Preliminary Hydrology and Low-Impact Design Report

24460 Calabasas Road

Calabasas, CA 91302

Prepared for:

Hello Auto Group

24095 Creekside Road

Santa Clarita, CA 91355

Prepared by:

DIAMOND WEST
INCORPORATED

23801 Calabasas Road, Suite 1034
Calabasas, California 91302
Phone: (818) 591-1050

April 5, 2023

PRELIMINARY

REGISTERED PROFESSIONAL ENGINEER
WILLIAM C. CUNNINGHAM III
No. 80129
Exp. 09/30/24
CIVIL
STATE OF CALIFORNIA

Contact Person:

William C. Cunningham III, P.E.
RCE #80129, Expires: September 30, 2024

Table of Contents

Appendices

PROJECT SUMMARY

The purpose of this Report is to facilitate the planning and implementation of drainage infrastructure improvements and provide a hydrology analysis of predevelopment and post-development conditions for the proposed redevelopment of the project site for a proposed KIA Calabasas car dealership. The results of this Report will be the basis for stormwater improvements solely the proposed KIA Calabasas car dealership.

Objective

The objective of the hydrology study is to show that the development of the proposed KIA car dealership will not create additional runoff or significantly impact the historic runoff patterns.

Project Location

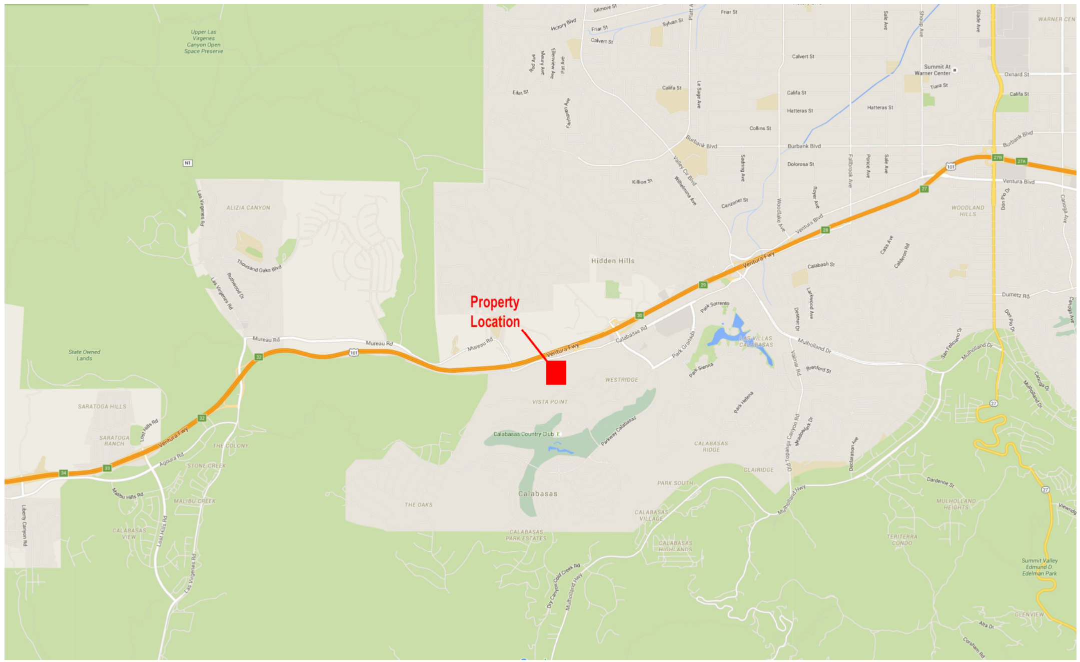
The project (KIA Calabasas) is located on the east side of the City of Calabasas. It is bounded to the north by Calabasas Road, to the south by an existing natural hillside, to the west by an existing automobile dealership, and to the east by existing commercial developments and natural hillside (see Figure 2 and Exhibit 1: Vicinity Map). The project is situated on Assessor's Parcel Numbers 2069-009-008 and 2069-009-020, encompassing approximately 10.94 acres. The existing land use for the project site is a commercial landscape/tree nursery with several structures, paved driveways, courtyards, and parking areas. Approximately one-third of the existing project site is already developed. The remaining two-thirds of the project site maintains its natural hillside. The immediate, adjacent areas of the project site are commercial uses that include automobile dealerships and inventory parking areas.

This conceptual drainage report discusses the impacts from the 85^{th} percentile, 10-year, and 50-year 24-hour design storm events. Its intended use is for the development of drainage infrastructure solely for the project.

Authorization

This Report has been performed at the request of Hello Auto Group to determine the existing drainage patterns and the drainage impacts from the proposed development on the study area. This Report does not intend to suggest remediation for any regional drainage issues outside of the project area.

Figure 1. Regional Location Map



METHODOLOGY

The methodology described in the Los Angeles County Department of Public Works (LACDPW) Hydrology Manual was used to compute the stormwater runoff at the proposed project site. LACDPW HydroCalc program was used to calculate each subarea's time of concentration (Tc). This software uses the modified rational method as defined in the County Hydrology Manual. The LA County-approved Watershed Modeling System (WMS v 11.0) computer software was used to combine and route peak runoff.

EXISTING WATERSHED CHARACTERISTICS

The existing project site was previously Sperling Nursery and Gift Shop, a commercial retail area consisting of several greenhouse structures, stocking areas for landscape and bedding material, and a parking area. The nursery facility is no longer in business, and the property has been vacated. The existing structures have been demolished; however, the building foundations, internal paved roads, and parking areas are still present. The surrounding area is a mix between natural hillside areas and commercial developments with several major automobile dealership facilities along Calabasas Road.

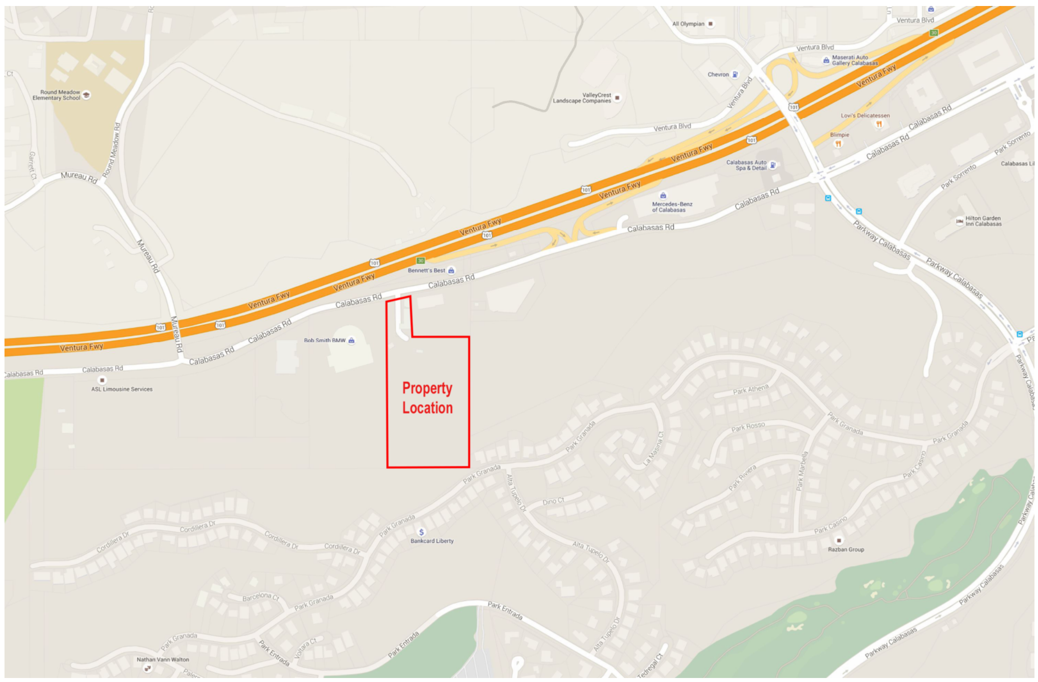
The project area currently surface drains in a northerly direction onto Calabasas Road and is then conveyed easterly along Calabasas Road. An existing, grated storm drain inlet is approximately 320 feet east of the property, along Calabasas Road. Based on visual inspection of the existing asphalt pavement and available topographic information, it appears that much of the existing stormwater runoff generated from the project site bypasses the grated inlet and continues further in the easterly direction along Calabasas Road, where it discharges into Line "D" of Private Drain No. 2120. Private Drain No. 2120 is an existing storm drain system owned and maintained by the County of Los Angeles.

The study area for the existing condition consists of approximately 10.32 acres with a single-basin watershed. The delineated watershed is defined by the existing topographic terrain and developed boundaries. The land uses within the study area is primarily commercial. The natural slopes within the area are relatively moderate to steep, with the grades ranging between 5 – 45% (See Exhibit 2: Existing Conditions Hydrology Map.)

Stormwater runoff generated from the study area generally drains northerly as overland flow through the existing, developed site. Once the runoff enters Calabasas Road, it continues to travel in an easterly direction as concentrated flow before an existing catch basin collects it.

Using the project survey and aerial imagery, the calculated percent impervious area for the existing watershed resulted in 15.8%.

Figure 2. Vicinity Map



Flood Insurance Study

The detailed study area is located on the following FEMA FIRMs (See Appendix A: FEMA Flood Insurance Rate Map).

Los Angeles County, California and Incorporated Areas, panel 1268 of 2350 map number 06037C1268F effective date September 26, 2008. According to these maps, the detailed study area is located in the "Area of Minimal Flood Hazard" Zone X.

ASSUMPTIONS

The soil type, rainfall, and runoff parameters are based on the County Hydrology Manual and the County Design Standards. The project site has a 50-year 24-hour rainfall depth of approximately 7.35 inches, soil type 4, and is in debris potential area (DPA) zone 4 (See Appendix B: Rainfall and Soils Maps). The LACDPW Sedimentation manual 2006 was used to calculate the bulk flow rate of 1.67 per the Appendix B-4 graph for "peak bulking factors," and a fire factor of 0.83 was determined. Debris production for the existing and proposed conditions will be evaluated. All proposed debris conveyance devices will be designed with self-cleaning velocities.

EXISTING CONDITION ANALYSIS

The existing condition peak runoff was analyzed for the 2-, 5-, 10-, 25- and 50-year 24-hour storm events. Confluence node 2A is used as the basis of comparison for the proposed development. The analysis includes the consideration of clear, burned, and burned and bulked runoff. Table-1 below summarizes the ***un-mitigated*** predevelopment vs. post-development runoff comparison (See Appendix C: Existing Peak Runoff Calculations and Appendix H: Debris Potential and Bulk flow Rate Calculations).

PROPOSED WATERSHED CHARACTERISTICS

The proposed project is for a 3-level KIA automobile dealership facility at 24460 Calabasas Road in the City of Calabasas. The project site is situated on Assessor's Parcel Numbers 2069-009-008 and 2069-009-020, encompassing approximately 10.94 acres. The proposed developed area is approximately 3.6 acres. The remaining property area will stay as undeveloped, natural hillside.

The proposed project development will maintain much of the existing natural hillsides. Due to the proposed site layout, building elevations, drainage devices, and catchment areas, the upstream natural hillside area was modeled separately using a sub-basin. The sub-basin approach also allowed us to isolate the natural hillside areas with less than 15% impervious areas and model a burned watershed for the 50-year/24-hour rainfall event. Subarea area 1A encompasses approximately 6.75 ac with an impervious area of approximately 1.1 %. Subarea 3A (which contains the entire proposed redevelopment area encompasses approximately 3.57 ac with a calculated impervious area of approximately 49.6 %. As expected, the peak runoff value for each storm event is greater than the existing peak runoff flow rates (See Exhibit 4: Hydrology Study Proposed Conditions and Exhibit 3: Conceptual Grading Plans).

The flows from undeveloped areas upstream from the project will be captured by the proposed debris/detention basin that will remove and store all potential debris. A trapezoidal earthen channel is also proposed along the existing roadway in subarea 1A to capture and divert/convey potentially debris-laden flows to the proposed debris/detention basin. The basins will also be utilized to provide detention to attenuate the peak flow rates to below the predevelopment flow rates. The mitigated flow rates from Basin #1 will outlet to a 36-inch RCP that will convey discharge through subarea 3A and discharge to Calabasas Road via a proposed parkway drain and allowed to follow historical drainage patterns, ultimately being collected by the beforementioned existing grated storm drain inlet and private storm drain line.

PROPOSED CONDITIONS ANALYSIS

The proposed conditions peak runoff was analyzed for the 2-, 5-, 10-, 25- and 50-year 24-hour storm events. The results of the **un-mitigated** routed runoff showed that the 5-, 10-, 25-, and 50-yr 24-hour storm events produced runoff greater than the predevelopment conditions (See Appendix D: Proposed Peak Runoff Calculations). Hydromodification control will be required to reduce the flow rate to predevelopment conditions. See table 1 below for a summary of the **unmitigated** pre vs. post-comparison.

Table 1.
Un-Mitigated Pre- vs. Post-Development Runoff

Point of Concentration	Node 2A (PRE)	Node 4A (POST)
24-hr, Design Storm Event	Existing Condition	Proposed Condition
50-year (clear)	30.30 cfs	34.43 cfs
50year (burned)	33.42 cfs	37.11 cfs
25-year (clear)	24.24 cfs	28.91 cfs
25-year (burned)	27.06 cfs	31.41 cfs
10-year (clear)	17.86 cfs	20.95 cfs
10-year (burned)	20.28 cfs	23.02 cfs
5-year (clear)	12.36 cfs	14.50 cfs
5-year (burned)	14.40 cfs	16.21 cfs
2-year (clear)	5.34 cfs	6.57 cfs
2-year (burned)	6.76 cfs	7.77 cfs

PROPOSED DEBRIS PRODUCTION AND BULKED FLOW

The LACDPW Sedimentation manual 2006 was used to calculate the debris production potential and bulked flow rate. The 50-yr burned frequency storm is used for this analysis. Based on the debris production rates graph found in Appendix B-1 of the sedimentation manual, a debris production rate of approximately 112.5 cy/ac was calculated. Each subarea was analyzed to determine areas considered debris-producing and non-debris-producing (i.e., developed, landscaped, fire mod/maintained, etc.). Developed subareas with impervious areas greater than 15% are assumed to be non-debris producing per the LA County Sedimentation manual; however, the additional debris production for subarea 3A was considered in this analysis to confirm potential debris-laden flows that might warrant the inclusion of bulk flow inlets where needed. The analysis resulted in a proposed debris production for each subarea (See Appendix H: Debris Potential and Bulk Flow Rate Calculations). Subareas 1A has enough potential debris production to necessitate a sediment control structure.

Table 2.
Debris Production Summary

Subarea	DPA Zone	Debris Production (cy/mi2)	DPR (cy/ac)	A (ac)	DP (cy)
1A	4	72000 *	112.50	6.42	722
3A**	4	72000 *	112.50	144.25	144

* SEDIMENTATION MANUAL - APPENDIX B: DEBRIS PRODUCTION RATE CURVES
** DEVELOPED AREA- NOT SUBJECT TO DEBRIS PRODUCTION

A proposed desilting inlet debris basin/detention structure has been designed to capture the calculated potential debris and provide stormwater detention. Basin #1 is located at the outlet of Subarea 1A. The proposed basin is designed with adequate level fill capacity to store 100% of the calculated potential debris and provide detention capacity (See Appendix E: Proposed Basins).

As previously mentioned, each basin serves as a desilting inlet that store sediment that may be transported during a rainfall event. The sediment accumulation will occur at the bottom of each basin, while the stormwater detention will occur at the top. The basin is sized to store all the potential debris and provide detention for each storm event while also maintaining a 1' min freeboard during a capital storm event. The freeboard is measured from the calculated Q50b detained water surface elevation to the emergency spillway elevation. The stage-storage relationship in Table-4 shows the storage of sedimentation and stormwater runoff detention volumes (See Appendix E: Proposed Basins).

The resulting "Burned and Bulked" flow rates for each subarea are summarized below in Table 3: Proposed Bulk Flow Rates. The clear water flow rates for the developed subarea 3A are shown since these areas are not subject to burned or burned and bulked flows. Additionally, for subareas 1A, the burned flow rate is shown since the proposed debris basins remove the bulked flow. The total unrouted "burned and bulked" flow rates are 38.26cfs, less than the predeveloped "burned and bulked" flow rate of 51.78 cfs.

Table 3.

Proposed Bulked Flow Rates

Subarea Description	DPA Zone	Q50 (cfs)	Q50b (cfs)	Bulk Factor, BF	Ai (ac)	Au (ac)	Ad (ac)	QBB (cfs)
1A ***	4	23.5	26.1	1.67 *	6.75	6.42	0.33346	26.1
3A**	4	12.1	12.1	1.67 *	3.57	1.28	2.28	12.1

* SEDIMENTATION MANUAL - APPENDIX B: PEAK BULKING FACTOR CURVES
** DEVELOPED AREA- NOT SUBJECT TO BULK AND BURN
*** BULKED FLOWS REMOVED BY DEBRIS BASIN

HYDROMOD

The developed flows for each storm event were analyzed and routed into the proposed debris/detention basins to reduce the proposed flow rates to pre-developed conditions. The Modified Plus method was used to route flows through the detention basin storage areas. See tables-4 below for the basin rating tables. (See Appendix F: Detention Analysis).

Table 4.

Basin#1 Rating Table

STAGE (msl)	***VOLUME*** (ac-ft)	***DETENTION*** (ac-ft)	***DISCHARGE*** (cfs)
1130(sediment)	0.00	-	-
1132	0.106	-	-
1134	0.255	-	-
1136	0.443	-	-
1137 (detention)	0.552	0.109	19.88
1138 (freeboard)	0.671	0.228	54.71

The overall reduction of the proposed flows was compared at the project out location node 4A. The results show that the detention and routing of flows successfully reduced flow rates to below pre-developed conditions for every storm event. Table 7 below summarizes the mitigated flows.

Table 7.

Mitigated Pre- vs. Post-Development Runoff

Point of Concentration	Node 2A (PRE)	Node 4A (POST)
24-hr, Design Storm Event	Existing Condition	Proposed Condition
50-year (clear)	30.30 cfs	26.93 cfs
50year (burned)	33.42 cfs	29.26 cfs
25-year (clear)	24.24 cfs	22.90 cfs
25-year (burned)	27.06 cfs	24.92 cfs
10-year (clear)	17.86 cfs	16.94 cfs
10-year (burned)	20.28 cfs	18.82 cfs
5-year (clear)	12.36 cfs	12.07 cfs
5-year (burned)	14.40 cfs	13.61 cfs
2-year (clear)	5.34 cfs	5.70 cfs
2-year (burned)	6.76 cfs	6.80 cfs

STORMWATER QUALITY DESIGN

The proposed development and paved areas will exceed 1 acre of disturbed area and add more than 10,000 square feet of impervious surface area. This redevelopment is considered a "designated project," Stormwater management will be required as defined under the NPDES 2012 MS4 permit and the 2014 County of Los Angeles Department of Public Works Low Impact Development (LID) Standards Manual. This drainage report's soil type, rainfall, and runoff parameters are based on and conform to the Los Angeles County Department of Public Works Hydrology (LACDPW) Manual 2006.

Stormwater treatment shall be based on the LACDPW Low Impact Development Standards Manual to comply with County LID requirements. The Stormwater Quality Volume is based on the 85th percentile, 24-hour rain event, since it produces a larger stormwater volume than the 0.75-inch event. The LACDPW HydroCalc computer program was again utilized to evaluate these design storms(See Appendix D: Rainfall and Soils Maps and Appendix K: Stormwater Quality Calculations). Per the Geotechnical Report, the tested infiltration rate is below the minimum 0.33 inch/hr, indicating that infiltration is not feasible (See Appendix J: Geotechnical Reports). Therefore, the project will utilize underground storage chambers to capture and reuse the calculated stormwater quality volume as the stormwater quality BMP (See Appendix I: Stormwater Quality Design BMP).

For the governing 85th percentile storm, the total water quality treatment volume of **6,385 cf** for subarea 3A.

For the capture and reuse to be feasible in meeting LID requirements, an evaluation of the Estimated Total Water Usage (ETWU) for irrigation must be determined. The ETWU for irrigation from October 1 to April 30 must be greater than or equal to the volume of water produced by the stormwater quality storm event. The ETWU will be evaluated as the project design progresses into final engineering, and the LID design will be updated accordingly. Actual design and feasibility of stormwater quality mitigation and Low Impact Design is pending geotechnical analysis for recommendations. Alternate stormwater treatment BMP may be considered. BMPs can be provided in various ways, varying from catch basin filters, proprietary treatment devices placed in the main storm drain infrastructure, and grass swale filters. The flow-based treatment device will be required to treat the peak mitigated runoff of 0.4298 cfs, which is the total stormwater runoff from the 85th percentile/24-hour rainfall event.

Additionally, this project will require preparing and implementing a Storm Water Pollution Prevention Plan (SWPPP) for the construction site by the requirements for construction sites greater than 1 acre per part VI.D.8.e-j of the NPDES permit order No. R4-2012-0175 as amended by order WQ2015-0075. A SWPPP will be provided per the state water board requirements, and construction BMPs will be implemented to reduce pollutants in stormwater. The types of construction BMPs can be found in Tables 13 and 14 in appendix K of the MS4 permit. The development of the SWPPP and selection of the construction BMPs will be determined as the project's design progresses into final engineering.

CONCLUSION

The preliminary analysis concludes that the proposed development will not adversely impact the historic runoff conditions. The mitigated post – redevelopment flows are decreased over the predevelopment. The equivalent of the entire stormwater quality volumes has been treated adequately via Capture and Reuse or flow-thru treatment BMPs.

References

1) County of Los Angeles (2006). *Hydrology Manual for Department of Public Works*, Los Angeles, CA.

2) County of Los Angeles (2006). *Sedimentation Manual for Department of Public Works*, Los Angeles, CA.

Exhibit 1:

Vicinity Map

VICINITY MAP

Project Address:
24460 Calabasas Road
Calabasas, CA 91302

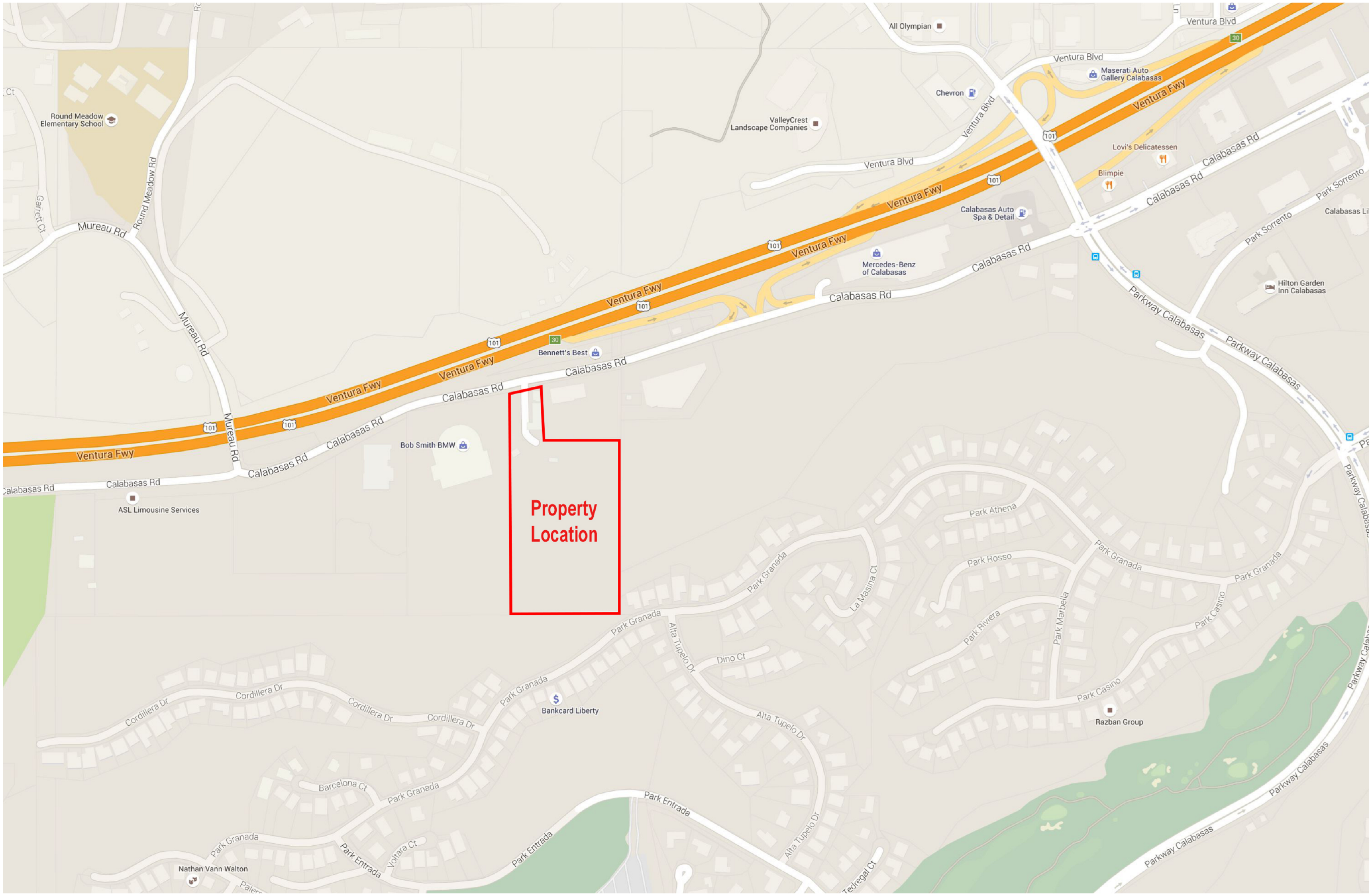
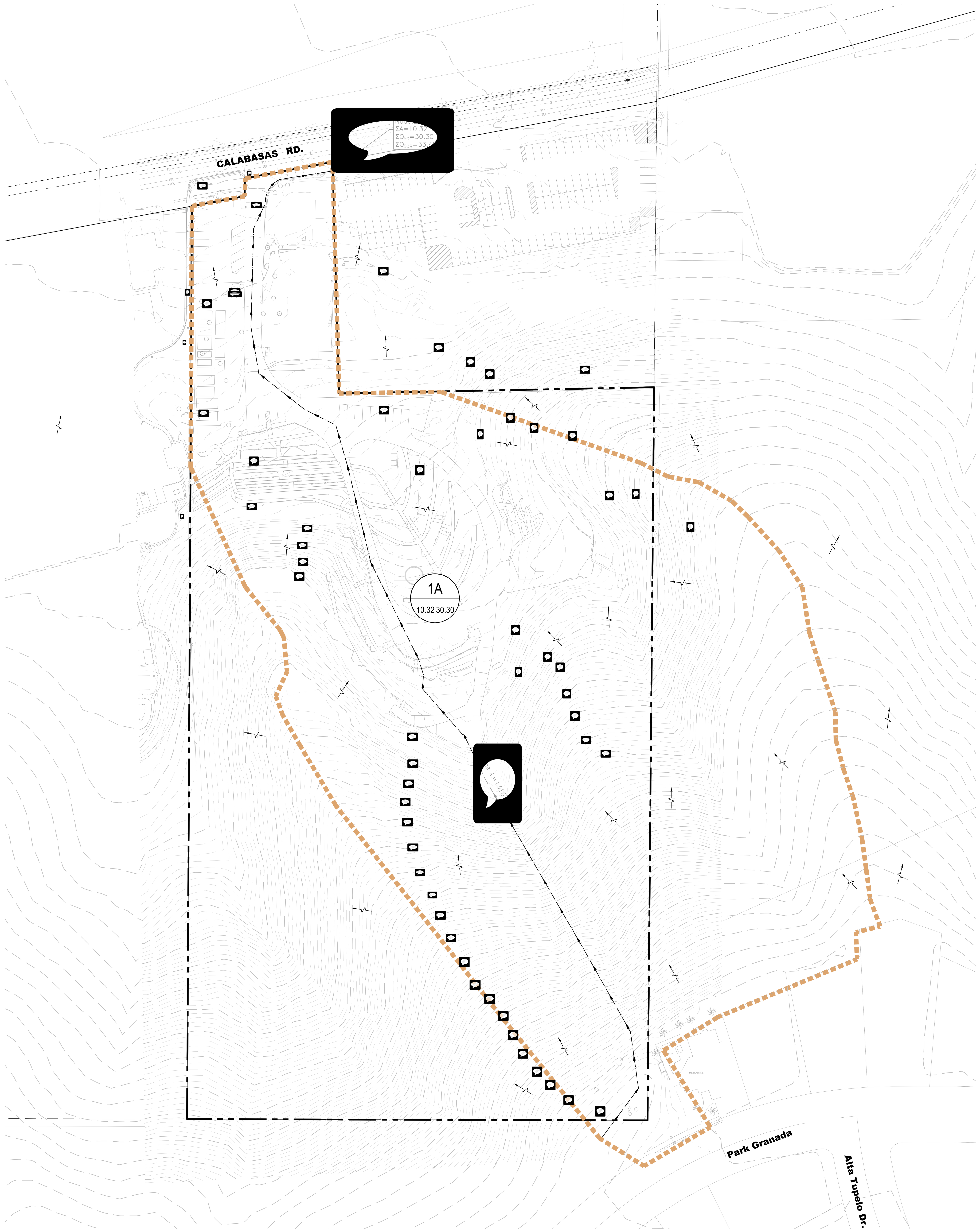


Exhibit 2:

Existing Conditions Hydrology Map



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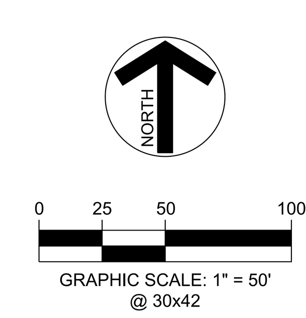
Hydrology Existing Conditions	
Sub Area	**1A**
Area (Sf)	449,519
Area (Acres)	10.32
Flowpath (ft)	1,313
Slope	0.22
% Impervious*	0.158
50 yr Rain Fall (in)	7.35
10 yr Rainfall (in)	5.25
Soil Type	4
Debris Zone	4
Cu - 50yr	0.7668
Cd - 50 yr	0.7879
Tc (min) - 50yr	7
Q (cfs) - 50yr	30.30
Qb (cfs) - 50yr	33.42
V (cf) - 50yr	87,340

Bulk Flow Rate		
Subarea	1A	Total
DPA Zone	4	-
Q50 (cfs)	30.30	30.3
Q50b(cfs)	33.42	33.4
BFi(Ai)	1.67 *	-
Ai (ac)	10.32	-
Au (ac)	8.69	-
Ad (ac)	1.63	-
QBB(cfs)	51.78	**51.8**

*Sedimentation Manual - Appendix B-4: Peak Bulking Factor Curves

Debris Potential		
Subarea	1A	Total
DPA Zone	4	-
Debris Production (cy/mi2)	72000 *	-
DPR(A) (cy/ac)	112.50	-
A (ac)	8.7	8.69
DP (cy)	**977**	**977**

*Sedimentation Manual - Appendix B-1: Debris Production Rate Curves



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DIAMOND WEST
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23801 Calabasas Road, Suite 1034
Calabasas, CA 91302
Phone: (818) 591-1050
www.diamondwest.net

[redacted]

CALABASAS KIA
24460 CALABASAS ROAD,
CALABASAS CA, 91302

[redacted]

HYDROLOGY EXISTING CONDITIONS MAP

[redacted]

1

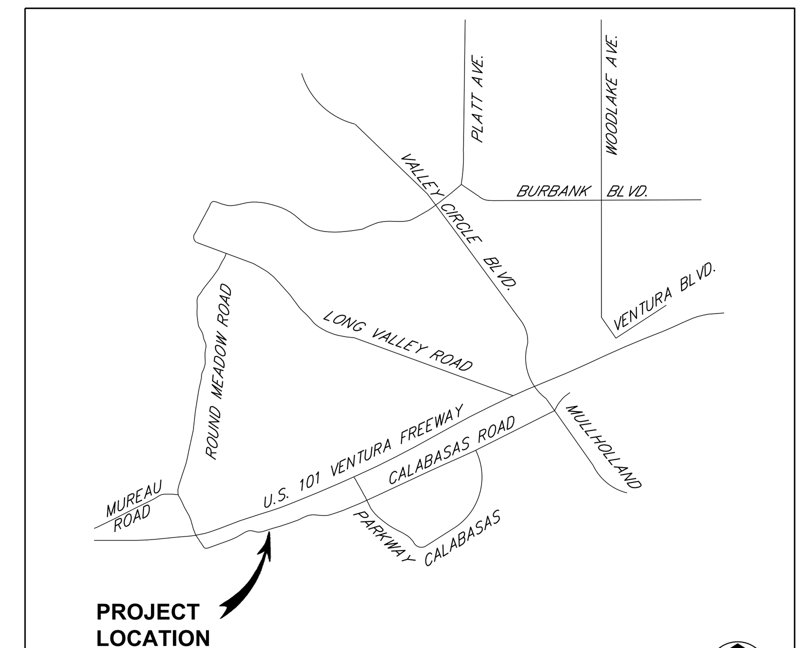
[redacted]

<u>Exhibit 3:</u>

Conceptual Grading Plan – For Reference

CITY *of* CALABASAS

CONCEPTUAL GRADING AND DRAINAGE PLAN
24460 CALABASAS ROAD



VICINITY MAP
N.T.S.

PROJECT APPLICANT/OWNER:

HELLO AUTO GROUP
24095 CREEKSIDE ROAD
SANTA CLARITA, CA 91355

PROPERTY INFORMATION:

24460 CALABASAS ROAD
CALABASAS CA, 91302
APN: 2069-009-009 & 2069-009-020

ARCHITECT:

AHT ARCHITECTS, INC.
2120 WILSHIRE BLVD, SUITE 200
SANTA MONICA CA, 90403
(310) 453 - 4431

CIVIL ENGINEER:

DIAMOND WEST, INC.
23801 CALABASAS ROAD, SUITE 1034
CALABASAS, CA 91302
(818) 591-1050

LANDSCAPE/ARCHITECT:

L NEWMAN DESIGN GROUP, INC.
31300 VIA COLINAS, SUITE 104
WESTLAKE VILLAGE, CA 91362
(818) 991-5056

ARBORIST:

TREE CARE CONSULTING
1534 N. MOORPARK ROAD
THOUSAND OAKS, CA 91360
(818) 512-3135

LEGAL DESCRIPTION:

ALL THAT CERTAIN REAL PROPERTY SITUATED IN THE COUNTY OF LOS ANGELES, STATE OF CALIFORNIA, DESCRIBED AS FOLLOWS:

PARCEL A: ASSESSORS PARCEL NO: 2069-009-020

PARCEL 1 OF PARCEL MAP NO. 5932, IN THE CITY OF CALABASAS, COUNTY OF LOS ANGELES, STATE OF CALIFORNIA, AS PER MAP RECORDED IN BOOK 67, PAGES 40 AND 41 OF PARCEL MAPS, IN THE OFFICE OF THE COUNTY RECORDER OF SAID COUNTY.

PARCEL B: ASSESSORS PARCEL NO: 2069-009-008

THAT PORTION OF THE SOUTHEAST QUARTER OF SECTION 1, TOWNSHIP 1 NORTH, RANGE 17 WEST, SAN BERNARDINO BASE AND MERIDIAN, IN THE COUNTY OF LOS ANGELES, STATE OF CALIFORNIA, ACCORDING TO THE OFFICIAL PLAT OF SAID LAND FILED IN THE DISTRICT LAND OFFICE, SEPTEMBER 25, 1896, BOUNDED EASTERLY AND SOUTHERLY BY THE EASTERLY AND SOUTHERLY LINES OF SAID SOUTHEAST QUARTER, BOUNDED NORTHERLY BY THE SOUTHERLY LINE OF CALABASAS ROAD, AS IT NOW EXISTED OF RECORD ON MARCH 9, 1965, AND BOUNDED WESTERLY BY THE FOLLOWING DESCRIBED LINE:

BEGINNING AT THE INTERSECTION OF THE SOUTHERLY LINE OF SAID SOUTHEAST QUARTER WITH A LINE PARALLEL WITH AND 60 FEET WESTERLY MEASURED AT RIGHT ANGLES, FROM THE SOUTHERLY PROLONGATION OF THE WESTERLY LINE OF THE LAND DESCRIBED IN THE DEED TO THE CALABASAS SCHOOL DISTRICT OF LOS ANGELES RECORDED FEBRUARY 13, 1930 AS INSTRUMENT NO. 1724, IN BOOK 9719, PAGE 244, OF OFFICIAL RECORDS, IN THE OFFICE OF THE COUNTY RECORDER OF SAID COUNTY, SAID POINT OF INTERSECTION BEING 528.29 FEET
WESTERLY MORE OR LESS FROM THE SOUTHEAST CORNER OF SAID SOUTHEAST QUARTER; THENCE NORTHERLY ALONG SAID PARALLEL LINE, TO SAID SOUTHERLY LINE OF THE LAND DESCRIBED IN SAID DEED RECORDED IN BOOK 14302, PAGE 296 OF SAID OFFICIAL RECORDS.

EXCEPT ANY PORTION THEREOF OWNED BY THE CALABASAS SCHOOL DISTRICT OF LOS ANGELES COUNTY AS SET FORTH IN DEED RECORDED FEBRUARY 13, 1930 IN BOOK 9719, PAGE 244, OF OFFICIAL RECORDS.

EXCEPT TWO-FIFTHS INTEREST IN AND TO ALL OIL, GAS AND/OR OTHER HYDROCARBON SUBSTANCES AND MINERALS, IN, ON OR UNDER SAID LAND, AS RESERVED IN THE DEED FROM VIOLA E. MIDDAGH, RECORDED IN BOOK 22560, PAGE 440, OFFICIAL RECORDS OF SAID COUNTY.

BY QUITCLAIM DEED DATED SEPTEMBER 11, 1975 AND RECORDED SEPTEMBER 12, 1975 AS INSTRUMENT NO. 5982, LOUIS HARSON QUITCLAIMED TO CAROL SCHINDELHEIM ALL HIS INTEREST IN ALL OIL, GAS AND OTHER HYDROCARBON SUBSTANCES, LYING IN AND UTILITIES UNDER A DEPTH OF 500 FEET BELOW THE SURFACE OF SAID LAND, BUT WITHOUT ANY RIGHT OF ENTRY UPON THE SURFACE OF SAID LAND.

ALSO EXCEPT THEREFROM ANY PORTION OF SAID LAND LYING WITHIN PARCEL MAP NO. 5932, AS PER MAP RECORDED IN BOOK 67, PAGES 40 AND 41 OF PARCEL MAPS.

BENCH MARK AND VERTICAL DATUM:

LOS ANGELES COUNTY PUBLIC WORKS
VERTICAL CONTROL DATA
DATUM: NAVD 1988
ADJUSTMENT: 2008 MALIBU QUAD

PRIMARY BM:
B.M. Y 11262
REC./MEAS.ELEVATION = 1,057.396 FEET
L&T IN CTR HDWL CULV 23' S/O C/L CALABASAS RD &
265' W/O C/L CALABASAS PKWY OFF-RAMP (101 FWY)

SECONDARY BM:
B.M. Y 12350
MEAS. ELEVATION = 1,085.264 FEET
REC. ELEVATION = 1,085.301 FEET
L&DPW TAG IN CB CALABASAS ROAD NR LARGE METAL
PP#2367217E 96' E/O & OPP WLY MOST DR TO BLDG#24500

ZONING AND LAND USE:

ZONING: COMERCIAL LIMITED (CL)

FLOOD ZONE:

FLOOD ZONE "X" - AREAS OF MINIMAL FLOODING PER COMMUNITY PANEL NO. 06037C1268F, DATED 09-26-2008.

