-3, dated October 21, 2015.
Plans:	Ali Iqbal (2016), "Oakmont of Agoura Hills" Sheets A0, R1 to R3, A1.0 through A1.2, A2.1 through A2.3, A3, A4.1 through A4.3 and A5, dated April 30, 2106 LandDesign Group (2016), "Oakmont of Agoura Hills, 29353 Canwood Street, Agoura Hills, California", Sheets 1 through 5, dated April, 2016 Huitt-Zollars (undated), "Grading Plan, Oakmont of Agoura Hills, 29353 Canwood Street, Agoura Hills, CA 91301", Sheets 1 and 2 of 2. Huitt-Zollars (2016), "Conceptual LID/Drainage Report for Oakmont of Agoura Hills, 29353 Canwood Street, Agoura Hills, CA 91301" J.N. R305871.01, dated April 12, 2016.
Previous Reviews:	May 20, 2016.

FINDINGS

Planning/Feasibility Issues	Geotechnical Report
☒ Acceptable as Presented	☐ Acceptable as Presented
☐ Response Required	☒ Response Required

REMARKS

CHJ Consultants (CHJ; consultant) provided a response to the review letter by the city of Agoura Hills dated May 20, 2016 regarding the proposed development at the site located at 29353 Canwood Street, in the City of Agoura Hills, California. According to the above-referenced reports, the site will be developed with a two- to three-story, 80-unit, senior facility of approximately 80,000 square feet. Grading will be required to create the level building pad using series of stacked retaining walls to support fill along the south edge of the pad and bedrock cut along the north edge of the pad. Based on the grading plans included as part of the submittal package, the overall height of the retaining wall stacks will reach heights of about 30 feet with individual walls as high as eight feet.

The City of Agoura Hills – Planning Department reviewed the referenced report from a geotechnical perspective for compliance with applicable codes, guidelines, and standards of practice. GeoDynamics, Inc. (GDI) performed the geotechnical review on behalf of the City. Based upon a review of the submitted report, we recommend the Planning Commission consider approval of Case Nos. CUP-001231-2016, SIGN-01232-2016, OAK-01233-2016. The Consultant should respond to the following Report Review comments prior to Building Plan-Check Approval. Plan-Check comments should be addressed in Building & Safety Plan Check. A separate geotechnical submittal is not required for plan-check comments.

Note to the City: The consultant indicates that the proposed development includes the construction of high retaining walls (higher than 6 ft), which might not be consistent with the current City building code and zoning ordinances.

Report Review Comments

1. The consultant should review development plans as they become available to verify compliance with recommendations in the above-referenced reports. A geotechnical map using the proposed grading plan as base map should be included. Cross-sections should be updated as necessary to reflect changes in the proposed grading relative to the current grading concept. Additional geotechnical recommendations should be provided as necessary.

   Note: The reviewers appreciate that the consultant addressed this comment, but this comment should be addressed during the design stage of the project, when final development plans become available. Note that ALL geologic data – bedding attitudes in particular - should be plotted on the geologic map.

2. The consultant should discuss and evaluate the potential for interaction between closely located retaining walls (example: stacked retaining walls) using an appropriate method of analyses. Please note that the 1:1 criterion is not acceptable for lateral surcharge unless substantiated with analyses and/or references.

   Note: Comment #6 of the Planning/Feasibility Comments does not address this comment. This comment is about the potential for lateral surcharge on the lower retaining wall due to the foundation load of the upper retaining wall.

3. The consultant should provide recommendations for the foundation to slope setback in accordance with the City of Agoura Hills building ordinance.

   Note: The consultant provided setback recommendations based on the California Building Code (CBC). But the City of Agoura Hills has more stringent recommendations for foundation to slope setback. As requested in the above comment, the consultant should provide recommendations for the foundation to slope setback in accordance with the City of Agoura Hills building ordinance.

4. The consultant should provide recommendations for the minimum depth of embedment of footings below lowest adjacent grade, with due considerations to the highly expansive nature of on-site soils.

   Note: the consultant responded to this comment by stating that "*Due to the high expansive nature of the on-site soils and the volume of expansive soil to be replaced, conventional spread foundation is not considered to be suitable footing type.*" Thereupon, the consultant should provide recommendations for alternative foundation system.

Plan-Check Comments

1. The name, address, and phone number of the Consultant and a list of all the applicable geotechnical reports shall be included on the building/grading plans.

2. The grading plan should include the limits and depths of overexcavation as recommended by the Consultant.

3. The following note must appear on the grading and foundation plans: "*Excavations shall be made in compliance with CAL/OSHA Regulations.*"

4. The following note must appear on the foundation plans: "*All foundation excavations must be observed and approved, in writing, by the Project Geotechnical Consultant prior to placement of reinforcing steel.*"

5. Foundation plans and foundation details shall clearly depict the embedment material and minimum depth of embedment for the foundations.

6. Drainage plans depicting all surface and subsurface non-erosive drainage devices, flow lines, and catch basins shall be included on the building plans.

7. Final grading, drainage, and foundation plans shall be reviewed, signed, and wet stamped by the consultant.

8. Provide a note on the grading and foundation plans that states: "*An as-built report shall be submitted to the City for review. This report prepared by the Geotechnical Consultant must include the results of all compaction tests as well as a map depicting the limits of fill, locations of all density tests, outline and elevations of all removal bottoms, keyway locations and bottom elevations, locations of all subdrains and flow line elevations, and location and elevation of all retaining wall backdrains and outlets. Geologic conditions exposed during grading must be depicted on an as-built geologic map.*"

If you have any questions regarding this review letter, please contact GDI at (805) 496-1222.

Respectfully Submitted,

GeoDynamics, INC.

Ali Abdel-Haq
Geotechnical Engineering Reviewer
GE 2308 (exp. 12/31/17)

Christopher J. Sexton
Engineering Geologic Reviewer
CEG 1441 (exp. 11/30/16)

Fuel Modification Plan Approval Letter

APPENDIX H

COUNTY OF LOS ANGELES

FIRE DEPARTMENT

605 N. ANGELENO AVENUE
AZUSA, CA 91702
(626) 969-5205

DARYL L. OSBY
FIRE CHIEF
FORESTER & FIRE WARDEN

July 20, 2017

Gregg Wanke
9240 Old Redwood Highway Suite 200
Windsor, CA 95492

Dear Mr. Wanke:

FUEL MODIFICATION PLAN – 29353 CANWOOD STREET, AGOURA HILLS
PARCEL #2053-001-005 - FM PROJECT #6300 - FFFM #201600423

The Revised Final Fuel Modification Plan has been reviewed and approved. Occupancy is subject to the on-site inspection and approval of required fuel modification. Inspections are to be performed by Forestry Division personnel.

Questions regarding this response should be directed to the Fuel Modification Unit. Office hours are Monday through Thursday, from 8:00 a.m. to 4:00 p.m. for plan submittal and general questions. Plan checkers are available 8:00 a.m. to 10:00 a.m. and by appointment. The Fuel Modification Unit may be reached at (626) 969-5205.

Very truly yours,

FOR KTJ

KEVIN T. JOHNSON, ASSISTANT CHIEF, FORESTRY DIVISION
PREVENTION SERVICES BUREAU

KTJ:lp

RECEIVED JUL 25 2017

Conceptual Low Impact Development/ Drainage Report

APPENDIX I

Conceptual LID/Drainage Report
for

Oakmont of Agoura Hills

29 353 Canwood Street
Agoura Hills, CA 91301

April 12, 2016
Revised June 24, 2016

Prepared for:

Oakmont Senior Living
9240 Old Redwood Highway
Suite 200
Windsor, CA 95492

Prepared by:

HUITT-ZOLLARS

Huitt-Zollars, Inc. Thousand Oaks
90 E Thousand Oaks Blvd, Suite 201
Thousand Oaks, CA 91360
Phone (805) 418-1802 Fax (805) 418-1819

REGISTERED PROFESSIONAL ENGINEER
JEREMY EPLEY
No. 057241
CIVIL
STATE OF CALIFORNIA

J. Epley

06-24-2016

Jeremy Epley, P.E. Date
HZ Job No. R305871.01

Table of Contents

Introduction

This report has been prepared to provide an analysis of drainage patterns and improvements related to the development of the proposed Oakmont of Agoura Hills located at 29353 Canwood Street. The project site is bounded by Canwood Street and the 101 Northbound Freeway to the south, by a medical office building to the west, vacant land to the east, and single family residential homes to the north. The project is located in in the City of Agoura Hills and therefore falls under City and Los Angeles County jurisdiction.

The lot is currently vacant. It has not been previously graded nor have utilities connections been installed. When complete, this project will contain a two-story 75,000 square-feet assisted living and memory care senior building with 55 parking spaces,

Runoff from this project will be collected by onsite storm drain infrastructure. The proposed storm drain infrastructure will convey onsite flows in a southerly direction. Onsite runoff will ultimately be treated by biofiltration systems and be discharged to the existing 36" Canwood Street CMP. Overall, the drainage patterns are characterized by steep gradients from north to south, and will largely remain unchanged upon project completion. Onsite flows will be controlled and placed in underground storm drain infrastructure.

Objectives

The objective of this report is to perform a conceptual evaluation of proposed stormwater flow rates based on conceptual project grading and infrastructure changes resulting from development. This report will address the following items:

- **Drainage Concept –** This report will discuss the proposed drainage concept for the site. Compliance with the existing drainage patterns will be demonstrated in the Final Drainage Report.
- **Detention –** Peak and volume mitigations, if necessary, will be addressed in the Final Drainage Report.
- **Low Impact Development –** This report will determine the guiding factors in implementing LID design on the project to comply with the requirements of the current MS4 Permit.

Methodology

This hydrology study was prepared using the design criteria and methodology developed by the Los Angeles County Department of Public Works and is in accordance with the 2006 Hydrology Manual. Calculations presented within this study were determined using the LA County HydroCalc program to determine time of concentration (TC) and onsite flows. The 50-year, 24-hour rainfall depth for the site is approximately 7.37 inches. The project site is located within the Debris Production Area 6. Since the watershed is already urbanized and has an imperviousness higher than 15%, and project slopes will be maintained, sediment production is not taken into consideration in both existing and proposed hydrology computations. No fire and bulking effects were considered when computing the peak discharges.

Drainage Concept

The proposed drainage concept for this site involves intercepting upstream slope flow (north) with area drains located behind the proposed retaining walls and routing offsite runoff in swales that bypass the site on both eastern (Drainage Area 2A) and western (Drainage Area 1A) boundaries of the project site. This flow will not be combined with onsite flow collected from parking areas and roof drains, and will not require water quality treatment. Onsite runoff from parking areas and roof drains will ultimately be conveyed in underground storm drain infrastructure that discharges into the existing 36" Canwood Street CMP. This system is part of PD 1645. Stormwater quality treatment is proposed for all onsite drainage areas.

Prior to discharge, flows will be biotreated in biofiltration systems. Further discussed in the LID portion of this report, the system will divert first flush flows into the biofiltration units while maintaining the ability of large storm flows to bypass the unit.

The proposed slope areas are engineered slopes that will be maintained by the property owner. In addition, there is no substantial offsite area that is tributary to this project. As a result, a burned and bulked factor will not be applied to the peak discharge computations.

Hydrology

Due to the nature of this site, the existing flow calculation (using the HydroCalc program) generates a 5-minute time of concentration. As a result, for existing flow purposes, no routing will be performed for site areas as subarea times of concentration will be less than that value and would produce an overly conservative result. The overall site area will be used as a comparison point for existing and proposed flow.

The proposed drainage concept for this site involves intercepting upstream slope flow (north) with area drains located behind the proposed retaining walls and routing offsite runoff in swales that bypass the site on both eastern (Drainage Area 2A) and western (Drainage Area 1A) boundaries of the project site. Runoff from Drainage Area 1A (3.29 acres) is collected by a proposed concrete swale that flows westerly and discharges into a natural, unimproved, vegetated swale that drains along the western boundary of the project site in a southerly direction. The natural, unimproved, vegetated swale collects runoff from the unimproved, pervious slopes of Drainage Area 5A (0.30 acres) and discharges to a downstream inlet located at node 104. Runoff from Drainage Area 2A (0.71 acres) is collected by a concrete swale that flows successively along the northern boundary and the eastern boundary of the project site. All runoff is then collected by an inlet and conveyed through an underground storm drain pipe to node 104. Runoff from Drainage Areas 1A, 2A, and 5A will not be combined with onsite flow collected from parking areas and roof drains, and will not require water quality treatment.

Onsite drainage patterns are designed to allow all onsite runoff to gravity drain to required biofiltration systems for adequate water quality treatment. Walkways and landscape areas located on the northern side of the project (Drainage Areas 3A and 4A) drain westerly and runoff is conveyed upon biotreatment through onsite storm drain infrastructure. Parking areas located on the western side of the project (Drainage Areas 4A and 6A) drain southerly and runoff is conveyed upon biotreatment through onsite storm drain infrastructure. Driveways and walkway areas located on both southern and eastern sides of the project drain southerly to a biofiltration system located at the southeast corner of the project site. Runoff from the building itself (Drainage Area 7A) is captured and discharged through roof drains into localized biofiltration areas before connecting to the onsite storm drain infrastructure. All runoff from the project area is ultimately conveyed through onsite storm drain infrastructure to Node 104, where it connects to the existing 36" Canwood Street CMP.

Seven biofiltration treatment systems will be installed throughout the project site to meet the requirements set forth in the 2012 MS4 Permit. The water quality treatment devices are designed to provide adequate treatment to the water quality flows and volumes generated by the 85-th percentile storm event. The treatment systems are also designed to bypass higher flows.

Maintenance of the onsite storm drain facilities, including cleaning of the catch basins and conveyance systems, will be of the responsibility of the owner.

FEMA Flood Insurance Rate Map #06037C1244F (dated September 26, 2008) identifies that the project site is not located within a floodplain.

Hydraulics

The County of Los Angeles Hydraulic Design Manual requires that storm drain systems in sump conditions be designed to a 50-year storm event. Other drains are required to be designed for a storm frequency of not less than 10-year.

Analysis of the proposed drainage facilities will be provided in the Final Drainage Report. Analysis will include the following:

Storm drain pipe sizing – The 10-year HGL will be developed using County-approved WSPG computer software and compared to the proposed finished surface in the final drainage report. The 10-year ultimate flow rates from HydroCalc will be used in the analysis of the proposed in-tract storm drain.

Catch basin or Drop inlet sizing – The 10-year and 50-year ultimate HydroCalc flow rates will be utilized in both flow-by and sump conditions in the final report, respectively. Calculations for capture flow rates will be performed using the County approved HydroCalcs computer software in the final report.

LID

The project triggers the LID requirements for New Development Projects over 5,000 square-feet, as established in the 2012 Los Angeles Regional MS4 Permit.

The project site is not subject to the hydromodification requirements, as defined in Section 8 of the LADPW Low Impact Development Standards Manual (February 2014). A review of the downstream channel on the Los Angeles County Storm Drain System Inventory (http://dpw.lacounty.gov/fcd/stormdrain/index.cfm) identified that runoff from the project is initially conveyed through a series of concrete-lined and engineered channels that are not susceptible to hydromodification impacts. A summary of the successive conveyance systems is provided in Table 1.

Table 1 – Downstream Channels & Susceptibility to Hydromodification

System	Material	Engineered?
PD1645	30" RCP	Yes
PD1605	60" RCP	Yes
Lindero Canyon Channel	126" RCB	Yes

Runoff from the upstream adjacent native slopes is collected by concrete swales, bypasses the project site, and is ultimately discharged at the downstream receiving point #104. Because the offsite flows are being bypassed and not combined with onsite flows, those undisturbed and natural areas are exempt from the LID requirements do not need to be treated.

The project site, once developed, will contain asphalt paving, concrete walks, rooftops and other impervious constructions. Several planters that can incorporate biofiltration systems are included in the design. These impervious areas will be directed to seven individual biofiltration systems that are laid out per the LID site design principles to meet the requirements of the 2012 MS4 Permit.

Consistent with the 2014 LID Standard Manual, the Stormwater Quality Design Volume (SWQDv) was computed for each tributary drainage area using Hydrocalc. Runoff rates and volumes for the 85th percentile storm event are summarized in Appendix C. Table 2 identifies the SWQDv for each drainage area.

Table 2 – Storm Water Quality Design Volume

			LID-85th percentile	
Subarea ID	**Footprint (ac)**	**Imperviousness (%)**	**Q (cfs)**	**Volume (cu-ft)**
		Offsite Areas		
1A	*3.29*	*1.0%*	*n/a*	*n/a*
2A	*0.71*	*1.0%*	*n/a*	*n/a*
5A	*0.30*	*1.0%*	*n/a*	*n/a*
Total	***4.30***	***1.0%***	***n/a***	***n/a***
		Onsite Areas		
3A	0.09	9.0%	0.01	53
4A	0.69	51.0%	0.10	1,199
6A	0.46	68.0%	0.11	1,013
7A	1.13	84.0%	0.45	2,983
8A	0.71	40.0%	0.09	1,020
Total	***3.08***	***61.9%***	***0.76***	***6,268***

Infiltration-based retention systems were ruled out by the geotechnical engineer because of the presence of colluvial fill and bedrock at the project site. Onsite geotechnical explorations revealed the presence of bedrock encountered at depths of 3 to 10 feet below existing ground, and the presence of medium dense to dense clayey sand (SC) to stiff to hard fat clay (CH) at depths 3 to 5 feet below ground surface. A copy of the geotechnical findings, along with the NRCS Soil Survey Report, is provided in Appendix D. Since infiltration is deemed infeasible onsite, onsite biofiltration systems were sized to treat 1.5 times the SWQDv volume consistent with the design guidelines defined in Appendix E of the 2014 LA County LID Standard Manual. Table 3 summarizes the treatment capacity of each of the seven distributed biofiltration systems.

Table 3 – Biofiltration Systems Treatment Capacity

Subarea ID	Footprint (ac)	SWQDv (cu-ft)	Minimum Facility Surface Area (sq.ft)*	Surface Area Provided (sq.ft)
3A	0.09	53	5,504	350
4A	0.69	1,199		950
6A	0.46	1,013		1,000
7A	1.13	2,983		2,050
8A	0.71	1,020		1,300
Total	***3.08***	***6,268***		***5,650***

** Assumes a 1-hour routing time and media design infiltration rate of 2.5 inch per hour.*

The seven biofiltration systems meet the requirements and treat the water quality volume to the maximum extent practicable.