AREA:

GROSS AREA: 476,553 SF (10.94AC)

TOTAL DISTURBED:

PROPOSED: 2.4 AC

TOTAL IMPERVIOUS AREA:

EXISTING: 71,110 SF
PROPOSED: 80,149 SF

GROSS YARDAGE VOLUMES:

CUT:	20,700 CY
FILL:	700 CY
TOTAL:	20,000 (EXPORT)

PUBLIC UTILITIES / SERVICES:

WATER: LAS VIRGENES MUNICIPAL WATER DISTRICT
4232 LAS VIRGENES ROAD
CALABASAS, CA 91302
(818) 880-4110

ELECTRICAL: SOUTHERN CALIFORNIA EDISON
3589 FOOTHILL DRIVE
THOUSAND OAKS, CA 91361
(818) 494-7016

SHEET INDEX:

FOR CONCEPTUAL APPROVAL ONLY

Prepared By:

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Calabasas, CA 91302
Phone: (818) 591-1050
www.diamondwest.net

PLAN REVISION DESCRIPTIONS

PREPARED BY OR UNDER THE DIRECTION OF:

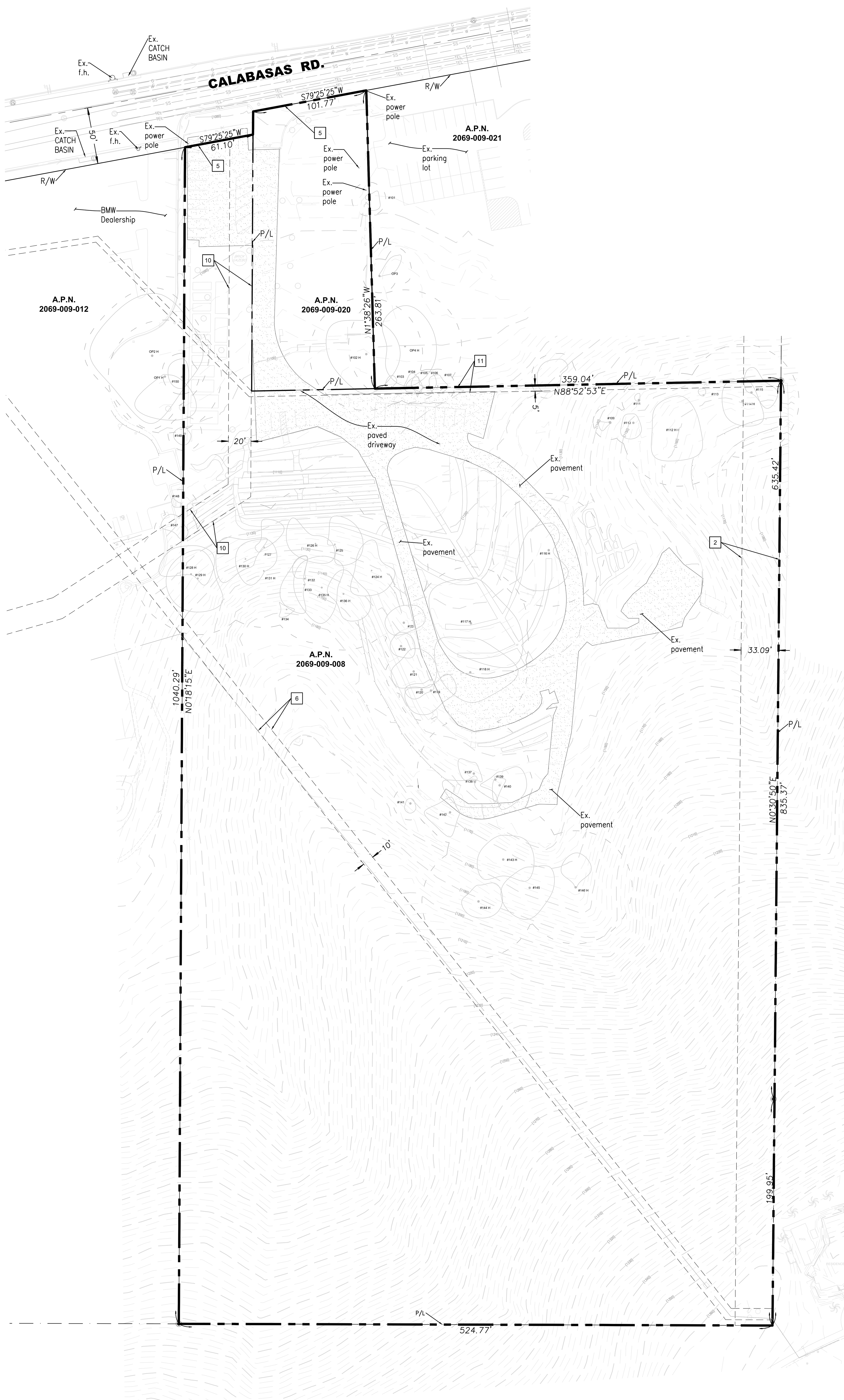
REGISTERED PROFESSIONAL ENGINEER
WILLIAM C. CUNNINGHAM III
No. 80129
Exp. 09/30/24
CIVIL
STATE OF CALIFORNIA
PRELIMINARY

SIGNATURE: DATE:

ADDRESS:
CALABASAS KIA
24460 CALABASAS ROAD,
CALABASAS CA, 91302

SHEET TITLE:
TITLE SHEET

SHEET NO.
1
OF 7 CIVIL SHEETS
DATE: 2023-04-05



LEGEND:

- EXISTING CONTOUR
- 1300.0 EXISTING SPOT ELEVATION
- CANOPY
- TREE NUMBER
- EXISTING OAK TREE
- PROTECTION ZONE
- EXISTING TREE
- EXISTING POWER POLE
- EXISTING PAVEMENT
- Existing Right-of-Way
- Property Line
- Existing Easements
- Existing Street Centerlines
- Water (Existing)
- Sanitary Sewer (Existing)
- Natural Gas (Existing)
- Telephone (Existing)

EASEMENTS:	
2	POLES
5	PRIVILEGE AND RIGHT—EXTEND AND MAINTAIN DRAINAGE STRUCTURES AND EMBANKMENT SLOPES
6	PIPELINES
10	UNDERGROUND TELEPHONE, TELEGRAPH, AND COMMUNICATION STRUCTURES
11	PIPELINES

FOR CONCEPTUAL APPROVAL ONLY

Prepared By:

DIAMOND WEST

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PLAN REVISION DESCRIPTIONS

PREPARED BY OR UNDER THE DIRECTION OF:

REGISTERED PROFESSIONAL ENGINEER
PRELIMINARY
No. 80129
Exp. 09/30/24
CIVIL
STATE OF CALIFORNIA

SIGNATURE: DATE:

ADDRESS:
CALABASAS KIA
24460 CALABASAS ROAD,
CALABASAS CA, 91302

SHEET TITLE:
EXISTING CONDITIONS PLAN

SHEET NO.

2

OF 7 CIVIL SHEETS

DATE: 2023-04-05

P:\22-1015 - Hello Auto Group re Calabasas Kia\230-Entitlement Planning\dwg\22-1015 - Calabasas Kia - 2 - Existing Conditions Plan.dwg
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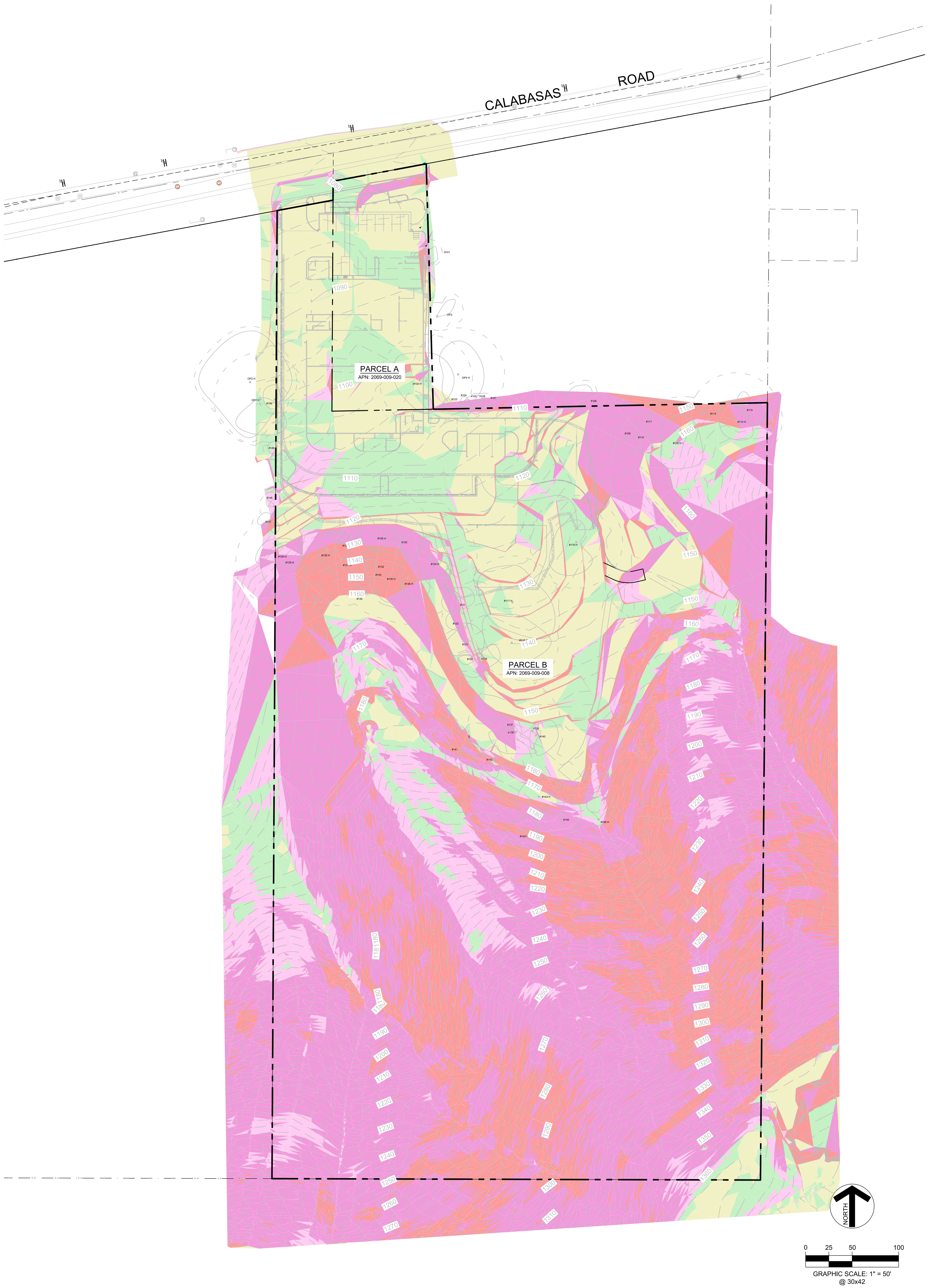

Slopes Table			
Number	Minimum Slope	Maximum Slope	Color
1	0.00%	10.00%	
2	10.00%	20.00%	
3	20.00%	30.00%	
4	30.00%	50.00%	
5	50.00%	5336873.77%	

LEGEND:

- Subject Parcel Boundary
- Existing Right-of-Way
- Existing Parcels
- Existing Easements
- Existing Street Centerlines

FOR CONCEPTUAL APPROVAL ONLY

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PLAN REVISION DESCRIPTIONS

PREPARED BY OR UNDER THE DIRECTION OF:

PRELIMINARY
REGISTERED PROFESSIONAL ENGINEER
WILLIAM C. CUNNINGHAM
No. 80129
Exp. 09/30/24
CIVIL
STATE OF CALIFORNIA

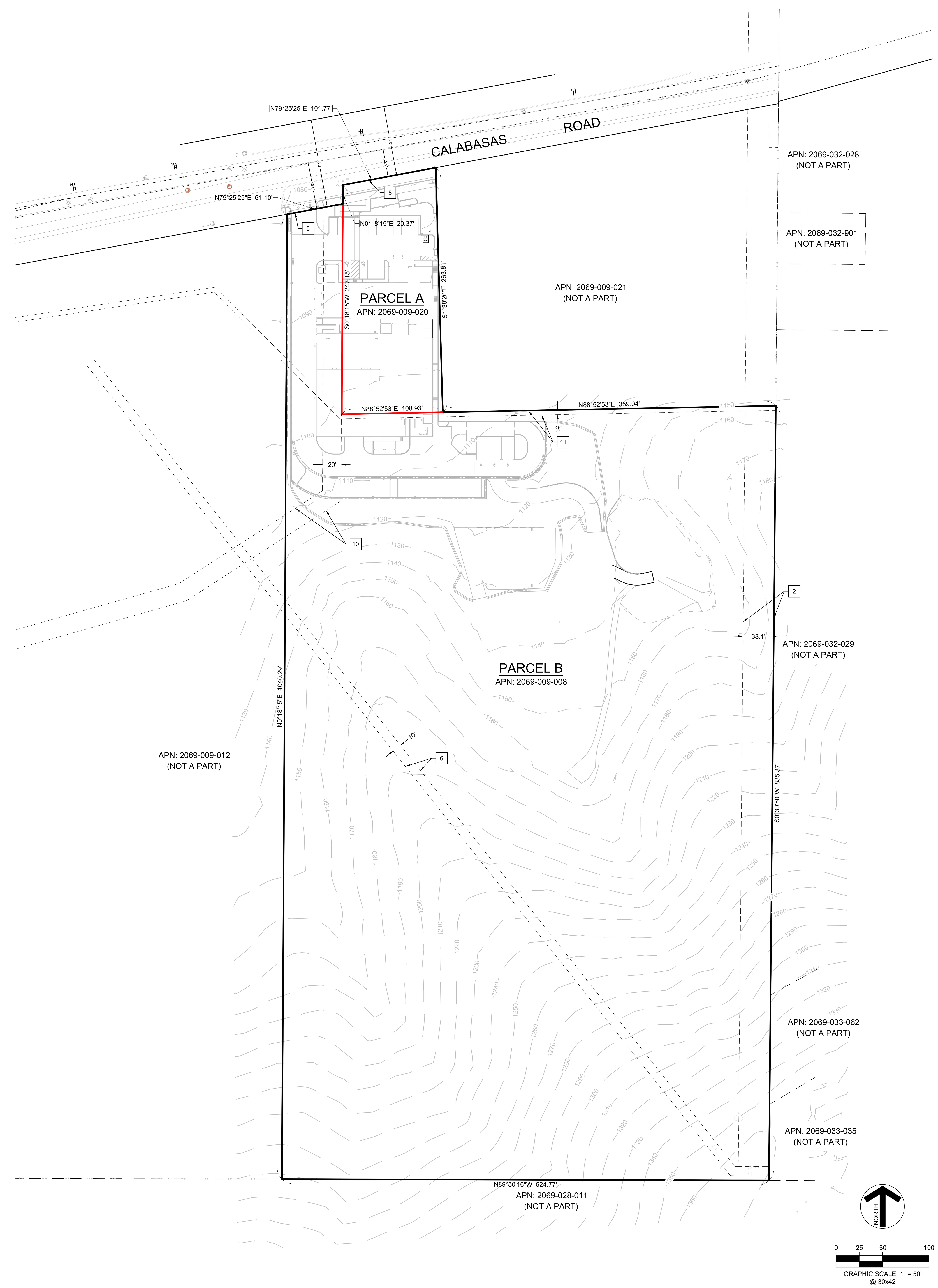
SIGNATURE: DATE:

ADDRESS:
CALABASAS KIA
24460 CALABASAS ROAD,
CALABASAS CA, 91302

SHEET TITLE:
SLOPE MAP

SHEET NO.
3
OF 7 CIVIL SHEETS
DATE: 2023-04-05

P:\22-1015 - Helio Auto Group re Calabasas Kia\230-Entitlement Planning\dwg\22-1015 - Calabasas Kia - 3 - Slope Map.dwg
04/05/23 3:04pm-SEAN



LEGEND:

- Existing Property Boundary, to Remain
- Existing Property Boundary, to be Removed
- Proposed Property Boundary
- Existing Right-of-Way
- Existing Parcels
- Existing Easements
- Existing Street Centerlines

LEGAL DESCRIPTION:

ALL THAT CERTAIN REAL PROPERTY SITUATED IN THE COUNTY OF LOS ANGELES, STATE OF CALIFORNIA, DESCRIBED AS FOLLOWS:

PARCEL A: ASSESSORS PARCEL NO: 2069-009-020

PARCEL 1 OF PARCEL MAP NO. 5932, IN THE CITY OF CALABASAS, COUNTY OF LOS ANGELES, STATE OF CALIFORNIA, AS PER MAP RECORDED IN BOOK 67, PAGES 40 AND 41 OF PARCEL MAPS, IN THE OFFICE OF THE COUNTY RECORDER OF SAID COUNTY.

PARCEL B: ASSESSORS PARCEL NO: 2069-009-008

THAT PORTION OF THE SOUTHEAST QUARTER OF SECTION 1, TOWNSHIP 1 NORTH, RANGE 17 WEST, SAN BERNARDINO BASE AND MERIDIAN, IN THE COUNTY OF LOS ANGELES, STATE OF CALIFORNIA, ACCORDING TO THE OFFICIAL PLAT OF SAID LAND FILED IN THE DISTRICT LAND OFFICE, SEPTEMBER 25, 1896, BOUNDED EASTERLY AND SOUTHERLY BY THE EASTERLY AND SOUTHERLY LINES OF SAID SOUTHEAST QUARTER, BOUNDED NORTHERLY BY THE SOUTHERLY LINE OF CALABASAS ROAD, AS IT NOW EXISTED OF RECORD ON MARCH 9, 1965, AND BOUNDED WESTERLY BY THE FOLLOWING DESCRIBED LINE:

BEGINNING AT THE INTERSECTION OF THE SOUTHERLY LINE OF SAID SOUTHEAST QUARTER WITH A LINE PARALLEL WITH AND 60 FEET WESTERLY MEASURED AT RIGHT ANGLES, FROM THE SOUTHERLY PROLONGATION OF THE WESTERLY LINE OF THE LAND DESCRIBED IN THE DEED TO THE CALABASAS SCHOOL DISTRICT OF LOS ANGELES RECORDED FEBRUARY 13, 1930 AS INSTRUMENT NO. 1724, IN BOOK 9719, PAGE 244, OF OFFICIAL RECORDS, IN THE OFFICE OF THE COUNTY RECORDER OF SAID COUNTY, SAID POINT OF INTERSECTION BEING 528.29 FEET
WESTERLY MORE OR LESS FROM THE SOUTHEAST CORNER OF SAID SOUTHEAST QUARTER; THENCE NORTHERLY ALONG SAID PARALLEL LINE, TO SAID SOUTHERLY LINE OF THE LAND DESCRIBED IN SAID DEED RECORDED IN BOOK 14302, PAGE 296 OF SAID OFFICIAL RECORDS.

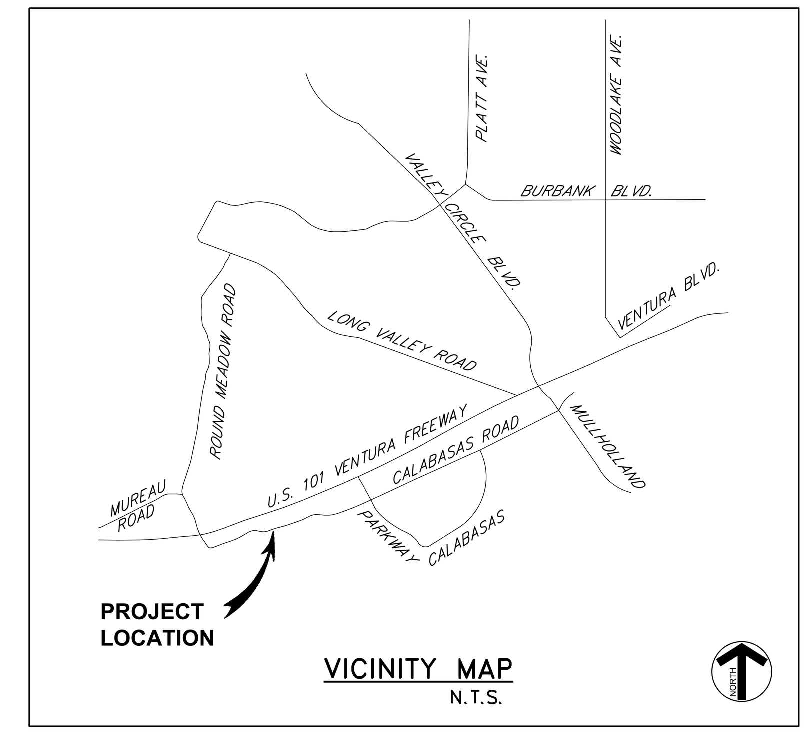
EXCEPT ANY PORTION THEREOF OWNED BY THE CALABASAS SCHOOL DISTRICT OF LOS ANGELES COUNTY AS SET FORTH IN DEED RECORDED FEBRUARY 13, 1930 IN BOOK 9719, PAGE 244, OF OFFICIAL RECORDS.

EXCEPT TWO-FIFTHS INTEREST IN AND TO ALL OIL, GAS AND/OR OTHER HYDROCARBON SUBSTANCES AND MINERALS, IN, ON OR UNDER SAID LAND, AS RESERVED IN THE DEED FROM VIOLA E. MIDDAGH, RECORDED IN BOOK 22560, PAGE 440, OFFICIAL RECORDS OF SAID COUNTY.

BY QUITCLAIM DEED DATED SEPTEMBER 11, 1975 AND RECORDED SEPTEMBER 12, 1975 AS INSTRUMENT NO. 5982, LOUIS HARSON QUITCLAIMED TO CAROL SCHINDELHEIM ALL HIS INTEREST IN ALL OIL, GAS AND OTHER HYDROCARBON SUBSTANCES, LYING IN AND UTILITIES UNDER A DEPTH OF 500 FEET BELOW THE SURFACE OF SAID LAND, BUT WITHOUT ANY RIGHT OF ENTRY UPON THE SURFACE OF SAID LAND.

ALSO EXCEPT THEREFROM ANY PORTION OF SAID LAND LYING WITHIN PARCEL MAP NO. 5932, AS PER MAP RECORDED IN BOOK 67, PAGES 40 AND 41 OF PARCEL MAPS.

EASEMENTS:	
2	POLES
6	PIPELINES
10	UNDERGROUND TELEPHONE, TELEGRAPH, AND COMMUNICATION STRUCTURES TO BE VACATED.
11	PIPELINES TO BE VACATED.



VICINITY MAP
N.T.S.

EXISTING PARCEL SUMMARY

PARCEL	EXISTING APN	EXISTING AREA	EXISTING ZONING	EXISTING PLAN DESIGNATION
Parcel A	2069-009-020	0.61 ac	CL (Commercial Limited)	CL (Commercial Limited)
Parcel B	2069-009-008	10.33 ac	CL (Commercial Limited)	CL (Commercial Limited)

PROPOSED PARCEL SUMMARY

PARCEL	PROPOSED AREA	PROPOSED ZONING	PROPOSED PLAN DESIGNATION
Parcel 1	10.94 ac	No Change CL (Commercial Limited)	No Change CL (Commercial Limited)

SITE MAP OF CERTIFICATE OF COMPLIANCE
FOR LOT LINE ADJUSTMENT NO. LLAXX-XXXXX

FOR CONCEPTUAL APPROVAL ONLY

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PLAN REVISION DESCRIPTIONS

PREPARED BY OR UNDER THE DIRECTION OF:

REGISTERED PROFESSIONAL ENGINEER
No. 80129
Exp. 09/30/24
CIVIL
STATE OF CALIFORNIA

PRELIMINARY

SIGNATURE: DATE:

ADDRESS:
CALABASAS KIA
24460 CALABASAS ROAD,
CALABASAS CA, 91302

SHEET TITLE:
TENTATIVE LOT LINE ADJUSTMENT

SHEET NO.

4

OF 7 CIVIL SHEETS

DATE: 2023-04-05

DETAIL ENLARGMENT
SCALE 1"=50'

EASEMENTS:

2	POLES
5	PRIVILEGE AND RIGHT–EXTEND AND MAINTAIN DRAINAGE STRUCTURES AND EMBANKMENT SLOPES
6	PIPELINES
10	UNDERGROUND TELEPHONE, TELEGRAPH, AND COMMUNICATION STRUCTURES TO BE VACATED.
11	PIPELINES TO BE VACATED.

LEGEND:

- 98.00 TC — SPOT ELEVATION / DESCRIPTION OF ELEVATION
- (98.00) EG — EXISTING SPOT ELEVATION / DESCRIPTION OF ELEVATION
- 1.0% — PROPOSED GRADIENT AND DIRECTION OF FLOW
- (1.0%) — EXISTING GRADIENT AND DIRECTION OF FLOW
- 1 — SITE PLAN NOTE
- 1150 — PROPOSED CONTOUR
- (1150) — EXISTING CONTOUR
- 986.4 — EXISTING SPOT ELEVATION
- 2 — EASEMENT NOTE
- PROPOSED STEP
- PROPOSED DRAIN
- EXISTING TREE
- TREE TO BE REMOVED
- #2 — CANOPY / TREE NUMBER / EXISTING OAK TREE / PROTECTION ZONE
- #2 — CANOPY / TREE NUMBER / DEMO OAK TREE / PROTECTION ZONE
- #2 — CANOPY / TREE NUMBER / ENCROACH OAK TREE / PROTECTION ZONE

Subject Parcel Boundary
Existing Right-of-Way
Proposed Property Line
Existing Parcels
Existing Easements
Existing Street Centerlines
Building Setback
Proposed Building
Existing Retaining Wall
Proposed Retaining Wall
Proposed Fence
Proposed Curb
GB — Grade Break
Storm Drain (Existing)
Storm Drain (Proposed)
SS — Sanitary Sewer (Existing)
SS — Sanitary Sewer (Proposed)
W — Water (Existing)
W — Water (Proposed)
IR — Recycled Water (Existing)
IR — Recycled Water (Proposed)
G — Natural Gas (Existing)
G — Natural Gas (Proposed)
CTV — Communications (Existing)
TEL — Telephone (Existing)

A.C. PAVEMENT
CONCRETE
LANDSCAPING
EXISTING PAVEMENT
TRUNCATED DOMES

ABBREVIATIONS:

1F	FIRST FLOOR	HP	HIGH POINT
APN	ASSESSOR'S PARCEL NUMBER	INV	INVERT
BLDG	BUILDING	MH	MANHOLE (UTILITY)
CB	CATCH BASIN	PA	PLANTER AREA
CL	CENTER LINE	P/L	PROPERTY LINE
CONC	CONCRETE	PP	POWER POLE
D/W	DRIVEWAY	PR	PROPOSED
EG	EXISTING GROUND	PROP	PROPERTY
ELEC	ELECTRICAL	ROW	RIGHT OF WAY
EP	EDGE OF PAVEMENT	SD	STORM DRAIN
EX	EXISTING	SDMH	STORM DRAIN MANHOLE
FF	FINISHED FLOOR	SMH	SEWER MANHOLE
FG	FINISHED GROUND	SWR	SEWER
FH	FIRE HYDRANT	TC	TOP OF CURB
FL	FLOW LINE	TG	TOP OF GRATE
FS	FINISHED SURFACE	TMH	TELEPHONE MANHOLE
GB	GRADE BREAK	TW	TOP OF WALL
H	HEIGHT	(TYP)	TYPICAL
		UNKN	UNKNOWN

SITE PLAN NOTES:

1. PROPOSED PAVEMENT.
2. PROPOSED SIDEWALK.
3. PROPOSED CURB.
4. PROPOSED RETAINING WALL.
5. PROPOSED RAMP.
6. PROPOSED DRAINAGE SWALE.
7. PROPOSED DRAIN INLET.
8. PROPOSED STEP.
9. PROPOSED CURB & GUTTER.
10. PROPOSED WALKWAY.
11. PROPOSED PARKWAY DRAIN.
12. PROPOSED UNDERGROUND STORM WATER DETENTION/LID FACILTY.
13. PROPOSED BACKFLOW ASSEMBLY.
14. PROPOSED TRASH ENCLOSURE.
15. PROPOSED WATER LINE.
16. PROPOSED ELECTRIC TRANSFORMER.
17. PROPOSED SEWER LINE.
18. PROPOSED STORM DRAIN LINE.
19. EXISTING OAK TREE TO BE REMOVED.
20. PROPOSED DEBRIS BASIN.
21. PROPOSED VEGETATED DRAINAGE CHANNEL.
22. PROPOSED ROLLED CURB.
23. PROPOSED TREE WELL.
24. PROPOSED FIRE LINE.
25. EXISTING POWER POLE TO BE RELOCATED/REMOVED.
26. EXISTING PAVEMENT TO REMAIN.

NORTH

GRAPHIC SCALE: 1" = 30' @ 30x42
0 15 30 60

FOR CONCEPTUAL APPROVAL ONLY

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PLAN REVISION DESCRIPTIONS

PREPARED BY OR UNDER THE DIRECTION OF:

REGISTERED PROFESSIONAL ENGINEER — C. CUNNINGHAM — No. 80129 — Exp. 09/30/24 — CIVIL — STATE OF CALIFORNIA

PRELIMINARY

SIGNATURE: DATE:

ADDRESS:
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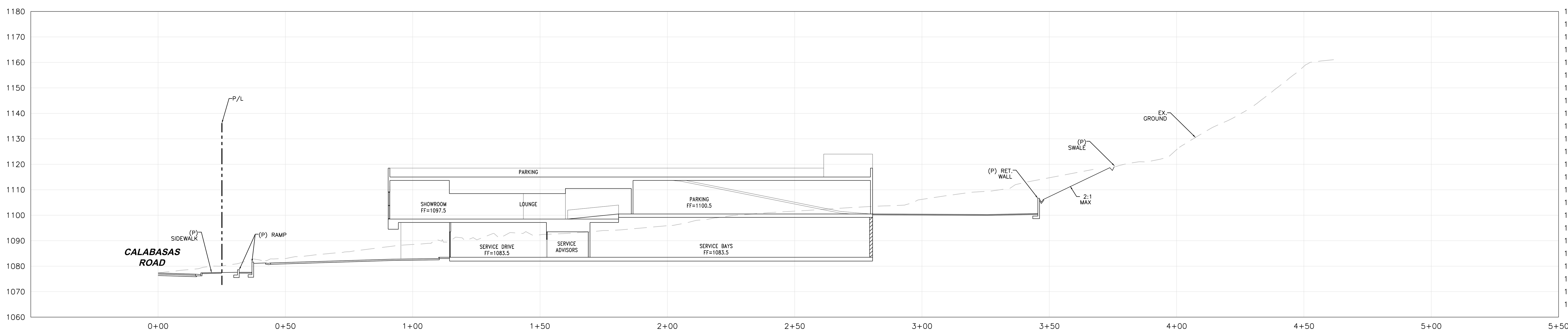
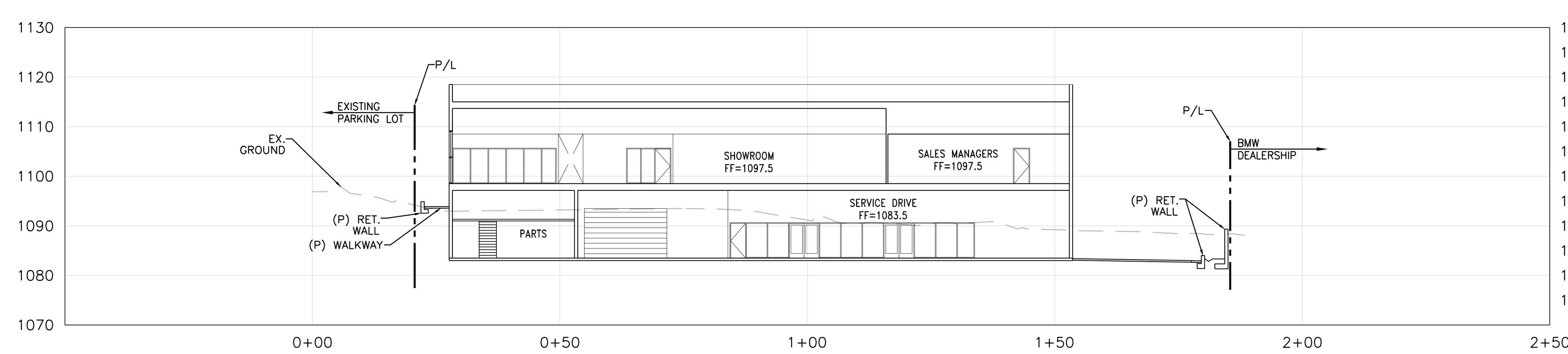
24460 CALABASAS ROAD,
CALABASAS CA, 91302

SHEET TITLE:
CONCEPTUAL GRADING AND
DRAINAGE PLAN

SHEET NO.
5
OF 7 CIVIL SHEETS
DATE: 2023-04-05



SECTION A-A



SECTION B-B

0 10 20 40
GRAPHIC SCALE: 1" = 20'
@ 30x42

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No. 80129
Exp. 09/30/24
CIVIL
STATE OF CALIFORNIA

SIGNATURE: DATE:

ADDRESS:

CALABASAS KIA
24460 CALABASAS ROAD,
CALABASAS CA, 91302

SHEET TITLE:

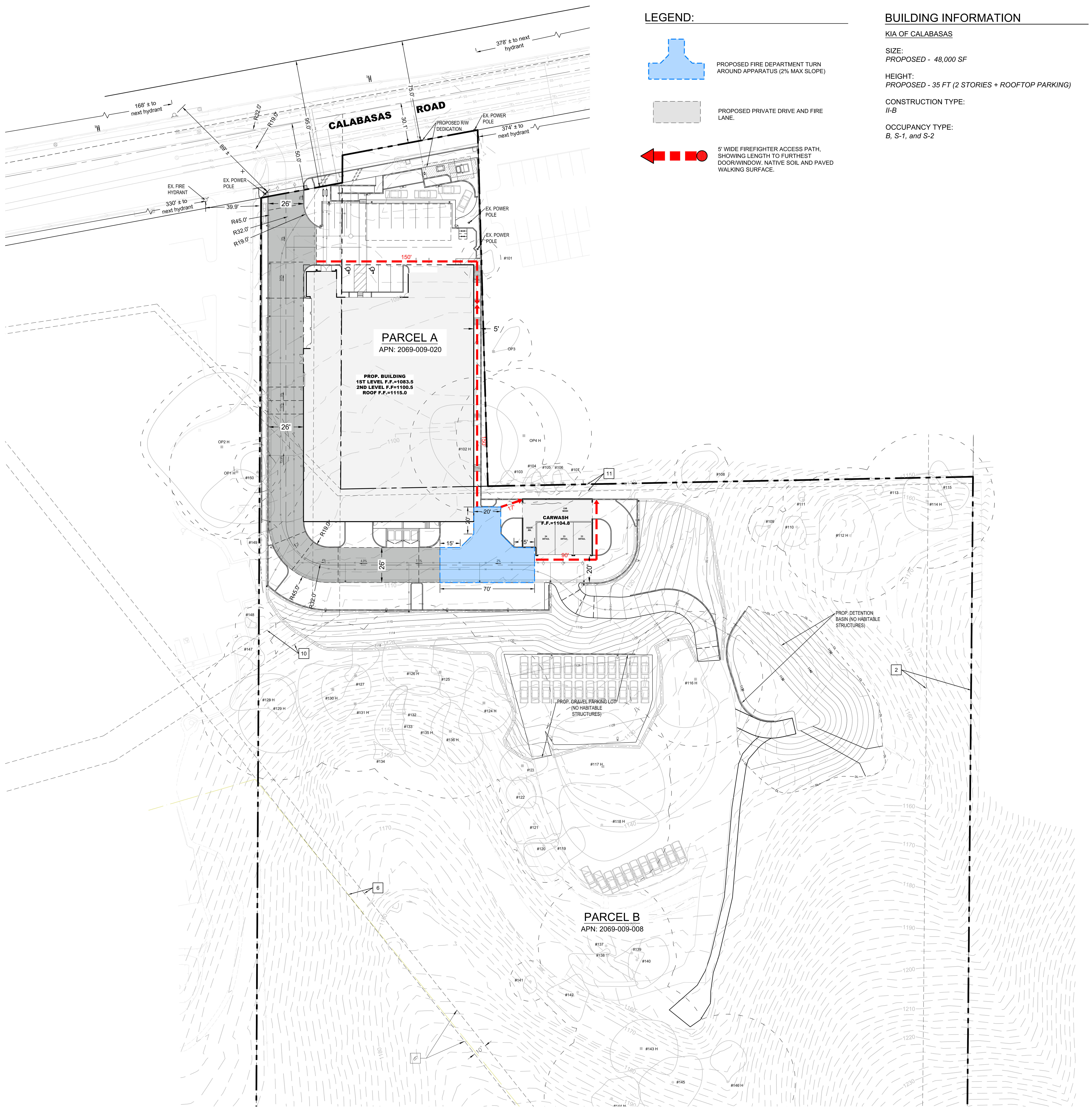
SECTIONS

SHEET NO.

6

OF 7 CIVIL SHEETS

DATE: 2023-04-04



LEGEND:

PROPOSED FIRE DEPARTMENT TURN AROUND APPARATUS (2% MAX SLOPE)

PROPOSED PRIVATE DRIVE AND FIRE LANE.

5' WIDE FIREFIGHTER ACCESS PATH, SHOWING LENGTH TO FURTHEST DOOR/WINDOW, NATIVE SOIL AND PAVED WALKING SURFACE.

BUILDING INFORMATION

KIA OF CALABASAS

SIZE:
PROPOSED - 48,000 SF

HEIGHT:
PROPOSED - 35 FT (2 STORIES + ROOFTOP PARKING)

CONSTRUCTION TYPE:
II-B

OCCUPANCY TYPE:
B, S-1, and S-2

Fire Department Notes:

1. Approved building address numbers, building numbers or approved building identification shall be provided and maintained so as to be plainly visible and legible from the street fronting the property. The numbers shall contrast with their background, be Arabic numerals or alphabet letters, and be a minimum of 4 inches high with a minimum stroke width of 0.5 inch. (Fire code 505.1)
2. Provide a minimum unobstructed width of 26 feet, except for approved security gates in accordance with Section 503.6 and an unobstructed vertical clearance "clear to sky" Fire Department vehicular access within 150 feet of all portions of the exterior building walls. (Fire Code 503.2.1)
3. Fire Department vehicular access roads must be installed and maintained in a serviceable manner prior to and during the time of construction. (Fire Code 501.4)
4. A minimum 5 foot wide approved firefighter access walkway leading from the fire apparatus access road to the buildings exterior openings shall be provided for fire fighting and rescue purposes. Fire Code 504.1. piping shall be witnessed by an authorized Fire Department representative. No underground piping or thrust blocks shall be covered with earth or hidden from view until the Fire Department representative has been notified and given not less the 48 hours in which to inspect such installations. Fire code 901.5, County of Los Angeles Fire Department Regulation 7.
5. All required public fire hydrants shall be installed, tested and accepted prior to beginning construction. Fire Code 501.4
6. Provide an approved automatic fire sprinkler system as set forth by Building Code 903 and Fire Code 903. Plans shall be submitted to the Sprinkler Plan Check Unit for review and approval prior to installation. Reason: Residential code and Fire Code 903.1 and fire flow reduction. Type of sprinkler system: 903.1.1, 903.1.2, 903.3.1.3
7. Egress doors shall be readily openable from the egress side without the use of a key or any special knowledge or effort. Building Code 1008.1.8.
   See architectural plans for door schedule.
8. Clearance of brush and vegetative growth shall be maintained per Fire Code 325
9. Walls and soffits within enclosed usable spaces under enclosed and unenclosed stairways shall be protected by 1-hour fire-resistance-rated construction or the fire-resistance rating of the stairway enclosure, whichever is greater. Building Code 1009.5.3.
   See architectural floor plans and details.
10. Eaves and soffits shall meet on of the following:
    a. Noncombustible construction on the exposed underside OR
    b. Protected by ignition-resistant materials OR
    c. Meet the requirements of SFM 12-7A-3 (Fire Code 4710.2.3)
11. Roof valley flashings shall be not less than 0.019-inch (0.48 mm) No. 26 gage galvanized sheet corrosion-resistant metal installed over not less than one layer of minimum 72 pound (32.4 kg) mineral-surfaced non-perforated cap sheet complying with ASTM D3909, at least 36-inch wide (914mm) running the full length of the valley. R337.5.3 Building Code 705A.2.
    See architectural roof plans.
12. Roof and attic vents shall resist the intrusion of flame and embers into the attic area of the structure. Vent openings shall be protected by corrosion-resistant, noncombustible wire mesh with 1/4-inch openings. Vents shall NOT be installed in eaves or cornices. (Fire Code 4710.2.1 & 4710.2.2)
    See architectural roof plan and exterior elevations.
13. Exterior windows, window walls, glazed doors, and glazed openings within exterior doors shall meet one of the following:
    a. Mulit-pane glazing units with a minimum of one tempered pane OR
    b. Glass block units OR
    c. Have a fire-resitsance rating of not less than 20 minutes, when tested according to ASTM E 2010 OR
    d. Meet the performance standards of SFM 12-7A-2 (Fire Code 4715.2.2)
    See architectural window/door schedule and/or architectural floor plans.
14. Decking, surfaces, stair treads, risers, and landing of decks, porches, and balconies where any portion of such surface is within 10 feet of the primary structure shall comply with one of the following:
    a. Ignition resistant material AND meet SFM 12-7A-4 parts A and B OR
    b. Approved noncombustible construction OR
    c. Heavy timber construction OR
    d. Exterior fire retardant treated wood construction (Fire Code 4716.1.1)
       See architectural plans and details.
15. Exterior door assemblies shall meet one of the following:
    a. Approved noncombustible construction OR
    b. Solid core wood having stiles and rails not less than 1-3/8-inch thick with interior panel thickness not less than 1-1/4-inch thick OR
    c. Minimum 20 minute fire resistance rating when tested according to ASTM E 2074 OR
    d. Conform to performance standard SFM 12-7A-1(Fire Code 4715.2.3)
       See architectural window/door schedule and/or architectural floor plans.
16. Single or multiple station smoke alarms shall be installed in the locations described in Building Code 907-2.10.1.1 and 907.2.10.1.2. Smoke alarms shall receive their primary power from the building wiring where such wiring is served from a commercial source and shall be equipped with a battery backup. Where more than one smoke alarm is required to be installed within an individual dwelling unit, the smoke alarms shall be interconnected in such a manner that the activation of one alarm will activate all of the alarms in the individual unit. Building Code 907.2.10.2 and 907.2.10.3.
    See architectural floor plans and notes.
17. The gradient of fire department vehicle access roads shall not exceed 15 percent unless approved by the fire code official. Fire code 503.2.7
18. All fire hydrants shall measure 6"X4"X2.5", 8"X8"X6", brass or bronze, conforming to American Water Works Association Standard C503, or approved equal, and shall be installed in compliance with the County of Los Angeles Fire Department Regulation 8. Fire code 507.5 and Regulation 8.
19. Plans showing underground piping, fire department connection, or private on-site fire hydrant shall be submitted to the sprinkler plan check unit to review and approval prior to installation. Fire code 901.2 County of Los Angeles Fire Department Regulation 7.
20. Roof gutters shall be provided with a means to prevent the accumulation of leaves and debris in the gutter. (Residential Code R327.534 and building code 750A.4)
21. Single and multiple-station carbon monoxide alarms shall be listed as complying with the requirement of UL 2034. Carbon monoxide detectors shall be listed as complying with the requirements of UL 2075. Carbon monoxide alarms required by (Sections R315.1 and R315.2) or (Section 420.4.1 and 420.4.2) shall be installed in the following locations:
    1. Outside of each separate dwelling unit sleeping area in the immediate vicinity of bedroom(s).
    2. On every Level of a dwelling unit including basements.
    3. For R-1 only.
       a. On the ceiling of sleeping units with permanently installed fuel-burning appliances . Residential code R315.3, Building code 420.4.3
22. The required fire flow for a single private fire hydrant at this location is TBD gpm, at 20 psi residual pressure, for a duration of 2 hours over and above maximum daily domestic demand. Fire Code 106.1.
    The required fire flow is based on the flowing calculation:

Type of construction per the Building Code	Type II-B Yes_X_ No___
Fire flow based on the fire flow calculation area	TBD
Size of lot (acres)	_10.94_ acres
Reduction for fire sprinklers (Maximum 50%)	TBD
Total fire flow required:	TBD

23. Fire apparatus access roads must be identified with approved signs. temporary signs shall be installed at each street intersection when construction of new roadways allows passage by vehicles. Signs shall be of an approved size, weather resistant and be maintained unitl replaced by permanent signs. Fire code 505.2
24. Provide approved signs or other approved notices or markings that include the words NO PARKING-FIRE LANE. Signs shall have a minimum dimension of 12 inches wide by 18 inches high and have red letterson a white reflective background. Signs shall be provided for fire apparatus roads, to clearly indicate the entrance to such road, or prohibit the obstruction thereof and at intervals, as required by the Fire Inspector. Fire Code 503.3.
25. When security gates are provided, maintain a minimum access width of 26 feet. The security gate shall be provided with an approved means of emergency operation, and shall be maintained operational at all times and replaced or repaired when defective. Electric gate operators, where provided, shall be listed in accordance with UL 325. Gates intended for automatic operation shall be designed, constructed and installed to comply with the requirements of ASTM F220. Gates shall be of the swinging or sliding type. Construction of gates shall be of materials that allow manual operation by one person. Fire Code 503.6.

NORTH

0 15 30 60
GRAPHIC SCALE: 1" = 30'
@ 30x42

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PLAN REVISION DESCRIPTIONS

PREPARED BY OR UNDER THE DIRECTION OF:

PRELIMINARY

SIGNATURE: DATE:

ADDRESS: CALABASAS KIA
24460 CALABASAS ROAD,
CALABASAS CA, 91302

SHEET TITLE: FIRE ACCESS EXHIBIT

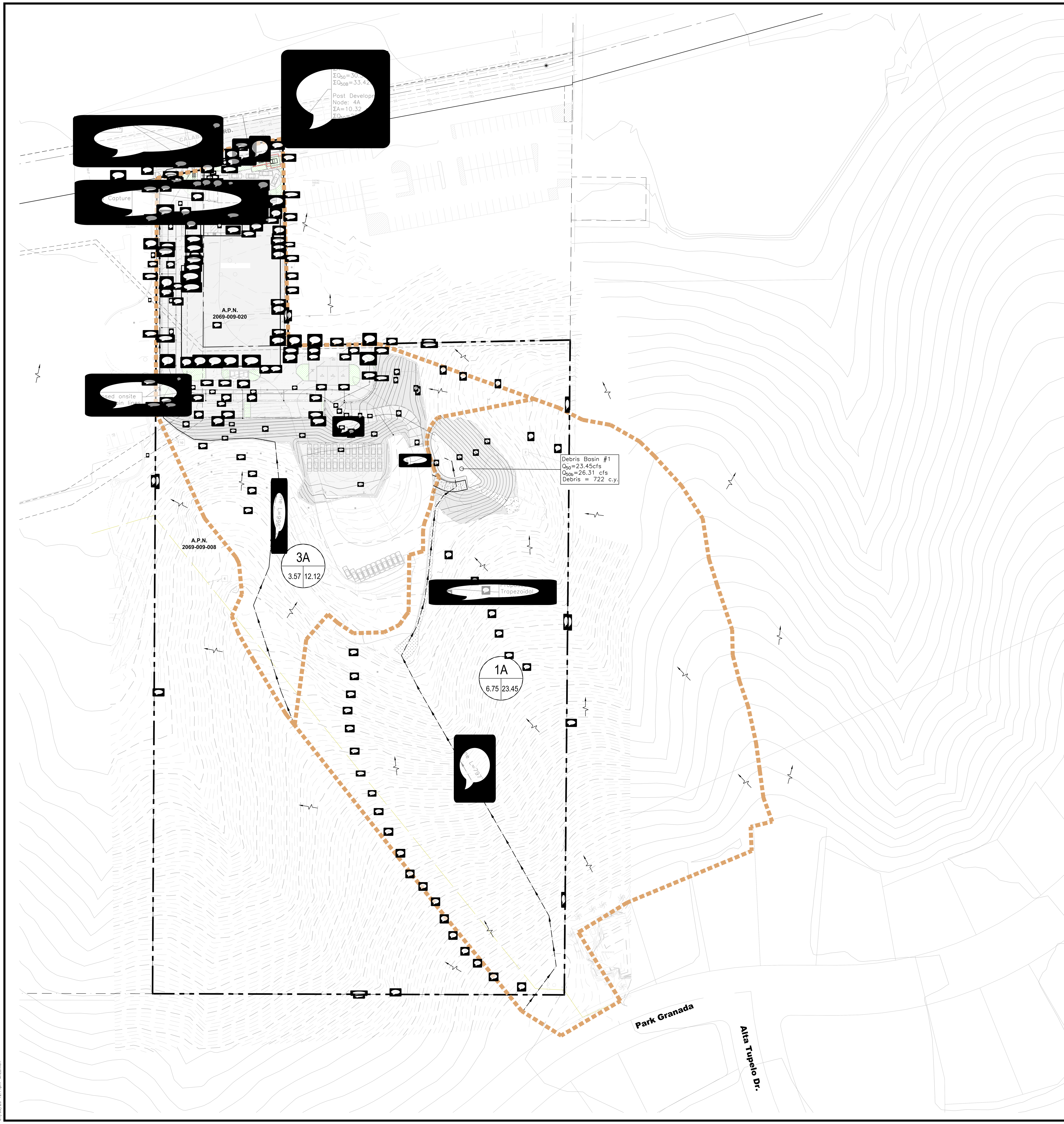
SHEET NO. 7

OF 7 CIVIL SHEETS

DATE: 2023-04-05

<u>Exhibit 4:</u>

Proposed Conditions Hydrology Map



[redacted]

Hydrology Proposed Conditions		
Sub Area	1A	3A**
Area (Sf)	294,158	155,332
Area (Arces)	6.75	3.57
Flowpath (ft)	797	996
Slope	0.29	0.14
% Impervious *	0.01	0.50
50 yr Rain Fall (in)	7.35	7.35
Soil Type	4	4
Debris Zone	4.00	4.00
Soil Type	4	4
Tc (min) - 50yr	5	6
Q (cfs) - 50yr	23.5	12.1
Qb (cfs) - 50yr	26.1	12.1
V (cf) - 50yr	39,058	52,215

* *Developed Area- Not Subject to Debris Production

Bulk Flow Rate			
Subarea Description	1A	3A**	Total
DPA Zone	4	4	-
Q50 (cfs)	23.5	12.1	-
Q50b (cfs)	26.1	12.1	-
Bulk Factor, BF	1.670 *	1.670 *	-
Ai (ac)	6.75	3.57	-
Au (ac)	6.42	0.00	-
Ad (ac)	0.33	3.57	-
QBB (cfs)	26.1	12.1	***38.26***

* Sedimentation Manual - Appendix B: Peak Bulking Factor Curves.
** Developed Area- Not Subject to Burn and Bulk.

Debris Production			
Subarea	1A	3A**	Total
DPA Zone	4	4	-
Debris Production (cy/mi2)	72000 *	72000 *	-
DPR (cy/ac)	112.50	112.50	75.00
A (ac)	6.42	1.28	7.70
DP (cy)	***722***	***144***	***866***

*Sedimentation Manual - Appendix B: Debris Production Rate Curves
**Developed Area- Not Subject to Debris Production

NORTH

0 25 50 100
GRAPHIC SCALE: 1" = 50'
@ 30x42

[redacted]

DIAMOND WEST
civil engineering • land surveying • land planning
23801 Calabasas Road, Suite 1034
Calabasas, CA 91302
Phone: (818) 591-1050
www.diamondwest.net

[redacted]

CALABASAS KIA
24460 CALABASAS ROAD,
CALABASAS CA, 91302

HYDROLOGY PROPOSED
CONDITIONS MAP

[redacted]

1

[redacted]

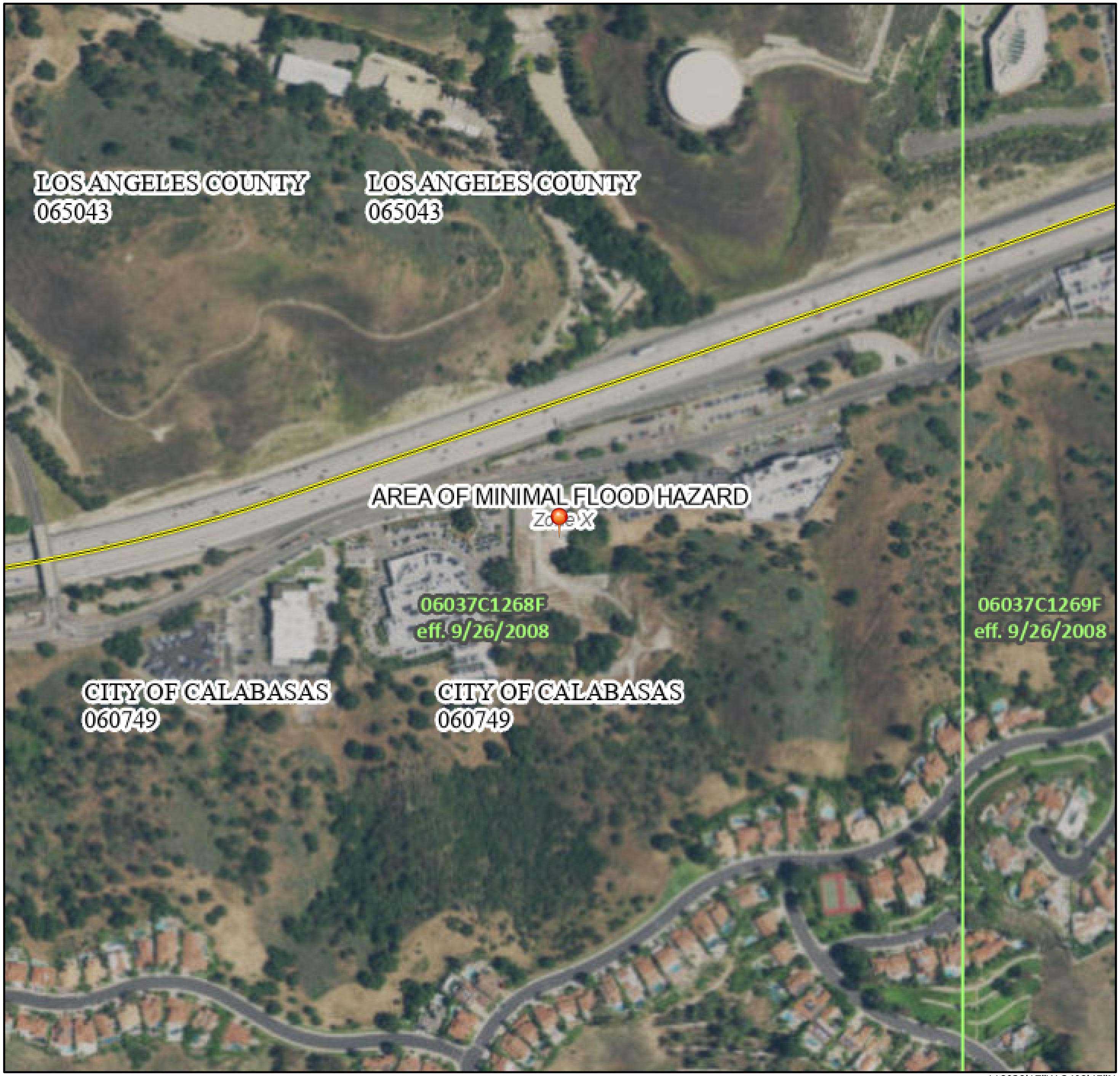
Appendix A:

FEMA Flood Insurance Rate Map

National Flood Hazard Layer FIRMette

FEMA

118°39'55"W 34°9'14"N



118°39'17"W 34°8'45"N

0 250 500 1,000 1,500 2,000 Feet

1:6,000

Basemap: USGS National Map: Orthoimagery: Data refreshed October, 2020

Legend

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

SPECIAL FLOOD HAZARD AREAS
- Without Base Flood Elevation (BFE) *Zone A, V, A99*
- With BFE or Depth *Zone AE, AO, AH, VE, AR*
- Regulatory Floodway

OTHER AREAS OF FLOOD HAZARD
- 0.2% Annual Chance Flood Hazard, Areas of 1% annual chance flood with average depth less than one foot or with drainage areas of less than one square mile *Zone X*
- Future Conditions 1% Annual Chance Flood Hazard *Zone X*
- Area with Reduced Flood Risk due to Levee. See Notes. *Zone X*
- Area with Flood Risk due to Levee *Zone D*

OTHER AREAS
- NO SCREEN Area of Minimal Flood Hazard *Zone X*
- Effective LOMRs
- Area of Undetermined Flood Hazard *Zone D*

GENERAL STRUCTURES
- Channel, Culvert, or Storm Sewer
- Levee, Dike, or Floodwall

OTHER FEATURES
- B 20.2 17.5 Cross Sections with 1% Annual Chance Water Surface Elevation
- 8 Coastal Transect
- 513 Base Flood Elevation Line (BFE)
- Limit of Study
- Jurisdiction Boundary
- Coastal Transect Baseline
- Profile Baseline
- Hydrographic Feature

MAP PANELS
- Digital Data Available
- No Digital Data Available
- Unmapped

N

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

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

The flood hazard information is derived directly from the authoritative NFHL web services provided by FEMA. This map was exported on 3/10/2023 at 10:37 AM and does not reflect changes or amendments subsequent to this date and time. The NFHL and effective information may change or become superseded by new data over time.

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

Appendix B:

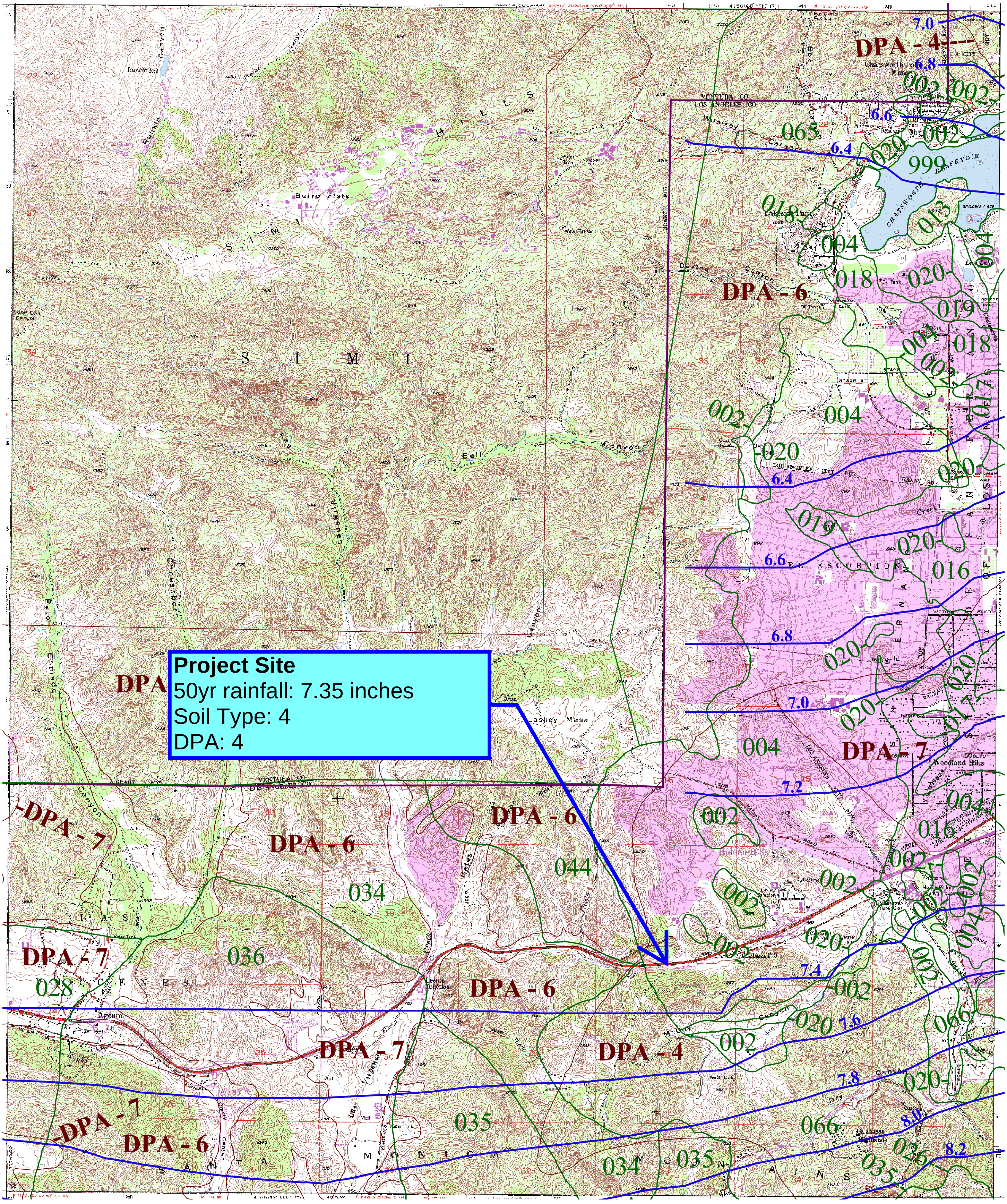
Rainfall & Soil Maps

34º 15' 00''

SANTA SUSANA 1-H1.34

-118º 45' 00''

THOUSAND OAKS 1-H1.24



CANOGA PARK 1-H1.26

-118º 37' 30''

MALIBU BEACH 1-H1.15

34º 07' 30''

016 SOIL CLASSIFICATION AREA

7.2 INCHES OF RAINFALL

DPA - 6 DEBRIS POTENTIAL AREA

1 0 1 2 Miles

25-YEAR 24-HOUR ISOHYET REDUCTION FACTOR: 0.878

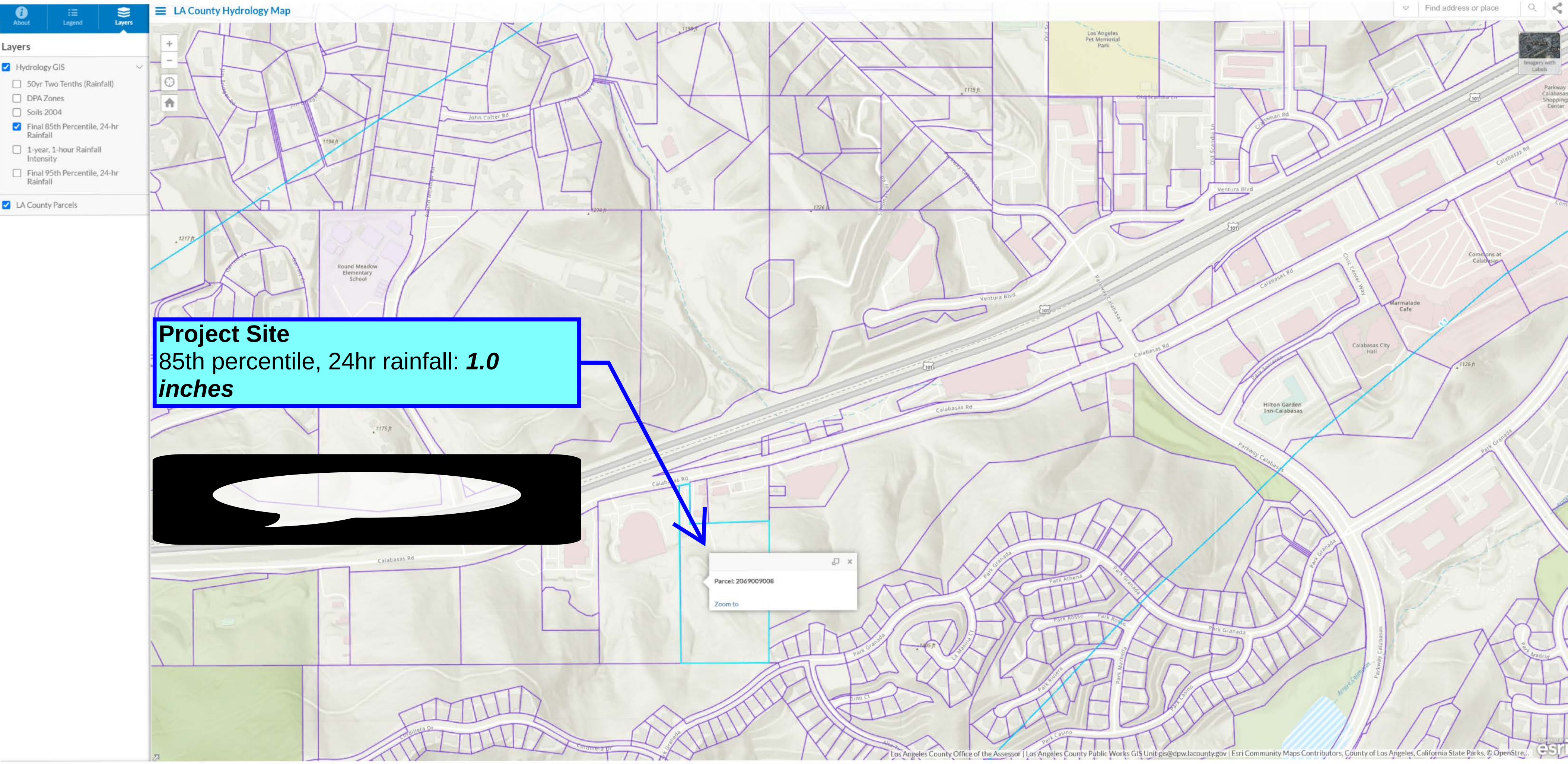
10-YEAR 24-HOUR ISOHYET REDUCTION FACTOR: 0.714

C A L A B A S A S

50-YEAR 24-HOUR ISOHYET

1-H1.25

LOS ANGELES COUNTY PUBLIC WORKS Public Service That Works



LA County Hydrology Map
About
Legend
Layers
Layers
Hydrology GIS
50yr Two Tenths (Rainfall)
DPA Zones
Soils 2004
Final 85th Percentile, 24-hr Rainfall
1-year, 1-hour Rainfall Intensity
Final 95th Percentile, 24-hr Rainfall
LA County Parcels
Find address or place
Project Site
85th percentile, 24hr rainfall: 1.0 inches
Parcel: 2069009008
Zoom to

Appendix C:

Existing Peak Runoff Calculations

Existing Conditions Hydrology

Sub Area	1A	Total	
Area (Sf)	449519	449519.00	
Area (Acres)	10.320	10.32	
Flowpath (ft)	1313	-	
Elevation @ start of Flowpath	1362	-	
Elevation @ end of Flowpath	1078	-	
Slope	0.22	-	
% Impervious	0.158	-	
50 yr Rain Fall (in)	7.35	-	
25 yr Rain Fall (in)	6.45	-	
10 yr Rain Fall (in)	5.25	-	
5 yr Rain Fall (in)	4.29	-	
2 yr Rain Fall (in)	2.84	-	
85th precentile	1		
Soil Type	4	-	
Debris Zone	4	-	
Peak Intensity (in/hr)	3.7438	-	
Undeveloped Runoff Coefficient (Cu)-50yr	0.7668	-	
Developed Runoff Coefficient (Cd)-50 yr	0.7879	-	
Time of Concentration 50yr (min)	7	Unrouted	**WMS Routed**
HYDROCALC- Clear Peak Flow Rate (cfs) -50yr	30.44	30.44	-
WMS- Clear Peak Flow Rate **(**cfs**)** -50yr	**30.30**	**30.30**	30.30
HYDROCALC-Burned Peak Flow Rate (cfs) -50yr	33.22	33.22	-
WMS-Burned Peak Flow Rate (cfs) -50yr	**33.42**	**33.42**	33.42
24-Hr Clear Runoff Volume (cf)- 50yr	87340	87340.05	
Peak Intensity (in/hr)	3.0871	-	
Undeveloped Runoff Coefficient (Cu)-25yr	0.7385	-	
Developed Runoff Coefficient (Cd)-25yr	0.764	-	
Time of Concentration 25yr (min)*	8	-	
HYDROCALC- Clear Peak Flow Rate (cfs) -25yr	24.34	24.34	-
WMS- Clear Peak Flow Rate (cfs) -25yr	**24.24**	**24.24**	24.2
HYDROCALC-Burned Peak Flow Rate (cfs) -25yr	26.81	26.81	-
WMS-Burned Peak Flow Rate (cfs) -25yr	**27.06**	**27.06**	27.1
24-Hr Clear Runoff Volume (cf)- 25yr	73561	73560.56	
Peak Intensity (in/hr)	2.753	-	
Undeveloped Runoff Coefficient (Cu)-10 yr	0.6977	-	
Developed Runoff Coefficient (Cd)-10 yr	0.7297	-	
Time of Concentration 10yr (min)	9	-	
HYDROCALC-Clear Peak Flow Rate (cfs) -10yr	17.89	17.89	-
WMS-Clear Peak Flow Rate (cfs) -10yr	**17.86**	**17.86**	17.86
HYDROCALC-Burned Peak Flow Rate (cfs) -10yr	19.98	19.98	-
WMS-Burned Peak Flow Rate (cfs) -10yr	**20.28**	**20.28**	20.3
24-Hr Clear Runoff Volume (cf)- 10yr	56492	56492	.
Peak Intensity (in/hr)	1.7679	-	
Undeveloped Runoff Coefficient (Cu)-5yr	0.6374	-	
Developed Runoff Coefficient (Cd)-5 yr	0.6789	-	
Time of Concentration 5yr (min)	11	-	
HYDROCALC-Clear Peak Flow Rate (cfs) -5yr	12.39	12.39	-
WMS-Clear Peak Flow Rate (cfs) -5yr	**12.36**	**12.36**	12.36
HYDROCALC-Burned Peak Flow Rate (cfs) -5yr	14.14	14.14	-
WMS-Burned Peak Flow Rate (cfs) -5yr	**14.40**	**14.40**	14.4
24-Hr Clear Runoff Volume (cf)- 5yr	44043	44043.42	
Peak Intensity (in/hr)	0.9548	-	
Undeveloped Runoff Coefficient (Cu)-2yr	0.4773	-	
Developed Runoff Coefficient (Cd)-2 yr	0.5441	-	
Time of Concentration 2yr (min)	17	-	
HYDROCALC-Clear Peak Flow Rate (cfs) -2yr	5.36	5.36	-
WMS-Clear Peak Flow Rate (cfs) -2yr	**5.34**	**5.34**	5.3
HYDROCALC-Burned Peak Flow Rate (cfs) -2yr	6.55	6.55	-
WMS-Burned Peak Flow Rate (cfs) -2yr	**6.76**	**6.76**	6.8
24-Hr Clear Runoff Volume (cf)- 2yr	26965	26965	

Non - Debris producing area	
	1A
Existing Developmed Area (sf)	71110
Landscaping / Maintained Area (sf)	
Total (sf)	71110
Total (Ac)	**1.63**

LA County HydroCalc

Time of Concentration Results

Peak Flow Hydrologic Analysis

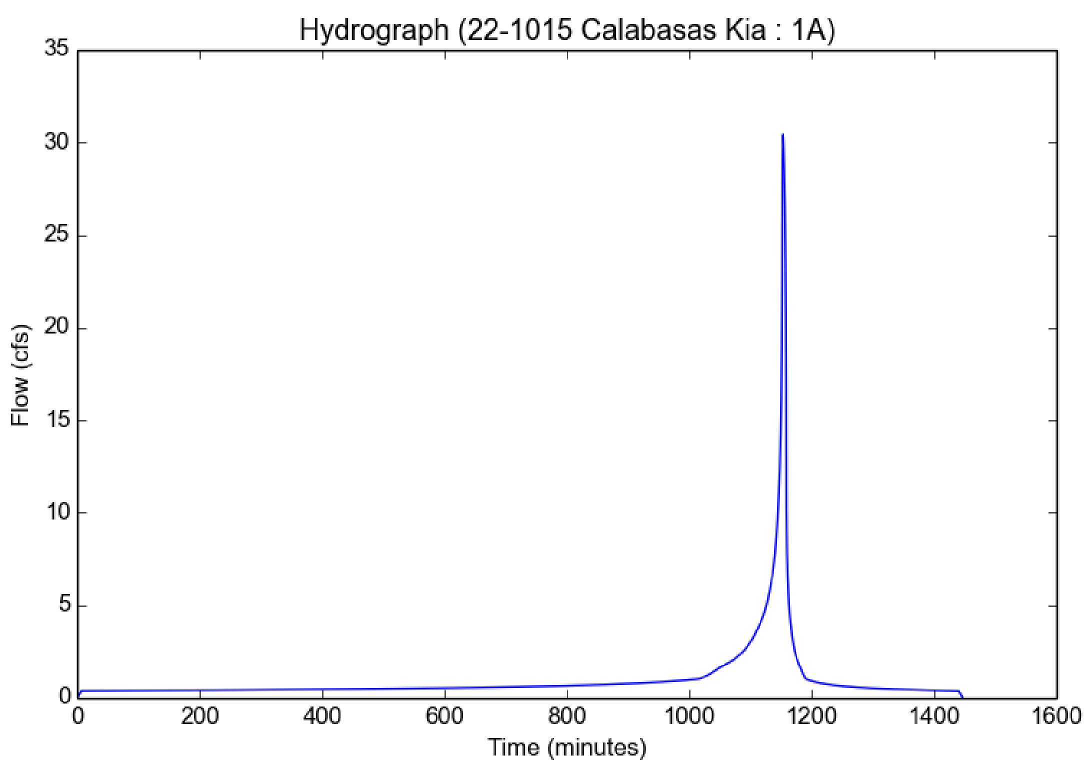
File location: P:/22-1015 - Hello Auto Group re Calabasas Kia/200-H&H/PDF/HydroCalc Results/2023-02-28 Existing All Storms/22-1015 Calabasas Kia
Version: HydroCalc 1.0.3

Input Parameters

Project Name	22-1015 Calabasas Kia
Subarea ID	1A
Area (ac)	10.32
Flow Path Length (ft)	1313.0
Flow Path Slope (vft/hft)	0.22
50-yr Rainfall Depth (in)	7.35
Percent Impervious	0.158
Soil Type	4
Design Storm Frequency	50-yr
Fire Factor	0.83
LID	False

Output Results

Modeled (50-yr) Rainfall Depth (in)	7.35
Peak Intensity (in/hr)	3.7438
Undeveloped Runoff Coefficient (Cu)	0.7668
Developed Runoff Coefficient (Cd)	0.7879
Time of Concentration (min)	[illegible]
Clear Peak Flow Rate (cfs)	30.44
Burned Peak Flow Rate (cfs)	33.2174
24-Hr Clear Runoff Volume (ac-ft)	2.0051
24-Hr Clear Runoff Volume (cu-ft)	87340.4638



Peak Flow Hydrologic Analysis

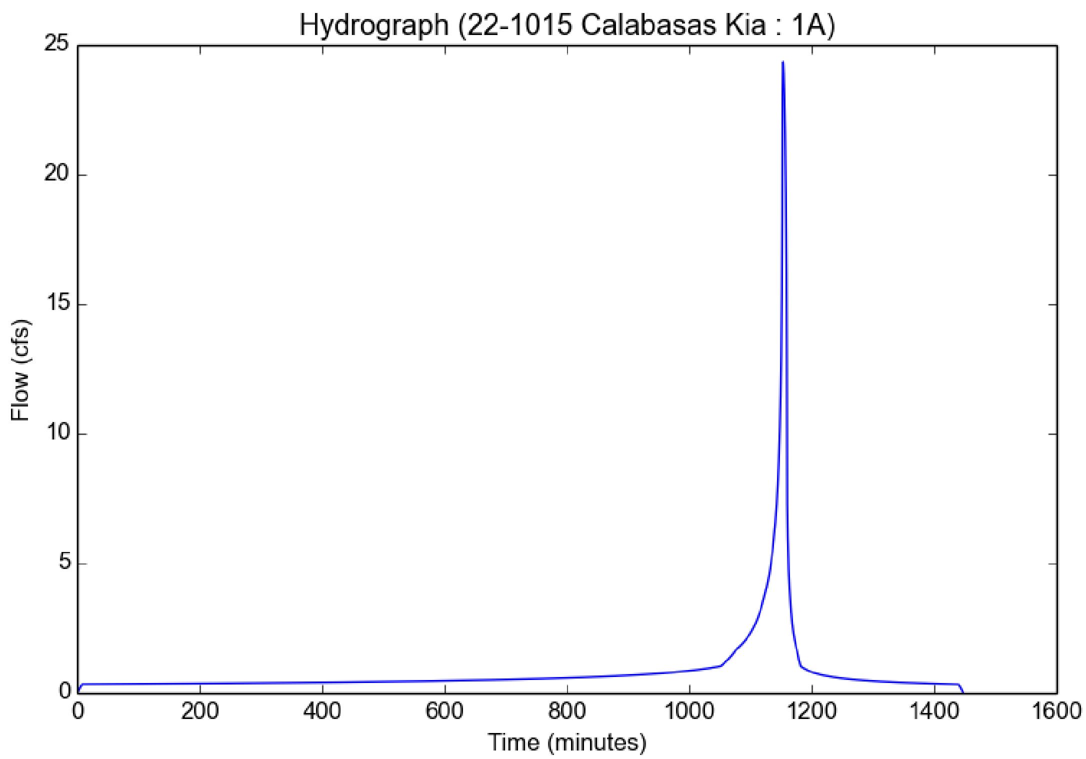
File location: P:/22-1015 - Hello Auto Group re Calabasas Kia/200-H&H/PDF/HydroCalc Results/2023-02-28 Existing All Storms/22-1015 Calabasas Kia
Version: HydroCalc 1.0.3

Input Parameters

Project Name	22-1015 Calabasas Kia
Subarea ID	1A
Area (ac)	10.32
Flow Path Length (ft)	1313.0
Flow Path Slope (vft/hft)	0.22
50-yr Rainfall Depth (in)	7.35
Percent Impervious	0.158
Soil Type	4
Design Storm Frequency	25-yr
Fire Factor	0.83
LID	False

Output Results

Modeled (25-yr) Rainfall Depth (in)	6.4533
Peak Intensity (in/hr)	3.0871
Undeveloped Runoff Coefficient (Cu)	0.7385
Developed Runoff Coefficient (Cd)	0.764
Time of Concentration (min)	[illegible]
Clear Peak Flow Rate (cfs)	24.3406
Burned Peak Flow Rate (cfs)	26.8149
24-Hr Clear Runoff Volume (ac-ft)	1.6887
24-Hr Clear Runoff Volume (cu-ft)	73560.564



Peak Flow Hydrologic Analysis

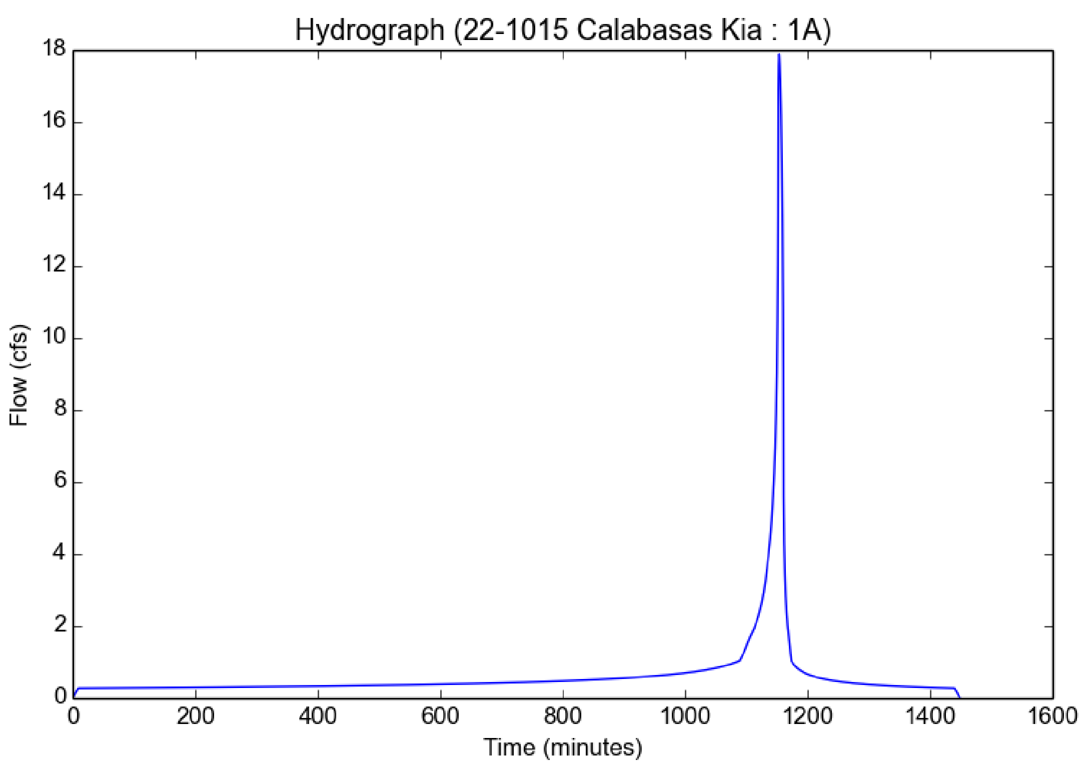
File location: P:/22-1015 - Hello Auto Group re Calabasas Kia/200-H&H/PDF/HydroCalc Results/2023-02-28 Existing All Storms/22-1015 Calabasas Kia
Version: HydroCalc 1.0.3

Input Parameters

Project Name	22-1015 Calabasas Kia
Subarea ID	1A
Area (ac)	10.32
Flow Path Length (ft)	1313.0
Flow Path Slope (vft/hft)	0.22
50-yr Rainfall Depth (in)	7.35
Percent Impervious	0.158
Soil Type	4
Design Storm Frequency	10-yr
Fire Factor	0.83
LID	False