Appendix

Appendix A. Existing Conditions Q_{50} Hydrology Calculations
Appendix B. Proposed Conditions Q_{50} Hydrology Calculations
Appendix C. Proposed Conditions Q_{10} Hydrology Calculations
Appendix D. Proposed Conditions SWQDv Hydrology Calculations
Appendix E. Geotechnical Explorations
Appendix F. Existing Conditions Hydrology Map
Appendix G. Proposed Condition Hydrology Map
Appendix H. Proposed Water Quality Map

Appendix A

Existing Conditions Q_{50} Hydrology Calculations

Peak Flow Hydrologic Analysis

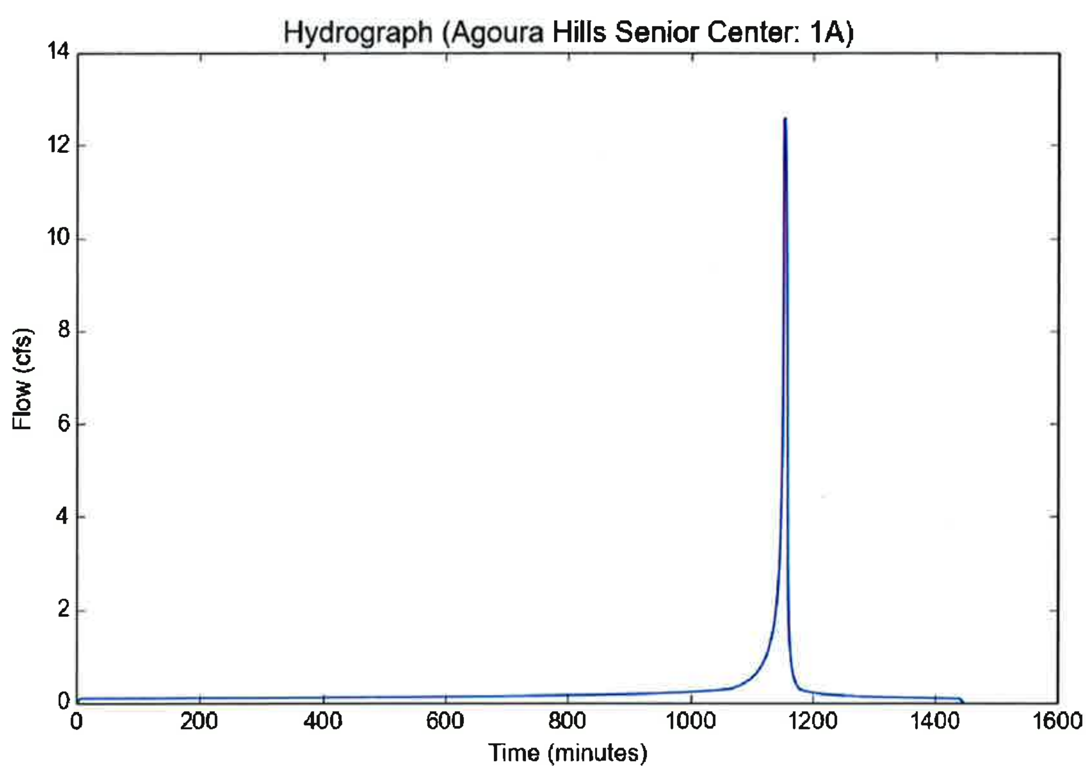
File location: R:/R305871.01 - Agoura Senior Village/10 CADD/10.1 AutoCAD/Hydrology/Agoura Hills Senior Center Report.50Ex.pdf
Version: HydroCalc 0.3.1-beta

Input Parameters

Project Name	Agoura Hills Senior Center
Subarea ID	1A
Area (ac)	5.09
Flow Path Length (ft)	952.0
Flow Path Slope (vft/hft)	0.098
50-yr Rainfall Depth (in)	7.37
Percent Impervious	0.01
Soil Type	28
Design Storm Frequency	50-yr
Fire Factor	0
LID	False

Output Results

Modeled (50-yr) Rainfall Depth (in)	7.37
Peak Intensity (in/hr)	3.754
Undeveloped Runoff Coefficient (Cu)	0.656
Developed Runoff Coefficient (Cd)	0.6585
Time of Concentration (min)	7.0
Clear Peak Flow Rate (cfs)	12.582
Burned Peak Flow Rate (cfs)	12.582
24-Hr Clear Runoff Volume (ac-ft)	0.5101
24-Hr Clear Runoff Volume (cu-ft)	22219.0502



Peak Flow Hydrologic Analysis

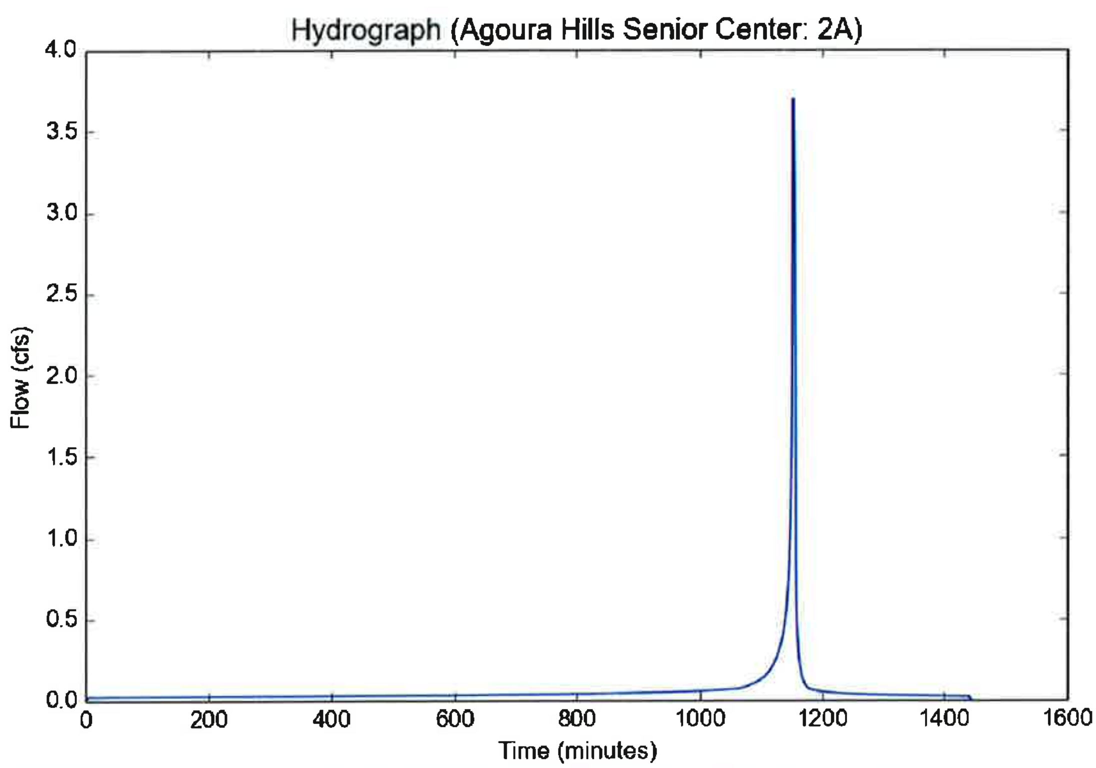
File location: R:/R305871.01 - Agoura Senior Village/10 CADD/10.1 AutoCAD/Hydrology/Agoura Hills Senior Center Report.50Ex.pdf
Version: HydroCalc 0.3.1-beta

Input Parameters

Project Name	Agoura Hills Senior Center
Subarea ID	2A
Area (ac)	1.21
Flow Path Length (ft)	584.0
Flow Path Slope (vft/hft)	0.204
50-yr Rainfall Depth (in)	7.37
Percent Impervious	0.01
Soil Type	28
Design Storm Frequency	50-yr
Fire Factor	0
LID	False

Output Results

Modeled (50-yr) Rainfall Depth (in)	7.37
Peak Intensity (in/hr)	4.3971
Undeveloped Runoff Coefficient (Cu)	0.692
Developed Runoff Coefficient (Cd)	0.6941
Time of Concentration (min)	5.0
Clear Peak Flow Rate (cfs)	3.693
Burned Peak Flow Rate (cfs)	3.693
24-Hr Clear Runoff Volume (ac-ft)	0.1216
24-Hr Clear Runoff Volume (cu-ft)	5297.4869



Peak Flow Hydrologic Analysis

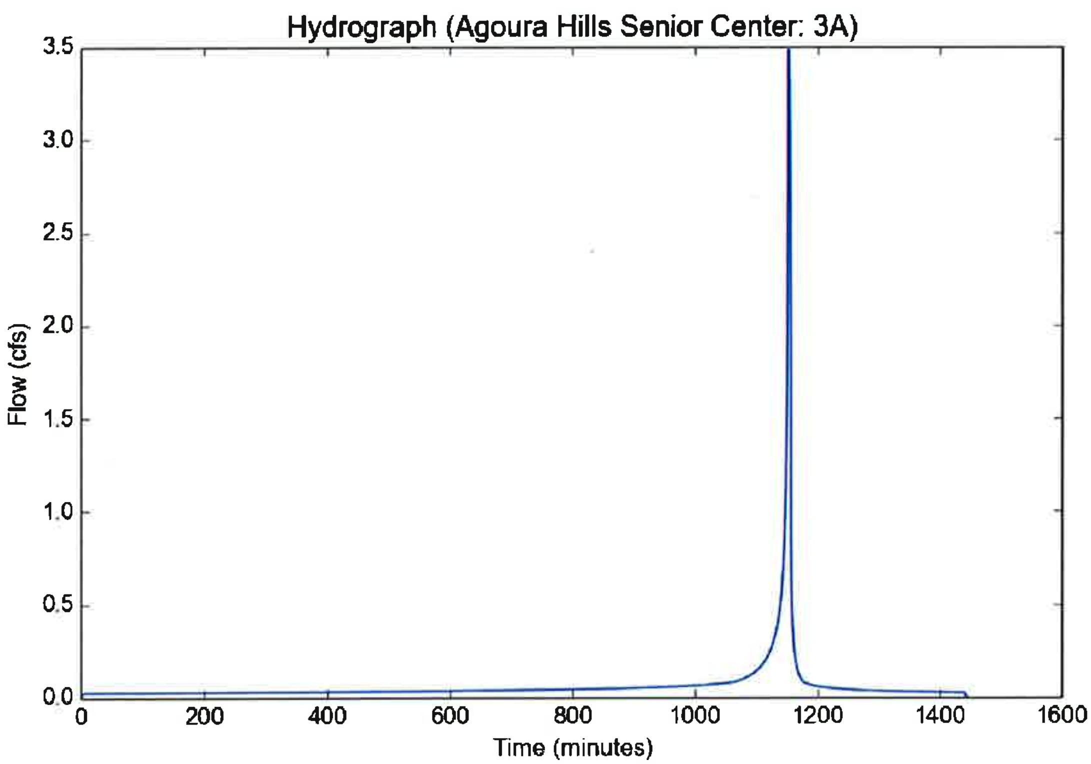
File location: R:/R305871.01 - Agoura Senior Village/10 CADD/10.1 AutoCAD/Hydrology/Agoura Hills Senior Center Report.50Ex.pdf
Version: HydroCalc 0.3.1-beta

Input Parameters

Project Name	Agoura Hills Senior Center
Subarea ID	3A
Area (ac)	1.13
Flow Path Length (ft)	489.0
Flow Path Slope (vft/hft)	0.217
50-yr Rainfall Depth (in)	7.37
Percent Impervious	0.04
Soil Type	28
Design Storm Frequency	50-yr
Fire Factor	0
LID	False

Output Results

Modeled (50-yr) Rainfall Depth (in)	7.37
Peak Intensity (in/hr)	4.3971
Undeveloped Runoff Coefficient (Cu)	0.692
Developed Runoff Coefficient (Cd)	0.7003
Time of Concentration (min)	5.0
Clear Peak Flow Rate (cfs)	3.4798
Burned Peak Flow Rate (cfs)	3.4798
24-Hr Clear Runoff Volume (ac-ft)	0.1289
24-Hr Clear Runoff Volume (cu-ft)	5614.9918



Appendix B

Proposed Conditions Q_{50} Hydrology Calculations

Peak Flow Hydrologic Analysis

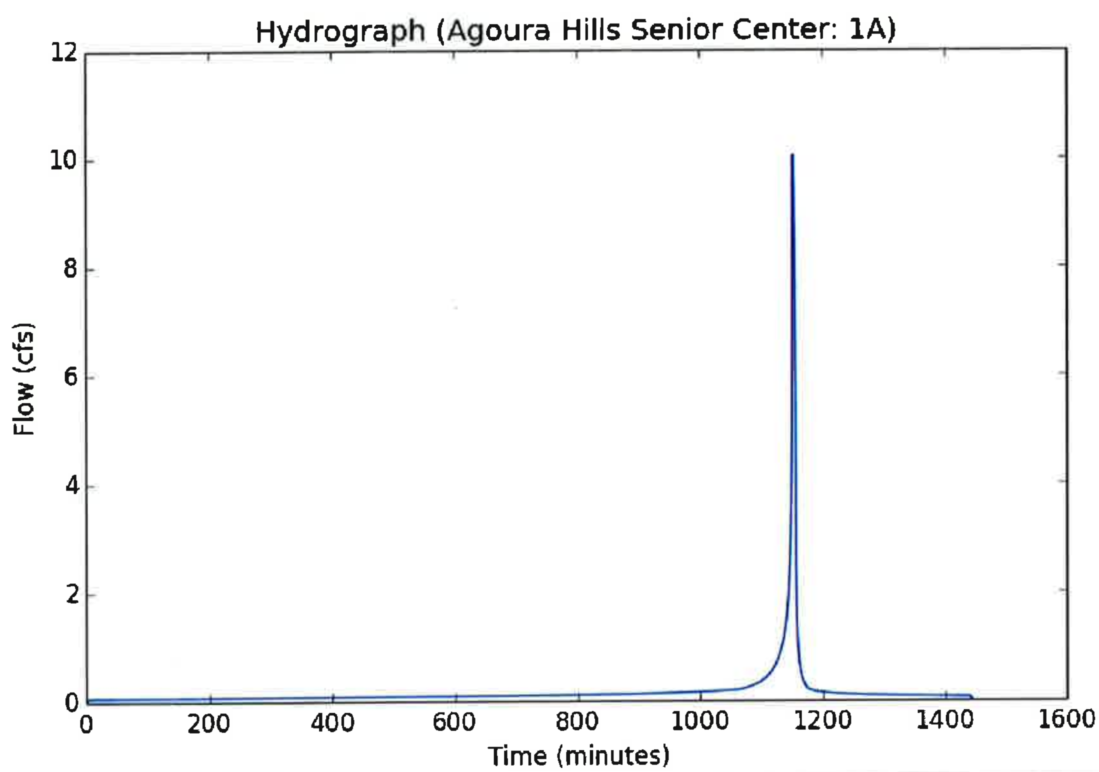
File location: R:/R305871.01 - Agoura Senior Village/10 CADD/10.1 AutoCAD/Hydrology/Agoura Hills Senior Center Report.50.pdf
Version: HydroCalc 0.3.1-beta

Input Parameters

Project Name	Agoura Hills Senior Center
Subarea ID	1A
Area (ac)	3.29
Flow Path Length (ft)	386.0
Flow Path Slope (vft/hft)	0.111
50-yr Rainfall Depth (in)	7.37
Percent Impervious	0.01
Soil Type	28
Design Storm Frequency	50-yr
Fire Factor	0
LID	False

Output Results

Modeled (50-yr) Rainfall Depth (in)	7.37
Peak Intensity (in/hr)	4.3971
Undeveloped Runoff Coefficient (Cu)	0.692
Developed Runoff Coefficient (Cd)	0.6941
Time of Concentration (min)	5.0
Clear Peak Flow Rate (cfs)	10.0413
Burned Peak Flow Rate (cfs)	10.0413
24-Hr Clear Runoff Volume (ac-ft)	0.3307
24-Hr Clear Runoff Volume (cu-ft)	14403.9108



Peak Flow Hydrologic Analysis

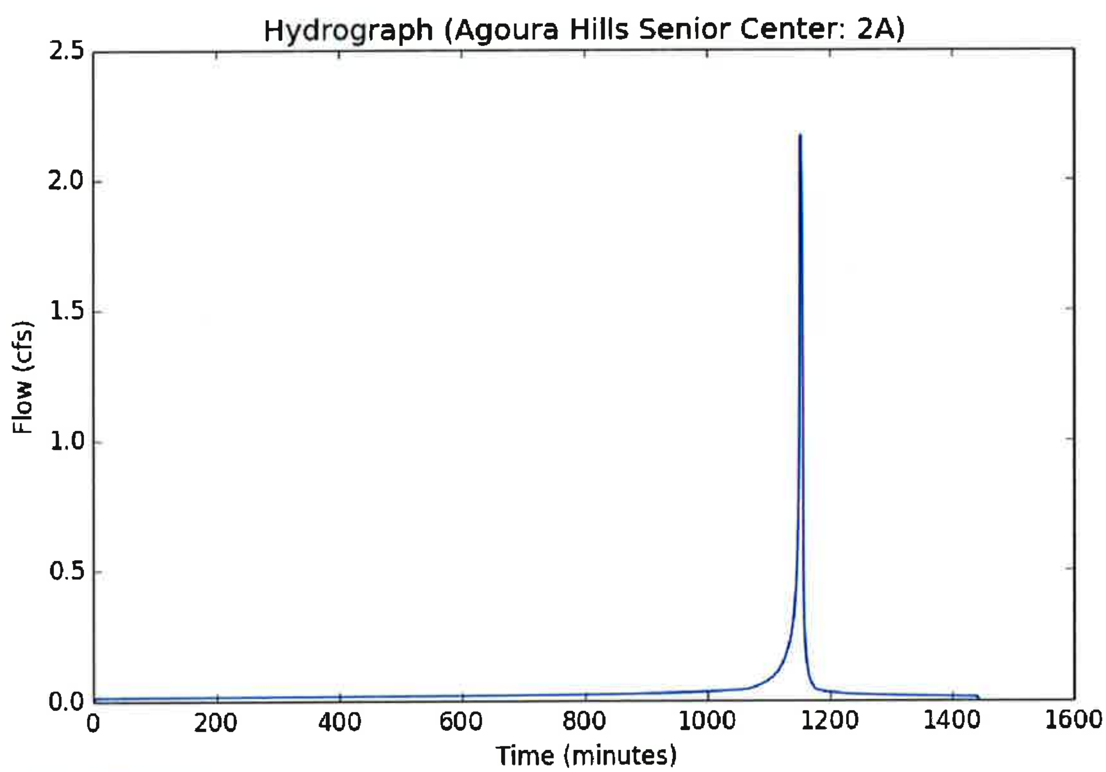
File location: R:/R305871.01 - Agoura Senior Village/10 CADD/10.1 AutoCAD/Hydrology/Agoura Hills Senior Center Report.50.pdf
Version: HydroCalc 0.3.1-beta

Input Parameters

Project Name	Agoura Hills Senior Center
Subarea ID	2A
Area (ac)	0.71
Flow Path Length (ft)	219.0
Flow Path Slope (vft/hft)	0.304
50-yr Rainfall Depth (in)	7.37
Percent Impervious	0.01
Soil Type	28
Design Storm Frequency	50-yr
Fire Factor	0
LID	False

Output Results

Modeled (50-yr) Rainfall Depth (in)	7.37
Peak Intensity (in/hr)	4.3971
Undeveloped Runoff Coefficient (Cu)	0.692
Developed Runoff Coefficient (Cd)	0.6941
Time of Concentration (min)	5.0
Clear Peak Flow Rate (cfs)	2.167
Burned Peak Flow Rate (cfs)	2.167
24-Hr Clear Runoff Volume (ac-ft)	0.0714
24-Hr Clear Runoff Volume (cu-ft)	3108.4427



Peak Flow Hydrologic Analysis

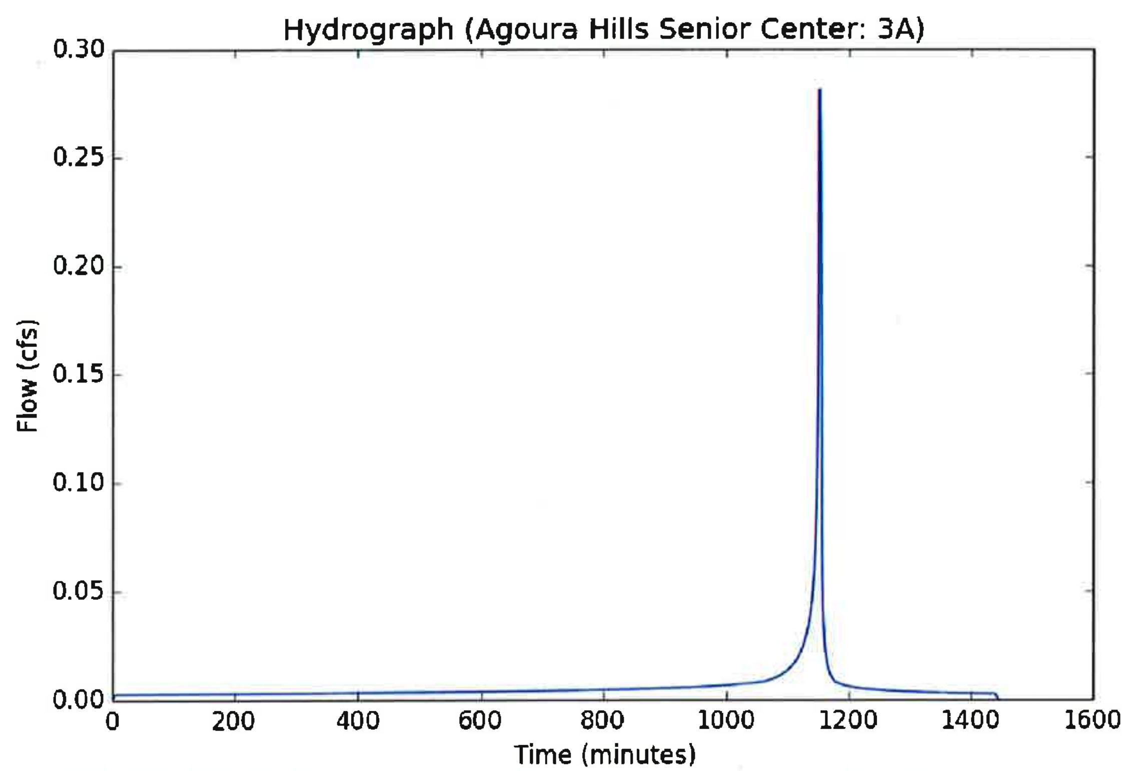
File location: R:/R305871.01 - Agoura Senior Village/10 CADD/10.1 AutoCAD/Hydrology/Agoura Hills Senior Center Report.50.pdf
Version: HydroCalc 0.3.1-beta