Output Results

Modeled (10-yr) Rainfall Depth (in)	5.2479
Peak Intensity (in/hr)	2.3753
Undeveloped Runoff Coefficient (Cu)	0.6977
Developed Runoff Coefficient (Cd)	0.7297
Time of Concentration (min)	[illegible]
Clear Peak Flow Rate (cfs)	17.8862
Burned Peak Flow Rate (cfs)	19.9772
24-Hr Clear Runoff Volume (ac-ft)	1.2969
24-Hr Clear Runoff Volume (cu-ft)	56492.2844



Peak Flow Hydrologic Analysis

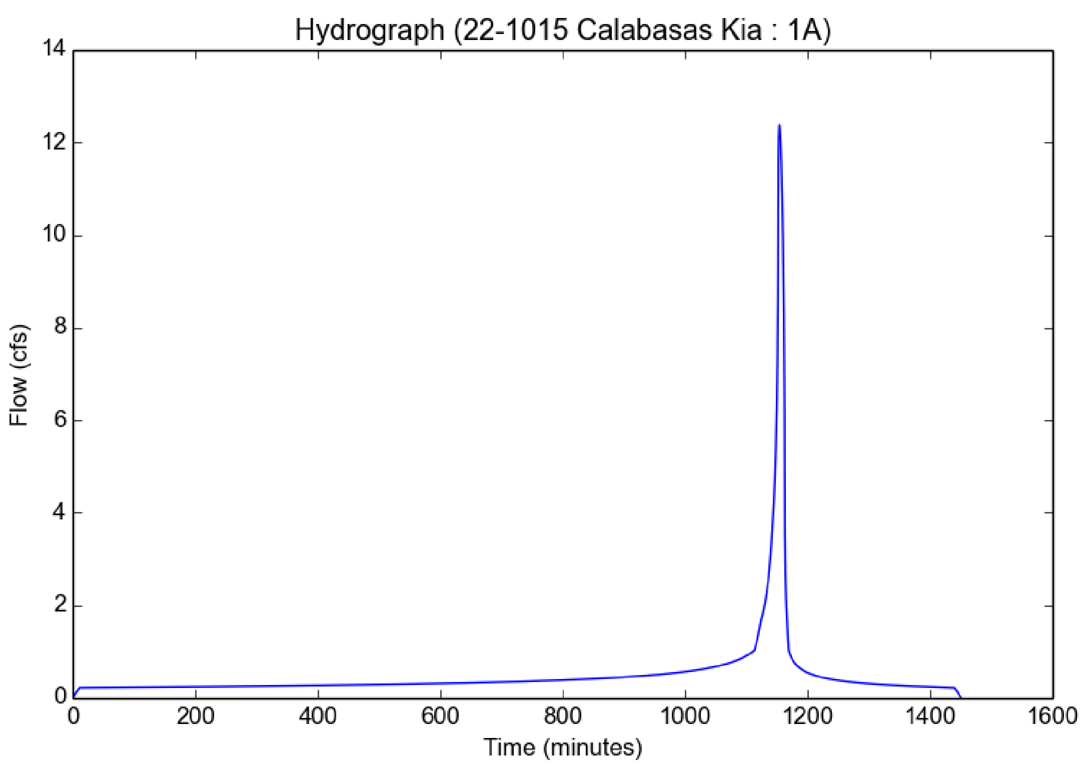
File location: P:/22-1015 - Hello Auto Group re Calabasas Kia/200-H&H/PDF/HydroCalc Results/2023-02-28 Existing All Storms/22-1015 Calabasas Kia
Version: HydroCalc 1.0.3

Input Parameters

Project Name	22-1015 Calabasas Kia
Subarea ID	1A
Area (ac)	10.32
Flow Path Length (ft)	1313.0
Flow Path Slope (vft/hft)	0.22
50-yr Rainfall Depth (in)	7.35
Percent Impervious	0.158
Soil Type	4
Design Storm Frequency	5-yr
Fire Factor	0.83
LID	False

Output Results

Modeled (5-yr) Rainfall Depth (in)	4.2924
Peak Intensity (in/hr)	1.7679
Undeveloped Runoff Coefficient (Cu)	0.6374
Developed Runoff Coefficient (Cd)	0.6789
Time of Concentration (min)	[illegible]
Clear Peak Flow Rate (cfs)	12.386
Burned Peak Flow Rate (cfs)	14.1426
24-Hr Clear Runoff Volume (ac-ft)	1.0111
24-Hr Clear Runoff Volume (cu-ft)	44043.4192



Peak Flow Hydrologic Analysis

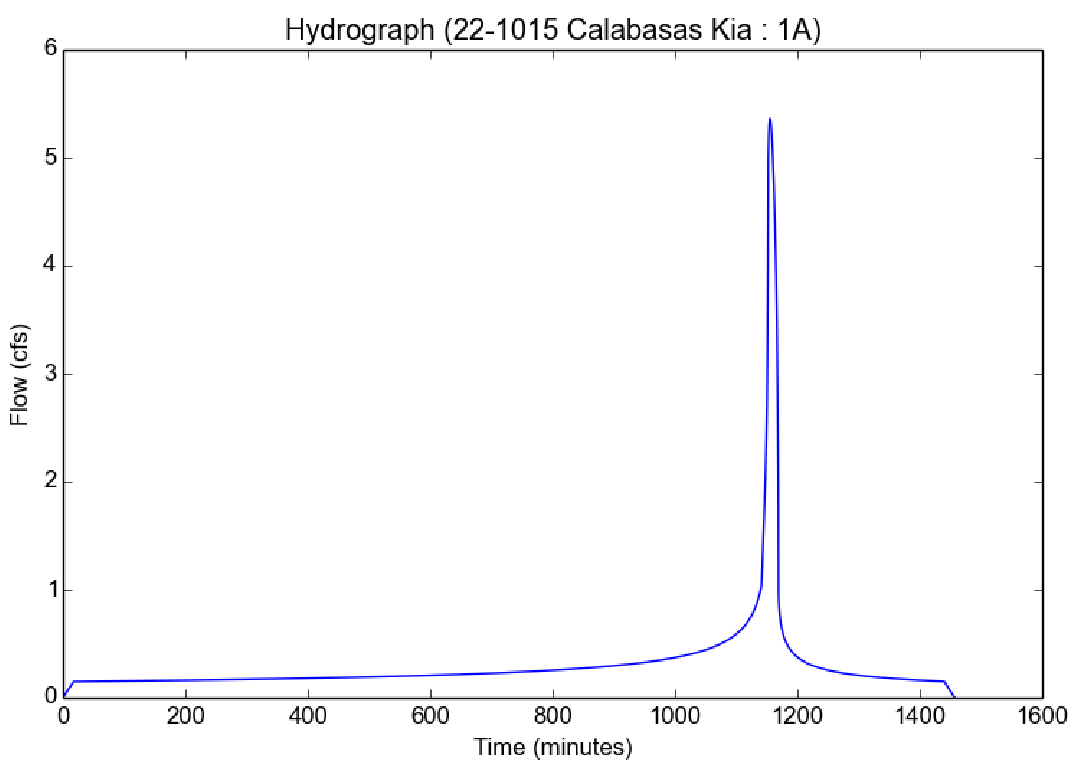
File location: P:/22-1015 - Hello Auto Group re Calabasas Kia/200-H&H/PDF/HydroCalc Results/2023-02-28 Existing All Storms/22-1015 Calabasas Kia
Version: HydroCalc 1.0.3

Input Parameters

Project Name	22-1015 Calabasas Kia
Subarea ID	1A
Area (ac)	10.32
Flow Path Length (ft)	1313.0
Flow Path Slope (vft/hft)	0.22
50-yr Rainfall Depth (in)	7.35
Percent Impervious	0.158
Soil Type	4
Design Storm Frequency	2-yr
Fire Factor	0.83
LID	False

Output Results

Modeled (2-yr) Rainfall Depth (in)	2.8445
Peak Intensity (in/hr)	0.9548
Undeveloped Runoff Coefficient (Cu)	0.4773
Developed Runoff Coefficient (Cd)	0.5441
Time of Concentration (min)	[illegible]
Clear Peak Flow Rate (cfs)	5.3613
Burned Peak Flow Rate (cfs)	6.5537
24-Hr Clear Runoff Volume (ac-ft)	0.619
24-Hr Clear Runoff Volume (cu-ft)	26964.8464



Watershed Modeling System

(WMS v11.0)

Routed Results

Existing 50 yr

File name: untitled.lac Run date: Fri Mar 10 08:01:06 2023

Los Angeles County Flood Control District
Modified Rational Method Hydrology

Storm Day 1 Storm Frequency 50

LOCATION		SUBAREA AREA (ACRES)	SUBAREA Q (CFS)	TOTAL AREA (ACRES)	TOTAL Q (CFS)	TOTAL VOLUME (AC-FT)	CONV TYPE	CONV LNGTH (FT)	CONV SLOPE (FT/FT)	CONV SIZE	CONV Z	CONTROL Q (CFS)	SOIL NAME	TC (MIN)	RAIN (IN)	PCT IMPV
1	1A	10.3	30.30	10.3	30.30	2.006	0	0	0.00000	0.00	0.00	0	4	7	7.35	0.16
1	2A	0.0	0.00	10.3	30.30	2.006	0	0	0.00000	0.00	0.00	0	4	0	7.35	0.00

Normal End of MODRAT

Existing 50 yr Burned

File name: untitled.lac Run date: Fri Mar 10 08:03:51 2023

Los Angeles County Flood Control District
Modified Rational Method Hydrology

Storm Day 1 Storm Frequency 50

LOCATION		SUBAREA AREA (ACRES)	SUBAREA Q (CFS)	TOTAL AREA (ACRES)	TOTAL Q (CFS)	TOTAL VOLUME (AC-FT)	CONV TYPE	CONV LNGTH (FT)	CONV SLOPE (FT/FT)	CONV SIZE	CONV Z	CONTROL Q (CFS)	SOIL NAME	TC (MIN)	RAIN (IN)	PCT IMPV
1	1A	10.3	33.42	10.3	33.42	2.555	0	0	0.00000	0.00	0.00	0	204	7	7.35	0.16
1	2A	0.0	0.00	10.3	33.42	2.555	0	0	0.00000	0.00	0.00	0	204	0	7.35	0.00

Normal End of MODRAT

Existing 25 yr

File name: untitled.lac Run date: Fri Mar 10 08:05:57 2023

Los Angeles County Flood Control District
Modified Rational Method Hydrology

Storm Day 1 Storm Frequency 25

LOCATION		SUBAREA AREA (ACRES)	SUBAREA Q (CFS)	TOTAL AREA (ACRES)	TOTAL Q (CFS)	TOTAL VOLUME (AC-FT)	CONV TYPE	CONV LNGTH (FT)	CONV SLOPE (FT/FT)	CONV SIZE	CONV Z	CONTROL Q (CFS)	SOIL NAME	TC (MIN)	RAIN (IN)	PCT IMPV
1	1A	10.3	24.24	10.3	24.24	1.689	0	0	0.00000	0.00	0.00	0	4	8	6.45	0.16
1	2A	0.0	0.00	10.3	24.24	1.689	0	0	0.00000	0.00	0.00	0	4	0	6.45	0.00

Normal End of MODRAT

Existing 25 yr Burned

File name: untitled.lac   Run date: Fri Mar 10 08:07:41 2023

Los Angeles County Flood Control District
Modified Rational Method Hydrology

Storm Day 1   Storm Frequency 25

LOCATION		SUBAREA AREA (ACRES)	SUBAREA Q (CFS)	TOTAL AREA (ACRES)	TOTAL Q (CFS)	TOTAL VOLUME (AC-FT)	CONV TYPE	CONV LNGTH (FT)	CONV SLOPE (FT/FT)	CONV SIZE	CONV Z	CONTROL Q (CFS)	SOIL NAME	TC (MIN)	RAIN (IN)	PCT IMPV
1	1A	10.3	27.06	10.3	27.06	2.135	0	0	0.00000	0.00	0.00	0	204	8	6.45	0.16
1	2A	0.0	0.00	10.3	27.06	2.135	0	0	0.00000	0.00	0.00	0	204	0	6.45	0.00

Normal End of MODRAT

Existing 10 yr

File name: untitled.lac   Run date: Fri Mar 10 08:10:49 2023

Los Angeles County Flood Control District
Modified Rational Method Hydrology

Storm Day 1   Storm Frequency 10

LOCATION		SUBAREA AREA (ACRES)	SUBAREA Q (CFS)	TOTAL AREA (ACRES)	TOTAL Q (CFS)	TOTAL VOLUME (AC-FT)	CONV TYPE	CONV LNGTH (FT)	CONV SLOPE (FT/FT)	CONV SIZE	CONV Z	CONTROL Q (CFS)	SOIL NAME	TC (MIN)	RAIN (IN)	PCT IMPV
1	1A	10.3	17.86	10.3	17.86	1.298	0	0	0.00000	0.00	0.00	0	4	9	5.25	0.16
1	2A	0.0	0.00	10.3	17.86	1.298	0	0	0.00000	0.00	0.00	0	4	0	5.25	0.00

Normal End of MODRAT

Existing 10 yr Burned

File name: untitled.lac Run date: Fri Mar 10 08:28:47 2023

Los Angeles County Flood Control District
Modified Rational Method Hydrology

Storm Day 1 Storm Frequency 10

LOCATION		SUBAREA AREA (ACRES)	SUBAREA Q (CFS)	TOTAL AREA (ACRES)	TOTAL Q (CFS)	TOTAL VOLUME (AC-FT)	CONV TYPE	CONV LNGTH (FT)	CONV SLOPE (FT/FT)	CONV SIZE	CONV Z	CONTROL Q (CFS)	SOIL NAME	TC (MIN)	RAIN (IN)	PCT IMPV
1	1A	10.3	20.28	10.3	20.28	1.619	0	0	0.00000	0.00	0.00	0	204	9	5.25	0.16
1	2A	0.0	0.00	10.3	20.28	1.619	0	0	0.00000	0.00	0.00	0	204	0	5.25	0.00

Normal End of MODRAT

Existing 5 yr

File name: untitled.lac   Run date: Fri Mar 10 08:41:14 2023

Los Angeles County Flood Control District
Modified Rational Method Hydrology

Storm Day 1   Storm Frequency 5

LOCATION		SUBAREA AREA (ACRES)	SUBAREA Q (CFS)	TOTAL AREA (ACRES)	TOTAL Q (CFS)	TOTAL VOLUME (AC-FT)	CONV TYPE	CONV LNGTH (FT)	CONV SLOPE (FT/FT)	CONV SIZE	CONV Z	CONTROL Q (CFS)	SOIL NAME	TC (MIN)	RAIN (IN)	PCT IMPV
1	1A	10.3	12.36	10.3	12.36	1.011	0	0	0.00000	0.00	0.00	0	4	11	4.29	0.16
1	2A	0.0	0.00	10.3	12.36	1.011	0	0	0.00000	0.00	0.00	0	4	0	4.29	0.00

Normal End of MODRAT

Existing 5 yr Burned

File name: untitled.lac Run date: Fri Mar 10 08:47:15 2023

Los Angeles County Flood Control District
Modified Rational Method Hydrology

Storm Day 1 Storm Frequency 5

LOCATION		SUBAREA AREA (ACRES)	SUBAREA Q (CFS)	TOTAL AREA (ACRES)	TOTAL Q (CFS)	TOTAL VOLUME (AC-FT)	CONV TYPE	CONV LNGTH (FT)	CONV SLOPE (FT/FT)	CONV SIZE	CONV Z	CONTROL Q (CFS)	SOIL NAME	TC (MIN)	RAIN (IN)	PCT IMPV
1	1A	10.3	14.40	10.3	14.40	1.242	0	0	0.00000	0.00	0.00	0	204	11	4.29	0.16
1	2A	0.0	0.00	10.3	14.40	1.242	0	0	0.00000	0.00	0.00	0	204	0	4.29	0.00

Normal End of MODRAT

Existing 2 yr

File name: untitled.lac   Run date: Fri Mar 10 08:50:15 2023

Los Angeles County Flood Control District
Modified Rational Method Hydrology

Storm Day 1   Storm Frequency 5

LOCATION		SUBAREA AREA (ACRES)	SUBAREA Q (CFS)	TOTAL AREA (ACRES)	TOTAL Q (CFS)	TOTAL VOLUME (AC-FT)	CONV TYPE	CONV LNGTH (FT)	CONV SLOPE (FT/FT)	CONV SIZE	CONV Z	CONTROL Q (CFS)	SOIL NAME	TC (MIN)	RAIN (IN)	PCT IMPV
1	1A	10.3	5.34	10.3	5.34	0.617	0	0	0.00000	0.00	0.00	0	4	17	2.84	0.16
1	2A	0.0	0.00	10.3	5.34	0.617	0	0	0.00000	0.00	0.00	0	4	0	2.84	0.00

Normal End of MODRAT

Existing 2 yr Burned

File name: untitled.lac Run date: Fri Mar 10 08:51:02 2023

Los Angeles County Flood Control District
Modified Rational Method Hydrology

Storm Day 1 Storm Frequency 2

LOCATION		SUBAREA AREA (ACRES)	SUBAREA Q (CFS)	TOTAL AREA (ACRES)	TOTAL Q (CFS)	TOTAL VOLUME (AC-FT)	CONV TYPE	CONV LNGTH (FT)	CONV SLOPE (FT/FT)	CONV SIZE	CONV Z	CONTROL Q (CFS)	SOIL NAME	TC (MIN)	RAIN (IN)	PCT IMPV
1	1A	10.3	6.76	10.3	6.76	0.736	0	0	0.00000	0.00	0.00	0	204	17	2.84	0.16
1	2A	0.0	0.00	10.3	6.76	0.736	0	0	0.00000	0.00	0.00	0	204	0	2.84	0.00

Normal End of MODRAT

Appendix D:

Proposed Peak Runoff Calculations

Hydrology Proposed Conditions

Sub Area	1A	3A**	Total			
Area (Sf)	294158	155332	449,490.6504			
Area (Acres)	6.75	3.57	10.32			
Flowpath (ft)	797	996	-			
Elevation @ start of Flowpath	1362	1210	-			
Elevation @ end of Flowpath	1130	1075	-			
Slope	0.29	0.14	-			
% Impervious	0.011	0.496	-			
50 yr Rain Fall (in)	7.35	7.35	-			
25 yr Rain Fall (in)	6.4533	6.4533	-			
10 yr Rain Fall (in)	5.2479	5.2479	-			
5 yr Rain Fall (in)	4.2924	4.2924	-			
2 yr Rain Fall (in)	2.8445	2.8445	-			
85th precentile	1	1				
Soil Type	4	4				
Debris Zone	4	4	-			
Peak Intensity (in/hr)	4.3852	4.0251	-			WMS Comparison Pre Vs Post
Undeveloped Runoff Coefficient (Cu)-50yr	0.7851	0.7748	-			
Developed Runoff Coefficient (Cd)-50 yr	0.7863	0.8369	**Unrouted Totals**	**WMS Routed Totals w/o Detention Total**	**WMS Routed w/ Detention Totals**	**Pre 1A vs. Post SubArea 1A + 3A**
Time of Concentration 50yr (min)*	5	6				
HYDROCALC-Clear Peak Flow Rate (cfs) -50yr	23.28	12.03	**35.3**	-	-	-
WMS-Clear Peak Flow Rate (cfs) -50yr	**23.45**	**12.12**	**35.6**	**34.43**	**26.93**	-3.37
HYDROCALC-Burned Peak Flow Rate (cfs) -50yr	25.47	12.83	**38.3**	-	-	-
WMS-Burned Peak Flow Rate (cfs) -50yr	**26.13**	**12.12**	**38.3**	**37.11**	**29.26**	-4.16
HYDROCALC-24-Hr Clear Runoff Volume (cf)- 50yr	39058	52215	**91273.2**	-		
Peak Intensity (in/hr)	3.8502	3.287	-			
Undeveloped Runoff Coefficient (Cu)-25yr	0.7699	0.7489	-			
Developed Runoff Coefficient (Cd)-25yr	0.7713	0.8238	-			
Time of Concentration 25yr (min)*	5	7	-			
HYDROCALC -Clear Peak Flow Rate (cfs) -25yr	20.04	9.67	**29.7**	-	-	-
WMS-Clear Peak Flow Rate (cfs) -25yr	**20.18**	**9.72**	**29.9**	**28.91**	**22.9**	-1.34
HYDROCALC-Burned Peak Flow Rate (cfs) -25yr	22.07	10.35	**32.4**	-	-	-
WMS-Burned Peak Flow Rate (cfs) -25yr	**22.66**	**9.72**	**32.4**	**31.41**	**24.9**	-2.14
24-Hr Clear Runoff Volume (cf)- 25yr	31928	45201	**31928.3**	-		
Peak Intensity (in/hr)	2.8739	2.5105	-			
Undeveloped Runoff Coefficient (Cu)-10 yr	0.7274	0.7086	-			
Developed Runoff Coefficient (Cd)-10 yr	0.7293	0.8035	-			
Time of Concentration 10yr (min)*	6	8	-			
HYDROCALC-Clear Peak Flow Rate (cfs) -10yr	14.15	7.20	**21.4**	-	-	-
WMS-Clear Peak Flow Rate (cfs) -10yr	**14.24**	**7.26**	**21.5**	**20.95**	**16.94**	-0.92
HYDROCALC-Burned Peak Flow Rate (cfs) -10yr	15.86	7.76	**23.6**	-	-	-
WMS-Burned Peak Flow Rate (cfs) -10yr	**16.35**	**7.26**	**23.6**	**23.020**	**18.82**	-1.46
24-Hr Clear Runoff Volume (cf)- 10yr	23404	36070	**59474.1**	-		
Peak Intensity (in/hr)	2.0534	1.9428	-			
Undeveloped Runoff Coefficient (Cu)-5yr	0.671	0.6594	-			
Developed Runoff Coefficient (Cd)-5 yr	0.6962	0.7787	-			
Time of Concentration 5yr (min)*	8	9	-			
HYDROCALC-Clear Peak Flow Rate (cfs) -5yr	9.65	5.40	**15.1**	-	-	-
WMS-Clear Peak Flow Rate (cfs) -5yr	**9.38**	**5.45**	**14.8**	**14.50**	**12.07**	-0.29
HYDROCALC-Burned Peak Flow Rate (cfs) -5yr	10.95	5.87	**16.8**	-	-	-
WMS -Burned Peak Flow Rate (cfs) -5yr	**11.10**	**5.45**	**16.6**	**16.2**	**13.61**	-0.79
24-Hr Clear Runoff Volume (cf)- 5yr	25149	29060	**54209.1**	-		
Peak Intensity (in/hr)	1.1246	1.0831	-			
Undeveloped Runoff Coefficient (Cu)-2yr	0.5218	0.5128	-			
Developed Runoff Coefficient (Cd)-2 yr	0.5634	0.7048	-			
Time of Concentration 2yr (min)	12	13	-			
HYDROCALC-Clear Peak Flow Rate (cfs) -2yr	4.28	2.73	**7.00**	-	-	-
WMS-Clear Peak Flow Rate (cfs) -2yr	**4.01**	**2.75**	**6.76**	**6.57**	**5.7**	0.36
HYDROCALC-Burned Peak Flow Rate (cfs) -2yr	5.19	3.04	**8.22**	-	-	-
WMS-Burned Peak Flow Rate (cfs) -2yr	**5.23**	**2.75**	**7.98**	**7.77**	**6.8**	0.04
24-Hr Clear Runoff Volume (cf)- 2yr	15176	18812	**33987.9**	-		

Per HydroCalc
****Developed Area- Not Subject to Debris Production***

Impervious and Non - Debris producing area

-	1A	3A**
Existing Development Area (sf)	3141	7670
Proposed Deveopment (sf)	0	69338
Landscaped_ Fire Modification Area (sf)	11385	22472
total Impervious Area (sf)	3,141	77,008
total impervious Area (ac)	**0.07**	**1.77**
Total non debris producing area(sf)	14526	99481
Total non debris producing area(ac)	***0.33***	***2.28***

LA County HydroCalc

Time of Concentration Results

Peak Flow Hydrologic Analysis

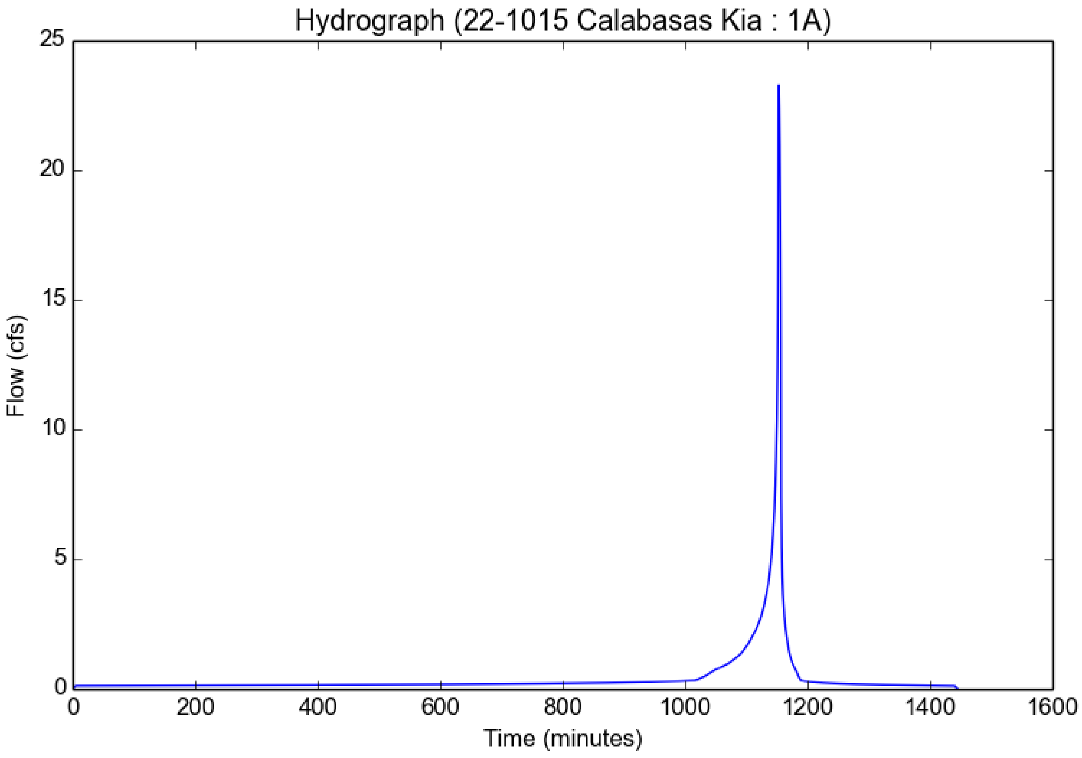
File location: P:/22-1015 - Hello Auto Group re Calabasas Kia/200-H&H/PDF/HydroCalc Results/2023-03-08 Proposed All stroms/22-1015 Calabasas Kia
Version: HydroCalc 1.0.3

Input Parameters

Project Name	22-1015 Calabasas Kia
Subarea ID	1A
Area (ac)	6.75
Flow Path Length (ft)	797.0
Flow Path Slope (vft/hft)	0.29
50-yr Rainfall Depth (in)	7.35
Percent Impervious	0.011
Soil Type	4
Design Storm Frequency	50-yr
Fire Factor	0.83
LID	False

Output Results

Modeled (50-yr) Rainfall Depth (in)	7.35
Peak Intensity (in/hr)	4.3852
Undeveloped Runoff Coefficient (Cu)	0.7851
Developed Runoff Coefficient (Cd)	0.7863
Time of Concentration (min)	5.0
Clear Peak Flow Rate (cfs)	23.2752
Burned Peak Flow Rate (cfs)	25.4683
24-Hr Clear Runoff Volume (ac-ft)	0.8966
24-Hr Clear Runoff Volume (cu-ft)	39058.0134



Peak Flow Hydrologic Analysis

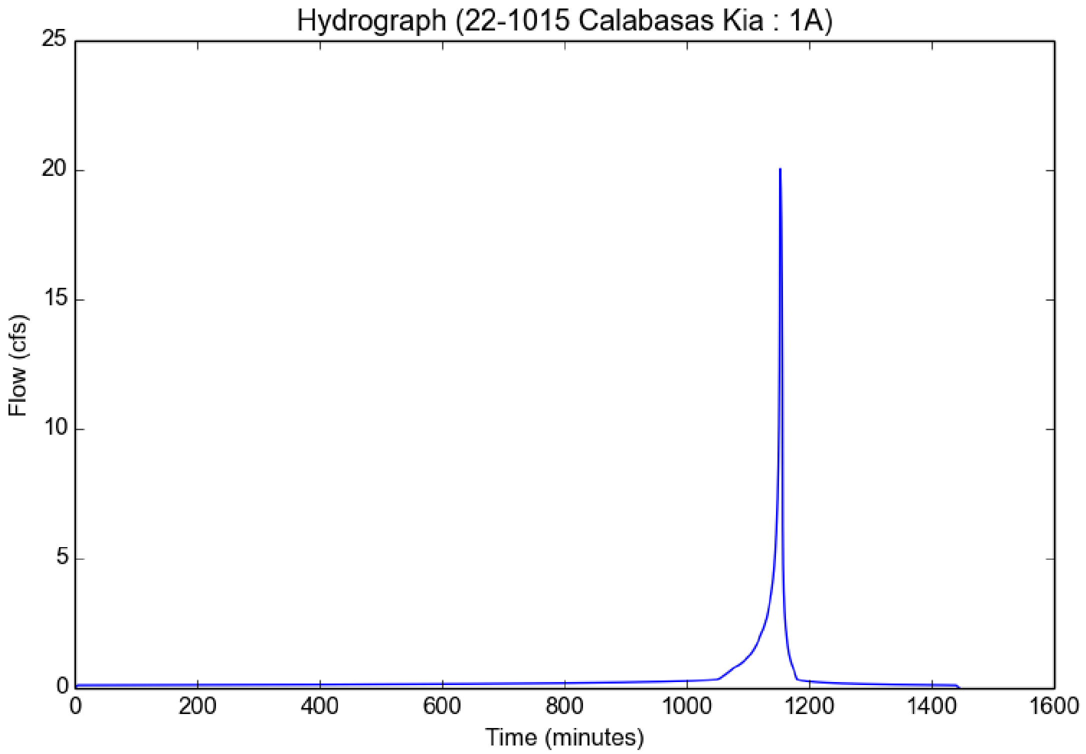
File location: P:/22-1015 - Hello Auto Group re Calabasas Kia/200-H&H/PDF/HydroCalc Results/2023-03-08 Proposed All stroms/22-1015 Calabasas Kia
Version: HydroCalc 1.0.3

Input Parameters

Project Name	22-1015 Calabasas Kia
Subarea ID	1A
Area (ac)	6.75
Flow Path Length (ft)	797.0
Flow Path Slope (vft/hft)	0.29
50-yr Rainfall Depth (in)	7.35
Percent Impervious	0.011
Soil Type	4
Design Storm Frequency	25-yr
Fire Factor	0.83
LID	False

Output Results

Modeled (25-yr) Rainfall Depth (in)	6.4533
Peak Intensity (in/hr)	3.8502
Undeveloped Runoff Coefficient (Cu)	0.7699
Developed Runoff Coefficient (Cd)	0.7713
Time of Concentration (min)	5.0
Clear Peak Flow Rate (cfs)	20.0449
Burned Peak Flow Rate (cfs)	22.0675
24-Hr Clear Runoff Volume (ac-ft)	0.733
24-Hr Clear Runoff Volume (cu-ft)	31928.2682



Peak Flow Hydrologic Analysis

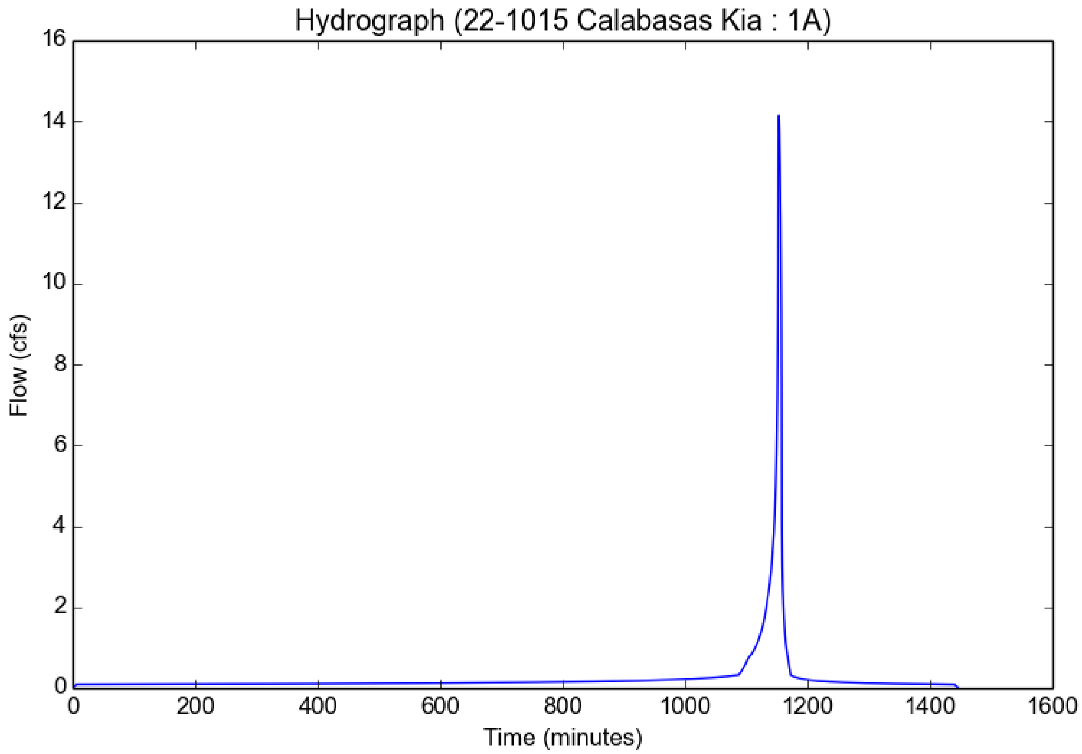
File location: P:/22-1015 - Hello Auto Group re Calabasas Kia/200-H&H/PDF/HydroCalc Results/2023-03-08 Proposed All stroms/22-1015 Calabasas Kia
Version: HydroCalc 1.0.3

Input Parameters

Project Name	22-1015 Calabasas Kia
Subarea ID	1A
Area (ac)	6.75
Flow Path Length (ft)	797.0
Flow Path Slope (vft/hft)	0.29
50-yr Rainfall Depth (in)	7.35
Percent Impervious	0.011
Soil Type	4
Design Storm Frequency	10-yr
Fire Factor	0.83
LID	False

Output Results

Modeled (10-yr) Rainfall Depth (in)	5.2479
Peak Intensity (in/hr)	2.8739
Undeveloped Runoff Coefficient (Cu)	0.7274
Developed Runoff Coefficient (Cd)	0.7293
Time of Concentration (min)	6.0
Clear Peak Flow Rate (cfs)	14.1485
Burned Peak Flow Rate (cfs)	15.8573
24-Hr Clear Runoff Volume (ac-ft)	0.5373
24-Hr Clear Runoff Volume (cu-ft)	23404.0968



Peak Flow Hydrologic Analysis

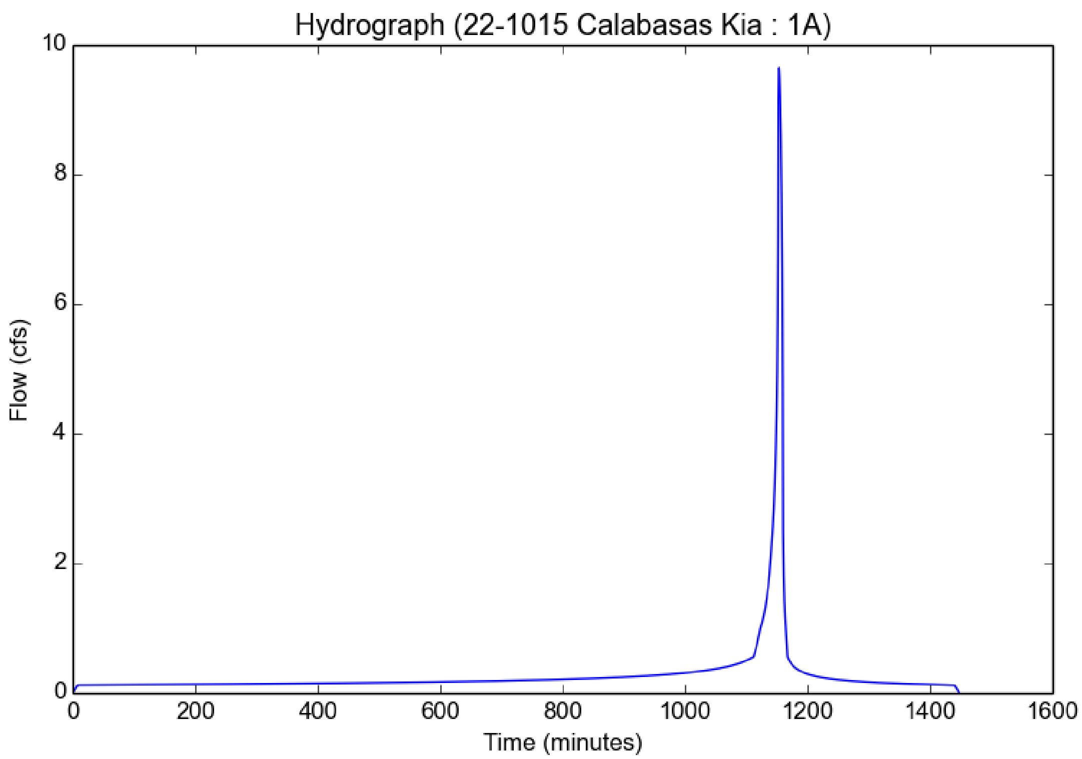
File location: P:/22-1015 - Hello Auto Group re Calabasas Kia/200-H&H/PDF/HydroCalc Results/2023-03-08 Proposed All stroms/22-1015 Calabasas Kia
Version: HydroCalc 1.0.3

Input Parameters

Project Name	22-1015 Calabasas Kia
Subarea ID	1A
Area (ac)	6.75
Flow Path Length (ft)	797.0
Flow Path Slope (vft/hft)	0.29
50-yr Rainfall Depth (in)	7.35
Percent Impervious	0.11
Soil Type	4
Design Storm Frequency	5-yr
Fire Factor	0.83
LID	False

Output Results

Modeled (5-yr) Rainfall Depth (in)	4.2924
Peak Intensity (in/hr)	2.0534
Undeveloped Runoff Coefficient (Cu)	0.671
Developed Runoff Coefficient (Cd)	0.6962
Time of Concentration (min)	8.0
Clear Peak Flow Rate (cfs)	9.6492
Burned Peak Flow Rate (cfs)	10.9456
24-Hr Clear Runoff Volume (ac-ft)	0.5773
24-Hr Clear Runoff Volume (cu-ft)	25149.0766



Peak Flow Hydrologic Analysis

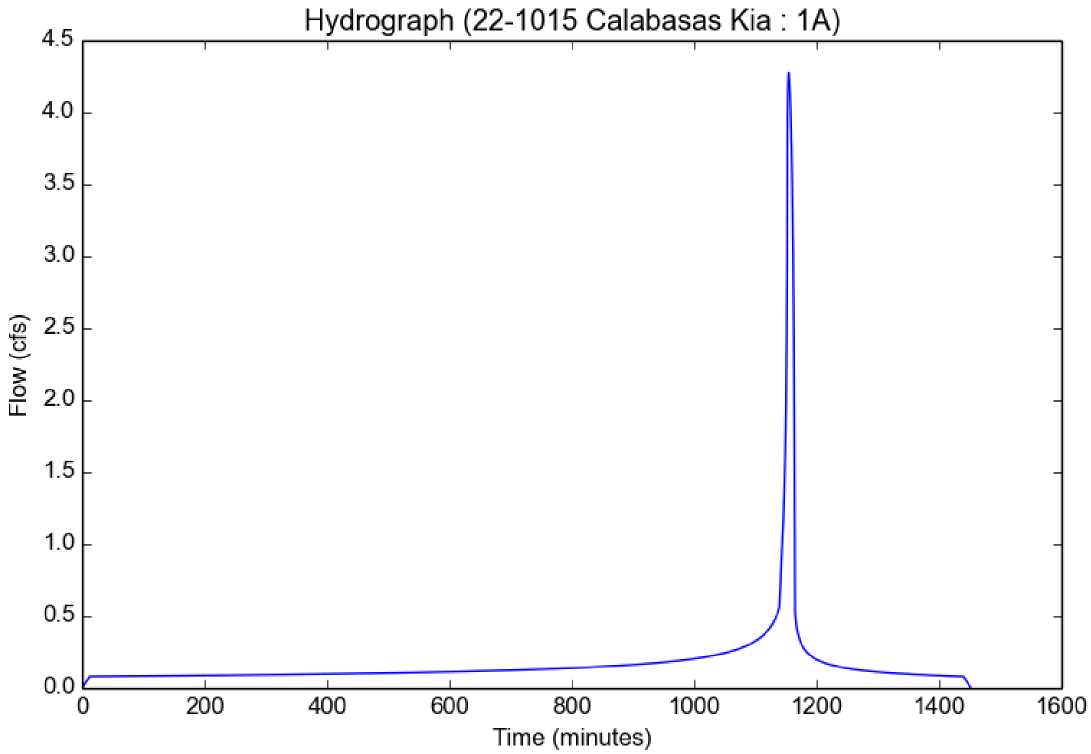
File location: P:/22-1015 - Hello Auto Group re Calabasas Kia/200-H&H/PDF/HydroCalc Results/2023-03-08 Proposed All stroms/22-1015 Calabasas Kia
Version: HydroCalc 1.0.3