Input Parameters

Project Name	Agoura Hills Senior Center
Subarea ID	3A
Area (ac)	0.09
Flow Path Length (ft)	88.0
Flow Path Slope (vft/hft)	0.087
50-yr Rainfall Depth (in)	7.37
Percent Impervious	0.09
Soil Type	28
Design Storm Frequency	50-yr
Fire Factor	0
LID	False

Output Results

Modeled (50-yr) Rainfall Depth (in)	7.37
Peak Intensity (in/hr)	4.3971
Undeveloped Runoff Coefficient (Cu)	0.692
Developed Runoff Coefficient (Cd)	0.7107
Time of Concentration (min)	5.0
Clear Peak Flow Rate (cfs)	0.2813
Burned Peak Flow Rate (cfs)	0.2813
24-Hr Clear Runoff Volume (ac-ft)	0.0123
24-Hr Clear Runoff Volume (cu-ft)	535.8514



Peak Flow Hydrologic Analysis

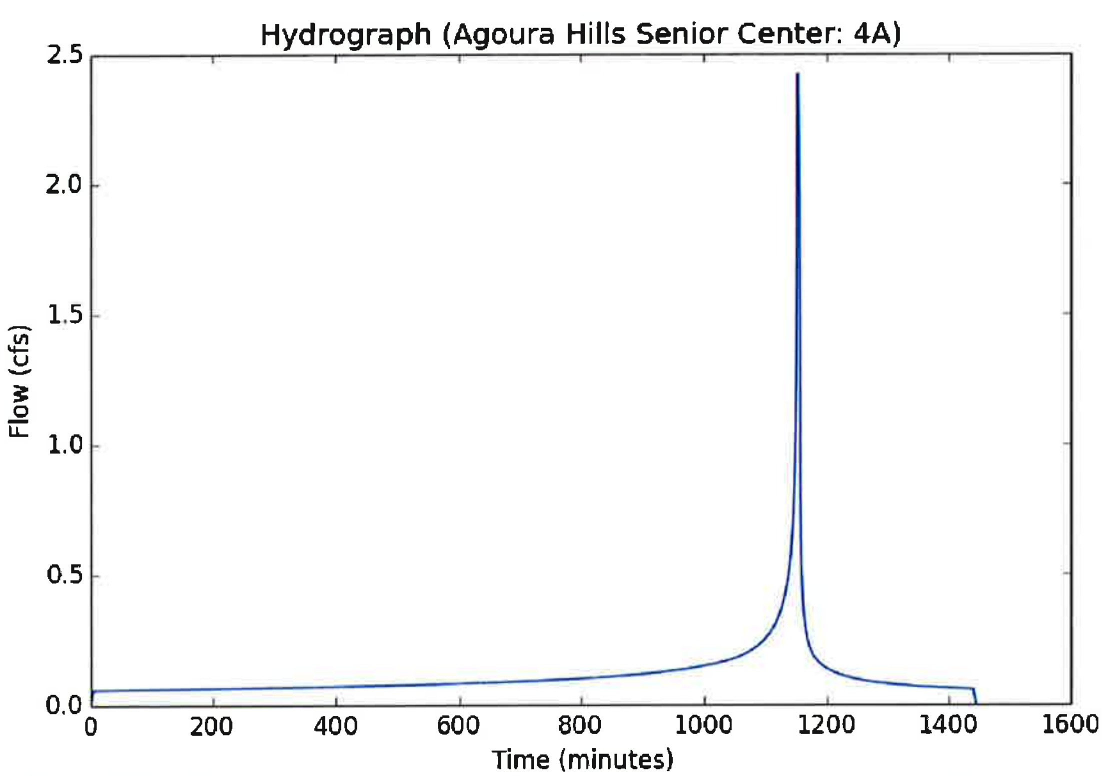
File location: R:/R305871.01 - Agoura Senior Village/10 CADD/10.1 AutoCAD/Hydrology/Agoura Hills Senior Center Report.50.pdf
Version: HydroCalc 0.3.1-beta

Input Parameters

Project Name	Agoura Hills Senior Center
Subarea ID	4A
Area (ac)	0.69
Flow Path Length (ft)	231.0
Flow Path Slope (vft/hft)	0.013
50-yr Rainfall Depth (in)	7.37
Percent Impervious	0.51
Soil Type	28
Design Storm Frequency	50-yr
Fire Factor	0
LID	False

Output Results

Modeled (50-yr) Rainfall Depth (in)	7.37
Peak Intensity (in/hr)	4.3971
Undeveloped Runoff Coefficient (Cu)	0.692
Developed Runoff Coefficient (Cd)	0.7981
Time of Concentration (min)	5.0
Clear Peak Flow Rate (cfs)	2.4214
Burned Peak Flow Rate (cfs)	2.4214
24-Hr Clear Runoff Volume (ac-ft)	0.2254
24-Hr Clear Runoff Volume (cu-ft)	9816.5861



Peak Flow Hydrologic Analysis

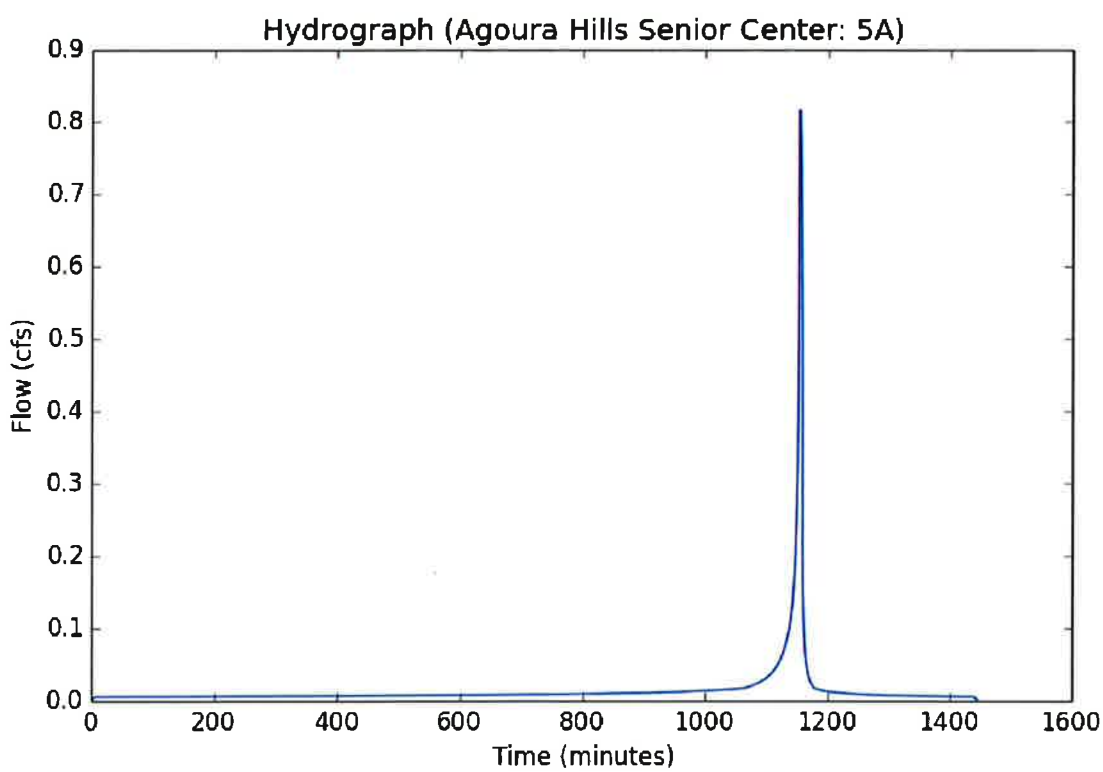
File location: R:/R305871.01 - Agoura Senior Village/10 CADD/10.1 AutoCAD/Hydrology/Agoura Hills Senior Center Report.50.pdf
Version: HydroCalc 0.3.1-beta

Input Parameters

Project Name	Agoura Hills Senior Center
Subarea ID	5A
Area (ac)	0.3
Flow Path Length (ft)	599.0
Flow Path Slope (vft/hft)	0.076
50-yr Rainfall Depth (in)	7.37
Percent Impervious	0.01
Soil Type	28
Design Storm Frequency	50-yr
Fire Factor	0
LID	False

Output Results

Modeled (50-yr) Rainfall Depth (in)	7.37
Peak Intensity (in/hr)	4.036
Undeveloped Runoff Coefficient (Cu)	0.6718
Developed Runoff Coefficient (Cd)	0.6741
Time of Concentration (min)	6.0
Clear Peak Flow Rate (cfs)	0.8162
Burned Peak Flow Rate (cfs)	0.8162
24-Hr Clear Runoff Volume (ac-ft)	0.0301
24-Hr Clear Runoff Volume (cu-ft)	1311.3344



Peak Flow Hydrologic Analysis

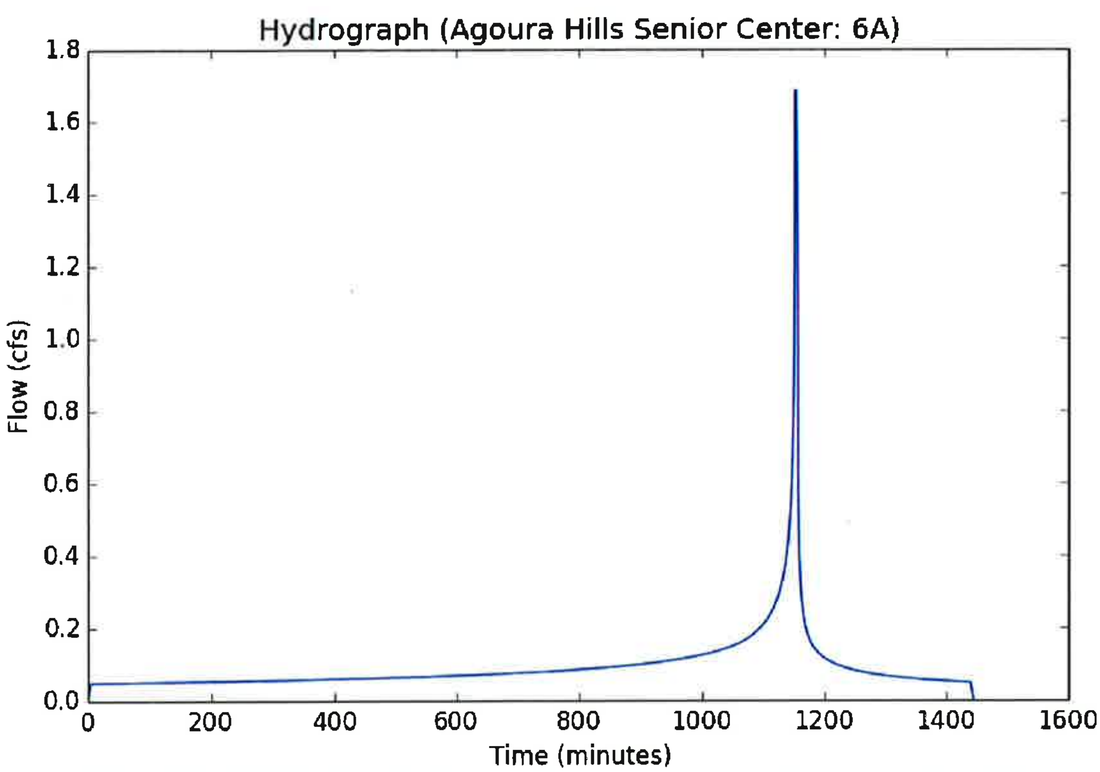
File location: R:/R305871.01 - Agoura Senior Village/10 CADD/10.1 AutoCAD/Hydrology/Agoura Hills Senior Center Report.50.pdf
Version: HydroCalc 0.3.1-beta

Input Parameters

Project Name	Agoura Hills Senior Center
Subarea ID	6A
Area (ac)	0.46
Flow Path Length (ft)	162.0
Flow Path Slope (vft/hft)	0.021
50-yr Rainfall Depth (in)	7.37
Percent Impervious	0.68
Soil Type	28
Design Storm Frequency	50-yr
Fire Factor	0
LID	False

Output Results

Modeled (50-yr) Rainfall Depth (in)	7.37
Peak Intensity (in/hr)	4.3971
Undeveloped Runoff Coefficient (Cu)	0.692
Developed Runoff Coefficient (Cd)	0.8334
Time of Concentration (min)	5.0
Clear Peak Flow Rate (cfs)	1.6858
Burned Peak Flow Rate (cfs)	1.6858
24-Hr Clear Runoff Volume (ac-ft)	0.1856
24-Hr Clear Runoff Volume (cu-ft)	8084.7506



Peak Flow Hydrologic Analysis

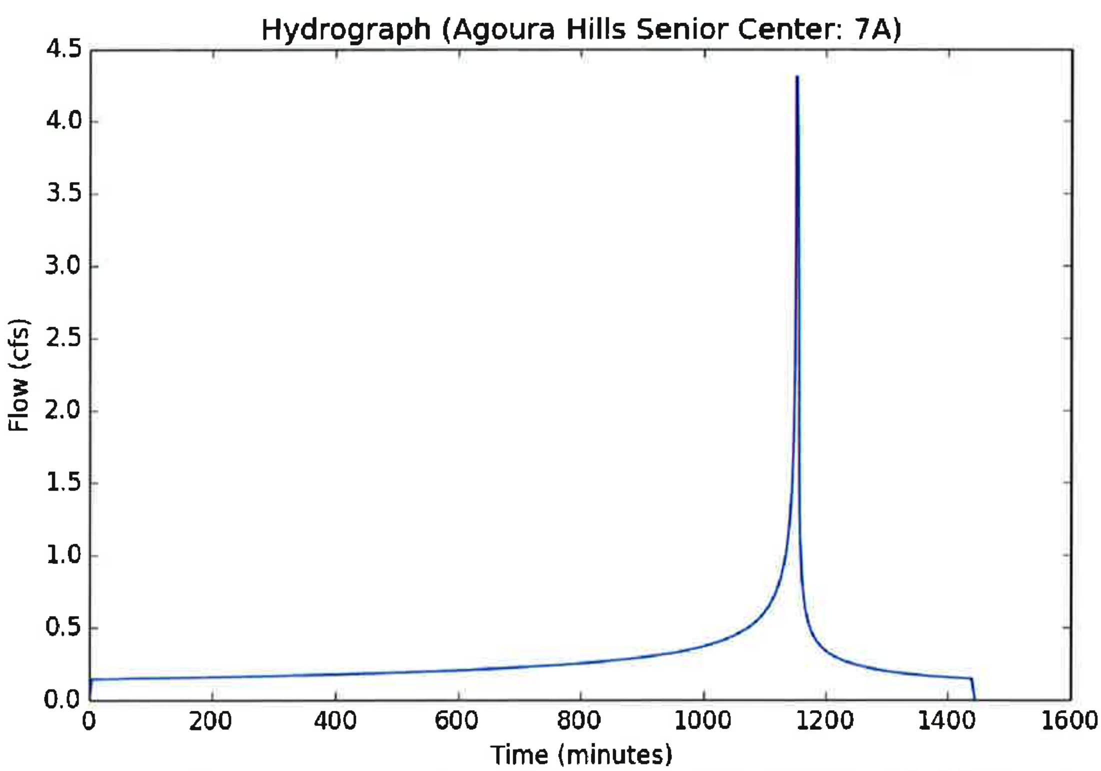
File location: R:/R305871.01 - Agoura Senior Village/10 CADD/10.1 AutoCAD/Hydrology/Agoura Hills Senior Center Report.50.pdf
Version: HydroCalc 0.3.1-beta

Input Parameters

Project Name	Agoura Hills Senior Center
Subarea ID	7A
Area (ac)	1.13
Flow Path Length (ft)	44.0
Flow Path Slope (vft/hft)	0.01
50-yr Rainfall Depth (in)	7.37
Percent Impervious	0.84
Soil Type	28
Design Storm Frequency	50-yr
Fire Factor	0
LID	False

Output Results

Modeled (50-yr) Rainfall Depth (in)	7.37
Peak Intensity (in/hr)	4.3971
Undeveloped Runoff Coefficient (Cu)	0.692
Developed Runoff Coefficient (Cd)	0.8667
Time of Concentration (min)	5.0
Clear Peak Flow Rate (cfs)	4.3066
Burned Peak Flow Rate (cfs)	4.3066
24-Hr Clear Runoff Volume (ac-ft)	0.5377
24-Hr Clear Runoff Volume (cu-ft)	23421.709



Peak Flow Hydrologic Analysis

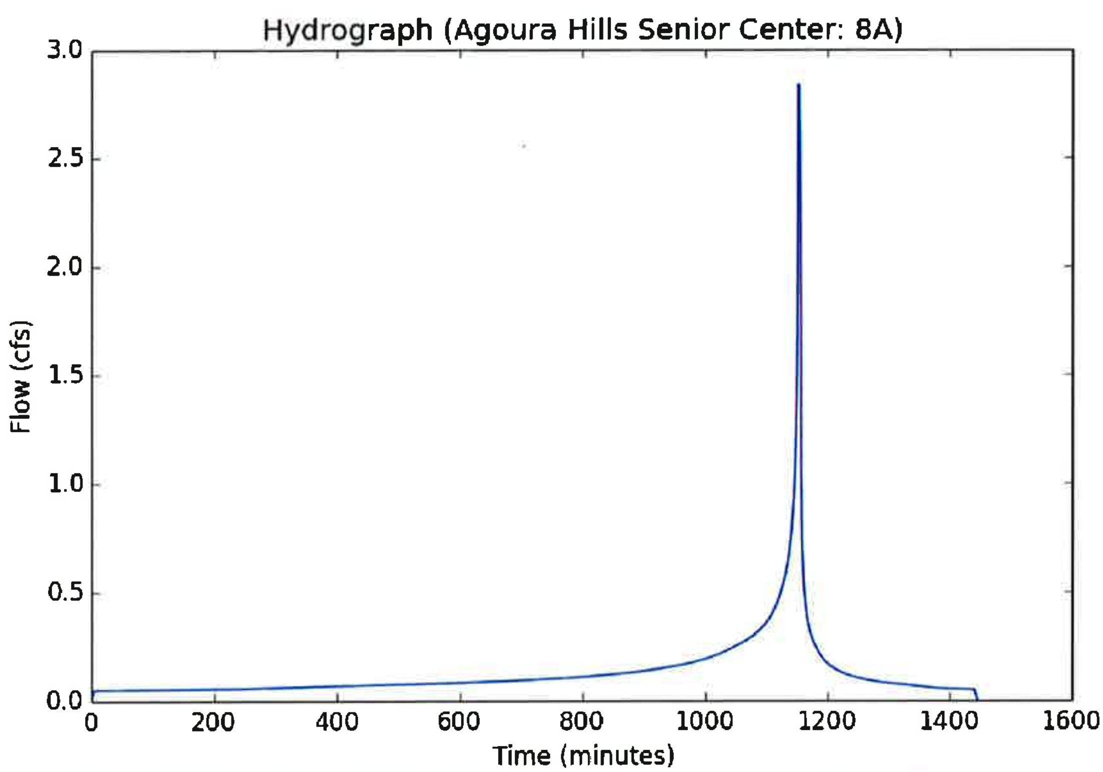
File location: R:/R305871.01 - Agoura Senior Village/10 CADD/10.1 AutoCAD/Hydrology/Agoura Hills Senior Center Report.50.pdf
Version: HydroCalc 0.3.1-beta

Input Parameters

Project Name	Agoura Hills Senior Center
Subarea ID	8A
Area (ac)	0.71
Flow Path Length (ft)	296.0
Flow Path Slope (vft/hft)	0.148
50-yr Rainfall Depth (in)	7.37
Percent Impervious	0.4
Soil Type	29
Design Storm Frequency	50-yr
Fire Factor	0
LID	False

Output Results

Modeled (50-yr) Rainfall Depth (in)	7.37
Peak Intensity (in/hr)	4.3971
Undeveloped Runoff Coefficient (Cu)	0.9147
Developed Runoff Coefficient (Cd)	0.9088
Time of Concentration (min)	5.0
Clear Peak Flow Rate (cfs)	2.8373
Burned Peak Flow Rate (cfs)	2.8373
24-Hr Clear Runoff Volume (ac-ft)	0.2579
24-Hr Clear Runoff Volume (cu-ft)	11233.7288



Appendix C

Proposed Conditions Q_{10} Hydrology Calculations

Peak Flow Hydrologic Analysis

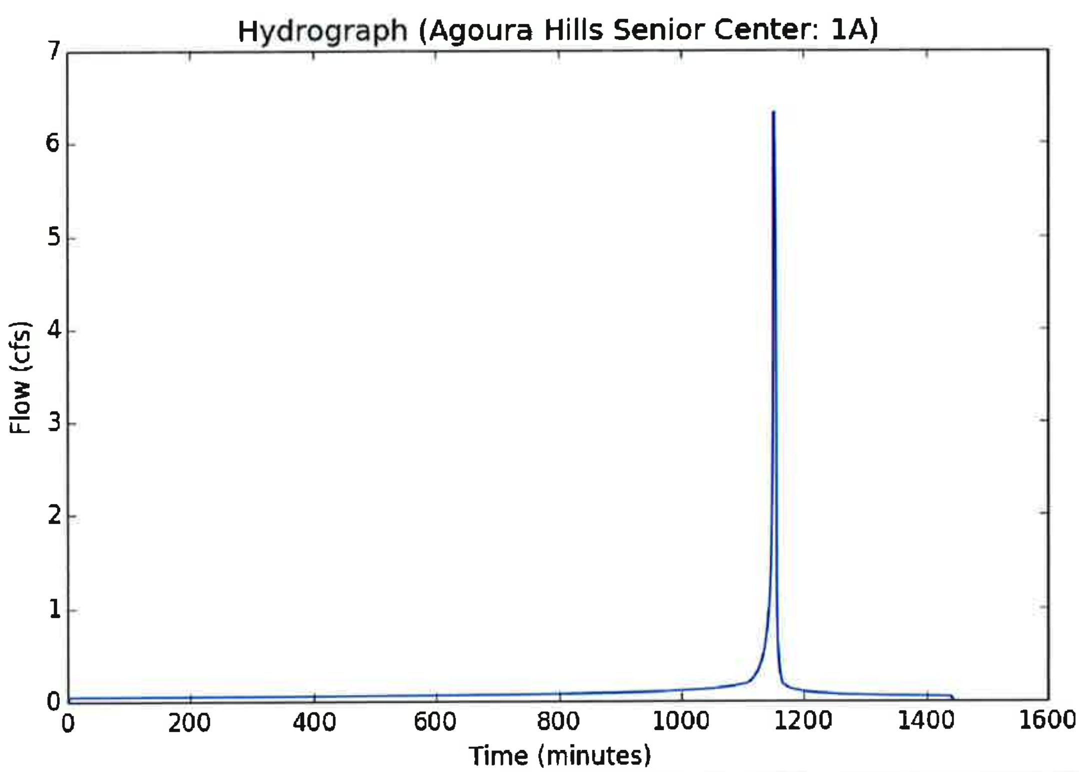
File location: R:/R305871.01 - Agoura Senior Village/10 CADD/10.1 AutoCAD/Hydrology/Agoura Hills Senior Center Report.10.pdf
Version: HydroCalc 0.3.1-beta

Input Parameters

Project Name	Agoura Hills Senior Center
Subarea ID	1A
Area (ac)	3.29
Flow Path Length (ft)	386.0
Flow Path Slope (vft/hft)	0.111
50-yr Rainfall Depth (in)	7.37
Percent Impervious	0.01
Soil Type	28
Design Storm Frequency	10-yr
Fire Factor	0
LID	False

Output Results

Modeled (10-yr) Rainfall Depth (in)	5.2622
Peak Intensity (in/hr)	3.1396
Undeveloped Runoff Coefficient (Cu)	0.6094
Developed Runoff Coefficient (Cd)	0.6123
Time of Concentration (min)	5.0
Clear Peak Flow Rate (cfs)	6.3243
Burned Peak Flow Rate (cfs)	6.3243
24-Hr Clear Runoff Volume (ac-ft)	0.2089
24-Hr Clear Runoff Volume (cu-ft)	9097.718



Peak Flow Hydrologic Analysis

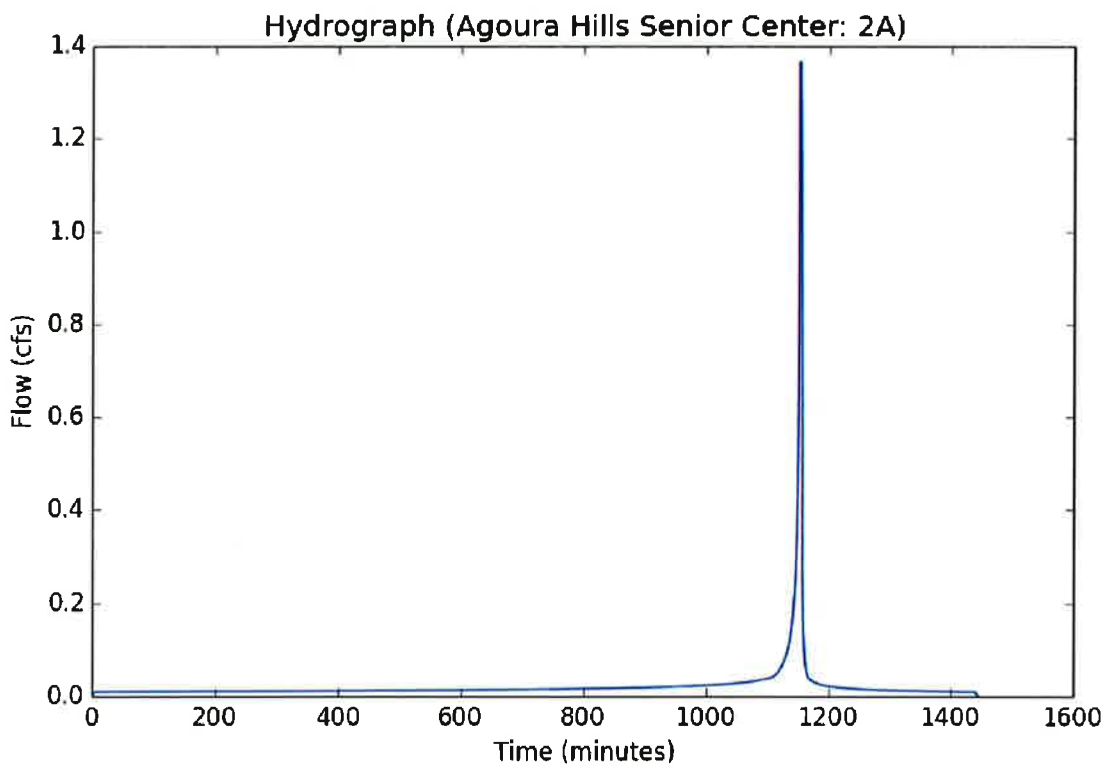
File location: R:/R305871.01 - Agoura Senior Village/10 CADD/10.1 AutoCAD/Hydrology/Agoura Hills Senior Center Report.10.pdf
Version: HydroCalc 0.3.1-beta

Input Parameters

Project Name	Agoura Hills Senior Center
Subarea ID	2A
Area (ac)	0.71
Flow Path Length (ft)	219.0
Flow Path Slope (vft/hft)	0.304
50-yr Rainfall Depth (in)	7.37
Percent Impervious	0.01
Soil Type	28
Design Storm Frequency	10-yr
Fire Factor	0
LID	False

Output Results

Modeled (10-yr) Rainfall Depth (in)	5.2622
Peak Intensity (in/hr)	3.1396
Undeveloped Runoff Coefficient (Cu)	0.6094
Developed Runoff Coefficient (Cd)	0.6123
Time of Concentration (min)	5.0
Clear Peak Flow Rate (cfs)	1.3648
Burned Peak Flow Rate (cfs)	1.3648
24-Hr Clear Runoff Volume (ac-ft)	0.0451
24-Hr Clear Runoff Volume (cu-ft)	1963.3373



Peak Flow Hydrologic Analysis

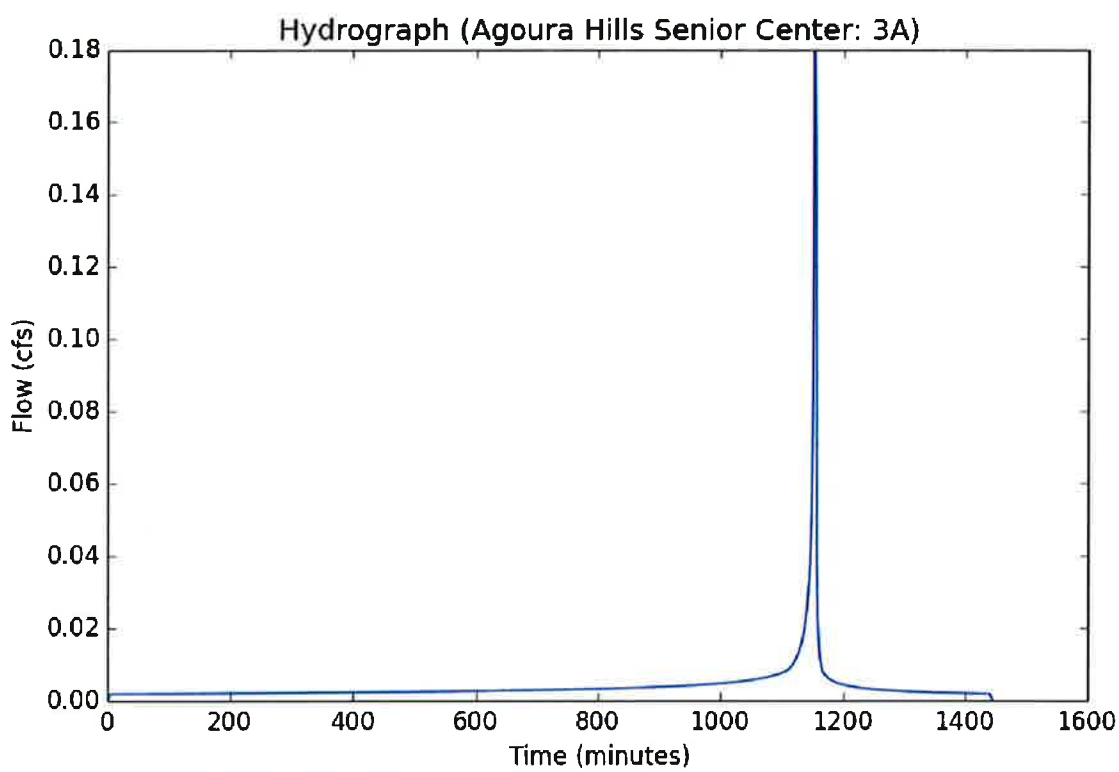
File location: R:/R305871.01 - Agoura Senior Village/10 CADD/10.1 AutoCAD/Hydrology/Agoura Hills Senior Center Report.10.pdf
Version: HydroCalc 0.3.1-beta

Input Parameters

Project Name	Agoura Hills Senior Center
Subarea ID	3A
Area (ac)	0.09
Flow Path Length (ft)	88.0
Flow Path Slope (vft/hft)	0.087
50-yr Rainfall Depth (in)	7.37
Percent Impervious	0.09
Soil Type	28
Design Storm Frequency	10-yr
Fire Factor	0
LID	False

Output Results

Modeled (10-yr) Rainfall Depth (in)	5.2622
Peak Intensity (in/hr)	3.1396
Undeveloped Runoff Coefficient (Cu)	0.6094
Developed Runoff Coefficient (Cd)	0.6355
Time of Concentration (min)	5.0
Clear Peak Flow Rate (cfs)	0.1796
Burned Peak Flow Rate (cfs)	0.1796
24-Hr Clear Runoff Volume (ac-ft)	0.0081
24-Hr Clear Runoff Volume (cu-ft)	352.7589



Peak Flow Hydrologic Analysis

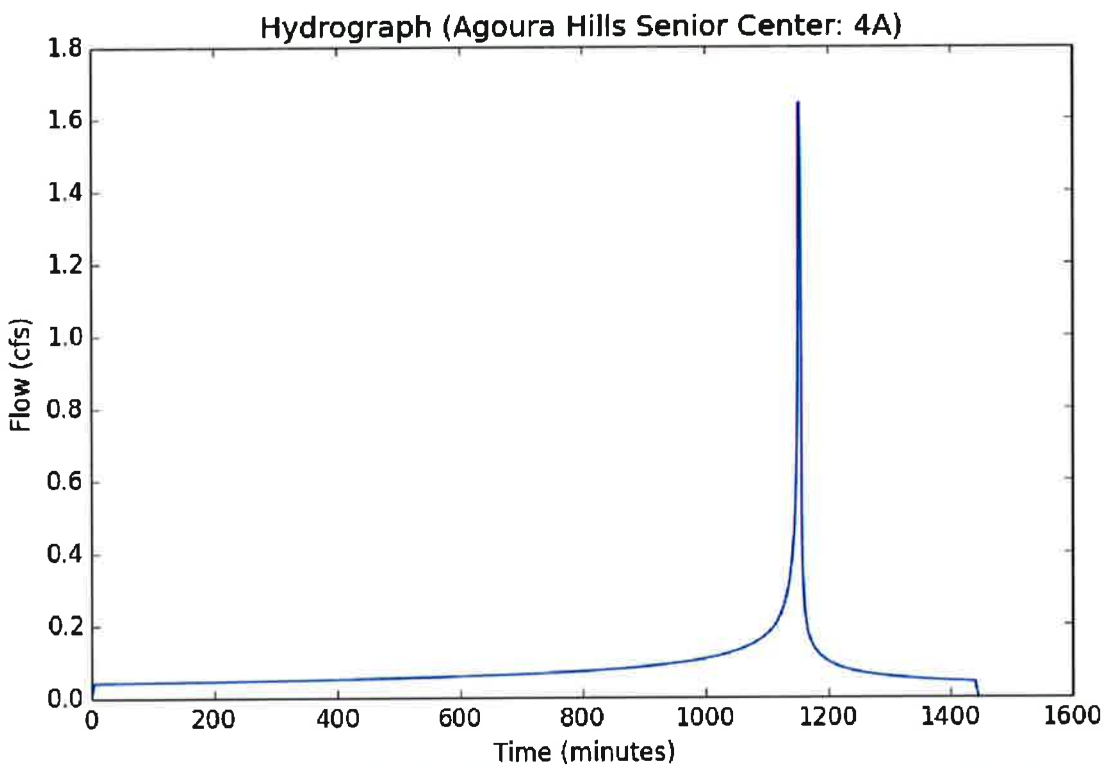
File location: R:/R305871.01 - Agoura Senior Village/10 CADD/10.1 AutoCAD/Hydrology/Agoura Hills Senior Center Report.10.pdf
Version: HydroCalc 0.3.1-beta

Input Parameters

Project Name	Agoura Hills Senior Center
Subarea ID	4A
Area (ac)	0.69
Flow Path Length (ft)	231.0
Flow Path Slope (vft/hft)	0.013
50-yr Rainfall Depth (in)	7.37
Percent Impervious	0.51
Soil Type	28
Design Storm Frequency	10-yr
Fire Factor	0
LID	False

Output Results

Modeled (10-yr) Rainfall Depth (in)	5.2622
Peak Intensity (in/hr)	3.1396
Undeveloped Runoff Coefficient (Cu)	0.6094
Developed Runoff Coefficient (Cd)	0.7576
Time of Concentration (min)	5.0
Clear Peak Flow Rate (cfs)	1.6412
Burned Peak Flow Rate (cfs)	1.6412
24-Hr Clear Runoff Volume (ac-ft)	0.1581
24-Hr Clear Runoff Volume (cu-ft)	6885.861



Peak Flow Hydrologic Analysis

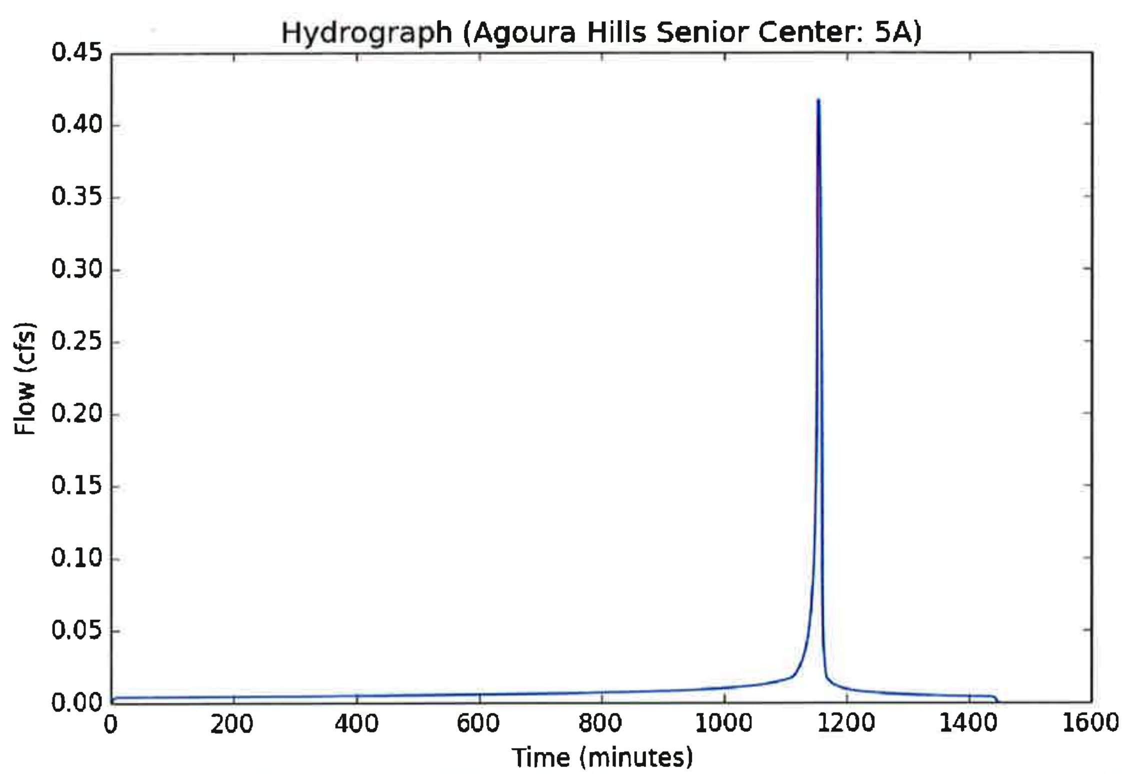
File location: R:/R305871.01 - Agoura Senior Village/10 CADD/10.1 AutoCAD/Hydrology/Agoura Hills Senior Center Report.10.pdf
Version: HydroCalc 0.3.1-beta

Input Parameters

Project Name	Agoura Hills Senior Center
Subarea ID	5A
Area (ac)	0.3
Flow Path Length (ft)	599.0
Flow Path Slope (vft/hft)	0.076
50-yr Rainfall Depth (in)	7.37
Percent Impervious	0.01
Soil Type	28
Design Storm Frequency	10-yr
Fire Factor	0
LID	False

Output Results

Modeled (10-yr) Rainfall Depth (in)	5.2622
Peak Intensity (in/hr)	2.5173
Undeveloped Runoff Coefficient (Cu)	0.5485
Developed Runoff Coefficient (Cd)	0.5521
Time of Concentration (min)	8.0
Clear Peak Flow Rate (cfs)	0.4169
Burned Peak Flow Rate (cfs)	0.4169
24-Hr Clear Runoff Volume (ac-ft)	0.0189
24-Hr Clear Runoff Volume (cu-ft)	822.3781