Input Parameters

Project Name	22-1015 Calabasas Kia
Subarea ID	1A
Area (ac)	6.75
Flow Path Length (ft)	797.0
Flow Path Slope (vft/hft)	0.29
50-yr Rainfall Depth (in)	7.35
Percent Impervious	0.11
Soil Type	4
Design Storm Frequency	2-yr
Fire Factor	0.83
LID	False

Output Results

Modeled (2-yr) Rainfall Depth (in)	2.8445
Peak Intensity (in/hr)	1.1246
Undeveloped Runoff Coefficient (Cu)	0.5218
Developed Runoff Coefficient (Cd)	0.5634
Time of Concentration (min)	12.0
Clear Peak Flow Rate (cfs)	4.2769
Burned Peak Flow Rate (cfs)	5.1876
24-Hr Clear Runoff Volume (ac-ft)	0.3484
24-Hr Clear Runoff Volume (cu-ft)	15176.1701



Peak Flow Hydrologic Analysis

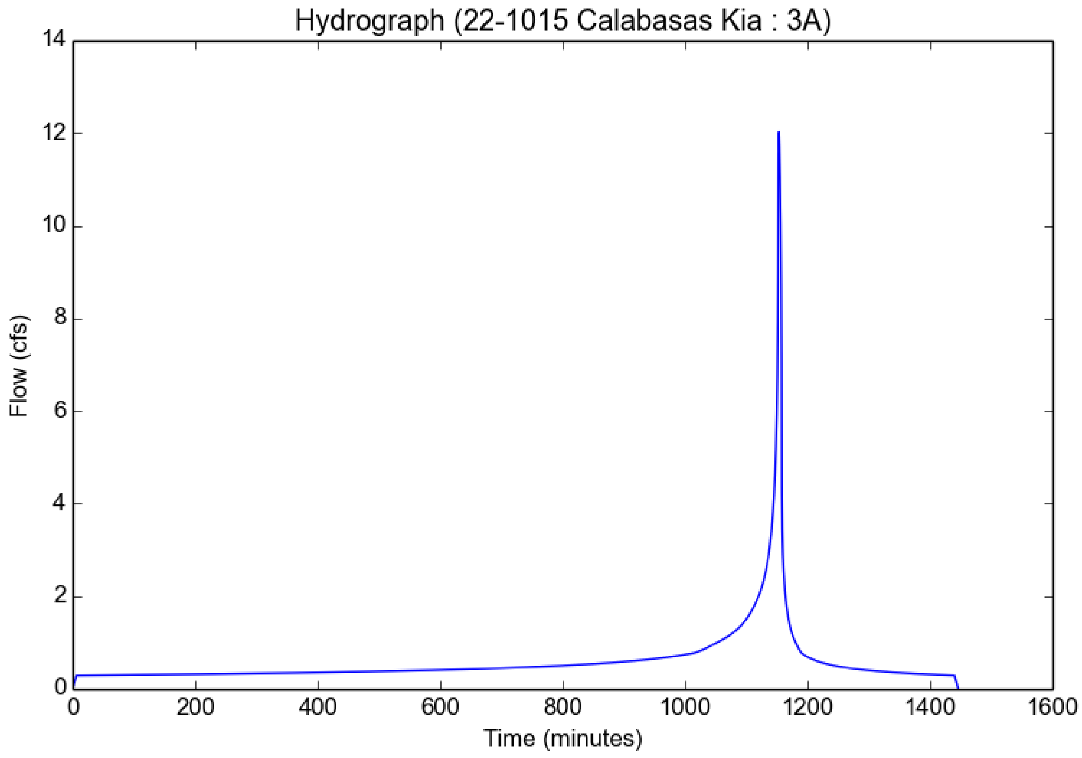
File location: P:/22-1015 - Hello Auto Group re Calabasas Kia/200-H&H/PDF/HydroCalc Results/2023-03-08 Proposed All stroms/22-1015 Calabasas Kia
Version: HydroCalc 1.0.3

Input Parameters

Project Name	22-1015 Calabasas Kia
Subarea ID	3A
Area (ac)	3.57
Flow Path Length (ft)	996.0
Flow Path Slope (vft/hft)	0.14
50-yr Rainfall Depth (in)	7.35
Percent Impervious	0.496
Soil Type	4
Design Storm Frequency	50-yr
Fire Factor	0.83
LID	False

Output Results

Modeled (50-yr) Rainfall Depth (in)	7.35
Peak Intensity (in/hr)	4.0251
Undeveloped Runoff Coefficient (Cu)	0.7748
Developed Runoff Coefficient (Cd)	0.8369
Time of Concentration (min)	6.0
Clear Peak Flow Rate (cfs)	12.026
Burned Peak Flow Rate (cfs)	12.8287
24-Hr Clear Runoff Volume (ac-ft)	1.1987
24-Hr Clear Runoff Volume (cu-ft)	52215.1809



Peak Flow Hydrologic Analysis

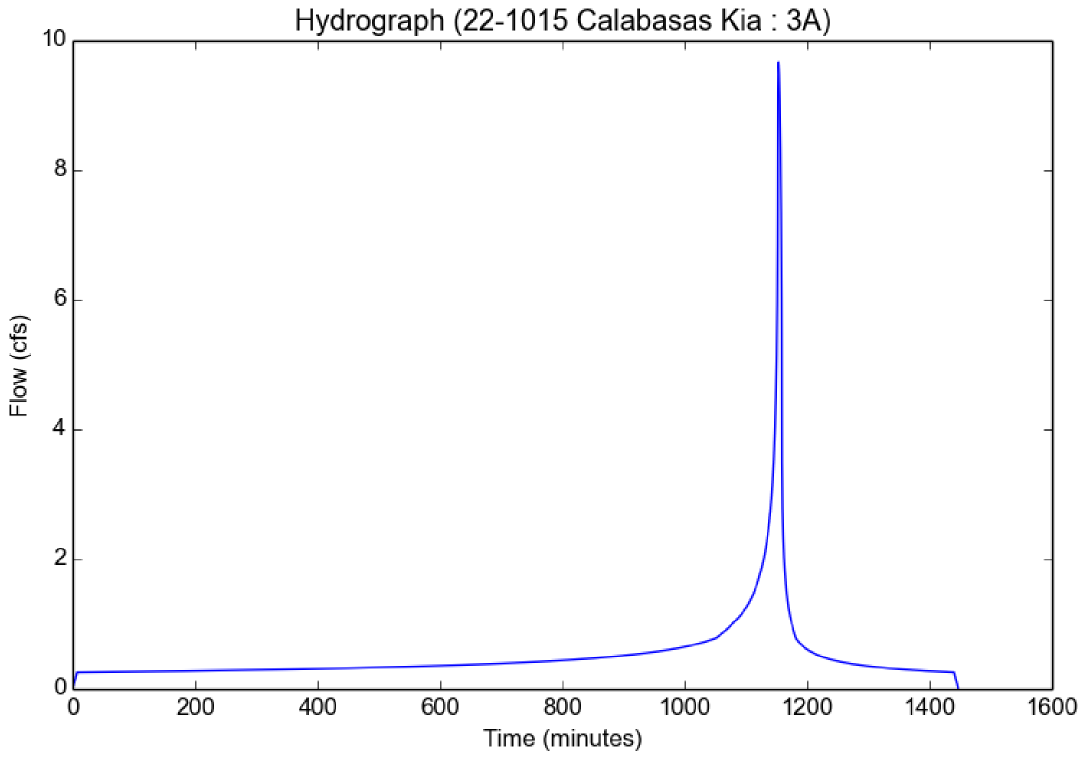
File location: P:/22-1015 - Hello Auto Group re Calabasas Kia/200-H&H/PDF/HydroCalc Results/2023-03-08 Proposed All stroms/22-1015 Calabasas Kia
Version: HydroCalc 1.0.3

Input Parameters

Project Name	22-1015 Calabasas Kia
Subarea ID	3A
Area (ac)	3.57
Flow Path Length (ft)	996.0
Flow Path Slope (vft/hft)	0.14
50-yr Rainfall Depth (in)	7.35
Percent Impervious	0.496
Soil Type	4
Design Storm Frequency	25-yr
Fire Factor	0.83
LID	False

Output Results

Modeled (25-yr) Rainfall Depth (in)	6.4533
Peak Intensity (in/hr)	3.287
Undeveloped Runoff Coefficient (Cu)	0.7489
Developed Runoff Coefficient (Cd)	0.8238
Time of Concentration (min)	7.0
Clear Peak Flow Rate (cfs)	9.6674
Burned Peak Flow Rate (cfs)	10.3544
24-Hr Clear Runoff Volume (ac-ft)	1.0377
24-Hr Clear Runoff Volume (cu-ft)	45200.8473



Peak Flow Hydrologic Analysis

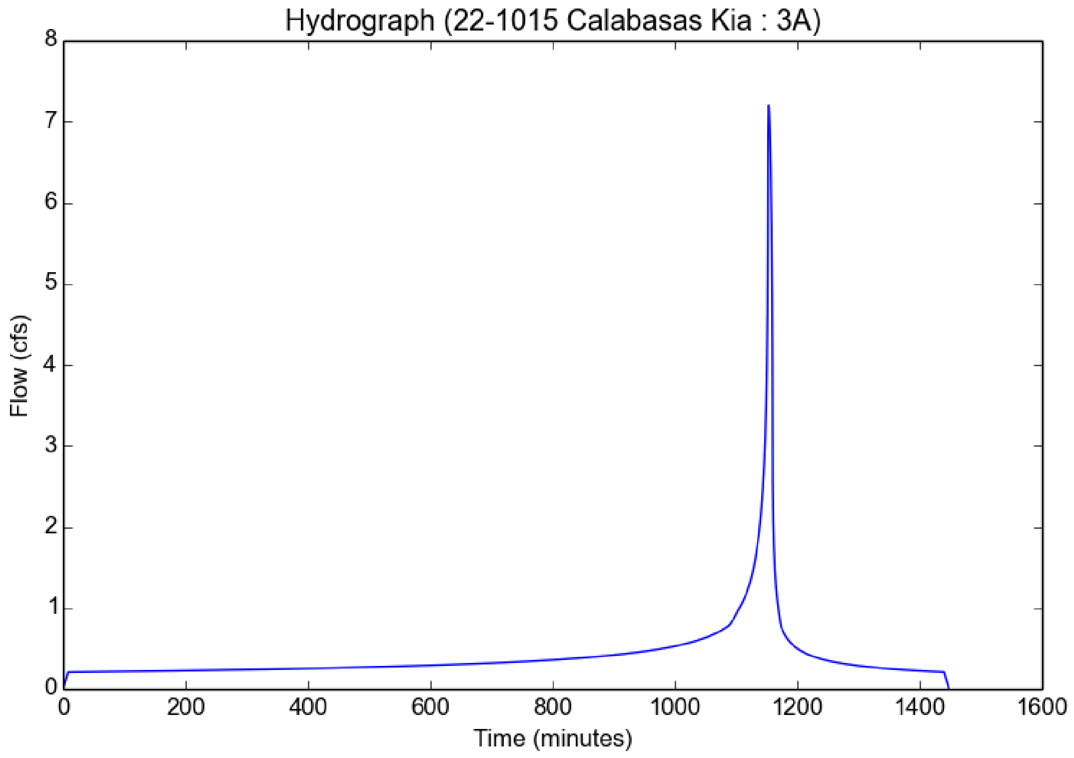
File location: P:/22-1015 - Hello Auto Group re Calabasas Kia/200-H&H/PDF/HydroCalc Results/2023-03-08 Proposed All stroms/22-1015 Calabasas Kia
Version: HydroCalc 1.0.3

Input Parameters

Project Name	22-1015 Calabasas Kia
Subarea ID	3A
Area (ac)	3.57
Flow Path Length (ft)	996.0
Flow Path Slope (vft/hft)	0.14
50-yr Rainfall Depth (in)	7.35
Percent Impervious	0.496
Soil Type	4
Design Storm Frequency	10-yr
Fire Factor	0.83
LID	False

Output Results

Modeled (10-yr) Rainfall Depth (in)	5.2479
Peak Intensity (in/hr)	2.5105
Undeveloped Runoff Coefficient (Cu)	0.7086
Developed Runoff Coefficient (Cd)	0.8035
Time of Concentration (min)	8.0
Clear Peak Flow Rate (cfs)	7.2016
Burned Peak Flow Rate (cfs)	7.7623
24-Hr Clear Runoff Volume (ac-ft)	0.8281
24-Hr Clear Runoff Volume (cu-ft)	36070.0775



Peak Flow Hydrologic Analysis

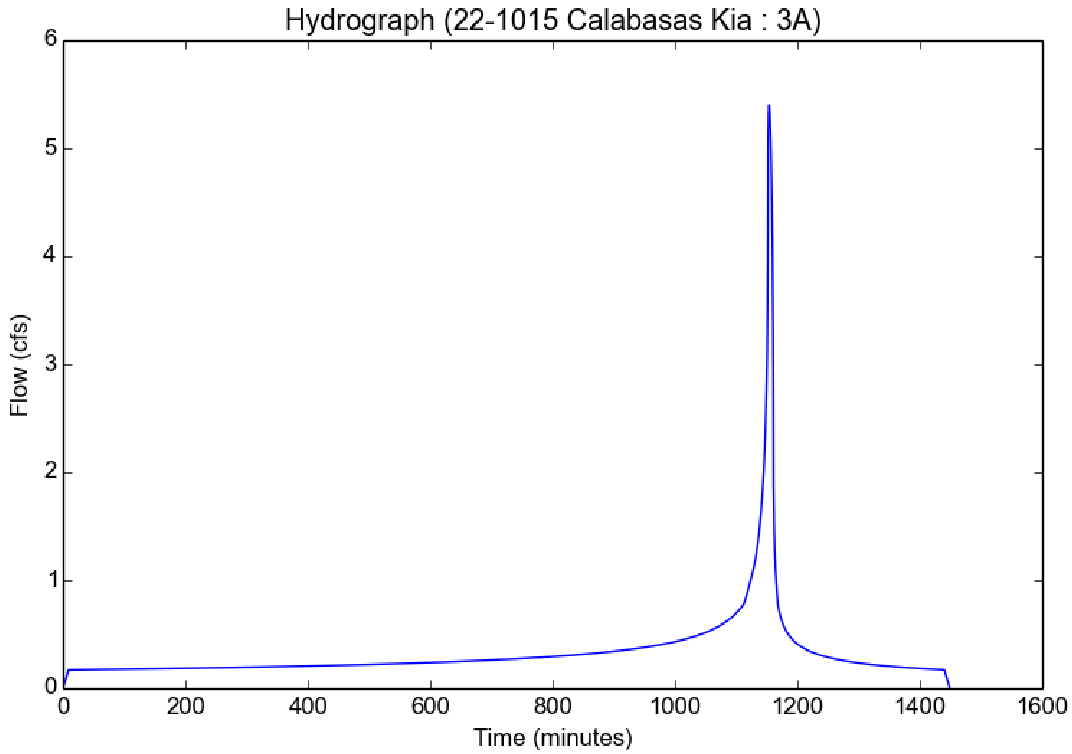
File location: P:/22-1015 - Hello Auto Group re Calabasas Kia/200-H&H/PDF/HydroCalc Results/2023-03-08 Proposed All stroms/22-1015 Calabasas Kia
Version: HydroCalc 1.0.3

Input Parameters

Project Name	22-1015 Calabasas Kia
Subarea ID	3A
Area (ac)	3.57
Flow Path Length (ft)	996.0
Flow Path Slope (vft/hft)	0.14
50-yr Rainfall Depth (in)	7.35
Percent Impervious	0.496
Soil Type	4
Design Storm Frequency	5-yr
Fire Factor	0.83
LID	False

Output Results

Modeled (5-yr) Rainfall Depth (in)	4.2924
Peak Intensity (in/hr)	1.9428
Undeveloped Runoff Coefficient (Cu)	0.6594
Developed Runoff Coefficient (Cd)	0.7787
Time of Concentration (min)	9.0
Clear Peak Flow Rate (cfs)	5.401
Burned Peak Flow Rate (cfs)	5.8689
24-Hr Clear Runoff Volume (ac-ft)	0.6671
24-Hr Clear Runoff Volume (cu-ft)	29060.0073



Peak Flow Hydrologic Analysis

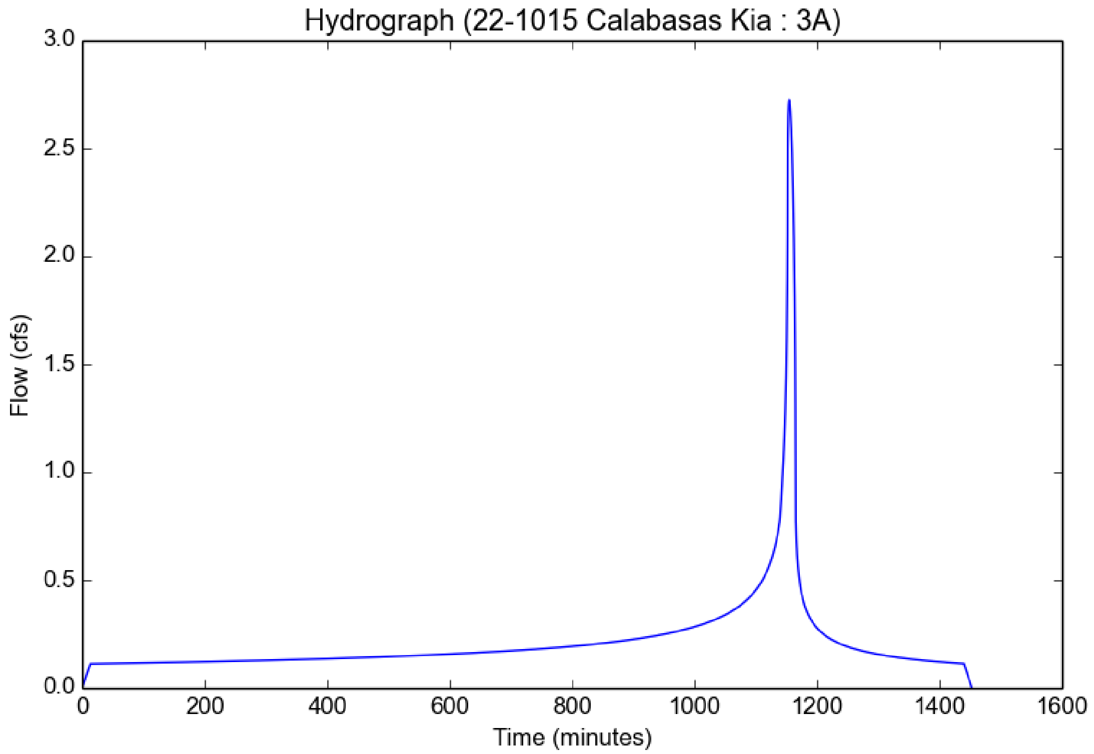
File location: P:/22-1015 - Hello Auto Group re Calabasas Kia/200-H&H/PDF/HydroCalc Results/2023-03-08 Proposed All stroms/22-1015 Calabasas Kia
Version: HydroCalc 1.0.3

Input Parameters

Project Name	22-1015 Calabasas Kia
Subarea ID	3A
Area (ac)	3.57
Flow Path Length (ft)	996.0
Flow Path Slope (vft/hft)	0.14
50-yr Rainfall Depth (in)	7.35
Percent Impervious	0.496
Soil Type	4
Design Storm Frequency	2-yr
Fire Factor	0.83
LID	False

Output Results

Modeled (2-yr) Rainfall Depth (in)	2.8445
Peak Intensity (in/hr)	1.0831
Undeveloped Runoff Coefficient (Cu)	0.5128
Developed Runoff Coefficient (Cd)	0.7048
Time of Concentration (min)	13.0
Clear Peak Flow Rate (cfs)	2.7253
Burned Peak Flow Rate (cfs)	3.0365
24-Hr Clear Runoff Volume (ac-ft)	0.4319
24-Hr Clear Runoff Volume (cu-ft)	18811.755



Peak Flow Hydrologic Analysis

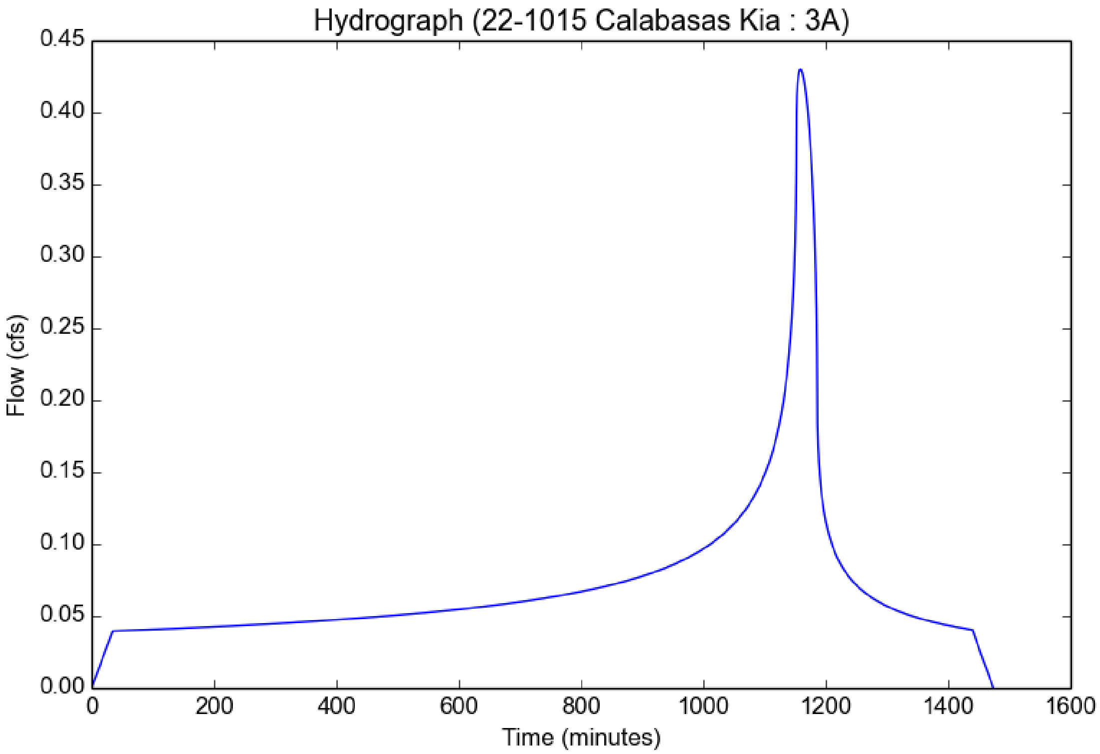
File location: P:/22-1015 - Hello Auto Group re Calabasas Kia/200-H&H/PDF/HydroCalc Results/2023-03-08 Proposed All stroms/22-1015 Calabasas Kia
Version: HydroCalc 1.0.3

Input Parameters

Project Name	22-1015 Calabasas Kia
Subarea ID	3A
Area (ac)	3.57
Flow Path Length (ft)	996.0
Flow Path Slope (vft/hft)	0.14
85th Percentile Rainfall Depth (in)	1.0
Percent Impervious	0.496
Soil Type	4
Design Storm Frequency	85th percentile storm
Fire Factor	0.83
LID	True

Output Results

Modeled (85th percentile storm) Rainfall Depth (in)	1.0
Peak Intensity (in/hr)	0.2423
Undeveloped Runoff Coefficient (Cu)	0.1
Developed Runoff Coefficient (Cd)	0.4968
Time of Concentration (min)	34.0
Clear Peak Flow Rate (cfs)	0.4298
Burned Peak Flow Rate (cfs)	0.5085
24-Hr Clear Runoff Volume (ac-ft)	0.1466
24-Hr Clear Runoff Volume (cu-ft)	6384.968



Watershed Modeling System

(WMS v11.0)

Routed Results

Proposed 50 yr

File name: untitled.lac Run date: Fri Mar 10 09:04:10 2023

Los Angeles County Flood Control District
Modified Rational Method Hydrology

Storm Day 1 Storm Frequency 50

LOCATION		SUBAREA AREA (ACRES)	SUBAREA Q (CFS)	TOTAL AREA (ACRES)	TOTAL Q (CFS)	TOTAL VOLUME (AC-FT)	CONV TYPE	CONV LNGTH (FT)	CONV SLOPE (FT/FT)	CONV SIZE	CONV Z	CONTROL Q (CFS)	SOIL NAME	TC (MIN)	RAIN (IN)	PCT IMPV
1	1A	6.8	23.45	6.8	23.45	0.900	0	0	0.00000	0.00	0.00	0	4	5	7.35	0.01
1	2A	0.0	0.00	6.8	23.45	0.900	4	725	0.07170	3.50	0.00	0	4	0	7.35	0.00
1	3A	3.6	12.12	10.4	26.93	1.914	0	0	0.00000	0.00	0.00	0	4	6	7.35	0.50
1	4A	0.0	0.00	10.4	26.93	1.914	0	0	0.00000	0.00	0.00	0	4	0	7.35	0.00

Normal End of MODRAT

Proposed 50 yr Burned

File name: untitled.lac Run date: Fri Mar 10 09:28:48 2023

Los Angeles County Flood Control District
Modified Rational Method Hydrology

Storm Day 1 Storm Frequency 50

LOCATION		SUBAREA AREA (ACRES)	SUBAREA Q (CFS)	TOTAL AREA (ACRES)	TOTAL Q (CFS)	TOTAL VOLUME (AC-FT)	CONV TYPE	CONV LNGTH (FT)	CONV SLOPE (FT/FT)	CONV SIZE	CONV Z	CONTROL Q (CFS)	SOIL NAME	TC (MIN)	RAIN (IN)	PCT IMPV
1	1A	6.8	26.13	6.8	26.13	1.326	0	0	0.00000	0.00	0.00	0	204	5	7.35	0.01
1	2A	0.0	0.00	6.8	26.13	1.326	4	725	0.07170	3.50	0.00	0	204	0	7.35	0.00
1	3A	3.6	12.12	10.4	29.26	2.340	0	0	0.00000	0.00	0.00	0	4	6	7.35	0.50
1	4A	0.0	0.00	10.4	29.26	2.340	0	0	0.00000	0.00	0.00	0	4	0	7.35	0.00

Normal End of MODRAT

Proposed 25 yr

File name: untitled.lac Run date: Fri Mar 10 09:34:37 2023

Los Angeles County Flood Control District
Modified Rational Method Hydrology

Storm Day 1 Storm Frequency 25

LOCATION		SUBAREA AREA (ACRES)	SUBAREA Q (CFS)	TOTAL AREA (ACRES)	TOTAL Q (CFS)	TOTAL VOLUME (AC-FT)	CONV TYPE	CONV LNGTH (FT)	CONV SLOPE (FT/FT)	CONV SIZE	CONV Z	CONTROL Q (CFS)	SOIL NAME	TC (MIN)	RAIN (IN)	PCT IMPV
1	1A	6.8	20.18	6.8	20.18	0.735	0	0	0.00000	0.00	0.00	0	4	5	6.45	0.01
1	2A	0.0	0.00	6.8	20.18	0.735	4	725	0.07170	3.50	0.00	0	4	0	6.45	0.00
1	3A	3.6	9.72	10.4	22.87	1.586	0	0	0.00000	0.00	0.00	0	4	7	6.45	0.50
1	4A	0.0	0.00	10.4	22.87	1.586	0	0	0.00000	0.00	0.00	0	4	0	6.45	0.00

Normal End of MODRAT

Proposed 25 yr Burned

File name: untitled.lac Run date: Fri Mar 10 09:36:40 2023

Los Angeles County Flood Control District
Modified Rational Method Hydrology

Storm Day 1 Storm Frequency 25

LOCATION		SUBAREA AREA (ACRES)	SUBAREA Q (CFS)	TOTAL AREA (ACRES)	TOTAL Q (CFS)	TOTAL VOLUME (AC-FT)	CONV TYPE	CONV LNGTH (FT)	CONV SLOPE (FT/FT)	CONV SIZE	CONV Z	CONTROL Q (CFS)	SOIL NAME	TC (MIN)	RAIN (IN)	PCT IMPV
1	1A	6.8	22.66	6.8	22.66	1.082	0	0	0.00000	0.00	0.00	0	204	5	6.45	0.01
1	2A	0.0	0.00	6.8	22.66	1.082	4	725	0.07170	3.50	0.00	0	204	0	6.45	0.00
1	3A	3.6	9.72	10.4	24.92	1.932	0	0	0.00000	0.00	0.00	0	4	7	6.45	0.50
1	4A	0.0	0.00	10.4	24.92	1.932	0	0	0.00000	0.00	0.00	0	4	0	6.45	0.00

Normal End of MODRAT

Proposed 10 yr

File name: untitled.lac   Run date: Fri Mar 10 09:39:35 2023

Los Angeles County Flood Control District
Modified Rational Method Hydrology

Storm Day 1   Storm Frequency 10

LOCATION		SUBAREA AREA (ACRES)	SUBAREA Q (CFS)	TOTAL AREA (ACRES)	TOTAL Q (CFS)	TOTAL VOLUME (AC-FT)	CONV TYPE	CONV LNGTH (FT)	CONV SLOPE (FT/FT)	CONV SIZE	CONV Z	CONTROL Q (CFS)	SOIL NAME	TC (MIN)	RAIN (IN)	PCT IMPV
1	1A	6.8	14.24	6.8	14.24	0.539	0	0	0.00000	0.00	0.00	0	4	6	5.25	0.01
1	2A	0.0	0.00	6.8	14.24	0.539	4	725	0.07170	3.50	0.00	0	4	0	5.25	0.00
1	3A	3.6	7.26	10.4	16.94	1.183	0	0	0.00000	0.00	0.00	0	4	8	5.25	0.50
1	4A	0.0	0.00	10.4	16.94	1.183	0	0	0.00000	0.00	0.00	0	4	0	5.25	0.00

Normal End of MODRAT

Proposed 10 yr Burned

File name: untitled.lac   Run date: Fri Mar 10 09:49:43 2023

Los Angeles County Flood Control District
Modified Rational Method Hydrology

Storm Day 1   Storm Frequency 10

LOCATION		SUBAREA AREA (ACRES)	SUBAREA Q (CFS)	TOTAL AREA (ACRES)	TOTAL Q (CFS)	TOTAL VOLUME (AC-FT)	CONV TYPE	CONV LNGTH (FT)	CONV SLOPE (FT/FT)	CONV SIZE	CONV Z	CONTROL Q (CFS)	SOIL NAME	TC (MIN)	RAIN (IN)	PCT IMPV
1	1A	6.8	16.35	6.8	16.35	0.788	0	0	0.00000	0.00	0.00	0	204	6	5.25	0.01
1	2A	0.0	0.00	6.8	16.35	0.788	4	725	0.07170	3.50	0.00	0	204	0	5.25	0.00
1	3A	3.6	7.26	10.4	18.82	1.428	0	0	0.00000	0.00	0.00	0	4	8	5.25	0.50
1	4A	0.0	0.00	10.4	18.82	1.428	0	0	0.00000	0.00	0.00	0	4	0	5.25	0.00

Normal End of MODRAT

Proposed 5 yr

File name: untitled.lac   Run date: Fri Mar 10 09:52:49 2023

Los Angeles County Flood Control District
Modified Rational Method Hydrology

Storm Day 1   Storm Frequency 5

LOCATION		SUBAREA AREA (ACRES)	SUBAREA Q (CFS)	TOTAL AREA (ACRES)	TOTAL Q (CFS)	TOTAL VOLUME (AC-FT)	CONV TYPE	CONV LNGTH (FT)	CONV SLOPE (FT/FT)	CONV SIZE	CONV Z	CONTROL Q (CFS)	SOIL NAME	TC (MIN)	RAIN (IN)	PCT IMPV
1	1A	6.8	9.38	6.8	9.38	0.402	0	0	0.00000	0.00	0.00	0	4	8	4.29	0.01
1	2A	0.0	0.00	6.8	9.38	0.402	4	725	0.07170	3.50	0.00	0	4	0	4.29	0.00
1	3A	3.6	5.45	10.4	12.07	0.898	0	0	0.00000	0.00	0.00	0	4	9	4.29	0.50
1	4A	0.0	0.00	10.4	12.07	0.898	0	0	0.00000	0.00	0.00	0	4	0	4.29	0.00

Normal End of MODRAT

Proposed 5 yr Burned

File name: untitled.lac Run date: Fri Mar 10 09:54:18 2023

Los Angeles County Flood Control District
Modified Rational Method Hydrology

Storm Day 1 Storm Frequency 5

LOCATION		SUBAREA AREA (ACRES)	SUBAREA Q (CFS)	TOTAL AREA (ACRES)	TOTAL Q (CFS)	TOTAL VOLUME (AC-FT)	CONV TYPE	CONV LNGTH (FT)	CONV SLOPE (FT/FT)	CONV SIZE	CONV Z	CONTROL Q (CFS)	SOIL NAME	TC (MIN)	RAIN (IN)	PCT IMPV
1	1A	6.8	11.10	6.8	11.10	0.581	0	0	0.00000	0.00	0.00	0	204	8	4.29	0.01
1	2A	0.0	0.00	6.8	11.10	0.581	4	725	0.07170	3.50	0.00	0	204	0	4.29	0.00
1	3A	3.6	5.45	10.4	13.61	1.060	0	0	0.00000	0.00	0.00	0	4	9	4.29	0.50
1	4A	0.0	0.00	10.4	13.61	1.060	0	0	0.00000	0.00	0.00	0	4	0	4.29	0.00

Normal End of MODRAT

Proposed 2 yr

File name: untitled.lac   Run date: Fri Mar 10 09:56:36 2023

Los Angeles County Flood Control District
Modified Rational Method Hydrology

Storm Day 1   Storm Frequency 2

LOCATION		SUBAREA AREA (ACRES)	SUBAREA Q (CFS)	TOTAL AREA (ACRES)	TOTAL Q (CFS)	TOTAL VOLUME (AC-FT)	CONV TYPE	CONV LNGTH (FT)	CONV SLOPE (FT/FT)	CONV SIZE	CONV Z	CONTROL Q (CFS)	SOIL NAME	TC (MIN)	RAIN (IN)	PCT IMPV
1	1A	6.8	4.01	6.8	4.01	0.228	0	0	0.00000	0.00	0.00	0	4	12	2.84	0.01
1	2A	0.0	0.00	6.8	4.01	0.228	4	725	0.07170	3.50	0.00	0	4	0	2.84	0.00
1	3A	3.6	2.75	10.4	5.74	0.525	0	0	0.00000	0.00	0.00	0	4	13	2.84	0.50
1	4A	0.0	0.00	10.4	5.74	0.525	0	0	0.00000	0.00	0.00	0	4	0	2.84	0.00

Normal End of MODRAT

Proposed 2 yr Burned

File name: untitled.lac Run date: Fri Mar 10 09:58:24 2023

Los Angeles County Flood Control District
Modified Rational Method Hydrology

Storm Day 1 Storm Frequency 2

LOCATION		SUBAREA AREA (ACRES)	SUBAREA Q (CFS)	TOTAL AREA (ACRES)	TOTAL Q (CFS)	TOTAL VOLUME (AC-FT)	CONV TYPE	CONV LNGTH (FT)	CONV SLOPE (FT/FT)	CONV SIZE	CONV Z	CONTROL Q (CFS)	SOIL NAME	TC (MIN)	RAIN (IN)	PCT IMPV
1	1A	6.8	5.23	6.8	5.23	0.319	0	0	0.00000	0.00	0.00	0	204	12	2.84	0.01
1	2A	0.0	0.00	6.8	5.23	0.319	4	725	0.07170	3.50	0.00	0	204	0	2.84	0.00
1	3A	3.6	2.75	10.4	6.84	0.595	0	0	0.00000	0.00	0.00	0	4	13	2.84	0.50
1	4A	0.0	0.00	10.4	6.84	0.595	0	0	0.00000	0.00	0.00	0	4	0	2.84	0.00

Normal End of MODRAT

Appendix E:

Proposed Basins

Basin #1

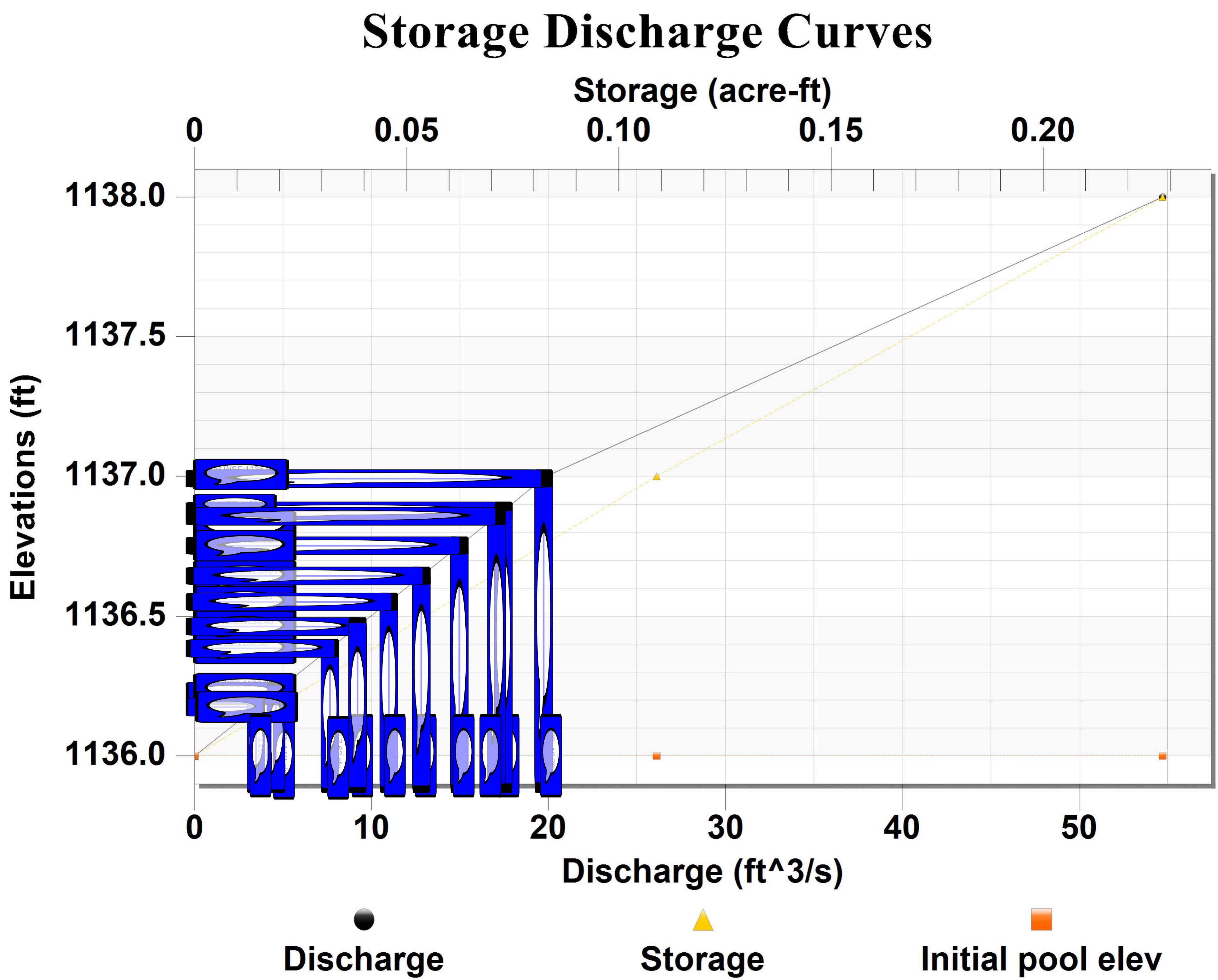
	elevation	area (ft^2)	total volume (ac-ft)	runoff volume cap. event (ac-ft) *	total volume (c.y.)	runoff volume available (c.y.)
Debris	1130.0	1796.9	0		0.0	
	1132.0	2862.4	0.106		171.0	
	1134.0	3665.2	0.255		412.2	
	1136.0	4531.5	0.443		715.2	
Detention	1137***	4953.7	0.552	0.108839	890.8	175.6
Freeboard	1138.0	5442.8	0.671	0.228130	1083.3	368.0

available debris storage = 0.44 ac-ft

see proposed hydrology map for debris basin details

* capital storm model subtracts 100% of the debris yield from the total basin volume before routing