Peak Flow Hydrologic Analysis

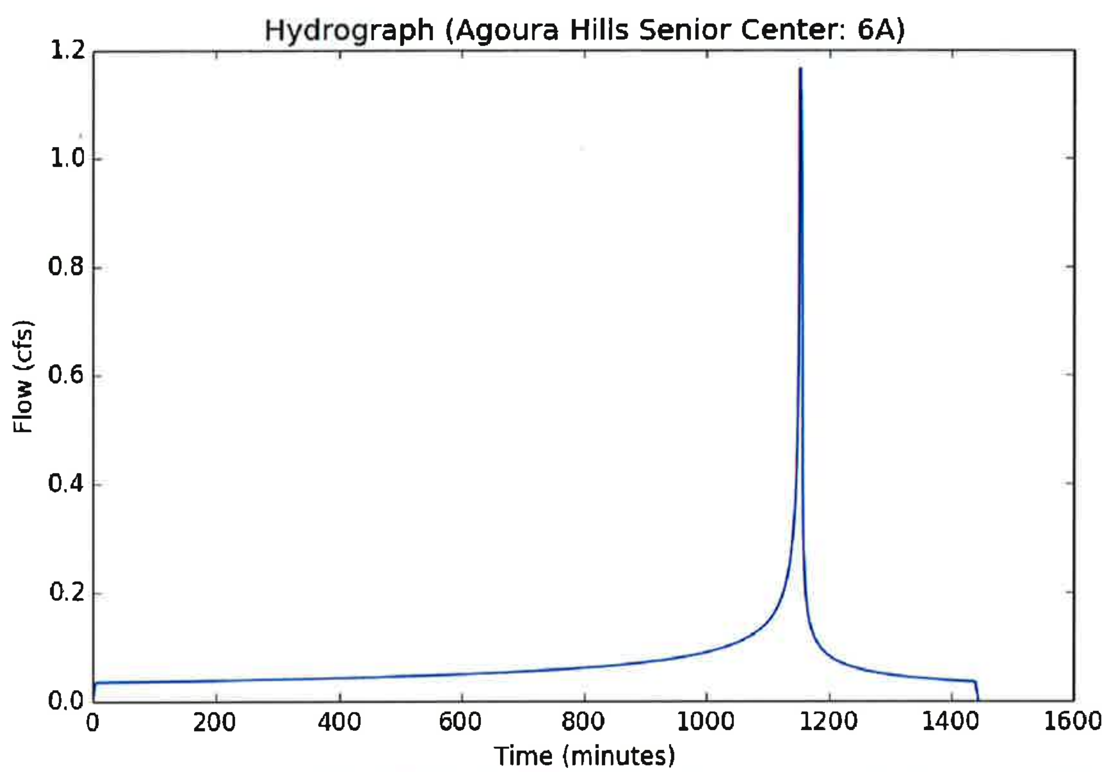
File location: R:/R305871.01 - Agoura Senior Village/10 CADD/10.1 AutoCAD/Hydrology/Agoura Hills Senior Center Report.10.pdf
Version: HydroCalc 0.3.1-beta

Input Parameters

Project Name	Agoura Hills Senior Center
Subarea ID	6A
Area (ac)	0.46
Flow Path Length (ft)	162.0
Flow Path Slope (vft/hft)	0.021
50-yr Rainfall Depth (in)	7.37
Percent Impervious	0.68
Soil Type	28
Design Storm Frequency	10-yr
Fire Factor	0
LID	False

Output Results

Modeled (10-yr) Rainfall Depth (in)	5.2622
Peak Intensity (in/hr)	3.1396
Undeveloped Runoff Coefficient (Cu)	0.6094
Developed Runoff Coefficient (Cd)	0.807
Time of Concentration (min)	5.0
Clear Peak Flow Rate (cfs)	1.1655
Burned Peak Flow Rate (cfs)	1.1655
24-Hr Clear Runoff Volume (ac-ft)	0.1313
24-Hr Clear Runoff Volume (cu-ft)	5718.8819



Peak Flow Hydrologic Analysis

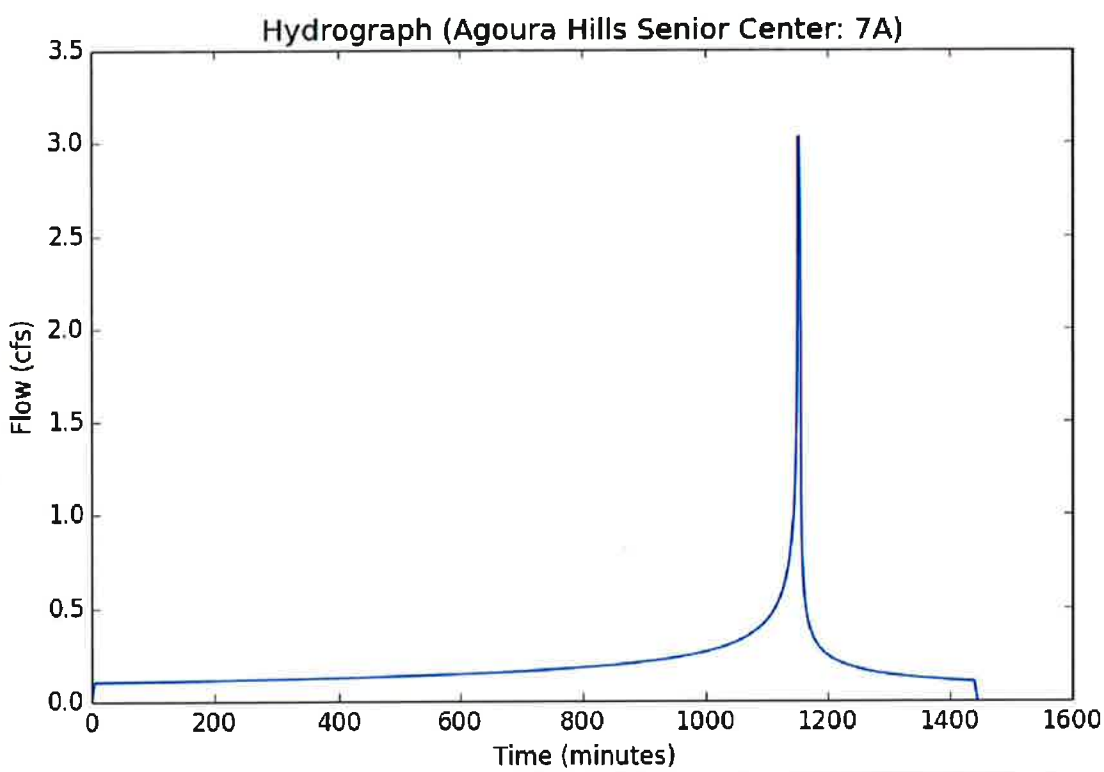
File location: R:/R305871.01 - Agoura Senior Village/10 CADD/10.1 AutoCAD/Hydrology/Agoura Hills Senior Center Report.10.pdf
Version: HydroCalc 0.3.1-beta

Input Parameters

Project Name	Agoura Hills Senior Center
Subarea ID	7A
Area (ac)	1.13
Flow Path Length (ft)	44.0
Flow Path Slope (vft/hft)	0.01
50-yr Rainfall Depth (in)	7.37
Percent Impervious	0.84
Soil Type	28
Design Storm Frequency	10-yr
Fire Factor	0
LID	False

Output Results

Modeled (10-yr) Rainfall Depth (in)	5.2622
Peak Intensity (in/hr)	3.1396
Undeveloped Runoff Coefficient (Cu)	0.6094
Developed Runoff Coefficient (Cd)	0.8535
Time of Concentration (min)	5.0
Clear Peak Flow Rate (cfs)	3.028
Burned Peak Flow Rate (cfs)	3.028
24-Hr Clear Runoff Volume (ac-ft)	0.3824
24-Hr Clear Runoff Volume (cu-ft)	16657.2285



Peak Flow Hydrologic Analysis

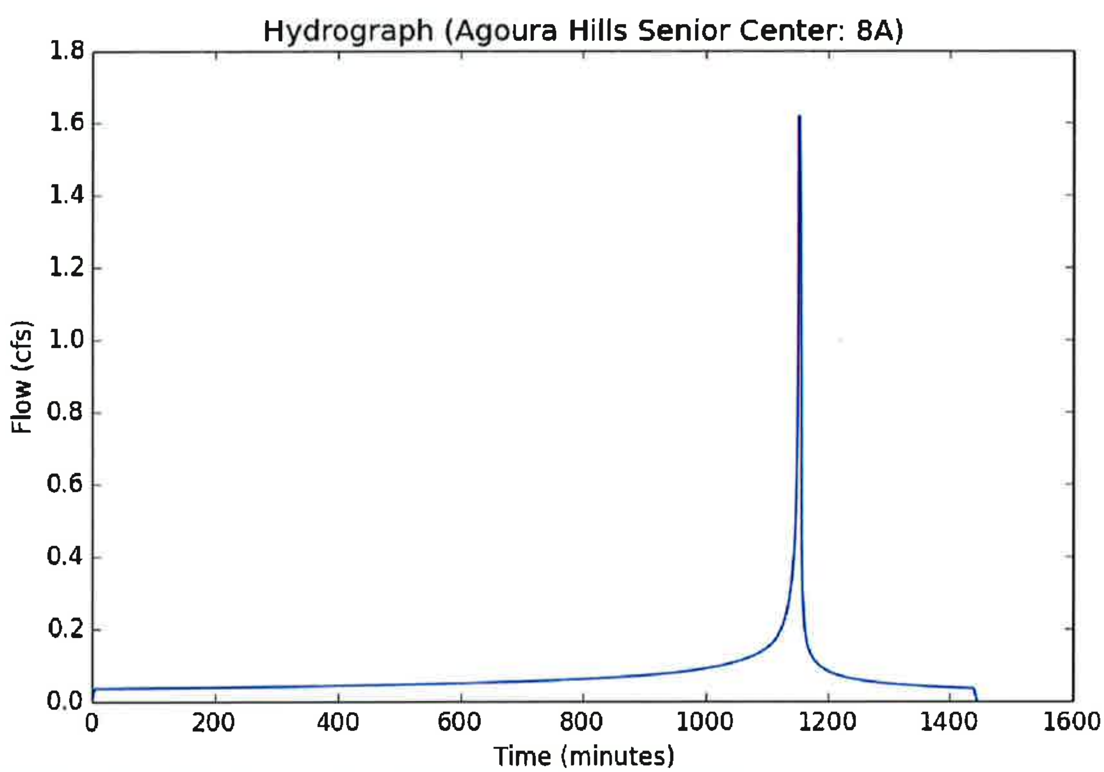
File location: R:/R305871.01 - Agoura Senior Village/10 CADD/10.1 AutoCAD/Hydrology/Agoura Hills Senior Center Report.10.pdf
Version: HydroCalc 0.3.1-beta

Input Parameters

Project Name	Agoura Hills Senior Center
Subarea ID	8A
Area (ac)	0.71
Flow Path Length (ft)	296.0
Flow Path Slope (vft/hft)	0.148
50-yr Rainfall Depth (in)	7.37
Percent Impervious	0.4
Soil Type	28
Design Storm Frequency	10-yr
Fire Factor	0
LID	False

Output Results

Modeled (10-yr) Rainfall Depth (in)	5.2622
Peak Intensity (in/hr)	3.1396
Undeveloped Runoff Coefficient (Cu)	0.6094
Developed Runoff Coefficient (Cd)	0.7256
Time of Concentration (min)	5.0
Clear Peak Flow Rate (cfs)	1.6175
Burned Peak Flow Rate (cfs)	1.6175
24-Hr Clear Runoff Volume (ac-ft)	0.1368
24-Hr Clear Runoff Volume (cu-ft)	5958.5861



Appendix D

Proposed Conditions SWQDv Hydrology Calculations

Peak Flow Hydrologic Analysis

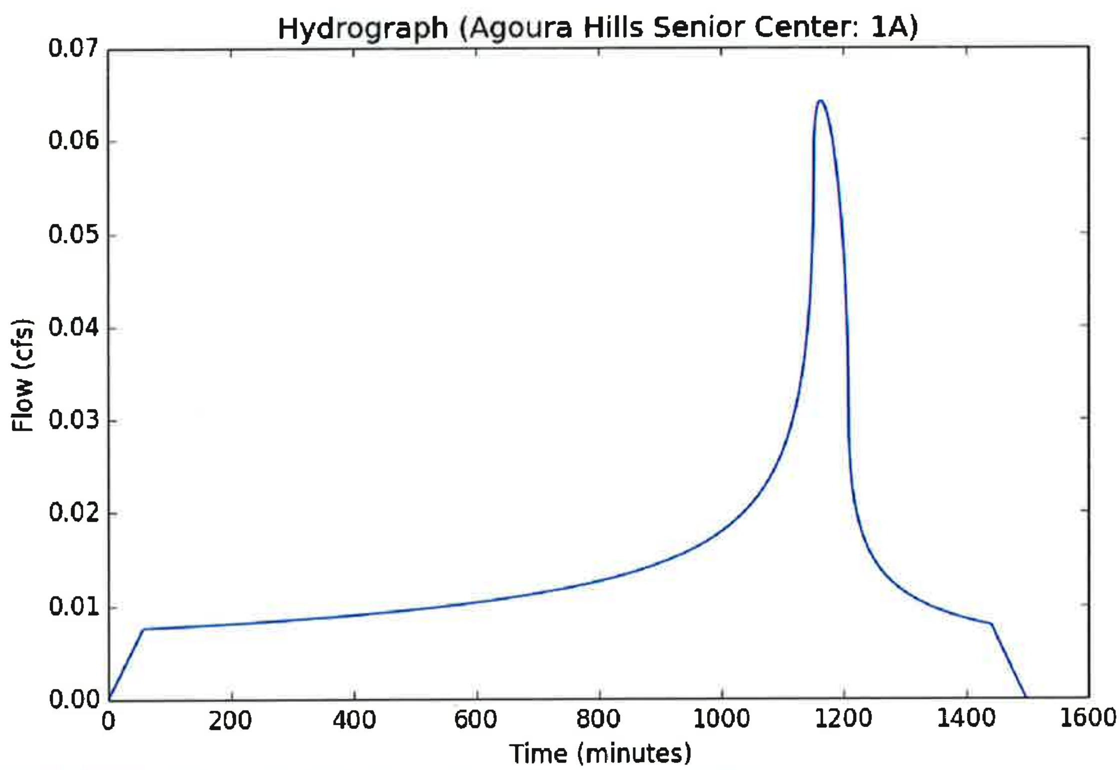
File location: R:/R305871.01 - Agoura Senior Village/10 CADD/10.1 AutoCAD/Hydrology/Agoura Hills Senior Center Report.WQ.pdf
Version: HydroCalc 0.3.1-beta

Input Parameters

Project Name	Agoura Hills Senior Center
Subarea ID	1A
Area (ac)	3.29
Flow Path Length (ft)	386.0
Flow Path Slope (vft/hft)	0.111
85th Percentile Rainfall Depth (in)	0.95
Percent Impervious	0.01
Soil Type	28
Design Storm Frequency	85th percentile storm
Fire Factor	0
LID	True

Output Results

Modeled (85th percentile storm) Rainfall Depth (in)	0.95
Peak Intensity (in/hr)	0.1806
Undeveloped Runoff Coefficient (Cu)	0.1
Developed Runoff Coefficient (Cd)	0.108
Time of Concentration (min)	57.0
Clear Peak Flow Rate (cfs)	0.0642
Burned Peak Flow Rate (cfs)	0.0642
24-Hr Clear Runoff Volume (ac-ft)	0.0279
24-Hr Clear Runoff Volume (cu-ft)	1215.2457



Peak Flow Hydrologic Analysis

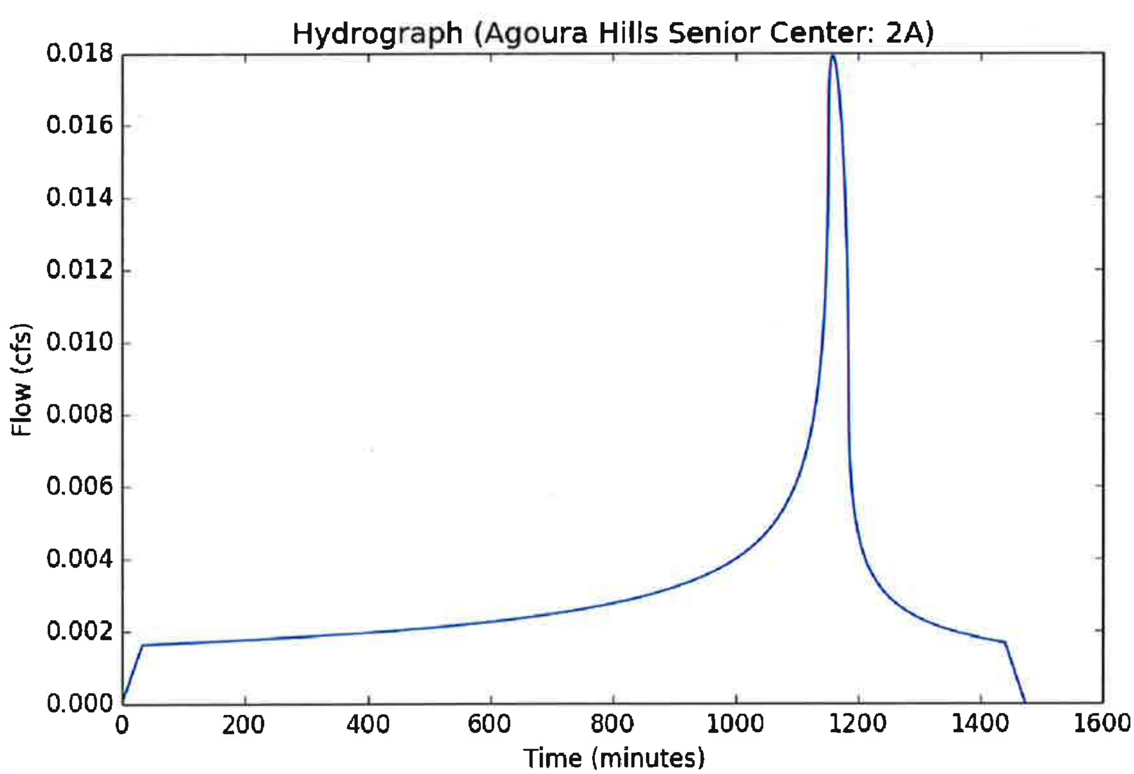
File location: R:/R305871.01 - Agoura Senior Village/10 CADD/10.1 AutoCAD/Hydrology/Agoura Hills Senior Center Report.WQ.pdf
Version: HydroCalc 0.3.1-beta

Input Parameters

Project Name	Agoura Hills Senior Center
Subarea ID	2A
Area (ac)	0.71
Flow Path Length (ft)	219.0
Flow Path Slope (vft/hft)	0.304
85th Percentile Rainfall Depth (in)	0.95
Percent Impervious	0.01
Soil Type	28
Design Storm Frequency	85th percentile storm
Fire Factor	0
LID	True

Output Results

Modeled (85th percentile storm) Rainfall Depth (in)	0.95
Peak Intensity (in/hr)	0.2335
Undeveloped Runoff Coefficient (Cu)	0.1
Developed Runoff Coefficient (Cd)	0.108
Time of Concentration (min)	33.0
Clear Peak Flow Rate (cfs)	0.0179
Burned Peak Flow Rate (cfs)	0.0179
24-Hr Clear Runoff Volume (ac-ft)	0.006
24-Hr Clear Runoff Volume (cu-ft)	262.2493



Peak Flow Hydrologic Analysis

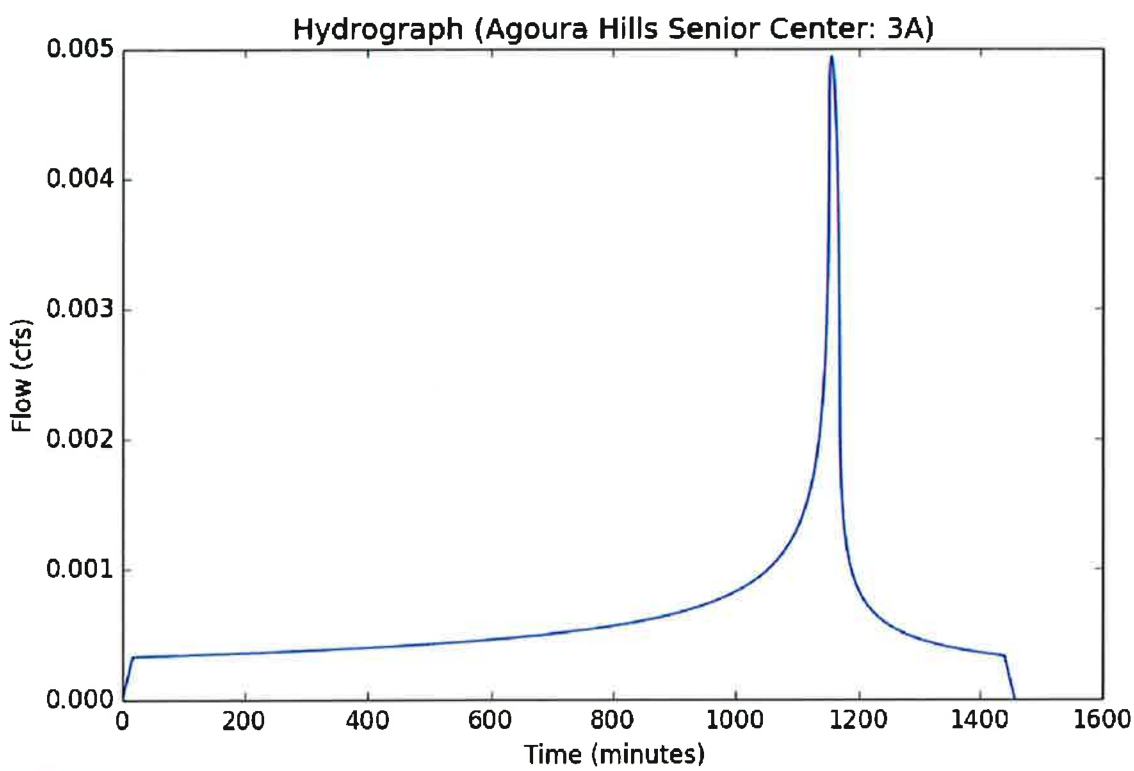
File location: R:/R305871.01 - Agoura Senior Village/10 CADD/10.1 AutoCAD/Hydrology/Agoura Hills Senior Center Report.WQ.pdf
Version: HydroCalc 0.3.1-beta

Input Parameters

Project Name	Agoura Hills Senior Center
Subarea ID	3A
Area (ac)	0.09
Flow Path Length (ft)	88.0
Flow Path Slope (vft/hft)	0.087
85th Percentile Rainfall Depth (in)	0.95
Percent Impervious	0.09
Soil Type	28
Design Storm Frequency	85th percentile storm
Fire Factor	0
LID	True

Output Results

Modeled (85th percentile storm) Rainfall Depth (in)	0.95
Peak Intensity (in/hr)	0.3189
Undeveloped Runoff Coefficient (Cu)	0.1
Developed Runoff Coefficient (Cd)	0.172
Time of Concentration (min)	17.0
Clear Peak Flow Rate (cfs)	0.0049
Burned Peak Flow Rate (cfs)	0.0049
24-Hr Clear Runoff Volume (ac-ft)	0.0012
24-Hr Clear Runoff Volume (cu-ft)	52.9418



Peak Flow Hydrologic Analysis

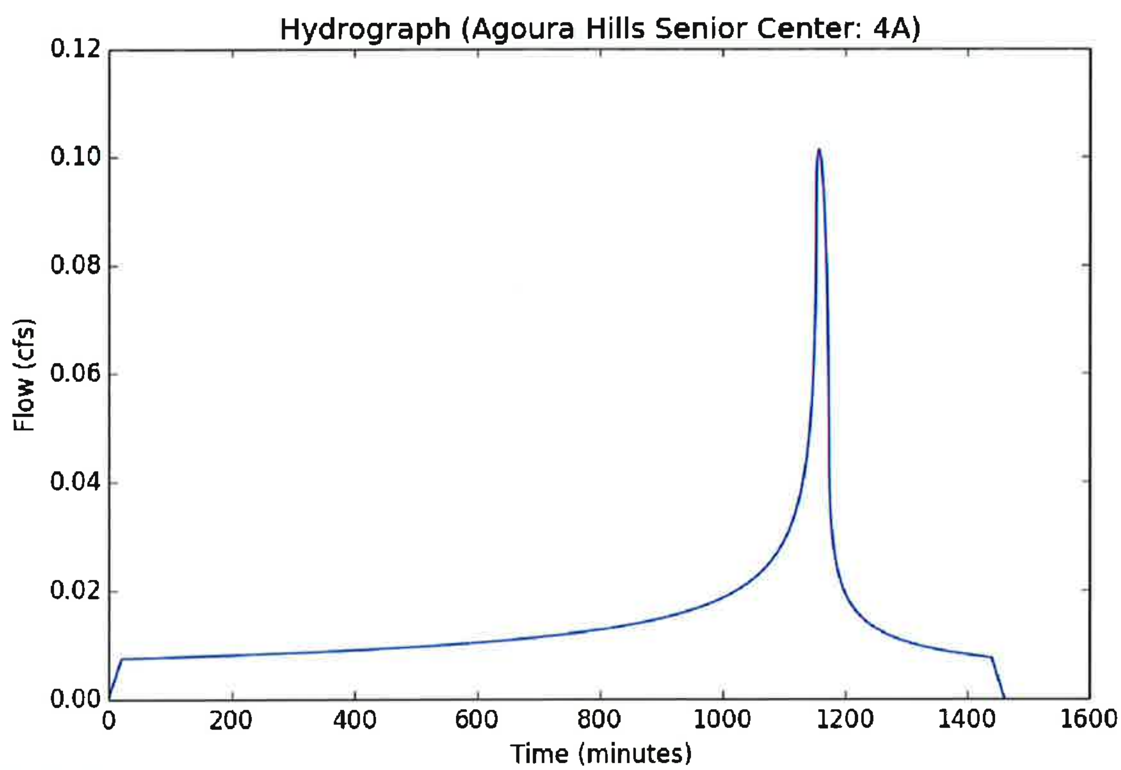
File location: R:/R305871.01 - Agoura Senior Village/10 CADD/10.1 AutoCAD/Hydrology/Agoura Hills Senior Center Report.WQ.pdf
Version: HydroCalc 0.3.1-beta

Input Parameters

Project Name	Agoura Hills Senior Center
Subarea ID	4A
Area (ac)	0.69
Flow Path Length (ft)	231.0
Flow Path Slope (vft/hft)	0.013
85th Percentile Rainfall Depth (in)	0.95
Percent Impervious	0.51
Soil Type	28
Design Storm Frequency	85th percentile storm
Fire Factor	0
LID	True

Output Results

Modeled (85th percentile storm) Rainfall Depth (in)	0.95
Peak Intensity (in/hr)	0.2887
Undeveloped Runoff Coefficient (Cu)	0.1
Developed Runoff Coefficient (Cd)	0.508
Time of Concentration (min)	21.0
Clear Peak Flow Rate (cfs)	0.1012
Burned Peak Flow Rate (cfs)	0.1012
24-Hr Clear Runoff Volume (ac-ft)	0.0275
24-Hr Clear Runoff Volume (cu-ft)	1198.7851



Peak Flow Hydrologic Analysis

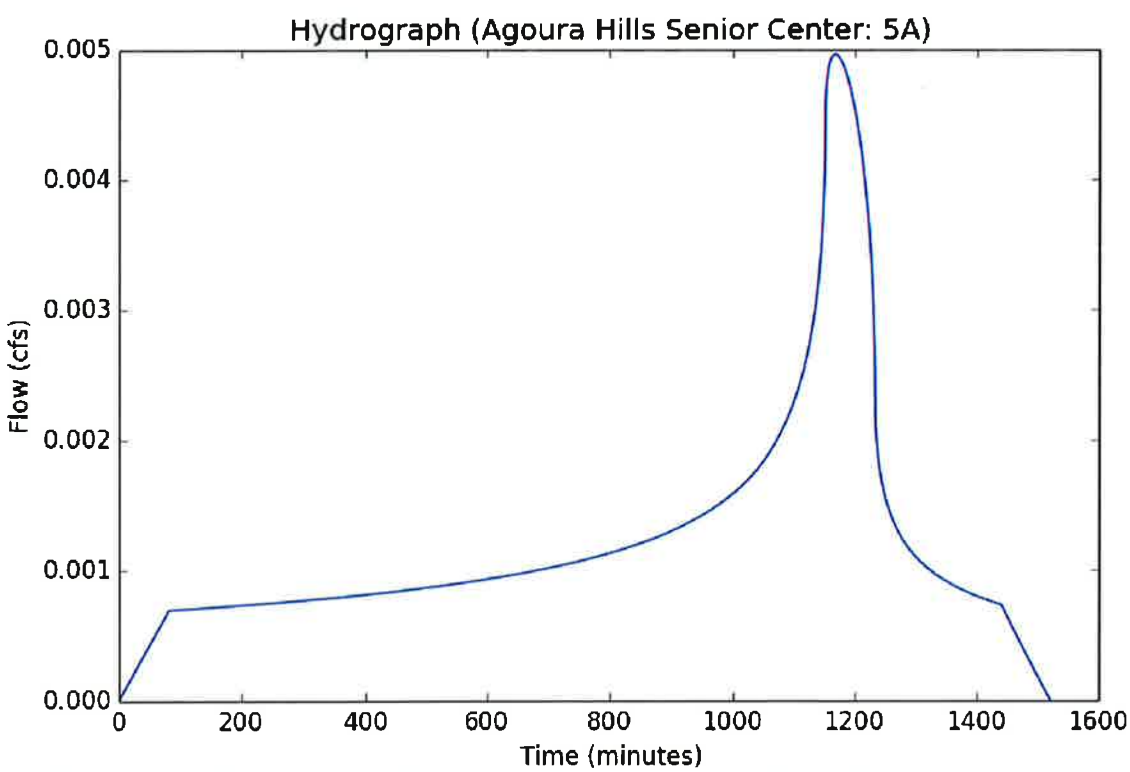
File location: R:/R305871.01 - Agoura Senior Village/10 CADD/10.1 AutoCAD/Hydrology/Agoura Hills Senior Center Report.WQ.pdf
Version: HydroCalc 0.3.1-beta

Input Parameters

Project Name	Agoura Hills Senior Center
Subarea ID	5A
Area (ac)	0.3
Flow Path Length (ft)	599.0
Flow Path Slope (vft/hft)	0.076
85th Percentile Rainfall Depth (in)	0.95
Percent Impervious	0.01
Soil Type	28
Design Storm Frequency	85th percentile storm
Fire Factor	0
LID	True

Output Results

Modeled (85th percentile storm) Rainfall Depth (in)	0.95
Peak Intensity (in/hr)	0.1531
Undeveloped Runoff Coefficient (Cu)	0.1
Developed Runoff Coefficient (Cd)	0.108
Time of Concentration (min)	81.0
Clear Peak Flow Rate (cfs)	0.005
Burned Peak Flow Rate (cfs)	0.005
24-Hr Clear Runoff Volume (ac-ft)	0.0025
24-Hr Clear Runoff Volume (cu-ft)	110.8176



Peak Flow Hydrologic Analysis

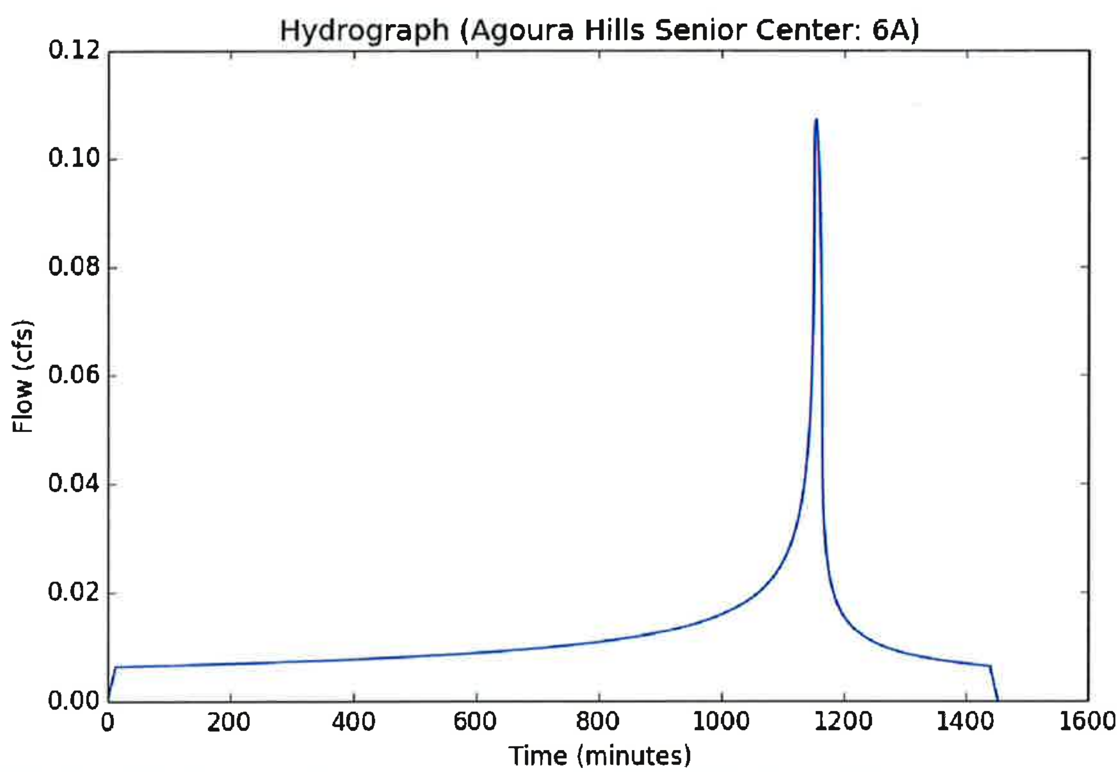
File location: R:/R305871.01 - Agoura Senior Village/10 CADD/10.1 AutoCAD/Hydrology/Agoura Hills Senior Center Report.WQ.pdf
Version: HydroCalc 0.3.1-beta

Input Parameters

Project Name	Agoura Hills Senior Center
Subarea ID	6A
Area (ac)	0.46
Flow Path Length (ft)	162.0
Flow Path Slope (vft/hft)	0.021
85th Percentile Rainfall Depth (in)	0.95
Percent Impervious	0.68
Soil Type	28
Design Storm Frequency	85th percentile storm
Fire Factor	0
LID	True

Output Results

Modeled (85th percentile storm) Rainfall Depth (in)	0.95
Peak Intensity (in/hr)	0.3617
Undeveloped Runoff Coefficient (Cu)	0.1
Developed Runoff Coefficient (Cd)	0.644
Time of Concentration (min)	13.0
Clear Peak Flow Rate (cfs)	0.1072
Burned Peak Flow Rate (cfs)	0.1072
24-Hr Clear Runoff Volume (ac-ft)	0.0233
24-Hr Clear Runoff Volume (cu-ft)	1013.143



Peak Flow Hydrologic Analysis

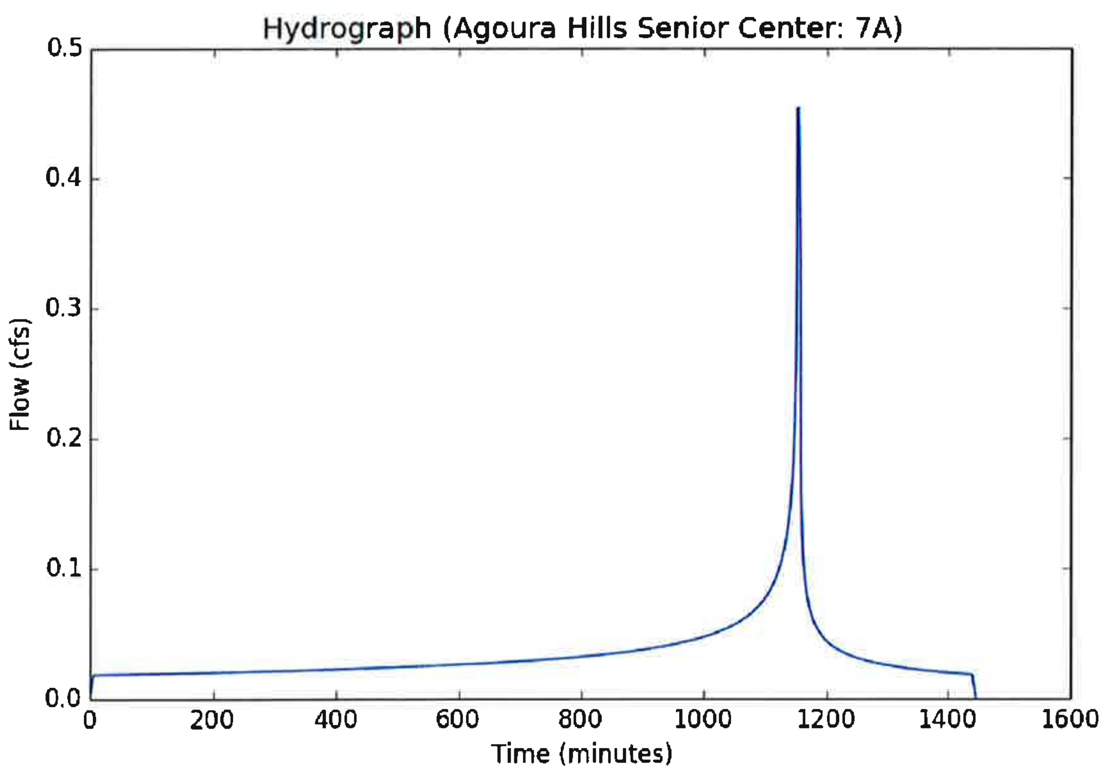
File location: R:/R305871.01 - Agoura Senior Village/10 CADD/10.1 AutoCAD/Hydrology/Agoura Hills Senior Center Report.WQ.pdf
Version: HydroCalc 0.3.1-beta

Input Parameters

Project Name	Agoura Hills Senior Center
Subarea ID	7A
Area (ac)	1.13
Flow Path Length (ft)	44.0
Flow Path Slope (vft/hft)	0.01
85th Percentile Rainfall Depth (in)	0.95
Percent Impervious	0.84
Soil Type	28
Design Storm Frequency	85th percentile storm
Fire Factor	0
LID	True

Output Results

Modeled (85th percentile storm) Rainfall Depth (in)	0.95
Peak Intensity (in/hr)	0.5202
Undeveloped Runoff Coefficient (Cu)	0.1
Developed Runoff Coefficient (Cd)	0.772
Time of Concentration (min)	6.0
Clear Peak Flow Rate (cfs)	0.4538
Burned Peak Flow Rate (cfs)	0.4538
24-Hr Clear Runoff Volume (ac-ft)	0.0685
24-Hr Clear Runoff Volume (cu-ft)	2983.4725



Peak Flow Hydrologic Analysis

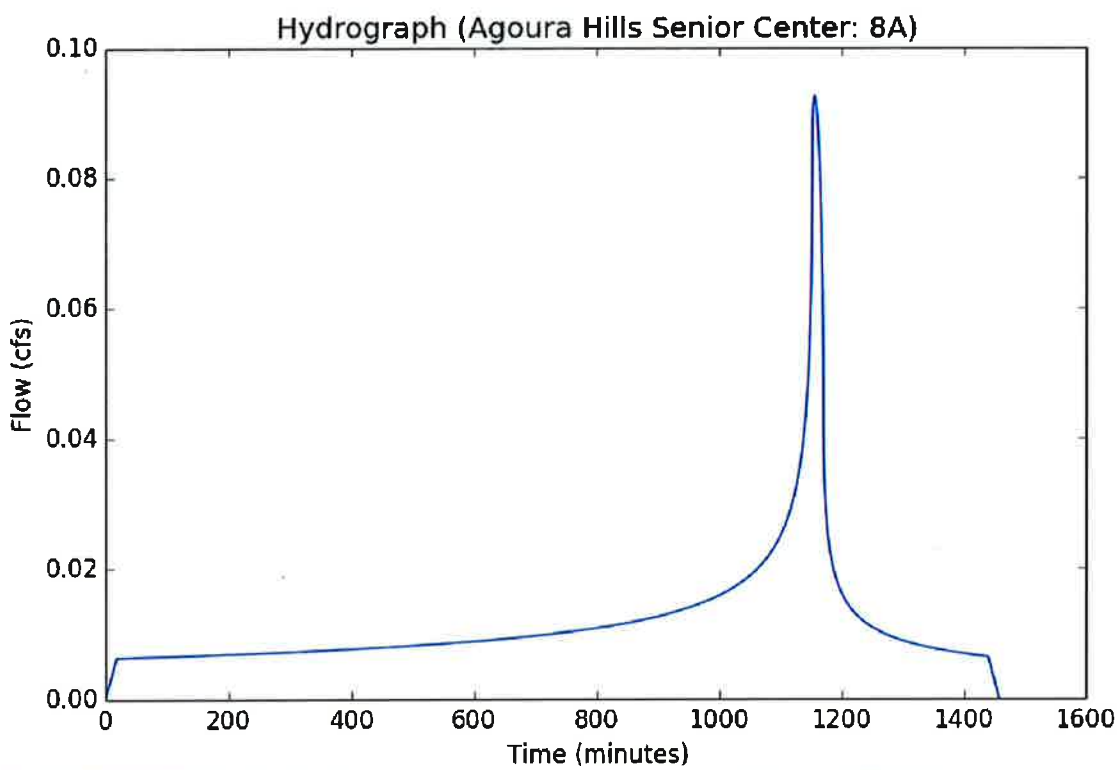
File location: R:/R305871.01 - Agoura Senior Village/10 CADD/10.1 AutoCAD/Hydrology/Agoura Hills Senior Center Report.WQ.pdf
Version: HydroCalc 0.3.1-beta

Input Parameters

Project Name	Agoura Hills Senior Center
Subarea ID	8A
Area (ac)	0.71
Flow Path Length (ft)	296.0
Flow Path Slope (vft/hft)	0.148
85th Percentile Rainfall Depth (in)	0.95
Percent Impervious	0.4
Soil Type	28
Design Storm Frequency	85th percentile storm
Fire Factor	0
LID	True

Output Results

Modeled (85th percentile storm) Rainfall Depth (in)	0.95
Peak Intensity (in/hr)	0.3104
Undeveloped Runoff Coefficient (Cu)	0.1
Developed Runoff Coefficient (Cd)	0.42
Time of Concentration (min)	18.0
Clear Peak Flow Rate (cfs)	0.0926
Burned Peak Flow Rate (cfs)	0.0926
24-Hr Clear Runoff Volume (ac-ft)	0.0234
24-Hr Clear Runoff Volume (cu-ft)	1019.8482



Appendix E

Geotechnical Explorations

GEOTECHNICAL INVESTIGATION
OAKMONT OF AGOURA HILLS
29353 CANWOOD STREET
AGOURA HILLS, CALIFORNIA
PREPARED FOR
OAKMONT SENIOR LIVING
JOB NO. 15473-3

GEOTECHNICAL INVESTIGATION
OAKMONT OF AGOURA HILLS
29353 CANWOOD STREET
AGOURA HILLS, CALIFORNIA
PREPARED FOR
OAKMONT SENIOR LIVING
JOB NO. 15473-3

INTRODUCTION

During October of 2015, this firm performed a geotechnical investigation for the proposed Oakmont of Agoura Hills senior facility, which is to be located at 29353 Canwood Street (APN 2053-001-005), in the city of Agoura Hills, California. The purposes of this investigation were to explore and evaluate the geotechnical engineering/engineering geologic conditions of the site and to provide appropriate geotechnical engineering recommendations for the design and construction of the subject project.