*** Q50B Water Surface elevation



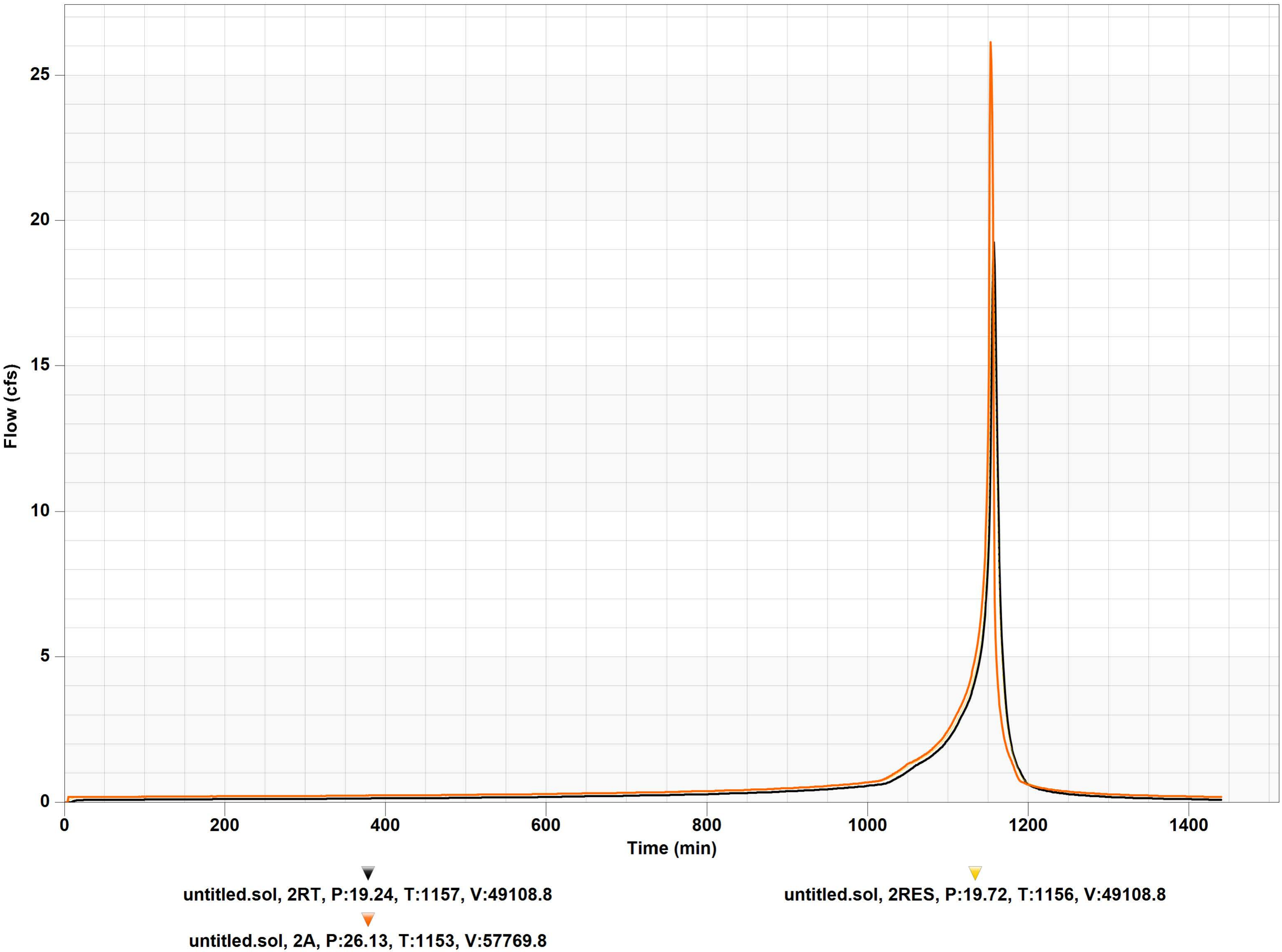
Storage Discharge Curves
Storage (acre-ft)
0
0.05
0.10
0.15
0.20
1138.0
1137.5
1137.0
1136.5
1136.0
Elevations (ft)
0
10
20
30
40
50
Discharge (ft^3/s)
Discharge
Storage
Initial pool elev

<u>Appendix F:</u>

Detention Analysis

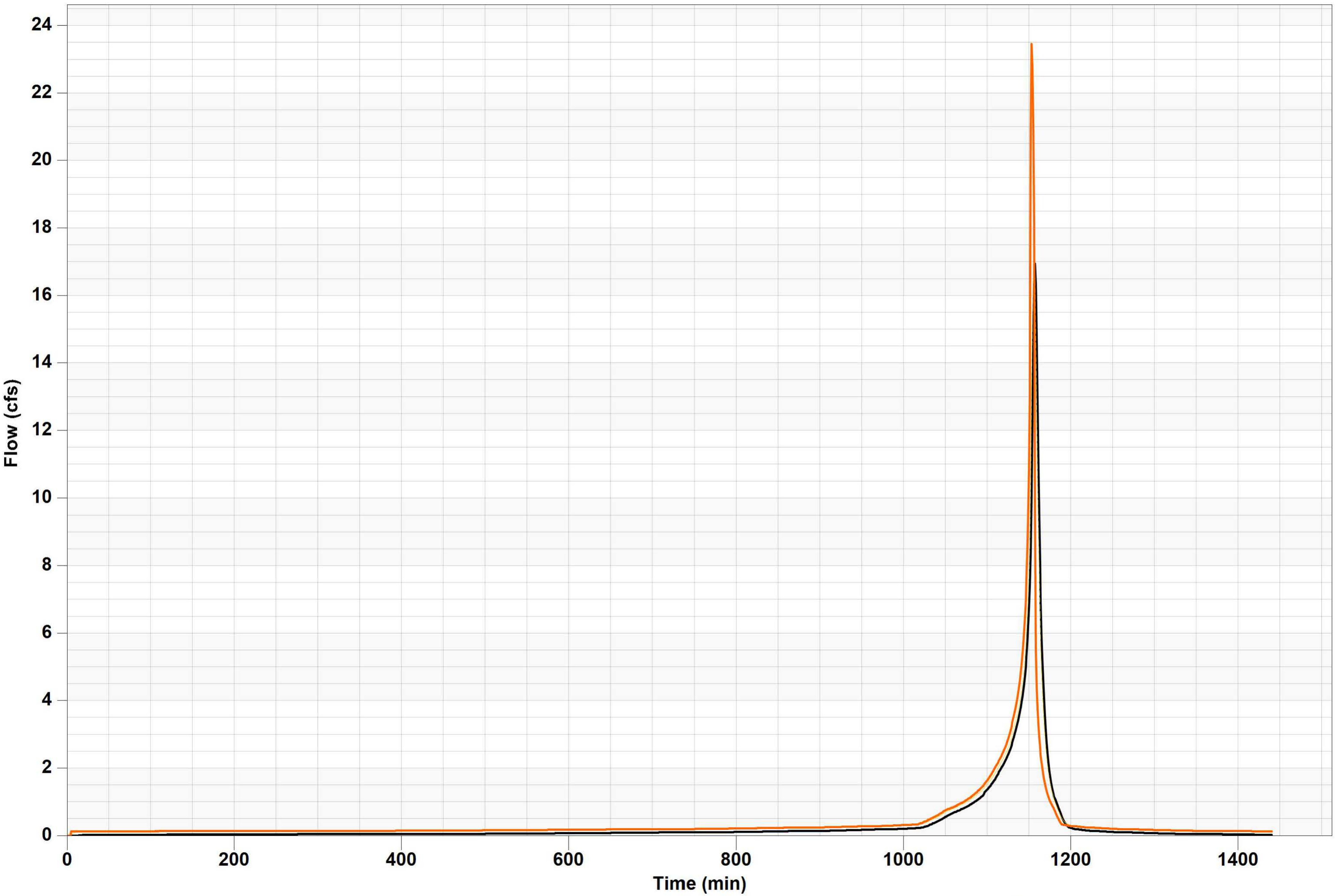
BASIN #1 50yr Burned Inflow - Outflow Hydrograph

PEAK: 26.13 cfs TIME OF PEAK: 1153 min VOLUME: 57769.80 ft^3



BASIN #1 50yr Inflow - Outflow Hydrograph

PEAK: 23.45 cfs TIME OF PEAK: 1153 min VOLUME: 39201.00 ft^3



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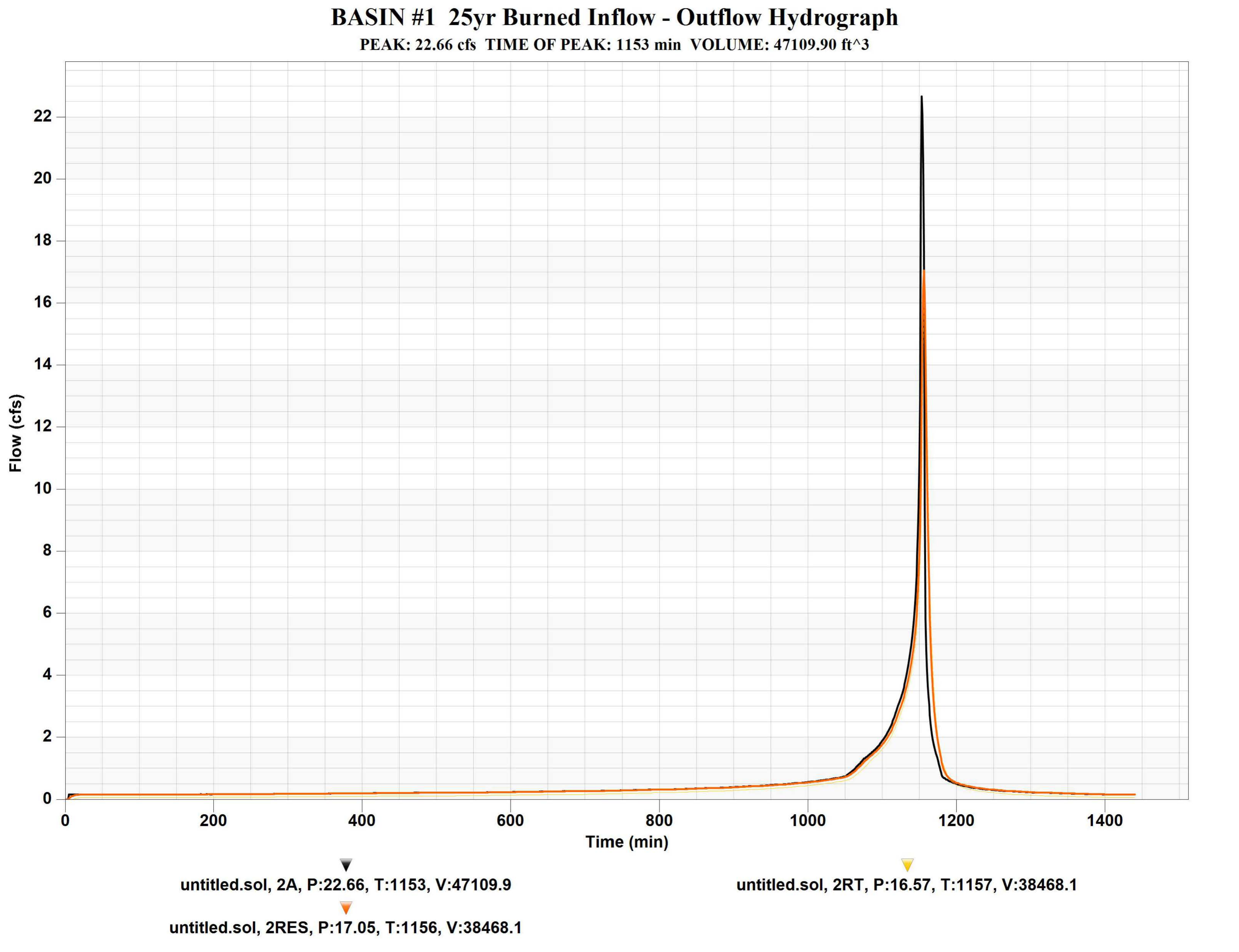
BASIN #1 25yr Burned Inflow - Outflow Hydrograph
PEAK: 22.66 cfs TIME OF PEAK: 1153 min VOLUME: 47109.90 ft^3
Flow (cfs)
Time (min)
0
2
4
6
8
10
12
14
16
18
20
22
0
200
400
600
800
1000
1200
1400
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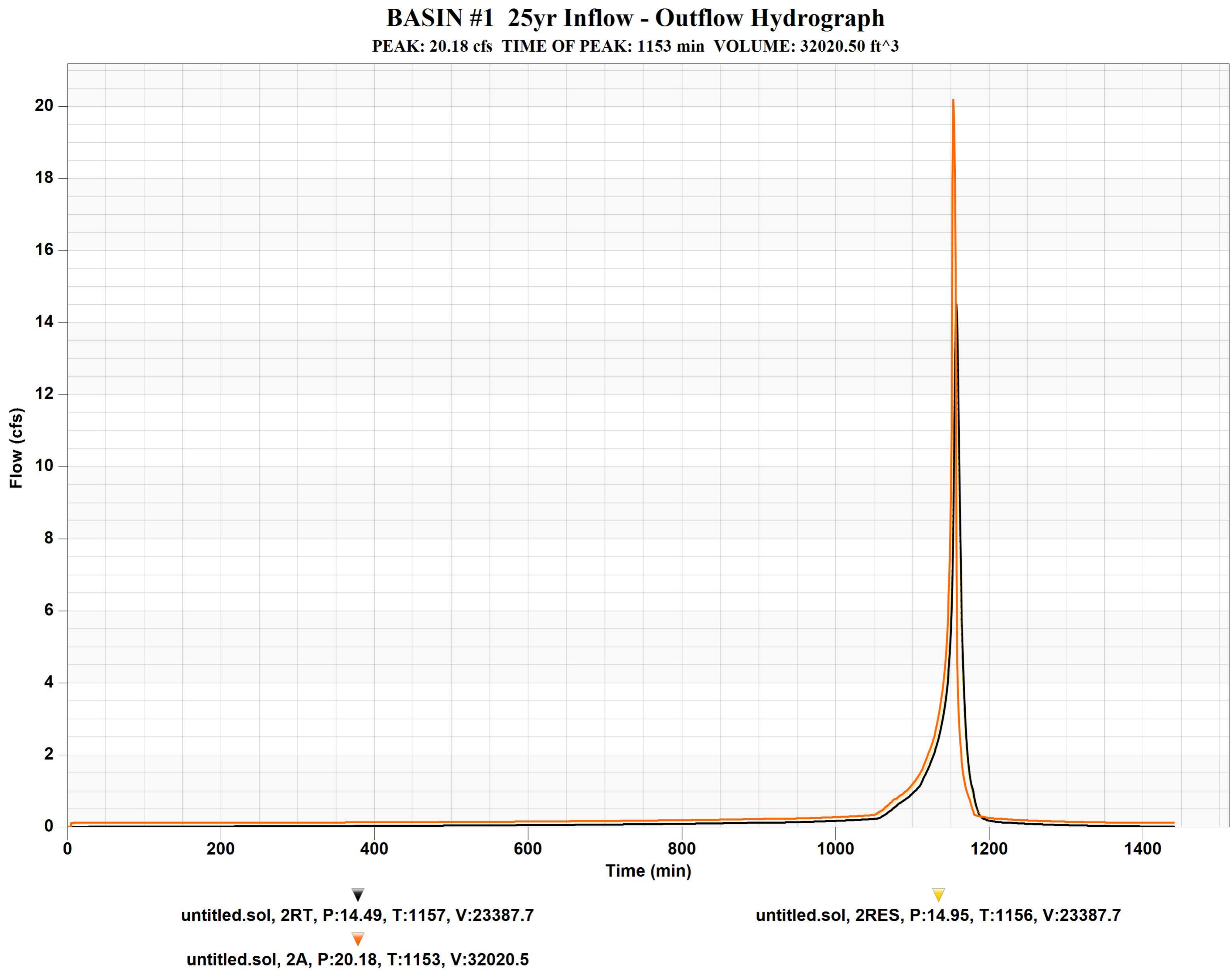
untitled.sol, 2RES, P:17.05, T:1156, V:38468.1


BASIN #1 25yr Inflow - Outflow Hydrograph
PEAK: 20.18 cfs TIME OF PEAK: 1153 min VOLUME: 32020.50 ft^3
Flow (cfs)
0
2
4
6
8
10
12
14
16
18
20
Time (min)
0
200
400
600
800
1000
1200
1400
untitled.sol, 2RT, P:14.49, T:1157, V:23387.7
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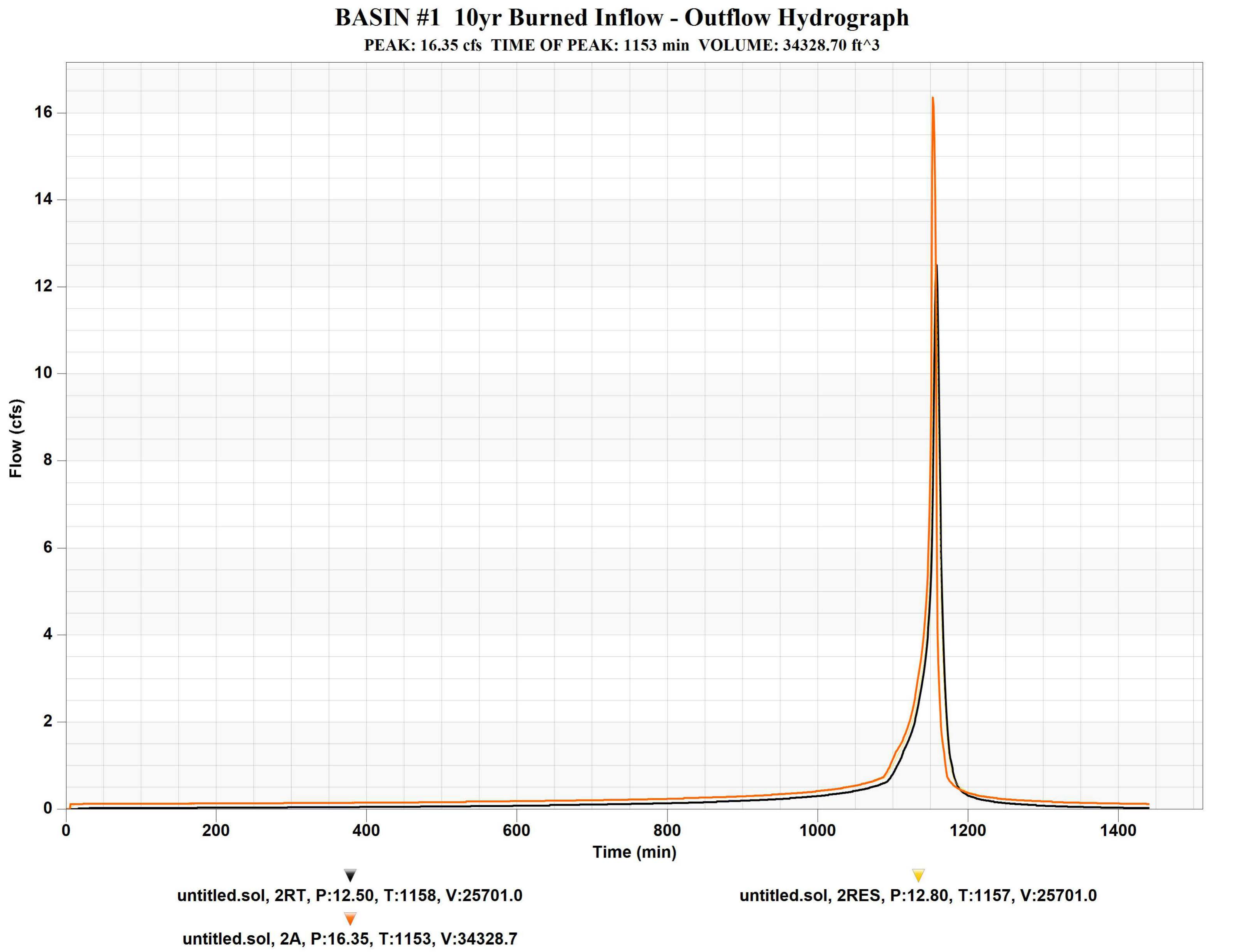
untitled.sol, 2A, P:20.18, T:1153, V:32020.5


BASIN #1 10yr Burned Inflow - Outflow Hydrograph
PEAK: 16.35 cfs TIME OF PEAK: 1153 min VOLUME: 34328.70 ft^3
Flow (cfs)
16
14
12
10
8
6
4
2
0
200
400
600
800
1000
1200
1400
Time (min)
untitled.sol, 2RT, P:12.50, T:1158, V:25701.0
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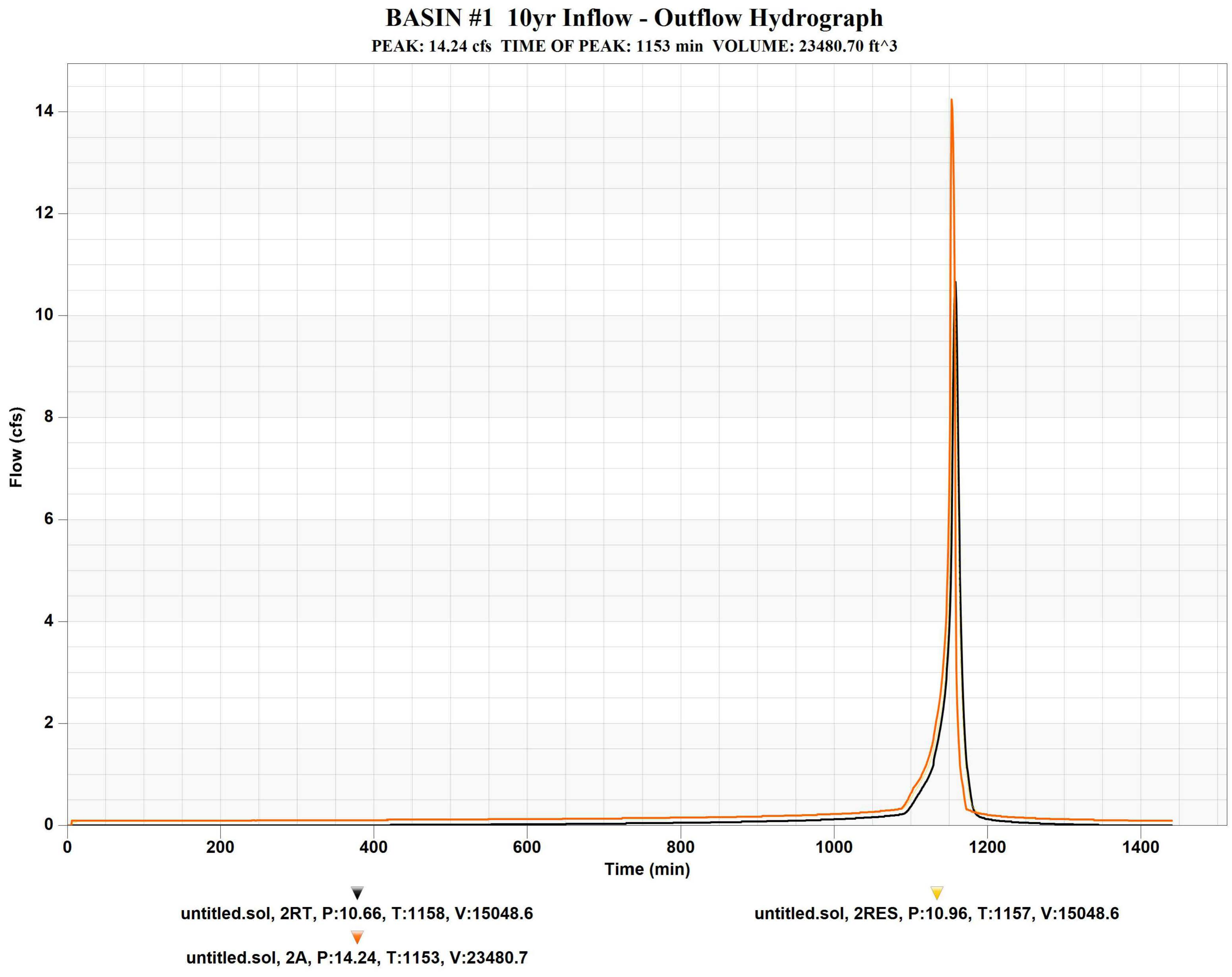
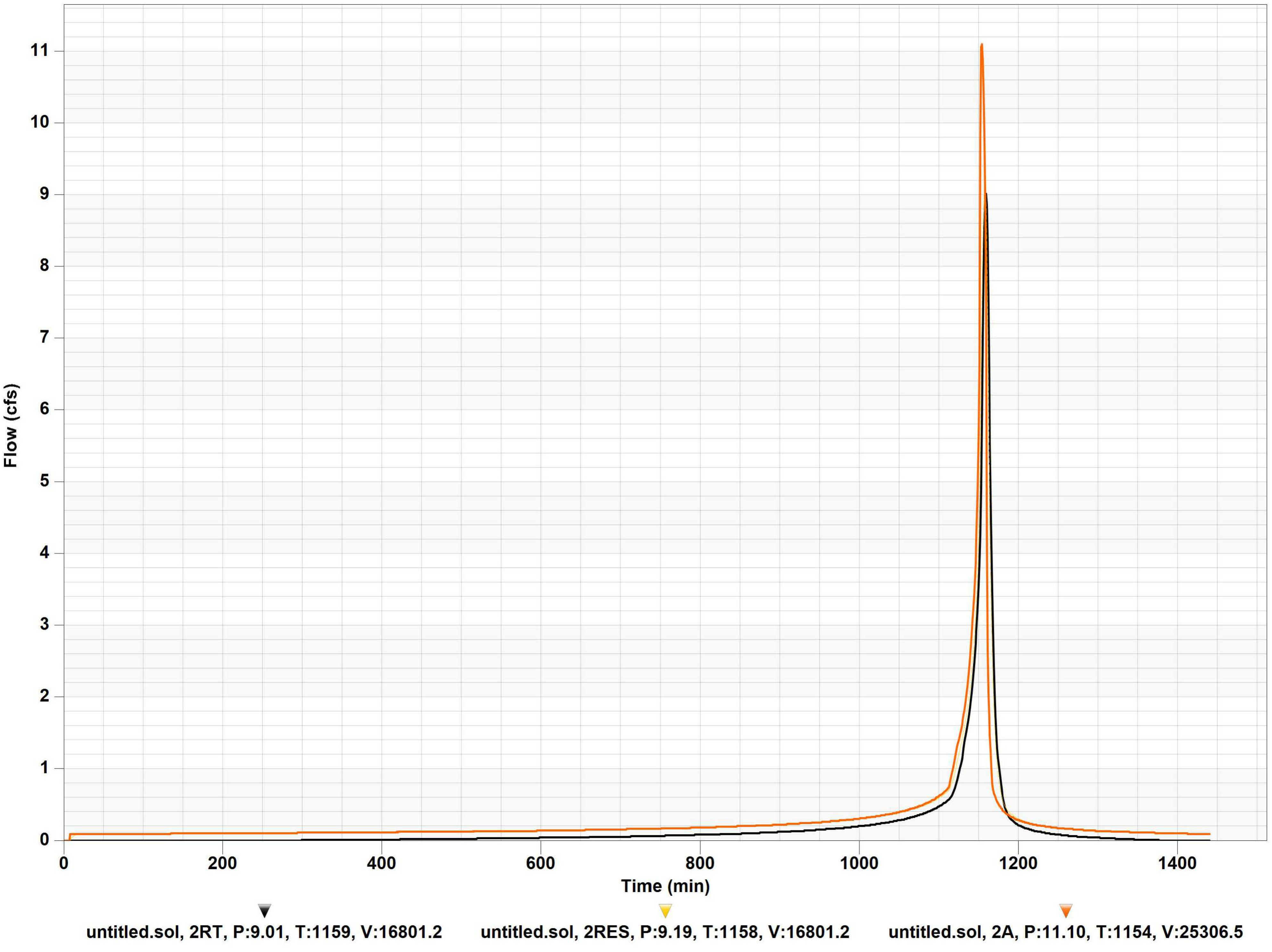


BASIN #1 10yr Inflow - Outflow Hydrograph
PEAK: 14.24 cfs TIME OF PEAK: 1153 min VOLUME: 23480.70 ft^3
Flow (cfs)
0
2
4
6
8
10
12
14
Time (min)
0
200
400
600
800
1000
1200
1400
untitled.sol, 2RT, P:10.66, T:1158, V:15048.6
untitled.sol, 2A, P:14.24, T:1153, V:23480.7
untitled.sol, 2RES, P:10.96, T:1157, V:15048.6

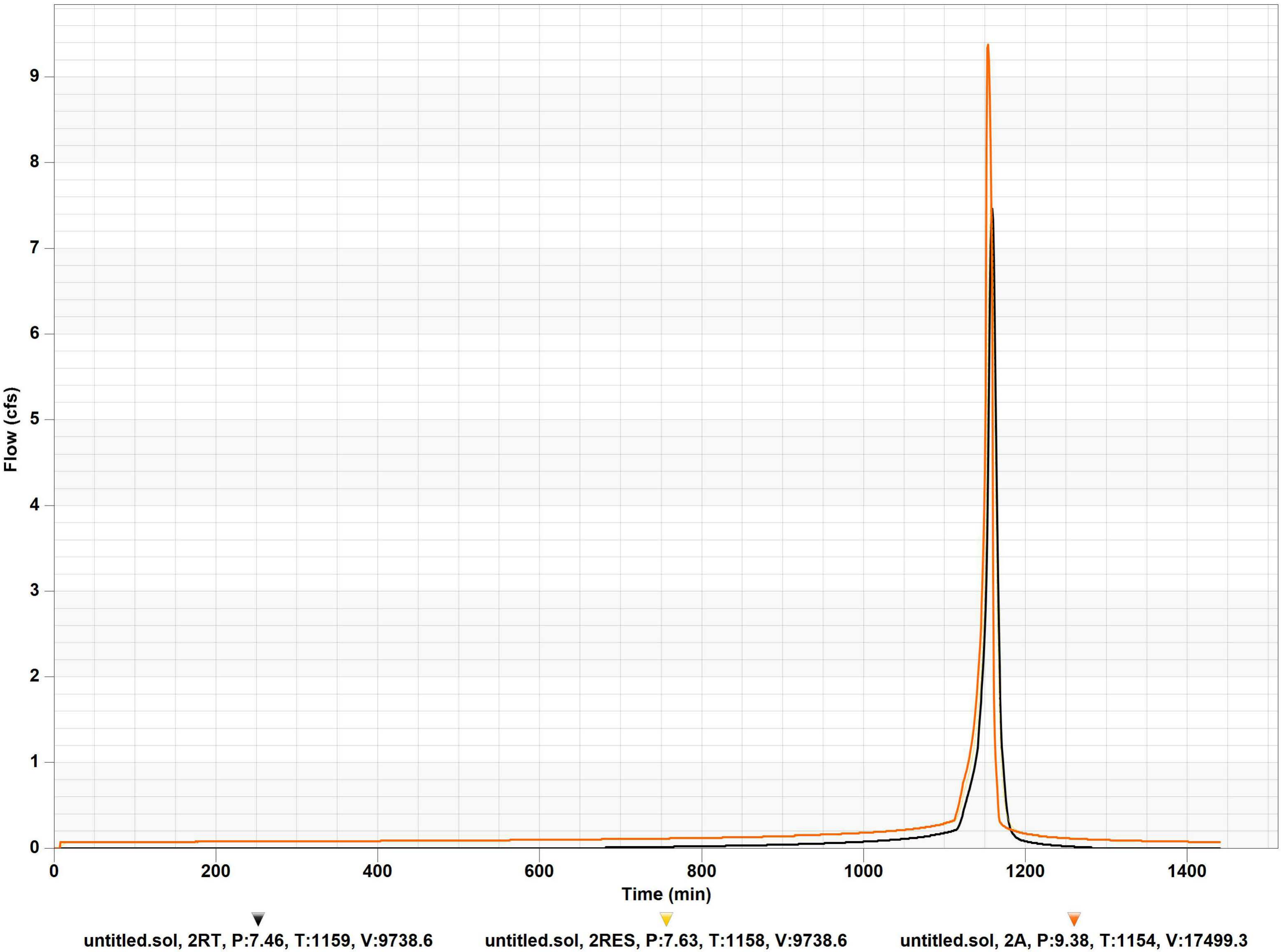
BASIN #1 5yr Burned Inflow - Outflow Hydrograph

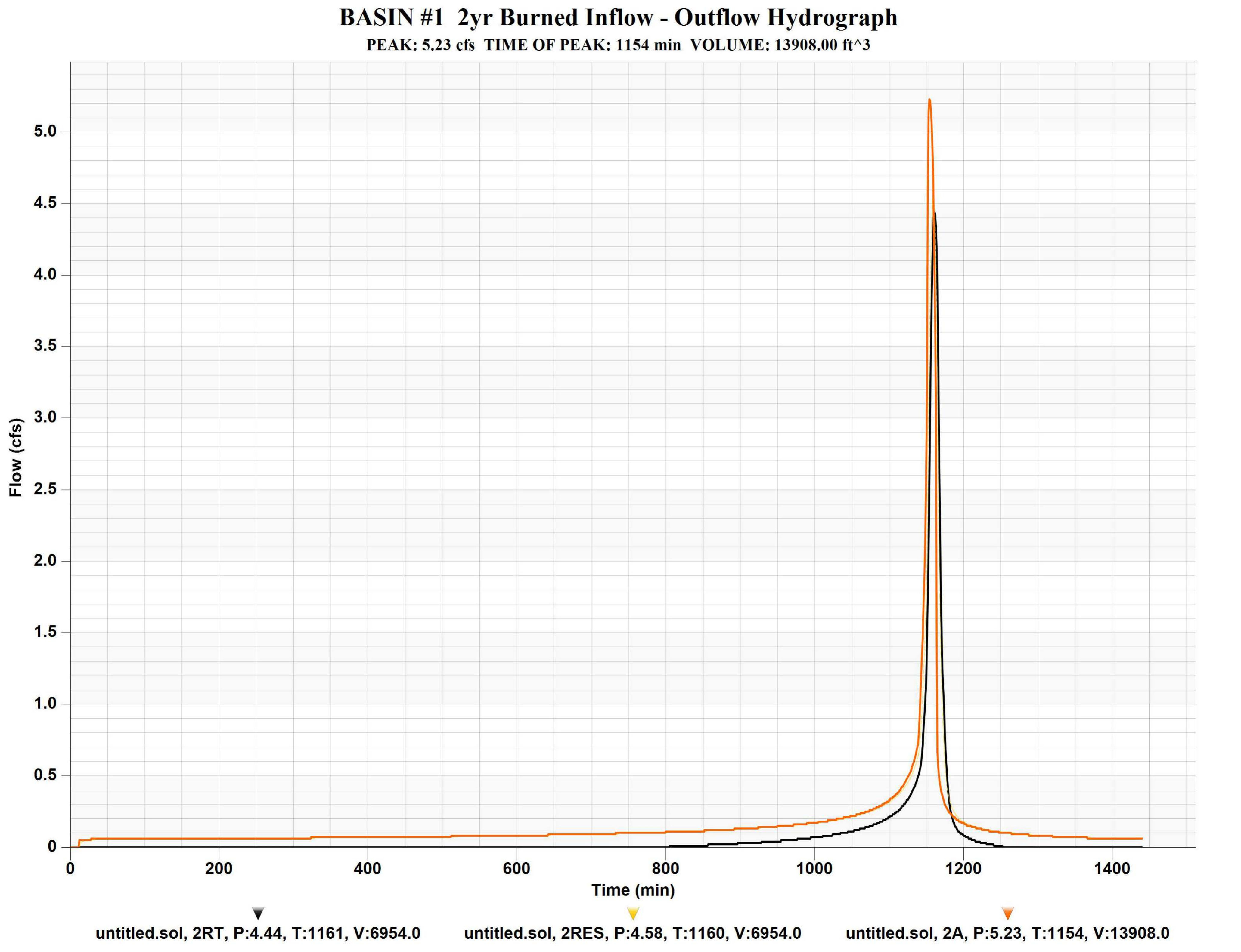
PEAK: 11.10 cfs TIME OF PEAK: 1154 min VOLUME: 25306.50 ft^3



BASIN #1 5yr Inflow - Outflow Hydrograph

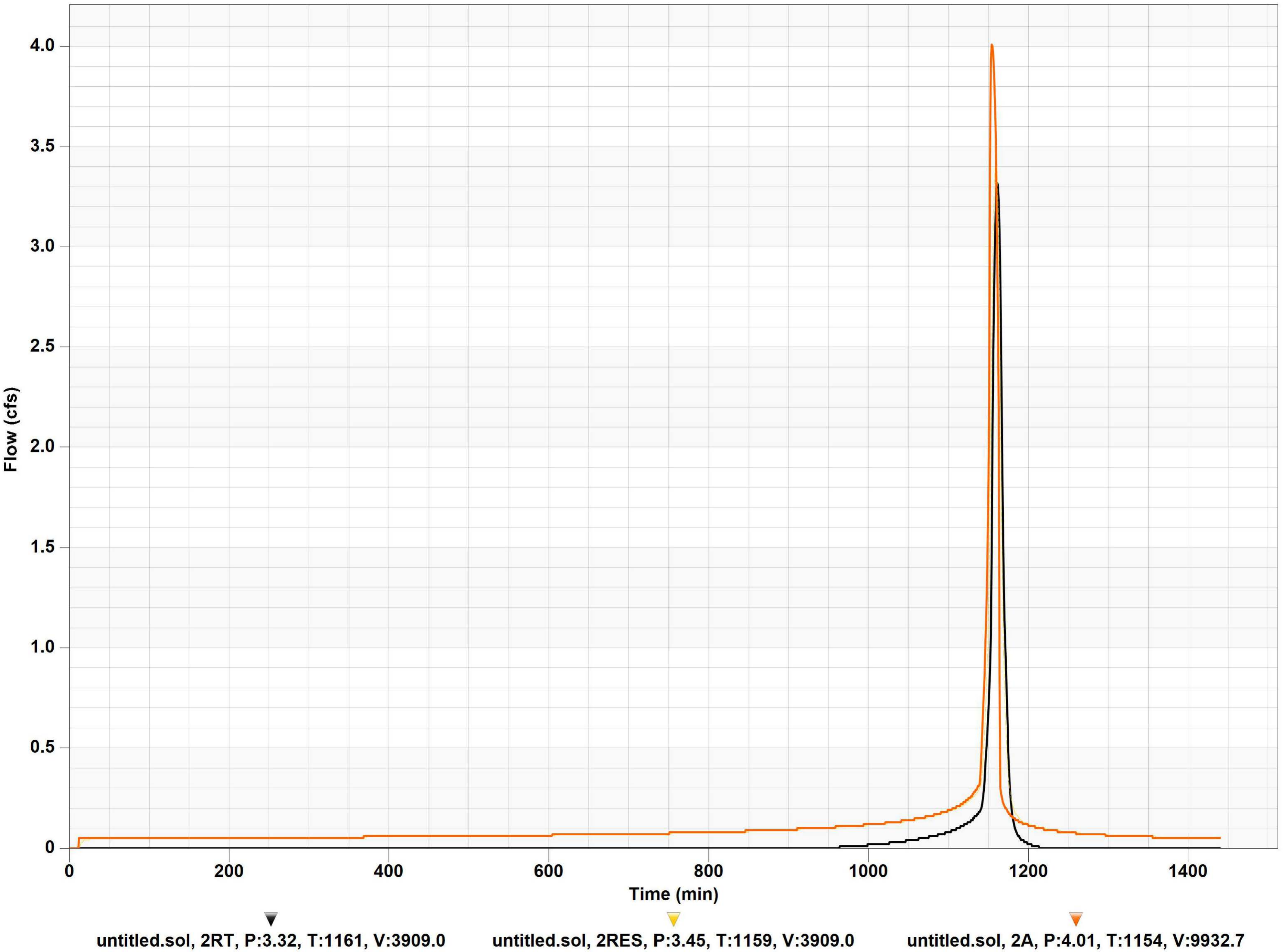
PEAK: 9.38 cfs TIME OF PEAK: 1154 min VOLUME: 17499.30 ft^3




BASIN #1 2yr Burned Inflow - Outflow Hydrograph
PEAK: 5.23 cfs TIME OF PEAK: 1154 min VOLUME: 13908.00 ft^3
Flow (cfs)
5.0
4.5
4.0
3.5
3.0
2.5
2.0
1.5
1.0
0.5
0
0
200
400
600
800
1000
1200
1400
Time (min)
untitled.sol, 2RT, P:4.44, T:1161, V:6954.0
untitled.sol, 2RES, P:4.58, T:1160, V:6954.0
untitled.sol, 2A, P:5.23, T:1154, V:13908.0

BASIN #1 2yr Inflow - Outflow Hydrograph

PEAK: 4.01 cfs TIME OF PEAK: 1154 min VOLUME: 9932.70 ft^3



Appendix G:

Stormwater Quality Calculations

Peak Flow Hydrologic Analysis

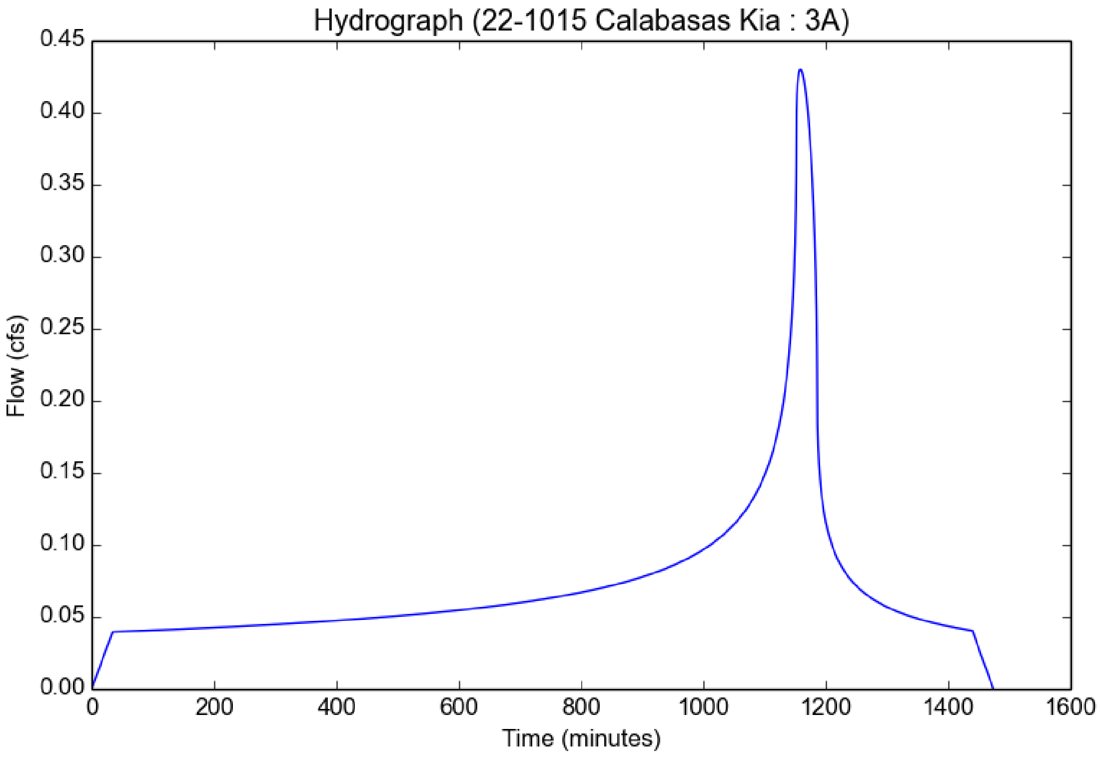
File location: P:/22-1015 - Hello Auto Group re Calabasas Kia/200-H&H/PDF/HydroCalc Results/2023-03-08 Proposed All stroms/22-1015 Calabasas Kia
Version: HydroCalc 1.0.3

Input Parameters

Project Name	22-1015 Calabasas Kia
Subarea ID	3A
Area (ac)	3.57
Flow Path Length (ft)	996.0
Flow Path Slope (vft/hft)	0.14
85th Percentile Rainfall Depth (in)	1.0
Percent Impervious	0.496
Soil Type	4
Design Storm Frequency	85th percentile storm
Fire Factor	0.83
LID	True

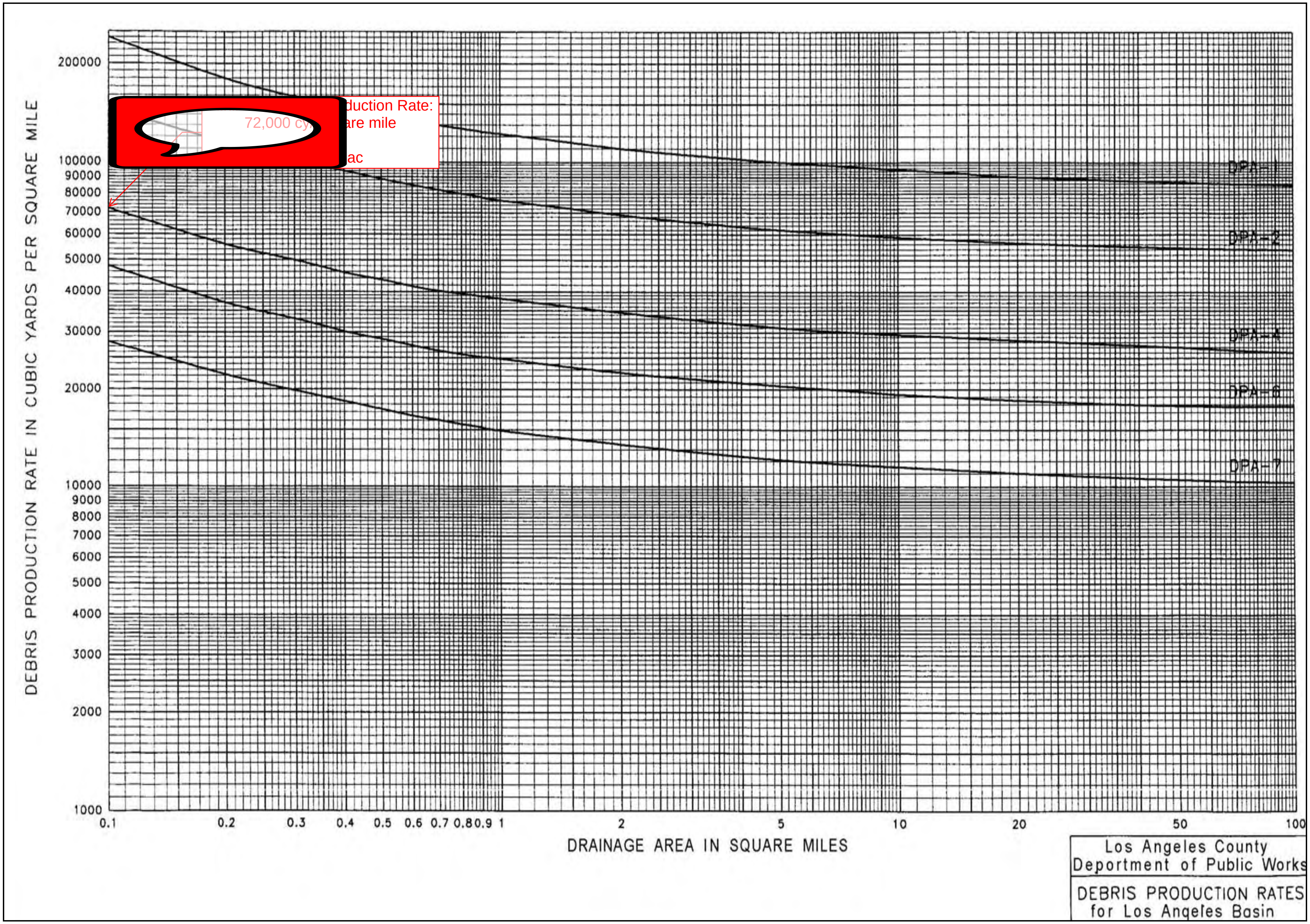
Output Results

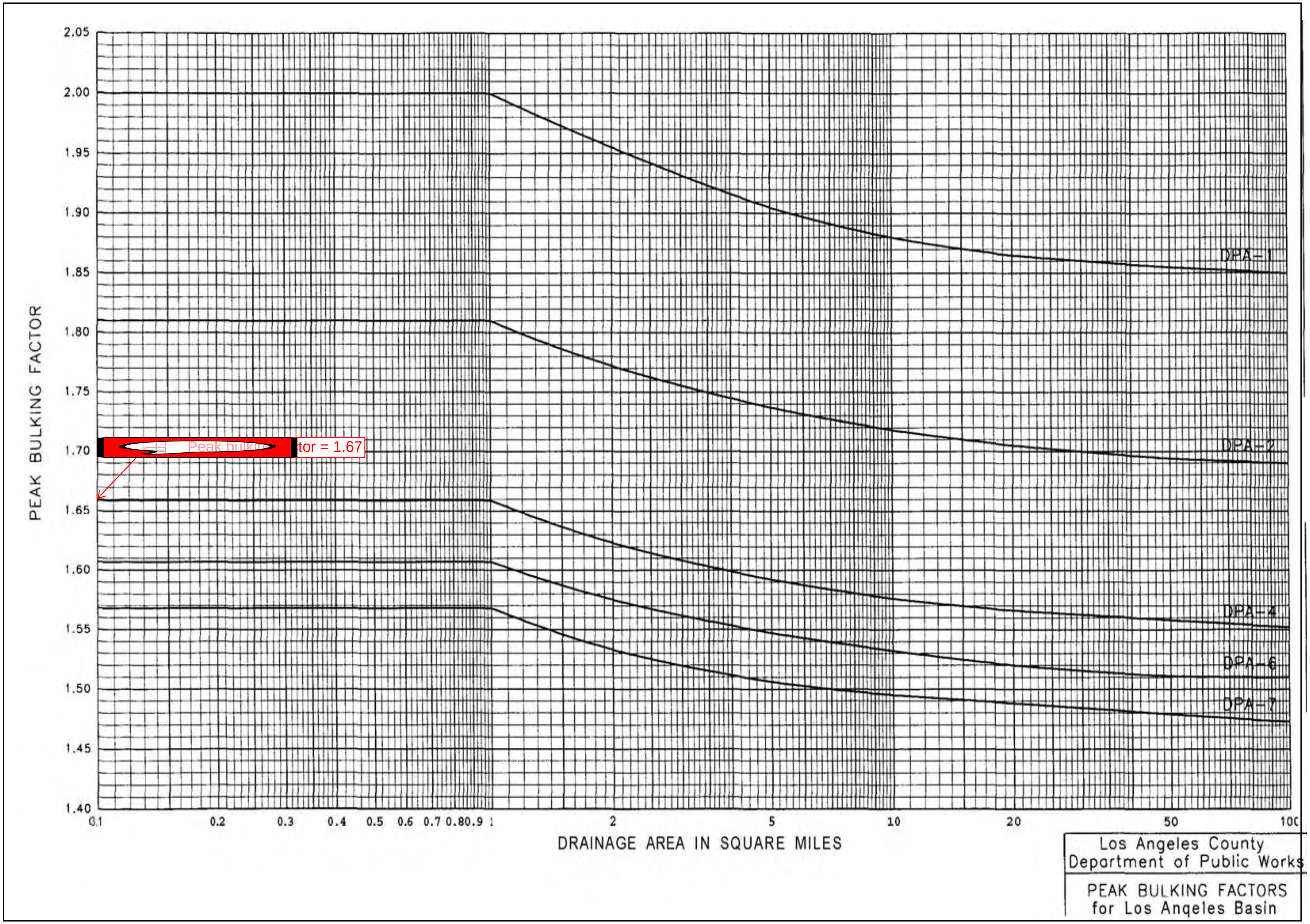
Modeled (85th percentile storm) Rainfall Depth (in)	1.0
Peak Intensity (in/hr)	0.2423
Undeveloped Runoff Coefficient (Cu)	0.1
Developed Runoff Coefficient (Cd)	0.4968
Time of Concentration (min)	34.0
Clear Peak Flow Rate (cfs)	0.4298
Burned Peak Flow Rate (cfs)	0.5085
24-Hr Clear Runoff Volume (ac-ft)	0.1466
24-Hr Clear Runoff Volume (cu-ft)	6384.968



Appendix H:

Debris Potential and Bulk Flow Rate Calculations


DEBRIS PRODUCTION RATE IN CUBIC YARDS PER SQUARE MILE
DRAINAGE AREA IN SQUARE MILES
DPA-1
DPA-2
DPA-4
DPA-6
DPA-7
duction Rate:
72,000 cy
are mile
ac
Los Angeles County Department of Public Works
DEBRIS PRODUCTION RATES for Los Angeles Basin


PEAK BULKING FACTOR
2.05
2.00
1.95
1.90
1.85
1.80
1.75
1.70
1.65
1.60
1.55
1.50
1.45
1.40
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1
2
5
10
20
50
100
DRAINAGE AREA IN SQUARE MILES
DPA-1
DPA-2
DPA-4
DPA-6
DPA-7
tor = 1.67
Los Angeles County
Department of Public Works
PEAK BULKING FACTORS
for Los Angeles Basin

Debris Production -Existing

3.3 SEDIMENT DELIVERY

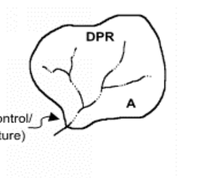
The following sections show the procedures to determir production from watersheds with different characteristics. production is used for the selection and sizing c control/conveyance structures. See Example 1 in Appendix D.

Undeveloped Watershed

Use the following procedure to determine sediment production at an undeveloped watershed that completely falls within the boun DPA zone:

1) Identify the DPA zone from the maps in Appendix A.
2) Determine the drainage area (A) in square miles.
3) Determine the Debris Production Rate (DPR) from curves in A 2, or 3, corresponding to the DPA zone and the drainage a steps 1 and 2 above. For areas smaller than 0.1 square same DPR for 0.1 square mile.
4) Calculate the total Debris Production by multiplying the Debr Rate, from step 3, by the drainage area, from step 2. Equ used for single undeveloped watersheds within a single DPA Z

For a single watershed use Equation 3.3.1:



Outlet (sediment control/ conveyance structure)

$$DP = DPR_{(A)} \times A$$

Where: DP = Debris Production in yd^3
DPR = Debris Production Rate in yd^3/mi^2

3.5 GENERAL FORM EQUATIONS – DEBRIS PRODUCTION RATES & BULKING FACTORS

These equations are the general form of the equations in Sections 3.3 and 3.4 and can be used for multiple DPA zones. The number to the right of each equation corresponds to the number of the equation in Section 3.3 or 3.4. The postscript "g" shows that this is the general form of the equation.

$$DP = DPR_{(A)} \times A \qquad \text{Equation 3.3.1g}$$

$$DP = \Sigma\ (DPR_{i(A_i)} \times A_i) \qquad \text{Equation 3.3.2g}$$

Where: DP = Debris production, in yd^3
$DPR_{i\,(Ai)}$ = Debris production rate based on area A_i in DPA zone i in yd^3/mi^2
A_i = Drainage area in mi^2

$$DP = DPR_{(A)} \times A_u\left(\frac{A_u}{A}\right) + DPR_{(A_u)} \times A_u\left(\frac{A_d}{A}\right) \qquad \text{Equation 3.3.3g}$$

$$A_d = \Sigma\ (\ A_{d_1} + A_{d_2} + A_{d_3} + \ldots + A_{d_n}\ )$$
$$A_u = A - A_d$$

Where: DP = Debris production in yd^3
$DPR_{(A)}$ = Debris production rate based on the total drainage area, A, in yd^3/mi^2
$DPR_{(Au)}$ = Debris production rate based on the total undeveloped drainage area, A_u, in yd^3/mi^2
A = Total drainage area including developments in mi^2
A_u = Total undeveloped area in mi^2
A_d = Total developed area (existing only) in mi^2

Debris Potential

Subarea	1A	Total
DPA Zone	4	-
Debris Production (cy/mi^2)	72000 *	-
$DPR_{(A)}$ (cy/ac)	112.50	-
A (ac)	8.7	8.69
DP (cy)	**977**	**977**

***Sedimentation Manual - Appendix B-1: Debris Production Rate Curves**

Bulk Flow Rate - Existing

debris basin does not exist. Example 1 in Appendix D illustrates use of these curves.

The procedures for determining bulking factors for watersheds with different characteristics are similar to the procedures for determining sediment production explained in Section 3.3. To determine bulked flow rates, Q_B, use the equation listed below for the appropriate case.

For single undeveloped watersheds (see Figure 3.3.1):

$$Q_B = BF_{(A)} \times Q_{(A)} \qquad \textbf{Equation 3.4.1}$$

For multiple undeveloped watersheds having a common outlet (see Figure 3.3.2):

$$Q_B = BF_{1(A_1)} \times \left(\frac{Q\ \ A_1}{A_1 + A_2}\right) + BF_{2(A_2)} \times \left(\frac{Q\ \ A_2}{A_1 + A_2}\right) \qquad \textbf{Equation 3.4.2}$$

For partially developed watersheds (see Figure 3.3.3):

$$Q_B = BF_{(A)} \times \left(\frac{Q_{(A)}\ \ A_u}{A}\right)\left(\frac{A_u}{A}\right) + BF_{(A_u)} \times \left(\frac{Q_{(A)}\ \ A_u}{A}\right)\left(\frac{A_d}{A}\right) + \left(\frac{Q_{(A)}\ \ A_d}{A}\right) \qquad \textbf{Equation 3.4.3}$$

For a watershed with multiple debris production zones (see Figure 3.3.4):

$$Q_B = BF_{1(A_1+A_2)} \times \left(\frac{Q\ \ A_1}{A_1 + A_2}\right)\left(\frac{A_1}{A_1 + A_2}\right) + BF_{1(A_1)} \times \left(\frac{Q\ \ A_1}{A_1 + A_2}\right)\left(\frac{A_2}{A_1 + A_2}\right) +$$
$$BF_{2(A_1+A_2)} \times \left(\frac{Q\ \ A_2}{A_1 + A_2}\right)\left(\frac{A_2}{A_1 + A_2}\right) + BF_{2(A_2)} \times \left(\frac{Q\ \ A_2}{A_1 + A_2}\right)\left(\frac{A_1}{A_1 + A_2}\right) \qquad \textbf{Equation 3.4.4}$$

$$Q = Q_{A_1+A_2}$$

Where:
- Q = Clear or burned discharge in cfs
- Q_B = Bulked or burned and bulked discharge in cfs
- $BF_{i(Ai)}$ = Bulking factor based on area A_i
- A_i = Drainage area in mi^2
- A_u = Total undeveloped area in mi^2
- A_d = Total developed area in mi^2

Bulk Flow Rate		
Subarea	1A	Total
DPA Zone	4	-
Q_{50} (cfs)	30.30	30.3
Q_{50b}(cfs)	33.42	33.4
$BF_{i(Ai)}$	1.67 *	-
Ai (ac)	10.32	-
Au (ac)	8.69	-
Ad (ac)	1.63	-
Q_{BB}(cfs)	51.78	**51.8**

***Sedimentation Manual - Appendix B-4: Peak Bulking Factor Curves**

Debris Production - Proposed

3.3 SEDIMENT DELIVERY

The following sections show the procedures to determine sediment production from watersheds with different characteristics. Sediment production is used for the selection and sizing of sediment control/conveyance structures. See Example 1 in Appendix D.

Undeveloped Watershed

Use the following procedure to determine sediment production at the outlet of an undeveloped watershed that completely falls within the boundaries of one DPA zone:

1) Identify the DPA zone from the maps in Appendix A.
2) Determine the drainage area (A) in square miles.
3) Determine the Debris Production Rate (DPR) from curves in Appendix B-1, 2, or 3, corresponding to the DPA zone and the drainage area found in steps 1 and 2 above. For areas smaller than 0.1 square mile, use the same DPR for 0.1 square mile.
4) Calculate the total Debris Production by multiplying the Debris Production Rate, from step 3, by the drainage area, from step 2. Equation 3.3.1 is used for single undeveloped watersheds within a single DPA Zone.

For a single watershed use Equation 3.3.1:

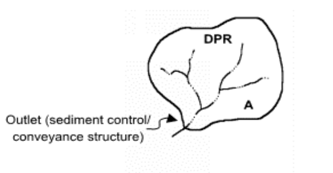


Fig
Deb
Wat

$$DP = DPR_{(A)} \times A$$

Equ

Where: DP = Debris Production in yd^3
DPR = Debris Production Rate in yd^3/mi^2

3.5 GENERAL FORM EQUATIONS – DEBRIS PRODUCTION RATES & BULKING FACTORS

These equations are the general form of the equations in Sections 3.3 and 3.4 and can be used for multiple DPA zones. The number to the right of each equation corresponds to the number of the equation in Section 3.3 or 3.4. The postscript "g" shows that this is the general form of the equation.

$$DP = DPR_{(A)} \times A \qquad \textbf{Equation 3.3.1g}$$

$$DP = \Sigma\ (DPR_{i(A_i)} \times A_i) \qquad \textbf{Equation 3.3.2g}$$

Where: DP = Debris production, in yd^3
$DPR_{i(Ai)}$ = Debris production rate based on area A_i in DPA zone i in yd^3/mi^2
A_i = Drainage area in mi^2

$$DP = DPR_{(A)} \times A_u\left(\frac{A_u}{A}\right) + DPR_{(A_u)} \times A_u\left(\frac{A_d}{A}\right) \qquad \textbf{Equation 3.3.3g}$$

$$A_d = \Sigma\ (A_{d_1} + A_{d_2} + A_{d_3} + \ldots + A_{d_n})$$
$$A_u = A - A_d$$

Where: DP = Debris production in yd^3
$DPR_{(A)}$ = Debris production rate based on the total drainage area, A, in yd^3/mi^2
$DPR_{(Au)}$ = Debris production rate based on the total undeveloped drainage area, A_u, in yd^3/mi^2
A = Total drainage area including developments in mi^2
A_u = Total undeveloped area in mi^2
A_d = Total developed area (existing only) in mi^2

Debris Production

Subarea	1A	3A**	Total
DPA Zone	4	4	-
Debris Production (cy/mi^2)	72000 *	72000 *	-
DPR (cy/ac)	112.50	112.50	75.00
A (ac)	6.42	1.28	7.70
DP (cy)	***722***	***144***	***866***

***Sedimentation Manual - Appendix B: Debris Production Rate Curves**

****Developed Area- Not Subject to Debris Production**

Bulk Flow Rate -Proposed

debris basin does not exist. Example 1 in Appendix D illustrates use of these curves.

The procedures for determining bulking factors for watersheds with different characteristics are similar to the procedures for determining sediment production explained in Section 3.3. To determine bulked flow rates, Q_B, use the equation listed below for the appropriate case.

For single undeveloped watersheds (see Figure 3.3.1):

$$Q_B = BF_{(A)} \times Q_{(A)} \qquad \textbf{Equation 3.4.1}$$

For multiple undeveloped watersheds having a common outlet (see Figure 3.3.2):

$$Q_B = BF_{1(A_1)} \times \left(\frac{Q\ \ A_1}{A_1 + A_2}\right) + BF_{2(A_2)} \times \left(\frac{Q\ \ A_2}{A_1 + A_2}\right) \qquad \textbf{Equation 3.4.2}$$

For partially developed watersheds (see Figure 3.3.3):

$$Q_B = BF_{(A)} \times \left(\frac{Q_{(A)}\ \ A_u}{A}\right)\left(\frac{A_u}{A}\right) + BF_{(A_u)} \times \left(\frac{Q_{(A)}\ \ A_u}{A}\right)\left(\frac{A_d}{A}\right) + \left(\frac{Q_{(A)}\ \ A_d}{A}\right) \qquad \textbf{Equation 3.4.3}$$

For a watershed with multiple debris production zones (see Figure 3.3.4):

$$\begin{aligned} Q_B = {} & BF_{1(A_1+A_2)} \times \left(\frac{Q\ \ A_1}{A_1 + A_2}\right)\left(\frac{A_1}{A_1 + A_2}\right) + BF_{1(A_1)} \times \left(\frac{Q\ \ A_1}{A_1 + A_2}\right)\left(\frac{A_2}{A_1 + A_2}\right) + \\ & BF_{2(A_1+A_2)} \times \left(\frac{Q\ \ A_2}{A_1 + A_2}\right)\left(\frac{A_2}{A_1 + A_2}\right) + BF_{2(A_2)} \times \left(\frac{Q\ \ A_2}{A_1 + A_2}\right)\left(\frac{A_1}{A_1 + A_2}\right) \end{aligned} \qquad \textbf{Equation 3.4.4}$$

$$Q = Q_{A_1+A_2}$$

Where:
- Q = Clear or burned discharge in cfs
- Q_B = Bulked or burned and bulked discharge in cfs
- $BF_{i(Ai)}$ = Bulking factor based on area A_i
- A_i = Drainage area in mi^2
- A_u = Total undeveloped area in mi^2
- A_d = Total developed area in mi^2

Bulk Flow Rate

Subarea Description	1A	3A**	Total
DPA Zone	4	4	-
Q_{50} (cfs)	23.5	12.1	-
Q_{50b} (cfs)	26.1	12.1	-
Bulk Factor, BF	1.670 *	1.670 *	-
Ai (ac)	6.75	3.57	-
Au (ac)	6.42	0.00	-
Ad (ac)	0.33	3.57	-
Q_{BB} (cfs)	26.1	12.1	***38.26***

*** Sedimentation Manual - Appendix B: Peak Bulking Factor Curves.**

**** Developed Area- Not Subject to Burn and Bulk.**

Appendix I:

Stormwater Quality Design BMP

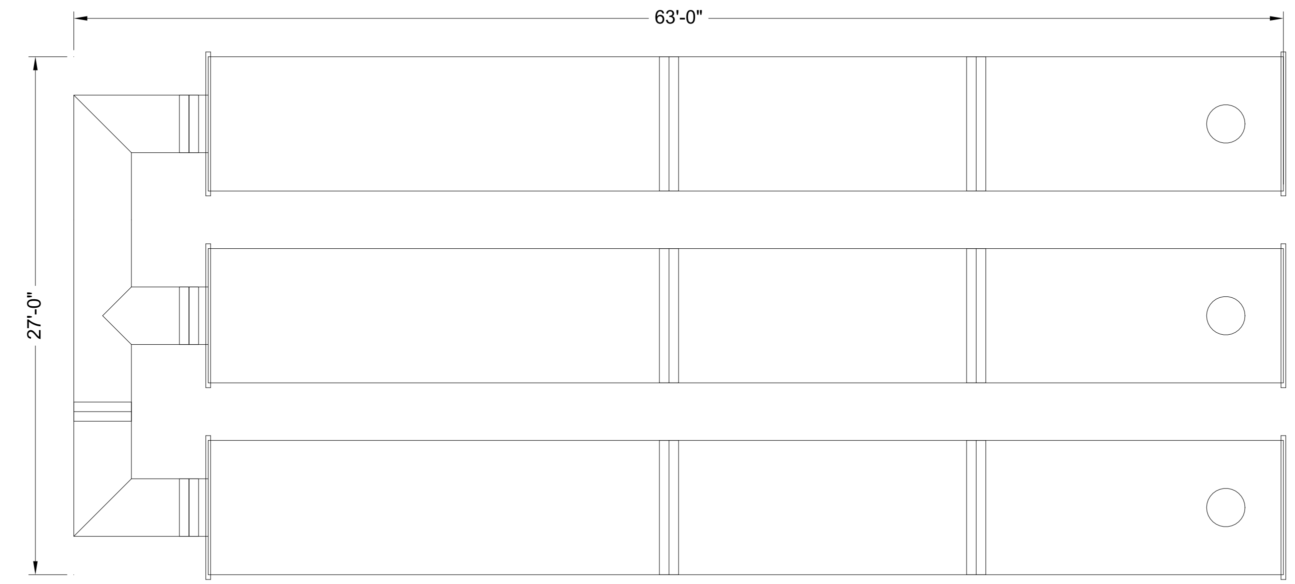
PROJECT SUMMARY

CALCULATION DETAILS
- LOADING = H20
- APPROX. LINEAR FOOTAGE = 200 LF

STORAGE SUMMARY
- STORAGE VOLUME REQUIRED = 6,384.983,931,598,338 CF
- PIPE STORAGE VOLUME = 6,481 CF
- BACKFILL STORAGE VOLUME = 0 CF
- TOTAL STORAGE PROVIDED = 6,481 CF

PIPE DETAILS
- DIAMETER = 84"
- JOINT TYPE = Soil Tight
- ALL RISERS TO HAVE REINFORCEMENT COLLAR
- WALL TYPE = SOLID
- BARREL SPACING = 36"

BACKFILL DETAILS
- WIDTH AT ENDS = 12"
- ABOVE PIPE = 0"
- WIDTH AT SIDES = 12"
- BELOW PIPE = 0"



ASSEMBLY
SCALE: 1" = 10'

NOTE:
THESE DRAWINGS ARE FOR CONCEPTUAL PURPOSES AND DO NOT REFLECT ANY LOCAL PREFERENCES OR REGULATIONS. PLEASE CONTACT YOUR LOCAL CONTECH REP FOR MODIFICATIONS.

NOTES

- ALL RISER AND STUB DIMENSIONS ARE TO CENTERLINE.
- ALL ELEVATIONS, DIMENSIONS, AND LOCATIONS OF RISERS AND INLETS, SHALL BE VERIFIED BY THE ENGINEER OF RECORD PRIOR TO RELEASING FOR FABRICATION.
- ALL RISERS AND STUBS ARE FABRICATED USING HDPE DR32.5 & M294 PIPE UNLESS OTHERWISE NOTED.
- RISERS TO BE FIELD TRIMMED TO GRADE.
- QUANTITY OF PIPE SHOWN DOES NOT PROVIDE EXTRA PIPE FOR CONNECTING THE SYSTEM TO EXISTING PIPE OR DRAINAGE STRUCTURES. OUR SYSTEM AS DETAILED PROVIDES NOMINAL INLET AND/OR OUTLET PIPE STUB FOR CONNECTION TO EXISTING DRAINAGE FACILITIES. IF ADDITIONAL PIPE IS NEEDED IT WILL HAVE TO BE REQUESTED BY THE REVIEWER/APPROVAL AUTHORITY PRIOR TO FABRICATION.

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DuroMaxx STEEL REINFORCED PE TECHNOLOGY

CONTECH
DYODS
DRAWING

DYO28355 Kia
SWQDv
Calabasas, CA
DETENTION SYSTEM

PROJECT No.:	SEQ. No.:	DATE:
18982	28355	3/13/2023

DESIGNED:	DRAWN:
DYO	DYO
CHECKED:	APPROVED:
DYO	DYO

SHEET NO.: D1

NOTES:

1. ALL ELEVATIONS, DIMENSIONS AND LOCATIONS OF RISERS AND INLETS, SHALL BE VERIFIED BY THE ENGINEER OF RECORD (E.O.R.) OR OWNER DURING DESIGN & CONTRACT PHASE OF PROJECT. IF ELEVATIONS AND DESIGN INFORMATION IS NOT PROVIDED BY E.O.R. OR OWNER, THE PROJECT CANNOT BE RELEASED FOR FABRICATION.
2. THIS PROJECT IS FOR DUROMAXX DIAMETER DESCRIBED IN PROJECT SUMMARY.
3. DESIGN SERVICE LIFE OF TANK IS TO BE SPECIFIED BY E.O.R. OR OWNER DURING DESIGN PHASE OF PROJECT, AND WILL BE REVIEWED BY MANUFACTURER.
4. ANY TESTING OF THE TANK WILL NEED TO BE SPECIFIED BY E.O.R. OR OWNER DURING DESIGN PHASE OF PROJECT.
5. ALL WATER TABLE ELEVATIONS ARE TO BE ACCURATELY PROVIDED BY E.O.R. OR OWNER PRIOR TO CONTRACT/FINAL DESIGN PHASE OF PROJECT.
6. TOTAL HEIGHT OF COVER AND DEPTHS OF THE TYPE OF PAVEMENT TO BE USED ARE TO BE PROVIDED BY E.O.R. OR OWNER DURING PROPOSAL & CONTRACT PHASES.
7. INTERNAL OPERATIONAL DESIGN PRESSURES (IF ANY) ARE TO BE PROVIDED BY E.O.R. OR OWNER TO THE MANUFACTURER DURING INITIAL DESIGN PHASE TO PROVIDE PROPER DESIGN OF THE TANK. (______ DESIGN PRESSURE)
8. DESIGN INFORMATION FOR SURGE PRESSURES THAT CAN BE SEEN DURING OPERATION OF THE SYSTEM AS REQUIRED BY E.O.R. OR OWNER WILL ALSO NEED TO BE PROVIDED DURING DESIGN PHASE OF THE PROJECT.
9. DESIGN INFORMATION AS TO WATER TEMPERATURE (IF ABOVE NORMAL EFFLUENT TEMPERATURES) WILL NEED TO BE PROVIDED BY E.O.R. OR OWNER DURING DESIGN PHASE OF PROJECT.
10. IF INLET OR OUTLET PIPES NEED TO HAVE GREATER THAN 4PSI JOINTS FOR FIELD PIPE CONNECTIONS, THE USE OF FLANGED ADAPTERS WILL NEED TO BE SPECIFIED BY E.O.R. OR OWNER DURING DESIGN PHASE OF PROJECT.
11. ALL TANK LATERALS (PIPE STUBS), RISERS & VENTS ARE CONSTRUCTED USING HDPE M294, DR32.5 & DR17 UNLESS OTHERWISE NOTED.
12. ALL PIPE DIMENSIONS ARE SUBJECT TO MANUFACTURERS TOLERANCES.
13. SYSTEM IS DESIGNED FOR H20 AND H25 LOADING.
14. CONSIDERATIONS FOR CONSTRUCTION EQUIPMENT LOADS MUST BE TAKEN INTO ACCOUNT. SEE DETAIL ON THIS PAGE.

SCOPE
THIS SPECIFICATION DESCRIBES DUROMAXX PIPE FOR USE IN MULTIPLE APPLICATIONS IN 30" (750 MM) THROUGH 120" (3000 MM) NOMINAL DIAMETERS.

DESCRIPTION
DUROMAXX IS A REINFORCED POLYETHYLENE PIPE WITH A SMOOTH WATERWAY WALL AND EXTERIOR PROFILE THAT IS REINFORCED WITH HIGH STRENGTH GALVANIZED STEEL RIBS. THE CONTINUOUS REINFORCING RIBS ARE COMPLETELY ENCASED WITHIN THE POLYETHYLENE PROFILE. DUROMAXX IS MANUFACTURED USING A HELICAL WINDING PROCESS THAT RESULTS IN A CONTINUOUSLY FUSION WELDED CIRCUMFERENTIAL LAP SEAM. THE PIPE PROFILE IS MANUFACTURED USING A HIGH QUALITY PRESSURE-RATED THERMOPLASTIC MEETING THE REQUIREMENTS OF ASTM F2562 “STANDARD SPECIFICATION FOR STEEL REINFORCED THERMOPLASTIC RIBBED PIPE AND FITTINGS FOR NON-PRESSURE DRAINAGE AND SEWERAGE”. FOR THE PURPOSE OF HYDRAULIC DESIGN, THE RECOMMENDED MANNING'S "N" VALUE SHALL BE 0.012 FOR PIPE DIAMETERS INCLUDED WITHIN THIS SPECIFICATION. PIPE LENGTH & ALL DIMENSIONS SHOWN ARE SUBJECT TO MANUFACTURERS TOLERANCES OF ±1% ACCORDING TO ASTM F2562.

MATERIAL PROPERTIES
VIRGIN HIGH DENSITY POLYETHYLENE PRESSURE-RATED RESINS ARE USED TO MANUFACTURE DUROMAXX PIPE. RESINS SHALL CONFORM TO THE MINIMUM REQUIREMENTS OF CELL CLASSIFICATION 345464 C AS DEFINED AND DESCRIBED IN THE LATEST VERSION OF ASTM D3350 “STANDARD SPECIFICATION FOR POLYETHYLENE PLASTICS PIPE AND FITTINGS MATERIALS”.

FITTINGS
ALL FITTINGS SHALL BE FABRICATED FROM DUROMAXX PIPE. ANY FITTINGS 30"Ø AND BELOW WILL BE HDPE PIPE.

ALL FABRICATION OF THE PRODUCT SHALL OCCUR WITHIN THE UNITED STATES.

INSTALLATION SPECIFICATION

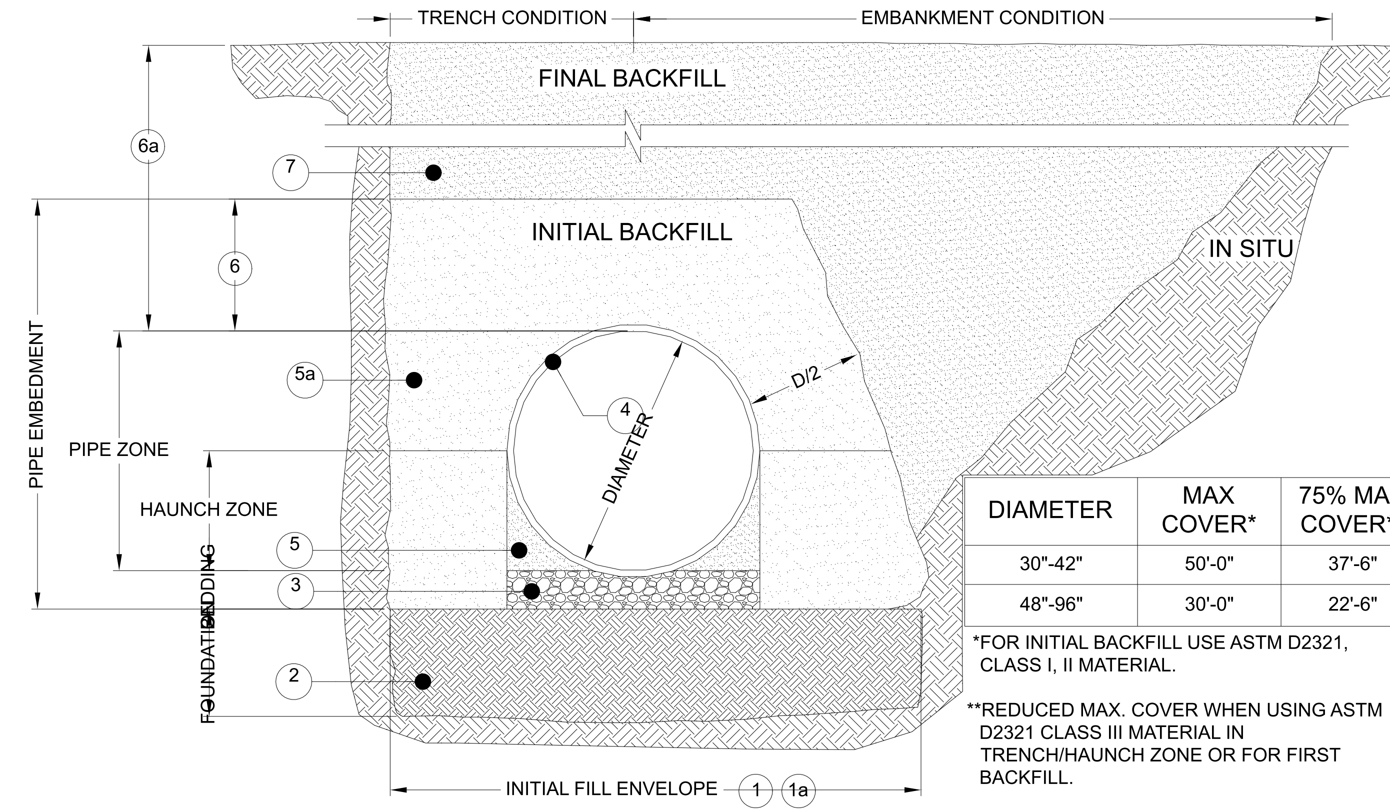
PRE-CONSTRUCTION MEETING

PRIOR TO INSTALLATION OF THE DRAINAGE SYSTEM A PRE-CONSTRUCTION MEETING SHALL BE CONDUCTED. THOSE REQUIRED TO ATTEND ARE THE SUPPLIER OF THE DRAINAGE SYSTEM, THE GENERAL CONTRACTOR, SUB CONTRACTORS AND THE ENGINEER.