The approximate location of the site is shown on the attached Index Map (Enclosure "A-1"). To orient our investigation, a site plan prepared by Landesign Group, Inc., showing the building location was provided for our use. The plan was utilized as a base map for our Site Plan (Enclosure "A-2").

The results of our investigation, together with our conclusions and recommendations, are presented in this report.

SCOPE OF SERVICES

The scope of services provided during this investigation included the following:

- Review of published and unpublished geologic literature and maps
- Field reconnaissance of the subject site and surrounding area and geologic mapping of the site
- Marking of exploration locations in the field and notification of Underground Service Alert
- Placement of four exploratory borings within the building pad area

- Placement of seven exploratory trenches within the site area
- Double-ring infiltrometer testing at two locations on the site
- Logging and sampling of the exploratory borings and test pits for testing and evaluation
- Laboratory testing on selected samples
- Evaluation of geologic hazards
- Seismic design parameters according to the 2013 California Building Code (CBC)
- Evaluation of the geotechnical data to develop site-specific recommendations for suitable foundation recommendations, including allowable bearing pressures, ultimate and allowable passive earth resistance and base friction, lateral earth pressures and mitigation of potential geotechnical concerns and hazards, such as expansive soils, liquefaction and seismic settlement, if encountered
- Preparation of this report summarizing our findings, professional opinions and recommendations for the geotechnical aspects of project design and construction

PROJECT CONSIDERATIONS

The proposed two- and three-story senior facility will include more than 80 units and will be approximately 80,000 square feet in plan area. We anticipate that the facility will be of wood frame and stucco or masonry construction. Light to moderate foundations loads are typically associated with structures of the type proposed.

Our review of furnished plans indicates that the site elevation varies approximately 120 feet, with the highest elevation of approximately 1,000 feet at the northeast corner and the lowest of approximately 880 feet at the southwest corner. The northern portion of the building pad (2-story portion) will be at elevation 912 feet and the southern portion of the building pad (3-story portion) will be at elevation 902 feet. Based on this information, we anticipate that the building pad and foundations will be stepped. Per our conversation with the client, post-tension slab foundations are anticipated. We expect that the slope on the north side of the building pad will be cut to provide a level building pad and that stepped retaining walls will be required for slope stability purposes. The slope cut will be on the order of 20 feet.

The final project grading plan should be reviewed by the geotechnical engineer to confirm that recommendations provided in this report have been properly implemented.

SITE DESCRIPTION

The site is located along a freeway frontage road on the north side of the 101 freeway, west of the Kanan Road off-ramp. At the time of our investigation, commercial buildings were located west of the site, and undeveloped land was located to the north and east. The site slopes up at a gentle grade north from Kanan Road to the toe of an approximately 2 horizontal (h) to 1 vertical (v) slope located north of the proposed building area. Debris and evidence of an abandoned structure and foundation area were present in the northeastern portion of the site.

Historic aerial imagery dating from 1947 was examined as part of this investigation. At the time of the 1947 aerial image, the site and surrounding area were undeveloped land. By the time of the 1959 aerial image, several structures were present on the north portion of the site. These structures remained on the site until the time of the 1980 aerial image, when the site appeared in its present condition, with debris in the northeastern portion of the site. Construction began on the commercial structures west of the site by the time of the image dated December 31, 2006, and was completed between the time of the image dated January 8, 2008, and May 24, 2009.

FIELD INVESTIGATION

Four exploratory borings were drilled to a maximum depth of 50-1/2 feet below the existing ground surface (bgs) using a limited-access (track mounted) hollow-stem auger drill rig equipped for soil sampling. In addition, seven trenches were excavated to depths ranging from 4 feet to 9-1/2 feet bgs. The exploratory trenches were used to evaluate the geologic structure of the bedrock. Two exploratory test pits were excavated in the proposed parking and driveway areas and were utilized to perform double-ring infiltrometer tests. The approximate locations of our exploratory borings, trenches and test pits are indicated on the attached Site Plan (Enclosure "A-2").

Continuous logs of the subsurface conditions, as encountered within the exploratory borings, were recorded at the time of drilling by a staff geologist from this firm. Both a standard penetration test (SPT) sampler (2-inch outer diameter and 1-3/8 inch inner diameter) and a modified California sampler (3-1/4-inch outer diameter and 2-3/8-inch inner diameter) were utilized in our investigation. Relatively undisturbed samples were obtained by driving the modified California sampler (a split-spoon ring sampler) ahead of the borings at selected levels. The penetration resistance was recorded on the boring logs as the number of hammer blows used to advance the sampler in 6-inch increments (or less if noted). The sampler is driven with an automatic hammer that drops a 140-pound weight 30 inches for each blow. After the required seating, the sampler is advanced up to 18 inches, providing up to three sets of blowcounts at each sampling interval. The recorded blows are raw numbers without any corrections for hammer type (automatic vs. manual cathead) or sampler size (California sampler vs. standard penetration test sampler). Both relatively undisturbed and bulk samples of typical soil types obtained were returned to the laboratory in sealed containers for testing and evaluation.

Our exploratory boring logs, together with our in-place blowcounts per 6-inch increment, are presented in Appendix "B". The stratification lines presented on the boring logs represent approximate boundaries between soil types, which may include gradual transitions.

LABORATORY INVESTIGATION

Included in our laboratory testing program were field moisture content tests on all samples returned to the laboratory and field dry density tests on all relatively undisturbed ring samples. The results are included on the boring logs. An optimum moisture content - maximum dry density relationship was established for a representative soil type. A direct shear test was performed on a selected remolded sample in order to provide shear strength parameters for bearing capacity and earth pressure evaluations. No. 200 wash, sieve analysis, sand equivalent and plasticity index testing was performed on selected samples in order to classify the subsurface soils encountered. Expansion index testing was performed on a selected sample to evaluate the expansion potential of the subsurface soils. Since the on-site soils are expansive, a sample was set up in the consolidation testing machine to determine expansive deformation strain and expansive pressure.

A selected sample of material was delivered to HDR for chemical/corrosivity testing.

Summaries of the laboratory test results appear in Appendix "C". Soil classifications provided in our geotechnical investigation are generally per the Unified Soil Classification System (USCS).

SITE GEOLOGY AND SUBSURFACE SOIL CONDITIONS

Regionally, the site is located in a valley within the Santa Monica Mountains of the Transverse Ranges geomorphic province. This province includes several discreet mountain ranges and intervening valleys including the Santa Monica, San Gabriel and San Bernardino Mountains and is so named because structural trends, such as the Simi-Santa Rosa fault zone, are oriented east-west in relation to the dominant northwest-southeast trend of adjoining provinces. The Transverse Ranges province extends from the Channel Islands eastward to the Eagle and Cottonwood Mountains of the Mojave Desert. As depicted on published geologic mapping, the site is underlain by the Upper Topanga formation, which is a Miocene-age sedimentary bedrock consisting of interbedded shale, siltstone and sandstone, and Miocene-age Conejo Volcanics (Dibblee, and Ehrenspeck, 1993, Enclosure "A-3").

As encountered in the explorations, the site is mantled by colluvial fill to depths from approximately 3 to 5 feet below ground surface. The fill materials encountered consisted of medium dense to dense clayey sand (SC) and stiff to hard fat clay (CH). The bedrock was encountered at depths of 3 to 10 feet bgs and consisted of Topanga Formation Siltstone recovered as silty and clayey sands (SM, SC), clays (CL, CH) and silt (ML).

Groundwater or seepage was not encountered in the explorations. Refusal was not encountered in the explorations to the maximum 50-1/2 foot depth. Caving was not encountered upon removal of the drilling augers.

More detailed descriptions of the subsurface soil conditions encountered are presented on the attached boring logs (Appendix "B").

FAULTING

The site does not lie within or immediately adjacent to an Alquist-Priolo Earthquake Fault Zone designated by the State of California to include traces of suspected active faulting. The closest known fault is a segment of the Chatsworth fault that is located approximately 4.5 miles to the northeast. The Malibu fault, Santa Monica fault, Sierra Madre fault zone and San Gabriel fault zone are the nearest known faults to the site and are located 7.6 miles south, 9.5 miles southeast, 14 miles northeast and 22.5 miles northeast of the site, respectively. No faults are shown on or in the immediate vicinity of the site on published geologic maps.

SEISMICITY

A map of recorded earthquake epicenters is included as Enclosure "A-4" (Epi Software, 2000). This map includes a database maintained by the Southern California Earthquake Center (University of Southern California) for earthquakes with magnitudes of 4.0 or greater from 1932 through 2012. The following table summarizes earthquakes that have occurred in the region of the site.

Summary of Historic Earthquakes				
Event ID	**Date**	**Magnitude**	**Distance from Site (miles)**	**Direction from Site**
Lake Matthews Area	4/21/1918	6.6	79	SE
Long Beach	3/10/1933	6.4	58	SE
Fish Creek Mountains	10/21/1942	6.6	178	SE
Borrego Mountain	4/9/1968	6.5	164	SE
West Hollywood	9/9/2001	5.9	21.5	SE
Whittier Narrows	10/1/1987	5.9	39	SE
Upland	2/28/1990	5.4	61	E
Sierra Madre	6/28/1991	5.8	46	NE
Mojave	7/11/1992	5.7	85	NE
Landers	6/28/1992	7.3	133	NE
Big Bear	6/28/1992	6.4	111	E
Northridge	1/17/1994	6.7	14	NE
Hector Mine	10/16/1999	7.1	147	NE
Fort Tejon	1/9/1857	7.9	134	NW
Chino Hills	7/29/2008	5.4	59	SE
Kern County (Tehachapi)	7/21/1952	7.3	62	NW
Inglewood	5/17/2009	4.7	28	SE
Upland	6/26/1988	4.8	60	E
Yorba Linda	9/3/1992	4.8	59	SE
Sylmar	2/9/1971	6.6	28	NE

SEISMIC DESIGN PARAMETERS

Based on the geologic setting and blowcount data from subsurface explorations, the soils underlying the site are classified as Site Class "C", according to the 2013 CBC.

The seismic design parameters in accordance with Section 1613A of 2013 CBC are presented in Table 2.1. These values were determined using the web-based application http://earthquake.usgs.gov/designmaps/us/application.php and the site coordinates 34.1475, W118.7659. The deaggregated modal earthquake magnitude was determined from the USGS website http://geohazards.usgs.gov/deaggint/2008 for evaluation of soil effects due to earthquake ground shaking.

2013 CBC - Seismic Design Parameters	
Mapped Spectral Acceleration Parameters	$S_s = 1.559$ and $S_1 = 0.600$
Site Coefficients	$F_a = 1.0$ and $F_v = 1.3$
Adjusted Maximum Considered Earthquake Spectral Response Parameters	$S_{MS} = 1.559$ and $S_{M1} = 0.780$
Design Spectral Acceleration Parameters	$S_{DS} = 1.039$ and $S_{D1} = 0.520$
Geometric Mean Peak Ground Acceleration (PGA_M)	0.579g
Deaggregated Modal Magnitude	7.03

GROUNDWATER AND LIQUEFACTION

Depth-to-groundwater data from the State of California Water Resources Control Board (2015) and groundwater contour mapping by CGS (2000) were examined for the area of the site. These data are summarized in the following table.

Depth to Groundwater				
Well No./ID	**Date Measured**	**Depth to Water (feet)**	**Measuring Point Elevation (feet amsl)**	**Location**
T06037041688-W-05DD	8/25/2009	6	871	1/4 mile S
	1/22/2010	6		
T0603703142-MW-K	9/1/2002	12	900	1/3 mile E
	10/1/2006	8		
	7/6/2009	6		
	4//2012	11		
T-0603703449-W-14	1/14/2004	14	886	1/3 mile SE
	10/10/2006	16		
	12/27/2014	15		
Contour Mapping	Historic High	10	--	--

Groundwater was not encountered within the maximum 50-1/2-foot depth of the explorations. Based on historical data and a site elevation of 900 feet, the historic high depth to groundwater in the area of the site is estimated at approximately 10 feet bgs.

Liquefaction is a process in which strong ground shaking causes saturated soils to lose their strength and behave as a fluid. Ground failure associated with liquefaction can result in severe damage to structures. The geologic conditions for increased susceptibility to liquefaction are: 1) shallow groundwater (generally less than 50 feet in depth), 2) the presence of unconsolidated sandy alluvium, typically Holocene in age, and 3) strong ground shaking. All three of these conditions must be present for liquefaction to occur.

The site is not included in a State of California Seismic Hazard Zone for liquefaction or earthquake-induced landslide (CGS, 2000). Based on the composition of the underlying soils encountered in our geotechnical investigation and the relatively shallow depths of bedrock encountered at the site, liquefaction is not considered a potential hazard, and further investigation is not warranted.

SEISMIC SETTLEMENT

Severe seismic shaking may cause dry and saturated sands to densify, resulting in settlement expressed at the ground surface. Seismic settlement in dry soils generally occurs in loose sands and silty sands, with cohesive and fine-grained soils being less prone to significant settlement. For saturated soils, significant settlement is anticipated if the soils are liquefied during seismic shaking. Soil types susceptible to liquefaction include sand, silty sand, sandy silt and silt, as well as clayey soils with clay content less than 15 percent.

Topanga Formation siltstone was encountered at depths of 3 to 10 feet below the existing ground surface. Little to no alluvial sands were encountered in our investigation. Therefore, seismic settlement at the site is considered negligible.

STATIC SETTLEMENT

Potential static settlement was evaluated utilizing field and laboratory data and foundation load assumptions. We anticipate a total static settlement of less than 1 inch beneath foundations. Differential settlement is anticipated to be less than one-half the total settlement in 40 feet. Most of the potential static settlement should occur during construction.

HYDROCONSOLIDATION

Based on the relatively dense nature of the underlying near-surface soils encountered in our investigation, the minimum mandatory removal requirements as provided in the "Recommendations" section of this report and the low potential for full saturation of the soil layers, it is our opinion that the potential for hydrocollapse settlement at the site is low.

SUBSIDENCE

The site is not located within an area identified by the State of California Seismic Hazard Zone as having a potential for subsidence. The potential for subsidence to affect the proposed structure is considered low.

SLOPE STABILITY AND LANDSLIDE POTENTIAL

Based on information provided by the project civil engineer, a finished floor elevation of approximately 912 feet above mean sea level (amsl) is estimated for the project. The slope located on the northern portion of the site consists of tight, well-bedded siltstone with sandstone interbeds. Bedding was measured to dip to the north. Landslides were not observed within the site. The site is not located within a State-designated area as having a potential for landslide, seismically induced landslide or lateral spreading (CGS, 2000). Therefore, the potential for landsliding or lateral spreading is considered low.

Grading of cut or fill slopes, if needed to achieve final site configurations, should be conducted in conformance with applicable grading codes. On-site soils may be considered Type "B" with regard to 2013 CAL/OSHA excavation standards.

FLOODING AND EROSION

The site is not located in an area designated by the Federal Emergency Management Agency (2008) as a flood hazard zone. A more accurate determination of the flood hazard to the site and the adequacy of existing flood and drainage improvements near the site is not within the scope of this investigation.

No large water storage facilities are known to exist within the area of the site. The site is not located within a coastal area; therefore, tsunami is not a potential hazard to the site.

EXPANSION POTENTIAL

ASTM D4829 test standard classifies expansion index (EI) of soils as follows:

Expansion Index	Expansion Potential
0-20	Very Low
21-50	Low
51-90	Medium
91-130	High
Greater than 130	Very High

According to Section 1803.5.3 of the 2013 CBC, soils having an EI greater than 20 are considered "expansive" and require foundation design to mitigate these conditions as per Section 1808.6 of the 2013 CBC.

EI analysis according to the ASTM standard was performed by this firm. The result indicates EI values of 150 and 157 ("very high"). Based on these results, construction procedures and/or special structural design to specifically mitigate the effects of expansive soil movements are necessary. Recommendations to mitigate expansive soil conditions are provided in the "Expansive Soils" section of this report.

DOUBLE-RING INFILTROMETER TESTS

Two double-ring infiltrometer tests were performed to evaluate the infiltration potential of the site soils located within the proposed water retention area. The test locations are indicated on Enclosure "A-2". The tests were performed in general conformance with ASTM D3385 at depths of 3 and 5 feet below the existing ground surface utilizing a rubber-tire backhoe to excavate the test pits. Exploratory test pit logs are provided in Appendix "B".

The data collected were used to calculate the infiltration rate of the soil. The infiltration test was performed until a steady-state infiltration velocity was reached. The steady-state infiltration velocity is presented as the infiltration rate.

The infiltration rates are presented in the following table and do not include safety factors.

Test Number/Depth	Infiltration Rate	
	cm. / hr.	**in. / hr.**
P-1	0.13	0.05
P-2	0.07	0.03

The measured infiltration rates are within the applicable range of the test method. The measured infiltration rate to use in design is discussed in the "Storm Water Infiltration" section of this report. It should be noted that infiltration rates determined by testing are ultimate rates based on short-duration field test results. The infiltration tests utilized clear water, and infiltration rates can be affected by buildup of silt, debris, the degree of soil saturation and other factors. An appropriate safety factor should be applied to measured infiltration rates prior to use in design to accommodate potential subsoil inconsistencies, possible compaction related to site grading and potential silting of the percolating soils. A safety factor should be determined with consideration to other factors in the storm water retention system design, particularly storm water volume estimates and the safety factors associated with those design components.

CONCLUSIONS

On the basis of our research and field and laboratory investigations, it is the opinion of this firm that the proposed project is feasible from a geological and geotechnical engineering standpoint, provided the recommendations contained in this report are implemented during design and construction.

As encountered in the explorations, the site is mantled by colluvial fill to depths from approximately 3 to 5 feet below ground surface. The fill materials encountered consist of medium dense to dense clayey sand (SC) and stiff to hard fat clay (CH). The bedrock was encountered at depths of 3 to 10 feet bgs and consisted of Topanga Formation Siltstone recovered as silty and clayey sands (SM, SC), clays (CL, CH) and silt (ML). Refusal to further advancement of the drilling augers was not experienced in the exploratory borings. Caving was not experienced within the exploratory borings utilized for this investigation.

The site does not lie within or immediately adjacent to an Alquist-Priolo Earthquake Fault Zone designated by the State of California to include traces of suspected active faulting.

Moderate to severe seismic shaking can be expected at the site.

Groundwater was not encountered within the maximum 50-1/2-foot depth of the explorations. Historic high groundwater is estimated to be at 10 feet bgs in the area of the site. Based on the composition of the underlying soils encountered in our geotechnical investigation and the relatively shallow depths of bedrock encountered at the site, liquefaction is not considered a potential hazard to the site.

Settlement resulting from seismic shaking is considered negligible. Hydroconsolidation potential is considered low for the site.

The potential for subsidence to affect the proposed structure is considered low.

The potential for landsliding or lateral spreading is considered low.

Expansion index testing yielded "very high" potential for expansion. Based on the EI test result, construction procedures and/or special structural design to specifically mitigate the effects of expansive soil movements are necessary.

Based on the classification, density and lack of significant soil cementation encountered in exploratory borings placed within the site, site grading and utility trenching are expected to be feasible with conventional heavy grading and trenching equipment, respectively.

RECOMMENDATIONS

The recommendations provided in this report assume that on-site expansive soils will be utilized and foundations and slabs-on-grade will be designed for expansive deformations and pressures provided herein. Retaining walls will require imported, very low expansive (EI<21), granular soils as backfill. If additional recommendations for use of imported soils or conventional foundations are required, this firm should be contacted.

GENERAL SITE GRADING:

It is imperative that no clearing and/or grading operations be performed without the presence of a representative of the geotechnical engineer. An on-site, pre-job meeting with the developer, the contractor and the geotechnical engineer should occur prior to all grading-related operations. Operations undertaken at the site without the geotechnical engineer present may result in exclusions of affected areas from the final compaction report for the project.

Grading of the subject site should be performed, at a minimum, in accordance with these recommendations and with applicable portions of the 2013 CBC. The following recommendations are presented for your assistance in establishing proper grading criteria.

INITIAL SITE PREPARATION:

All areas to be graded should be stripped or cleaned of significant vegetation, rocks greater than 6 inches in largest dimension and other deleterious materials. These materials should be removed from the site for disposal.

The cleaned soils may be reused as properly compacted fill if foundations, which include slabs-on-grade, are designed as indicated in the "Expansive Soils" section of this report.

If encountered, existing utility lines should be traced, removed and rerouted from areas to be graded.

Cavities created by removal of subsurface obstructions such as structures, individual effluent disposal systems and trees should be thoroughly cleaned of loose soil, organic matter and other deleterious materials, shaped to provide access for construction equipment, and backfilled as recommended for compacted fill.

MINIMUM MANDATORY REMOVAL AND RECOMPACTION OF EXISTING SOILS:

All areas to be graded should have at least the upper 5 feet of existing soils removed or expose siltstone bedrock, and the open excavation bottoms observed by our engineering geologist to verify and document in writing that all undocumented fill is removed prior to refilling with properly tested and documented compacted fill. The removed soils may only be used as compacted fill if foundations are designed as recommended in the "Expansive Soils" section of this report.

Further subexcavation may be necessary depending on the conditions of the underlying soils. The actual depth of removal should be determined at the time of grading by the project geotechnical engineer/geologist. The determination will be based on soil conditions exposed within the excavations.

Compaction tests may be taken in the removal bottom areas where appropriate to provide in-place moisture/density data for potential relative compaction evaluations and to help support and document the engineering geologist's decision. As such, all areas to be graded should have any undocumented fill, topsoil or other unsuitable materials removed and replaced with properly compacted fill. Fill may consist of suitable on-site material, imported material or a combination thereof depending on foundation design.

PREPARATION OF FILL AREAS:

Prior to placing fill, and after the mandatory subexcavation operation with all loose native and/or undocumented fill removed, the surfaces of all areas to receive fill should be scarified to a depth of 6 inches or more. The scarified soils should be brought to between optimum moisture content and 2 percent above optimum moisture content and recompacted to a minimum relative compaction of 90 percent in accordance with ASTM D1557.

PREPARATION OF FOUNDATION AREAS:

For foundations designed for expansive soils as recommended in the "Expansive Soils" section of this report, the thickness of compacted fill underneath footings should be at least 3 feet and the removed soils may be used as compacted fill. In areas where the required thickness of compacted fill is not accomplished by site rough grading, mandatory subexcavation operation and the undocumented fill removal, the footing areas should be further subexcavated to a depth of at least 3 feet below the proposed footing base grade. The required overexcavation should extend at least 10 feet laterally beyond the footing lines, where possible. The bottom of this excavation should then be scarified to a depth of at least 6 inches, brought to between optimum moisture content and 2 percent above optimum moisture content and recompacted to a minimum of 90 percent relative compaction in accordance with ASTM D1557 prior to refilling the excavation to the required grade as properly compacted fill.

Thickness of compacted fill underneath foundations should not be allowed to vary by more than 50 percent or 4 feet, whichever is less, for a single foundation system. In areas where, by virtue of grading, the fill thickness will exceed this maximum allowable differential, the subexcavation depths should be increased as necessary to reduce the differential fill thickness. This deepening of the subexcavation may involve additional removals of native soils. A determination of specific structural areas that require additional subexcavation should be performed at the time of grading.

Foundation concrete should be placed in neat excavations with vertical sides, or the concrete should be formed and the excavations properly backfilled as recommended for compacted fill.

COMPACTED FILLS:

The on-site soils should provide adequate quality fill material provided they are free from organic matter and other deleterious materials and foundations and slabs-on-grade are designed for expansive soils as indicated in the "Expansive Soils" section of this report. Unless approved by the geotechnical engineer, rock or similar irreducible material with a maximum dimension greater than 8 inches should not be buried or placed in fills.

If utilized, import materials should be inorganic, very low-expansive (EI<21), granular soil free from rocks or lumps greater than 6 inches in maximum dimension. The contractor shall notify the geotechnical engineer of import sources sufficiently ahead of their use so that the sources can be observed and approved as to the physical characteristic of the import material. For all import material, the contractor shall also submit current verified reports from a recognized analytical laboratory indicating that the import has a "not applicable" (Class S0) potential for sulfate attack based upon current American Concrete Institute (ACI) criteria and is not corrosive to ferrous metal and copper. The reports shall be accompanied by a written statement from the contractor that the laboratory test results are representative of all import material that will be brought to the job.

Fill should be spread in near-horizontal layers, approximately 8 inches thick. Thicker lifts may be approved by the geotechnical engineer if testing indicates that the grading procedures are adequate to achieve the required compaction. Each lift should be spread evenly, thoroughly mixed during spreading to attain uniformity of the material and moisture in each layer, brought to between optimum moisture content and 2 percent above optimum moisture content, and compacted to a minimum relative compaction of 90 percent in accordance with ASTM D1557.

It is crucial that the geotechnical engineer or representative be present to observe the grading operations. Monitoring of the soil expansion potential by the geotechnical engineer during the grading operation should be performed regularly. Further recommendations may be made in the field, depending on the actual conditions encountered.

SLOPE CONSTRUCTION:

Slopes should be constructed no steeper than 2(h):1(v). Fill slopes should be overfilled during construction and then cut back to expose fully compacted soil. A suitable alternative would be to compact the slopes during construction and then roll the final slopes to provide dense, erosion-resistant surfaces.

SLOPE PROTECTION:

Inasmuch as the native materials are susceptible to erosion by wind and running water, it is our recommendation that the slopes at the project be protected from erosion as soon as possible after completion. On permanent slopes the use of succulent ground covers, such as ice plant or sedum, is not recommended. If watering is necessary to sustain plant growth on slopes, then the watering operation should be monitored to assure proper operation of the water system and to prevent overwatering.

Measures should be provided to prevent surface water from flowing over slope faces.

FOUNDATION DESIGN:

Foundations and slabs-on-grades should be designed to resist the effects of expansive soils. Structural design measures including design of slab-on-grade foundations in accordance with "WRI/CRSI Design of Slab-On-Ground Foundations" or "PTI Standard Requirements for Design of Shallow Post-Tensioned Concrete Foundations of Expansive Soils" would be necessary. Foundations should also be designed to prevent uplift of the supported structure and resist forces exerted on the foundation due to soil volume change or shall be isolated from the expansive soil as indicated in Sections 1808.6.1 and 1808.6.2 of the 2013 California Building Code.

For foundations designed for expansive soils, bearing on a minimum of 3 feet of compacted fill, footings may be designed for a maximum safe soil bearing pressure of 1,000 pounds per square foot (psf) for dead plus live loads. The bearing values may be increased by one-third for wind or seismic loading.

For footings thus designed and constructed, we would anticipate a maximum static settlement of less than 1 inch. Differential static settlement between similarly loaded adjacent footings is expected to be approximately half the total settlement. Static settlement is expected to occur during construction or shortly after. Foundation concrete should be placed in neat excavations with vertical sides, or the concrete should be formed and the excavations properly backfilled as recommended for compacted fill.

LATERAL LOADING:

Resistance to lateral loads will be provided by passive earth pressure and cohesion. For footings bearing against on-site compacted fill, allowable passive earth pressure may be considered to be developed at a rate of 100 psf per foot of depth. Passive earth pressure only applies to level, properly drained backfill with no additional surcharge loadings. Cohesion may be computed as 130 psf. Cohesion and passive earth pressure may be combined without reduction.

Cohesion value is to be multiplied by the contact area, as limited by Section 1806.3.2 of the 2013 CBC. The lateral passive earth pressure and cohesion values are provided from Table 1806.2 of the 2013 CBC.

The resistance values provided do not consider expansive pressures of the on-site soils. Expansive pressures should be taken into account during design of foundations.

For preliminary retaining wall design, lateral active earth pressures indicated in the table below should be utilized for properly drained backfill with no additional surcharge loadings.

Lateral Active Earth Pressures	
Backfill Inclination	**Active (psf/ft)**
Level	40
3(h):1(v)	55
2(h):1(v)	65

For restrained conditions, an at-rest earth pressure of 65 psf per foot of depth should be utilized for level, properly drained backfill with no additional surcharge loadings.

The "at-rest" condition applies toward braced walls that are not free to tilt. The "active" condition applies toward unrestrained cantilevered walls where wall movement is anticipated. The structural designer should use judgment in determining the wall fixity and may utilize values interpolated between the "at-rest" and "active" conditions where appropriate.

The values for earth pressures are based on imported backfills consisting of inorganic, very low-expansive (EI<21), granular, compacted fill, and assume that soils will have a phi angle of 30 degrees and a unit weight of 120 pounds per cubic foot. These values should be verified by an engineer from this firm when import materials are selected. These values do not include a factor of safety other than conservative modeling of the soil strength parameters.

RETAINING WALL BACKFILL:

Backfill behind retaining walls should consist of a soil of sufficient granularity that the backfill will properly drain. The granular backfill shall extend from the bottom of the wall at a 1(h):1(v) plane to the surface. The granular soil should be classified per the USCS as GW, GP, SW, SP, SW-SM or SP-SM and should have a minimum phi angle of 30 degrees and a unit weight of 120 pounds per cubic foot. Surface drainage should be provided to prevent ponding of water behind walls. A drainage system should be installed behind all retaining walls consisting of either of the following:

1. A 4-inch-diameter perforated PVC (Schedule 40) pipe or equivalent at the base of the stem encased in 2 cubic feet of granular drain material per linear foot of pipe or

2. Synthetic drains such as Enkadrain, Miradrain, Hydraway 300 or equivalent.

Perforations in the PVC pipe should be 3/8 inch in diameter. Granular drain material should be wrapped with filter cloth such as Mirafi 140 or equivalent to prevent clogging of the drains with fines. Walls should be waterproofed to prevent nuisance seepage. Water should outlet to an approved drain.

SEISMIC LATERAL EARTH PRESSURE (CANTILEVERED WALL):

The seismic earth pressure acting on a cantilevered retaining wall was calculated using the Mononobe-Okabe ("M-O") method (Okabe, 1926; Mononobe and Matsuo, 1929). According to AASHTO (LRFD Bridge Design Specifications, Sixth Edition, 2012, Section C11.8.6.2 and A11.3.2), the resulting pseudostatic horizontal seismic coefficient, k_h, could be reduced by 50 percent when 1.0 to 2.0 inches of permanent ground deformation is permitted during the design seismic event, i.e., the pseudostatic horizontal seismic coefficient (k_h) can be taken as equal to one-half of the PGA, which equates to 0.29g. The pseudostatic vertical seismic coefficient (k_v) is usually taken as 0.0g. For retaining walls with imported backfills consisting of inorganic, very low-expansive (EI<21), granular, compacted fill, a unit weight of 120 pounds per cubic foot (pcf) and a friction angle of 30 degrees were utilized in the calculation. These values should be verified prior to construction when the backfill materials and conditions have been determined and are applicable only to properly drained backfill with no additional surcharge loadings.

The total lateral active seismic earth pressures (including static active earth pressures) to be utilized for unrestrained conditions are provided in the following table.

Lateral Active Seismic Earth Pressures	
Backfill Inclination	**Active Seismic (psf/ft)**
Level	70
3(h):1(v)	125
2(h):1(v)	135

A triangular distribution of total seismic earth pressure should be used in the design (Atik and Sitar, 2010).

SLABS-ON-GRADE:

Slabs-on-grade should be designed to resist the expansive soils as provided in the "Expansive Soils" section of this report.

Slabs to receive moisture-sensitive coverings should be provided with a moisture vapor retarder. We recommend that a vapor retarder be designed and constructed according to the American Concrete Institute 302.1R, Concrete Floor and Slab Construction, which addresses moisture vapor retarder construction. At a minimum, the vapor retarder/barrier should comply with ASTM E1745 and have a nominal thickness of at least 10 mils. The vapor retarder/barrier should be properly sealed, per the manufacturer's recommendations, and protected from punctures and other damage. Per the Portland Cement Association (www.cement.org/tech/cct_con_vapor_retarders.asp), for slabs with vapor-sensitive coverings, a layer of dry, granular material (sand) should be placed under the vapor retarder/barrier. For slabs in humidity-controlled areas, a layer of dry, granular material (sand) should be placed above the vapor retarder/barrier.

A modulus of vertical subgrade reaction of 100 kips per cubic foot can be utilized in the design of slabs-on-grade for the proposed project.

EXPANSIVE SOILS:

The expansion index testing performed for this report indicated a "very high" potential for expansion (EI of 150 and 157) in the upper soil layers. Based on these results, construction procedures and/or special structural design to specifically mitigate the effects of expansive soil movements are necessary, as recommended below.

Structural design measures, including design of slab-on-grade foundations in accordance with "WRI/CRSI Design of Slab-On-Ground Foundations" or "PTI Standard Requirements for Design of Shallow Post-Tensioned Concrete Foundations of Expansive Soils", should be taken into consideration for this project. Foundations should also be designed to prevent uplift of the supported structure and resist forces exerted on the foundation due to soil volume change or shall be isolated from the expansive soil as indicated in Sections 1808.6.1 and 1808.6.2 of the 2013 California Building Code.

The expansive potential deformation within the upper 5 feet of clayey soils is expected to be approximately 1-1/2 inches (expansive strain of 2.4%). An expansive pressure of 7,000 psf should be used in the design of the foundations and slab-on-grade.

Additional evaluation of soils for expansion potential should be conducted by the geotechnical engineer during grading in order to provide the geotechnical parameters required for the design. Utilities should also be designed for potential expansive deformation and pressure.

POTENTIAL EROSION AND DRAINAGE:

The potential for erosion should be mitigated by proper drainage design. The site should be graded in such a way that surface water flows away from structures. Water should not be allowed to flow over graded areas or natural areas so as to cause erosion. Graded areas should be planted or otherwise protected from erosion by wind or water.

STORM WATER INFILTRATION:

Based on the measured infiltration rates, we recommend that a design infiltration rate of 0.03 inches per hour be used for the design of the storm water disposal system(s) on site. An appropriate safety factor should be applied to the recommended infiltration rate prior to use in design to accommodate potential subsoil inconsistencies, possible compaction related to site grading and potential silting of the percolating soils. A safety factor should be determined with consideration to other factors in the storm water retention system design, particularly storm water volume estimates and the safety factors associated with those design components.

As the design infiltration rate is very low, alternative measures to storm water abatement should be considered.

TRENCH EXCAVATION:

The soils encountered within our exploratory borings are generally classified as a Type "B" soil in accordance with the CAL/OSHA excavation standards. Unless specifically evaluated by our engineering geologist, all the trench excavations should be performed following the recommendation of CAL/OSHA (State of California, 2013) for Type "B" soil. Based upon a soil classification of Type "B", the temporary excavation should not be inclined steeper than 1(h):1(v) for maximum trench depth of less than 20 feet. For trench excavation deeper than 20 feet or for conditions that differ from those described for Type "B" in the CAL/OSHA excavation standards, this firm should be contacted.

TRENCH BEDDING AND BACKFILLS:

Trench Bedding - Pipe bedding material should meet and be placed according to the current edition of the Standard Specifications for Public Works Construction "Greenbook" or other project specifications. Pipe bedding should be uniform, free-draining, granular material with a sand equivalent of at least 30. The pipe bedding material should be evaluated to confirm sand equivalent values by this firm prior to use as pipe bedding material.

Backfill - The on-site expansive soils may be utilized for trench backfill if utilities are designed to accommodate the expansive deformations and pressures provided in the "Expansive Soils" section of this report. Rock or similar irreducible material with a maximum dimension greater than 6 inches should not be buried or placed in backfills.

Fill to be compacted by heavy equipment should be spread in near-horizontal layers, approximately 8 inches in thickness. For fill to be compacted by hand-operated equipment, thinner lifts, 4 to 6 inches in thickness, should be utilized. Each lift should be spread evenly, brought to between optimum moisture content and 2 percent above optimum moisture content and compacted to a minimum relative compaction of 90 percent in accordance with ASTM D1557. To avoid pumping, backfill material should be mixed and moisture treated outside of the excavation prior to lift placement in the trench.

Soils required to be compacted to at least 95 percent relative compaction, such as pavement subgrade, should also be moisture treated to near optimum moisture content not exceeding 2 percent above optimum moisture content.

As an alternative, a controlled low-strength material (CLSM) could be considered to fill trenches, cavities, such as voids created by caving or undermining of soils beneath existing improvements or pavement to remain, or any other areas that would be difficult to properly backfill.

CHEMICAL/CORROSIVITY TESTING:

Selected samples of materials were delivered to HDR, Inc. for soil corrosivity testing. Laboratory testing consisted of pH, resistivity and major soluble salts commonly found in soils. The results of the laboratory tests performed by HDR, Inc. appear in Appendix "C".

These tests have been performed to screen the site for potentially corrosive soils. Values from the soil tested are considered "mildly corrosive" to ferrous metals at as-received moisture condition and "corrosive" at saturated condition. Specific corrosion control measures, such as coating of the pipe with non-corrosive material or alternative non-metallic pipe material, are considered necessary.

Ammonium and nitrate levels did not indicate a concern as to corrosion of buried copper.

Results of the soluble sulfate testing indicate a "not applicable" (Class S0) anticipated exposure to sulfate attack. Based on the criteria from Table 4.3.1. of the "American Concrete Institute Manual of Concrete Practice" (2011), no special measures, such as specific cement types or water-cement ratios, will be required.

The soluble chloride content of the soils tested was not at levels high enough to be of concern with respect to corrosion of reinforcing steel. The results should be considered in combination with the soluble chloride content of the hardened concrete in determining the effect of chloride on the corrosion of reinforcing steel.

CHJ Consultants does not practice corrosion engineering. If further information concerning the corrosion characteristics, or interpretation of the results submitted herein, is required, then a competent corrosion engineer could be consulted.

CONSTRUCTION OBSERVATION:

All grading operations, including site clearing and stripping, should be observed by a representative of the geotechnical engineer. The geotechnical engineer's field representative will be present to provide observation and field testing and will not supervise or direct any of the actual work of the contractor, his employees or agents. Neither the presence of the geotechnical engineer's field representative nor the observations and testing by the geotechnical engineer shall excuse the contractor in any way for defects discovered in his work. It is understood that the geotechnical engineer will not be responsible for job or site safety on this project, which will be the sole responsibility of the contractor.

LIMITATIONS

CHJ Consultants has striven to perform our services within the limits prescribed by our client and in a manner consistent with the usual thoroughness and competence of reputable geotechnical engineers and engineering geologists practicing under similar circumstances. No other representation, express or implied, and no warranty or guarantee is included or intended by virtue of the services performed or reports, opinion, documents, or otherwise supplied.

This report reflects the testing conducted on the site as the site existed during the investigation, which is the subject of this report. However, changes in the conditions of a property can occur with the passage of time, due to natural processes or the works of man on this or adjacent properties. Changes in applicable or appropriate standards may also occur whether as a result of legislation, application or the broadening of knowledge. Therefore, this report is indicative of only those conditions tested at the time of the subject investigation, and the findings of this report may be invalidated fully or partially by changes outside of the control of CHJ Consultants. This report is therefore subject to review and should not be relied upon after a period of one year.

The conclusions and recommendations in this report are based upon observations performed and data collected at separate locations, and interpolation between these locations, carried out for the project and the scope of services described. It is assumed and expected that the conditions between locations observed and/or sampled are similar to those encountered at the individual locations where observation and sampling was performed. However, conditions between these locations may vary significantly. Should conditions that appear different from those described herein be encountered in the field by the client or any firm performing services for the client or the client's assign, this firm should be contacted immediately in order that we might evaluate their effect.

If this report or portions thereof are provided to contractors or included in specifications, it should be understood by all parties that they are provided for information only and should be used as such.

The report and its contents resulting from this investigation are not intended or represented to be suitable for reuse on extensions or modifications of the project or for use on any other project.

CLOSURE

We appreciate this opportunity to be of service and trust this report provides the information desired at this time. Should questions arise, please do not hesitate to contact this firm at your convenience.

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