INSTALLATION OF PIPE:

IT IS THE RESPONSIBILITY OF THE CONTRACTOR AND/OR PROJECT ENGINEER TO ENSURE THAT ALL QUESTIONS ABOUT INSTALLATION ARE ADDRESSED PRIOR TO APPROVAL OF SYSTEM. ALL DETAILS FOR INSTALLATION ARE LOCATED IN THIS DRAWING PACKAGE ANY QUESTIONS CONCERNING THESE STANDARD DETAILS CAN BE ADDRESSED BY YOUR CONTECH REPRESENTATIVE PRIOR TO APPROVAL.

BACKFILL SHALL BE PLACED TO THE PROPER ELEVATION OVER THE SYSTEM AS OUTLINED IN THE PLANS. MINIMUM COVER FOR CONSTRUCTION LOADING NEEDS TO BE DETERMINED BASED ON THE TYPE OF EQUIPMENT THAT IS PLANNED FOR CONSTRUCTION. PROPER COVER FOR CONSTRUCTION EQUIPMENT IS DETAILED IN THE CONSTRUCTION LOADING DIAGRAM LOCATED ON THIS PAGE.



DIAMETER	MAX COVER*	75% MAX COVER**
30"-42"	50'-0"	37'-6"
48"-96"	30'-0"	22'-6"

*FOR INITIAL BACKFILL USE ASTM D2321, CLASS I, II MATERIAL.

**REDUCED MAX. COVER WHEN USING ASTM D2321 CLASS III MATERIAL IN TRENCH/HAUNCH ZONE OR FOR FIRST BACKFILL.

1. MINIMUM TRENCH WIDTH MUST ALLOW ROOM FOR PROPER COMPACTION OF HAUNCH MATERIALS UNDER PIPE. MIN. WIDTH = (1.25 x DIAMETER) + 12" (FOLLOW ASTM D2321)

1a. MINIMUM EMBANKMENT WIDTH IS 3 PIPE DIAMETERS BUT NO LESS THAN 2' OUTSIDE OF SPRINGLINE.

2. FOUNDATION SHALL BE WELL CONSOLIDATED & STABLE.

3. ENGINEER TO DETERMINE IF BEDDING IS REQUIRED. BEDDING MATERIAL SHALL BE A RELATIVELY LOOSE MATERIAL THAT IS ROUGHLY SHAPED TO FIT THE BOTTOM OF THE PIPE, 4" TO 6" IN DEPTH.

4. DUROMAXX STEEL REINFORCED (SRPE) PIPE.

5. HAUNCH ZONE MATERIAL SHALL BE HAND SHOVELED OR SHOVEL SLICED INTO PLACE TO ALLOW FOR PROPER COMPACTION.

5a. INITIAL BACKFILL FOR PIPE EMBEDMENT MATERIAL TO MEET ASTM D2321 CLASS I, II, III OR APPROVED EQUAL, COMPACTED TO 90% STANDARD PROCTOR (NATIVE MATERIAL CAN BE UTILIZED THAT MEETS ASTM D2321 OR APPROVED EQUAL).
- ALL LIFTS PLACED IN CONTROLLED MANNER. TO PREVENT UNEVEN LOADING, IT IS RECOMMENDED THAT LIFTS NOT EXCEED 8" UNCOMPACTED LIFT HEIGHTS.

6. INITIAL BACKFILL ABOVE PIPE MAY INCLUDE ROAD BASE MATERIAL AND RIGID PAVEMENT (IF APPLICABLE), MINIMUM COVER STILL APPLIES, OTHERWISE:
6" MINIMUM FOR PIPE DIAMETERS 30" - 60"
12" MINIMUM FOR PIPE DIAMETERS 66" - 96"

6a. HEIGHT OF COMPACTED COVER PER DIAMETER FOR CONVENTIONAL HIGHWAY LOADS (DISTANCE MEASURED FROM TOP OF PIPE TO BOTTOM OF FLEXIBLE PAVEMENT OR TOP OF RIGID PAVEMENT):
12" MINIMUM FOR PIPE DIAMETERS 30" - 60"
18" MINIMUM FOR PIPE DIAMETER 66" - 72"
24" MINIMUM FOR PIPE DIAMETERS 84" - 96"

7. FINAL BACKFILL MATERIAL SELECTION AND COMPACTION REQUIREMENTS PER THE PROJECT PLANS, SPECIFICATIONS. ENGINEER OF RECORD.

NOTES:
- ENGINEER TO DETERMINE IF GEOTEXTILE SHOULD BE USED TO PREVENT SOIL MIGRATION.
- FOR MULTIPLE BARREL INSTALLATION THE RECOMMENDED STANDARD SPACING BETWEEN PARALLEL PIPE RUNS SHALL BE = PIPE DIA./2 OR 3' FOR PIPE DIAMETERS 72" AND LARGER. CONTACT YOUR CONTECH REPRESENTATIVE FOR NONSTANDARD SPACING.
- BACKFILL REQUIREMENTS SHALL FOLLOW ASTM D2321. IN THE EVENT OF DISCREPANCIES, ASTM D2321 SHALL SUPERCEDE THIS DETAIL.

INSTALLATION/BACKFILL DETAIL

NOT TO SCALE

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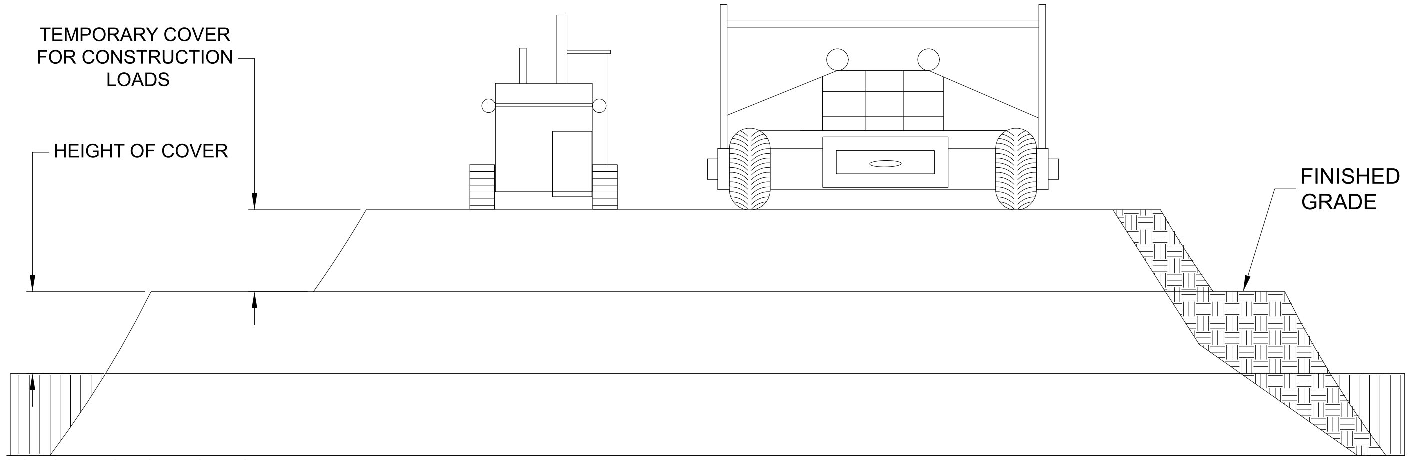
CONTECH DYODS DRAWING

DYO28355 Kia
SWQDv
Calabasas, CA
DETENTION SYSTEM

PROJECT No.: 18982	SEQ. No.: 28355	DATE: 3/13/2023
DESIGNED: DYO		DRAWN: DYO
CHECKED: DYO		APPROVED: DYO
SHEET NO.: D2		



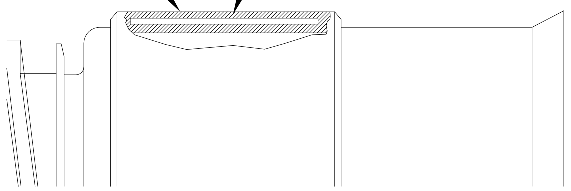
CONSTRUCTION LOADS:
FOR TEMPORARY CONSTRUCTION VEHICLE LOADS, AN EXTRA AMOUNT OF COMPACTED COVER MAY BE REQUIRED OVER THE TOP OF THE PIPE. THE HEIGHT-OF-COVER SHALL MEET THE MINIMUM REQUIREMENTS SHOWN IN THE TABLE BELOW. THE USE OF HEAVY CONSTRUCTION EQUIPMENT NECESSITATES GREATER PROTECTION FOR THE PIPE THAN FINISHED GRADE COVER MINIMUMS FOR NORMAL HIGHWAY TRAFFIC.

DIAMETERS	AXLE LOADS (kips)			
	18-50	50-75	75-110	110-150
	MINIMUM COVER (FT)			
12"-42"	2.0	2.5	3.0	3.0
48"-72"	3.0	3.5	3.5	4.0
84"-96"	3.0	3.5	4.0	4.5
108"-120"	3.5	4.0	4.5	5.0

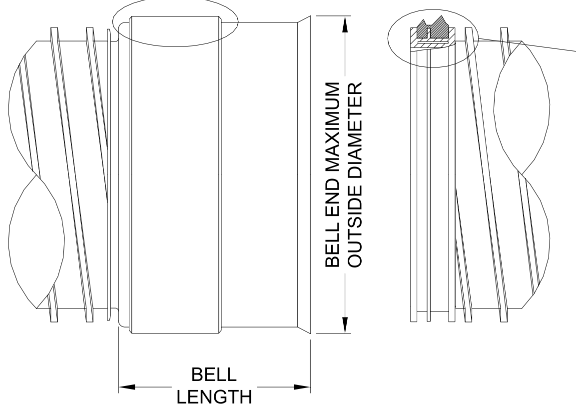
MINIMUM COVER MAY VARY, DEPENDING ON LOCAL CONDITIONS. THE CONTRACTOR MUST PROVIDE THE ADDITIONAL COVER REQUIRED TO AVOID DAMAGE TO THE PIPE. MINIMUM COVER IS MEASURED FROM THE TOP OF THE PIPE TO THE TOP OF THE** ***MAINTAINED **CONSTRUCTION ROADWAY SURFACE.**

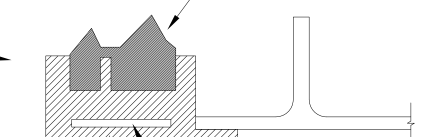
CONSTRUCTION LOADING DIAGRAM

N.T.S

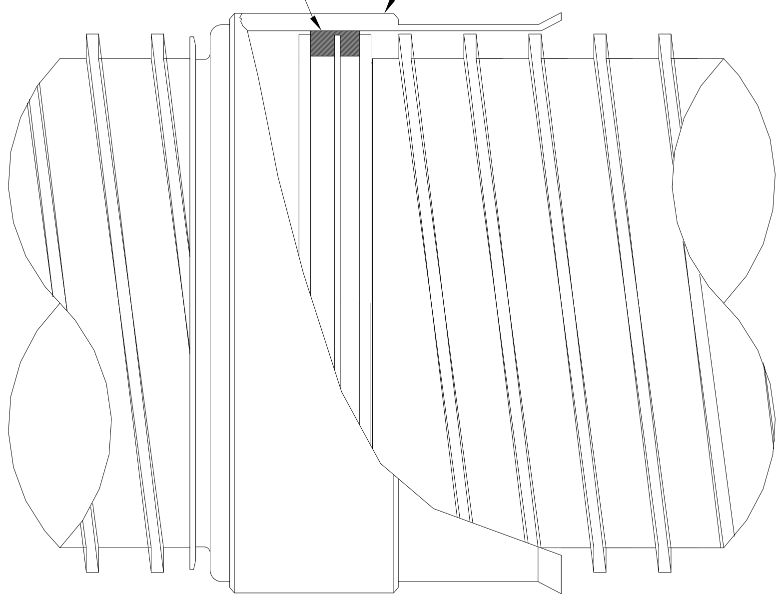


BELL DETAIL





SPIGOT DETAIL



NOMINAL PIPE DIA. (in)	BELL O.D. (in)	BELL LENGTH (in)-MAX
30	34.0	8.90
36	39.9	8.90
42	45.8	8.90
48	52.3	8.90
54	58.2	8.90
60	64.1	8.90
66	71.6	11.15
72	77.6	11.15
ABOVE JOINTS ARE 10.8 psi		
84	88.9	11.15
84" BELL & SPIGOT (SOIL TIGHT ONLY)		

HIGH PERFORMANCE (HP) JOINT DETAIL

NOT TO SCALE

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SWQDv
Calabasas, CA
DETENTION SYSTEM

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SHEET NO.: D3		

GEOTECHNICAL
PROFESSIONALS INC.

GEOTECHNICAL INVESTIGATION
PROPOSED KIA DEALERSHIP DEVELOPMENT
24460 CALABASAS ROAD
CALABASAS, CALIFORNIA

Prepared for:
Hello Auto Group
c/o Integrity Design & Construction Services
P.O. Box 1171
Solana Beach, California 92075

Prepared by:
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Project No. 3162.I

December 16, 2022

GEOTECHNICAL
PROFESSIONALS INC.

December 16, 2022

Hello Auto Group
c/o Integrity Design & Construction Services
PO Box 1171
Solana Beach, CA 92075

Attention: Jody Stout (jstout@integritydcs.com)

Subject: Report of Geotechnical Investigation
Proposed Kia Dealership Development
24460 Calabasas Road, Calabasas, California
GPI Project No. 3162.I

Dear Jody:

Transmitted herewith is an electronic copy of our geotechnical investigation report for the subject project. The report presents our evaluation of the foundation conditions at the site and recommendations for design and construction. This report will need to be submitted by you or your representative to the City for review and approval.

We appreciate the opportunity of offering our services on this project and look forward to seeing the project through its successful completion. Feel free to call us if you have questions regarding our report or need further assistance.

Very truly yours,
Geotechnical Professionals Inc.

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1.0 INTRODUCTION

1.1 GENERAL

This report presents the results of a geotechnical investigation performed by Geotechnical Professionals Inc. (GPI) for the proposed Kia Dealership in Calabasas, California. The geographical location of the site is shown on the Site Location Map, Figure 1.

1.2 PROPOSED DEVELOPMENT

The proposed development will consist of a new dealership development located at the south side of Calabasas Road, west of Parkway Calabasas. Based on our review of the provided conceptual plans by AHT Architects, the proposed development consists of a 2-story dealership building with rooftop parking, including showroom, offices, service department, and parking. The building footprint is approximately 18,375 square feet and will be founded at about the existing grade (Elevation 1080 feet) on the north end and extending to a depth of about 18 feet below grade on the south end.

Based on limited information provided, the maximum column and wall loads for the proposed structure are anticipated to be on the order of 400 kips and 12 kips per lineal foot, respectively.

The recommendations given in this report are based upon preliminary structural and grading information. We should be notified if the actual loads and/or grade changes are known to either confirm or modify our recommendations.

1.3 PURPOSE OF INVESTIGATION

The purpose of this investigation and report is to provide an evaluation of the existing geotechnical conditions at the site, as they relate to the design and construction of the proposed development. More specifically, this investigation was aimed at providing geotechnical recommendations for earthwork and design of foundations, retaining structures, and pavements.

1.4 PRIOR NEARBY GEOTECHNICAL INVESTIGATIONS

We performed a geotechnical investigation for a previously planned automobile dealership at the site and presented the results in a report dated July 6, 2016 (GPI, 2016). Our prior investigation included borings (B-1 to B-9), test pits (TP-1 to TP-6), and geologic mapping across the site. Two of the borings (B-1 and B-2) were drilled near the footprint of the currently planned dealership building with the remainder of the explorations performed to the south of the proposed building. We have incorporated our prior data and findings into this report.

Additionally, we requested the available on-site and nearby prior geotechnical investigations from the City of Calabasas. The City did not have records of prior on-site geotechnical studies but did provide copies of the prior geotechnical reports for nearby sites, which we reviewed as part of our investigation. These reports are included in the list of references herein.

2.0 SCOPE OF WORK

Our scope of work for this investigation consisted of review of available documentation, field exploration, laboratory testing, geologic and engineering analysis, and the preparation of this report.

The field exploration for this report consisted of two exploratory borings (B-101 and B-102). We have also included the field explorations from our previous investigation onsite which included nine exploratory borings and six test pits. The locations of the subsurface explorations are presented on the Site Plan, Figure 2.

The borings were drilled using predominately hollow-stem auger borings to depths ranging between 20 to 51 feet below existing grades. Two of the borings were drilled as part of our previous investigation using bucket auger equipment to depths of about 18 to 49 feet below existing grades. The larger diameter bucket auger borings were downhole logged by our Certified Engineering Geologist. The test pits were excavated with a backhoe to depths of 5 to 16 feet below grade to expose the subsurface soils for observation and sampling. A description of field procedures and logs of borings and test pits are presented in Appendix A.

Two infiltration tests were performed as part of our previous investigation in 2016. Test wells were installed adjacent to Borings B-1 and B-2. The wells were installed to depths of about 10 feet below the existing grade. Infiltration testing was performed in each well, with details of the testing presented herein, and test results presented in Tables 1.1 and 1.2, Borehole Infiltration Test Results.

Laboratory soil tests were performed on selected representative soil and bedrock samples as an aid in soil classification and to evaluate the engineering properties of the materials. The geotechnical laboratory testing program included determinations of moisture/density, shear strength (direct shear), compressibility (consolidation), expansion potential, compaction, gradation, R-value, and corrosion potential. Laboratory testing procedures and results are summarized in Appendix B.

R-value testing was performed by GeoLogic Associates, Inc. under subcontract to GPI. The test results are presented in Appendix B. Soil corrosivity testing was performed by HDR under subcontract to GPI. The results are provided in Appendix B of this report.

A geologic evaluation of the site was performed by our consulting Certified Engineering Geologist. The evaluation included downhole logging of two of our exploratory borings, observation during excavation of the test pits, surface mapping, and review of available published and unpublished data. The geologic-seismic evaluation is incorporated herein.

Engineering evaluations were performed to provide earthwork criteria, foundation, retaining structure, and slab design parameters, and assessments of seismic hazards. The results of our evaluations are presented in the remainder of this report.

3.0 SITE CONDITIONS

3.1 SURFACE CONDITIONS

The site is irregularly shaped and bounded by Calabasas Road and a separate lot to the north, an existing automobile dealership to the east, an ascending slope up to a street (Park Granada) to the south, and undeveloped land to the west. The site was formerly occupied by a commercial nursery, and several minor structures related to those operations are still present on the site. Also present on the site are both paved and unpaved access roads and numerous trees, including medium to very large oak trees.

The ground surface elevations within the area planned for development ranges from 1079 feet at the site entrance off Calabasas Road up to about Elevation 1110 feet at the southern edge of the planned development. The slope continues to ascend to the street above the planned development (Park Granada), to an elevation of about 1370 feet. The site topography is shown on the Site Plan, Figure 2, Site Geologic Plan, Figure 3, and Figure 4, Subsurface Cross Section.

The pavement observed during our fieldwork appeared to be in poor to fair condition. The existing pavement sections at our boring locations consisted of 2 to 5 inches of asphalt concrete over about 0 to 5 inches of aggregate base. The patio slab outside the former sales building at Boring B-1 consisted of 3 inches of portland cement concrete.

3.2 SUBSURFACE CONDITIONS

Our field explorations disclosed a subsurface profile generally consisting of undocumented fills overlying natural materials. Detailed descriptions of the materials encountered are shown on the Logs of Borings and Test Pits, Appendix A. The geologic conditions are also discussed in a following section.

Fill soils were encountered in some of our borings and test pits to depths of 1 to 10½ feet. The deeper fill soils were encountered in the lower elevations near Boring B-5 and Test Pit TP-2. The fill soils consisted predominantly of silty and sandy clays with varying amounts of shale fragments and caliche. The fill was not consistent with properly compacted fill in that densities were variable and moisture contents varied from slightly moist to very moist. Documentation regarding the placement and compaction of the fills was not available. The fill soils have a medium expansion potential and are anticipated to shrink and swell with changes in moisture content.

The natural materials encountered consisted of colluvium/alluvium over bedrock. The colluvium/alluvium extended to a depth of about 20 feet at the northern portion of the proposed building pad and about 6 to 10 feet on the southern end. The colluvium/alluvium consisted predominantly of sandy clay, silty clay, clayey silt, silt, and silty sand. These soils were generally stiff to very stiff and medium dense with relatively low densities and slightly porous in the upper 10 feet. Below 10 feet these materials were generally very stiff to hard. The moisture content of the soils ranged from slightly moist to wet. The underlying bedrock consisted of siltstone and sandstone that was found to be hard and dense to very dense (soil consistencies). The sandstone was found to generally be massive, poorly to moderately cemented, and contain infrequent layers of conglomerate. The siltstone was found to generally be massive to moderately bedded. Further details regarding the bedrock materials encountered are presented in the following Site Geologic Conditions section of this report.

The natural soils and siltstone have a medium expansion potential and will shrink and swell with changes in moisture content. Select corrosivity testing indicates that the on-site soils are moderately corrosive to concrete (sulfate content) and reinforcing steel (chloride content). These results are consistent with the findings at nearby sites.

3.3 SITE GEOLOGIC CONDITIONS

The project site consists of an irregular-shaped parcel in moderate relief hillside terrain in the Calabasas area of Los Angeles County. The property is on the south side of an unnamed canyon, where the 101 Freeway and Calabasas Road have been constructed. The property consists of relatively small ridgelines and canyons that descend from a higher ridgeline to the south to the main drainage where the freeway is located.

Regional geologic maps (Dibblee, T.W. Jr., 1992, Geologic Map of the Calabasas Quadrangle) indicate that the site area is underlain by Tertiary age Sedimentary rocks typical of this portion of the Santa Monica Mountains. Specifically, the site is mapped as being underlain by bedrock of the Upper Topanga formation, generally dipping at moderate inclinations to the northwest and northeast near the crest and east limb of a northerly plunging anticline, as indicated on the attached Regional Geologic Map, Figure 3.

Our field investigation, which consisted of a total of eleven borings, six backhoe pits, and geologic mapping of available bedrock exposures in road cuts on the site, generally confirmed the bedrock geology of the site. Soil and geologic conditions determined by our field investigation, as well as the locations of exploratory excavations, are shown on Figure 3, and Subsurface Cross Section, Figures 4. Five of the borings were drilled utilizing a hollow stem auger in the canyon bottoms to determine the thickness and engineering characteristics of the surficial deposits overlying the bedrock. The surficial deposits were mapped as colluvium/alluvium undifferentiated and consisted primarily of silty and sandy clays. These deposits were typically 5 to 10 feet thick in the higher portions of the site and reached a maximum thickness of approximately 20 feet in the area near Calabasas Road in Boring B-1 and B101. Three backhoe test pits were also excavated in the canyon areas to further delineate the occurrence and depth of the surficial deposits.

Two of the borings were drilled utilizing a 24-inch diameter bucket or spiral auger to facilitate downhole logging to determine geologic structure in bedrock areas. Geologic structure determined by downhole logging was augmented by logging of three backhoe excavations and geologic mapping of bedrock exposures. As shown on the attached Site Geologic Map, bedding in the bedrock generally strikes nearly east-west and dips at moderate inclinations to the north, northeast, and northwest. In Boring B-7, the dip of bedding was observed to be steeper than in other areas, possibly influenced by east-west trending faults observed in Borings B-4 and B-9. As observed in the surface outcrops, borings and backhoe excavations, the Upper Topanga formation consisted of generally fine-grained, massive to vaguely bedded sandstone with infrequent conglomerate beds, and massive to thinly bedded siltstone.

The native soil and bedrock are overlain in limited areas with generally thin fill deposits placed during the previous use of the site as a nursery. The fills should be considered as uncompacted.

3.4 GROUNDWATER AND CAVING

Groundwater seepage was observed at depths of 34 feet in Boring B-6, 31 feet in Boring B-8, and 46 feet in Boring B-9, corresponding to elevations of about 1103 to 1108 feet. Groundwater was not encountered in the remainder of our field explorations, including the four borings performed within the currently proposed building area. The seepage was observed in the sandstone and appears to be perched on underlying siltstone or more cemented sandstone layers. The possibility exists that some minor seepage of groundwater may be encountered in the excavation and walls below grade planned for the southern portion of the proposed building that will extend into the bedrock. The local groundwater source is likely from irrigation practices above the site and from infiltration of rain runoff. The depth to groundwater and groundwater seepage can be expected to vary annually.

Caving was not encountered in our borings.

4.0 CONCLUSIONS AND RECOMMENDATIONS

4.1 GENERAL

Based on the results of our investigation, it is our opinion that, from a geotechnical viewpoint, it is feasible to develop the site as proposed, provided the recommendations contained herein are incorporated into the design and construction of the project.

Furthermore, in accordance with the County of Los Angeles Statement 111, it is our opinion that the project will be safe for its intended use against hazard from landslide, settlement, or slippage and the project will have no adverse effect on the stability of the site or adjoining properties.

The major geotechnical constraints related to the proposed construction are as follows:

- Based on the sloping ground surface, the finished floor of the proposed building will be at about the existing grade at the northern building limit and at about 18 feet below the existing grade at the southern building limit. Given the gradual descending slope of the bedrock surface towards the north, the southern portion of the finished building pad will be subterranean and extending into the undisturbed bedrock while the northern portion of the building will be underlain by up to about 20 feet of soil over the bedrock surface. Given the differing foundation depths and supporting materials, we recommend supporting the proposed building on a mat foundation underlain by properly compacted fill to mitigate the potential differential settlement.

- Undocumented and low-density fills are present at the site ranging from 1 to 10½ feet below existing grades, with the deepest fill encountered in the vicinity of our Borings B-2, B-102 and B-5 and Test Pit TP-2, which are located north of the planned building. These fills are not suitable for support of the proposed structures or floor slabs. We recommend that undocumented fills be removed and replaced as properly compacted fill within the building pad, where not removed by cut. The depths of removal and details regarding excavations are provided in the “Earthwork” section of this report.

- The upper natural soils immediately beneath the fill soils have variable strength and consolidation characteristics within the upper 10 feet of the ground surface. To provide uniform support for the planned building and avoid a soil/bedrock contact immediately beneath the mat foundation, the upper natural materials encountered at the completion of the fill removal or excavation for the subterranean portion of the building should be removed and replaced with properly compacted fills to provide uniform support for the mat foundation. The depths of removal and details regarding excavations are provided in the “Earthwork” section of this report.

- Foundations for retaining walls and minor structures such as screen walls or retaining walls less than 5 feet high may be supported on shallow foundations established in properly compacted fill or undisturbed natural soils. In-place undocumented fill soils should be removed in their entirety beneath the foundations. The depths of removal and details regarding excavations are provided in the “Earthwork” section of this report.

- Clays encountered at the site exhibit a medium expansion potential (expansion index of 54). To help mitigate the expansion potential, the clay soils should not be placed as compacted fill within 2 feet of pedestrian concrete hardscape subgrade. Building floor slabs will not be impacted because of the plans for a mat foundation. The soils placed to support pedestrian hardscape should consist of granular, non-expansive (E.I. of 20 or less) on-site or imported soils. Such materials were encountered within our explorations (sandstone and silty sands) but selective grading will be required to identify and stockpile these soils for use in capping these areas.

- As noted in the geologic assessment of the site, the bedrock encountered in our explorations was noted to be dipping to the northwest and northeast, which will create an adverse condition for north facing retaining walls and excavations for the southern subterranean portion (upslope) of the building. As such, the stability of excavations extending into the bedrock material is anticipated to be adversely affected by the potential for adverse bedding. We have provided recommendations herein to address the lateral earth pressures related to the adverse bedding, as well as earthwork accommodations to maintain stable excavations. We recommend that our Geologist be on-site during the excavation to confirm the actual subsurface conditions encountered.

- Groundwater seepage was encountered at depths of 31 to 46 feet below the existing grades in our explorations, corresponding to Elevations 1103 to 1108 feet. Because the planned building will be located in the lowest area of the site with a partial subterranean condition established at about Elevation 1080 feet, the groundwater should be accounted for in performing the required excavations and in the long-term performance of the subterranean portions of the structure.

- Based on our field testing, the on-site soils and bedrock do not appear to be conducive to infiltration. Tested infiltration rates in wells established at depths of about 10 feet near our Borings B-1 and B-2 were found to be 0 to 0.1 inch/hour.

4.2 SEISMIC CONSIDERATIONS

4.2.1 General

The site is in a seismically active area typical of Southern California and is likely to be subjected to strong ground shaking due to earthquakes on nearby faults.

We assume the seismic design of the proposed development will be in accordance with the California Building Code, 2022 edition. For the 2022 CBC, a Site Class C may be used.

4.2.2 Strong Ground Motion Potential

Based on published information (geohazards.usgs.gov), the most significant active faults in the proximity of the site are the Malibu Coast and Santa Monica faults, which are located about 7½ and 9½ miles from the site, respectively.

During the life of the project, the site will likely be subject to strong ground motions due to earthquakes on nearby faults. Based on the USGS website (earthquake.usgs.gov), we computed that the site could be subjected to a peak ground acceleration (PGA_M) of 0.79g for a mean magnitude 6.8 earthquake. This acceleration has been computed using the mapped

Maximum Considered Geometric Mean peak ground acceleration from the ASCE 7-16 (for 2022 CBC) and a site coefficient (F_{PGA}) based on Site Class. The predominant earthquake magnitude was determined using a 2-percent probability of exceedance in a 50-year period, or an average return period of 2,475 years. The structural design will need to incorporate measures to mitigate the effects of strong ground motion.

The corresponding seismic design parameters from the CBC are as follows:

<u>2022 CBC:</u>

S_S = 1.59g	$S_{MS} = F_a * S_S$ = 1.91g	S_{DS} = 2/3 * S_{MS} = 1.27g
S_1 = 0.56g	$S_{M1} = F_V * S_1$ = 0.81g	S_{D1} = 2/3 * S_{M1} = 0.54g

The above seismic code values should be confirmed by the Project Structural Engineer using the value above and the pertinent internet website and tables from the building code.

4.2.3 Potential for Ground Rupture

There are no known active faults crossing or projecting through the site. The site is not located in an Alquist-Priolo Earthquake Fault Zone. Therefore, ground rupture due to faulting is considered unlikely at this site.

4.2.4 Liquefaction

The site is not located within an area mapped by the State of California as having a potential for soil liquefaction (CGS, 1997). Groundwater seepage was encountered at depths of 31 to 46 feet in three of our recent explorations. The seepage was encountered in dense sandstone. Historical information is limited as the bedrock material underlying the site is not considered to be water bearing. Excluding the site vicinity from the potential liquefaction zone was based primarily on the suggestion that the bedrock materials have likely reached a state of consolidation that would preclude liquefaction based on their geologic age, and the colluvium/alluvium soils are well above the encountered groundwater.

Based on our evaluation, we conclude that the potential for liquefaction to adversely impact the planned project is very low. The potential seismic-induced liquefaction settlement will be less than ¼-inch.

4.2.5 Seismic Ground Subsidence

Seismic ground subsidence (not related to liquefaction induced settlements), occurs when loose, granular (sandy) soils above the groundwater are densified during strong earthquake shaking. Earthquake-induced seismic subsidence during a strong earthquake is not anticipated to adversely affect the planned project because of the planned subterranean construction and stiff to very stiff cohesive soils overlying the bedrock.

4.3 SLOPE AND WALL STABILITY

The site is located within a north-south trending canyon extending up from Calabasas Road to Park Granada. The dealership development is planned in the lower portion of the site where the north-south ground surface slopes gradually up to the south at an inclination of about 8:1 (h:v), with localized slopes of up to 2:1. The area of the planned development was previously used as a commercial nursery, and much of the ground surface has been terraced and graded for access. The natural slopes at the site above the planned development are covered with moderate to heavy growth (brush and trees) and inclined at about 2¼:1 to 4:1.

The site is in an area designated as a hillside area by the County of Los Angeles (County of Los Angeles, 1990). The state (CGS, 1997) indicates that there are areas of potential earthquake-induced landslides in the vicinity of the site, but that the subject site is not within such a zone. There are no known landslides adjacent to the site, nor is the site within or in the path of known or potential landslides. The results of our site specific geotechnical and geologic investigation indicates that the on-site slopes are grossly stable.

4.3.1 Gross Stability

Based on our site reconnaissance and review of available data, there are no known landslides at the site or on immediately adjacent sites. Based on our geologic investigation, the material in the lower portions of the site consist of Upper Topanga sandstones and siltstones that were found to be predominantly massive to thinly bedded. Where measured, bedding in these deposits was generally to the northwest and northeast, which is generally adverse, but also predominantly at relatively flat inclinations (12 to 18 degrees to the horizontal). Steeper inclinations were encountered in our downhole logging of one boring (Boring B-7), but these inclinations were encountered in a siltstone layer within the upper 12 feet, which was underlain by a massive sandstone deposit. Bedding information is presented in Figure 3, with apparent bedding inclinations shown on Figure 4. Regional geologic mapping (Dibblee, 1992) indicates that the predominant bedding is to the north, but also indicates variable inclinations in the upper areas of the slopes at the site, including favorable inclinations dipping to the south. These conditions are presented on Figure 6.1 and 6.2, Regional Geologic Map and Cross Section.

Based on our findings, the natural slopes at the site are considered to be grossly stable, with the potential for slope instability to adversely affect the planned development to be low. The planned development will not require significant cuts to reach the planned finished grades except for the excavation required for the southern subterranean portion of the building, which will extend to about 20 feet in depth below the existing grades. Such excavations will expose both adverse and favorable bedding conditions. In general, we anticipate north facing cut slopes (southern building walls) to expose adverse bedding, with the predominant adverse bedding inclinations being relatively flat (12 to 18 degrees).

4.3.2 Surficial Stability

We did not observe evidence of surficial slope failure or debris flows during our geologic site reconnaissance or review of aerial photographs. We performed surficial stability analyses for the slopes that will remain above the planned development. We assumed that the in-place soils would be saturated with a failure surface extending parallel and 3 feet below the ground surface. Based on our analyses, the natural soils will have a factor of safety of at least 1.5 with respect to surficial stability. This finding corresponds to the conditions observed on-site.

4.3.3 Slope and Wall Stability Analyses

We used shear strength parameters in our slope stability analyses that were developed by testing saturated samples of the colluvium/alluvium and bedrock. We tested selected bedrock samples by re-shearing the samples multiple times to simulate along bedding strengths. We also reviewed the along-bedding shear strength values published in State documents for similar bedrock materials. The shear strength values used in our analyses are presented in the following table:

Material Type	Unit Weight (pcf)	Friction Angle (degrees)	Cohesion (psf)
Siltstone (along Bedding)	120	20	300
Siltstone (across bedding)	120	25	350
Sandstone (massive)	120	37	350
Colluvium/Alluvium	120	22	175
Compacted Fill	120	25	250

Our analyses of the planned slopes and walls/shoring included evaluating the subsurface materials using both wedge-type and circular failure surfaces. Using the soil shear strengths values above, we modeled the bedrock as having along bedding shear strengths for failure surfaces dipping out of slope between 12 and 18 degrees for north-facing slopes (south facing retaining walls). We considered east- and west-facing cuts as exposing favorable bedding and across bedding shear strengths.

For determination of lateral earth pressures, we determined the stability of the planned cut, and then determined the required point load to provide a factor of safety of 1.5. We then converted the point load into an equivalent fluid pressure to determine an appropriate lateral earth pressure to use for the design of temporary shoring and permanent retaining walls. Lateral earth pressures are presented in the Retaining Structures and Shoring section of this report.

4.4 EARTHWORK

The earthwork anticipated at the project site will consist of clearing and grubbing, excavations, subgrade preparation, and the placement and compaction of fill.

4.4.1 Clearing and Grubbing

Prior to grading, the areas to be developed should be stripped of landscaping, cleared of demolition debris, old foundations, pavements, and utilities. Deleterious material generated during the clearing operation should be removed from the site. Where appropriate, existing underground utilities should be removed in their entirety and properly capped. Should cesspools or other buried obstructions be encountered in the building areas during construction, they should be removed in their entirety. The resulting excavations should be backfilled as recommended in the "Subgrade Preparation," “Material for Fill,” and "Placement and Compaction of Fill" sections of this report. As an alternative, cesspools can be backfilled with lean sand-cement slurry. At the conclusion of the clearing operations, a representative of GPI should observe and accept the site prior to any further grading.

4.4.2 Excavations

Excavation at the site will include removal of existing undocumented fill and low density/compressible natural soils in building and conventional retaining wall areas, excavation to finish subgrade, footing excavations, and trenching for utility lines.

Building Pad, Pavement, and Minor Structures

Prior to construction of the building supported on a mat foundation, existing undocumented fills and the upper natural soils in the proposed building areas should be removed and replaced as properly compacted fill when not removed by the planned cuts. Undocumented fills were encountered in our explorations in the lower portion of the site (Borings B-1, B-2, B-5, B-101, B-102, and Test Pit TP-2) to depths of up to 10½ feet below existing grades. The upper natural soils encountered in the lower portion of the site have variable strength and consolidation characteristics. As such, these soils are not considered to be suitable for direct support of the planned mat foundations and should be included in the required overexcavation within the building areas. For planning purposes, removals should extend to depths of 12 feet below the existing grade or 4 feet below the base of the mat foundation, whichever is deeper. The purpose of the removals beneath the base of the mat foundation in the subterranean portion of the building is to reduce the potential differential settlements across the soil/bedrock contact. In doing so, the required depth of excavation for the southern portion of the building will be increased beneath the mat to allow for compacted fill placement with the result being more uniform support for the mat foundation.

For planning purposes, removals for retaining walls taller than 5 feet should remove the in-place undocumented fill and extend to depths of at least 7 feet below the existing grade or 3 feet below the base of the wall footing, whichever is deeper. As an alternative, tall retaining walls footings can be established directly within the undisturbed bedrock if the entire wall footing will extend deep enough to extend at least 1 foot into the bedrock.

Removals for minor structures, such as retaining walls less than 5 feet high or screen walls, should extend at least 4 feet below the existing grade or 2 feet below the base of the foundations, whichever is deeper. In concrete pedestrian hardscape areas, removals should extend at least 2 feet below the planned finished subgrade to allow for the placement of relative non-expansive, granular soils. In pavement areas, removals should extend at least 1 foot below the existing grade or the finished subgrade, whichever is deeper.

The actual depths of removals will need to be determined during grading in the field by a representative of GPI.

Lateral Limits

The base of removals should extend laterally beyond the building line or edge of footings a minimum distance of 5 feet or the depth of overexcavation/compaction below foundations (i.e., a 1:1 projection below the bottom edge of the mat foundation or footing), whichever is greater. For the northern portion of the building where relatively deep removals are anticipated, the lateral limits of the base of the removals will be governed by the 1:1 projection below the edge of the mat. Building lines include canopies, loading docks, and other foundation supported improvements. The lateral limits of removals should be confirmed and certified by the project surveyor. GPI does not practice surveying; therefore, we cannot confirm lines, grades, or limits of earthwork.

Existing Utilities

Where not removed by the aforementioned excavations, existing utility trench backfill should be removed and replaced as properly compacted fill within the building pad. The limits of removal should be confirmed in the field. We recommend known utilities be shown on the grading plan.

Caving Potential and Cuts

Temporary construction excavations may be made vertically without shoring to a depth of 5 feet below adjacent grade. The inclination required for deeper excavations will be dependent on the direction of the cut relative to the direction of bedding of the siltstone. We recommend deeper cuts be properly shored or inclined as follows:

- Within the alluvium/colluvium soils, cuts up to 9 feet can be made at an inclination of ¾:1 (horizontal to vertical), cuts up to 14 feet sloped back to at least 1:1, and cuts up to 19 feet sloped back to 1½:1 or flatter.

- For east- or west-facing cuts in the bedrock materials, cuts up to 16 feet within the existing bedrock may be inclined at ¾:1, cuts up to 24 feet may be inclined at 1:1, and cuts up to 32 feet may be inclined at 1¼:1. Our Geologist should observe the excavations as they proceed to confirm favorable bedding conditions and the absence of potentially adverse shears in the bedrock.

- For north-facing cuts in the bedrock materials, cuts up to 10 feet within the existing bedrock may be inclined at 1:1, cuts up to 16 feet may be inclined at 1¼:1, and cuts up to 27 feet may be inclined at 1½:1 or flatter. Our Geologist should observe the excavations as they proceed to confirm the bedding conditions exposed are consistent with our initial findings. Flatter slope inclinations may be required based on the conditions exposed.

Surcharge loads should not be permitted within a horizontal distance equal to the height of cut from the top of the excavation or 5 feet from the top of the slopes, whichever is greater, unless the cut is properly shored. Excavations that extend below an imaginary plane inclined at 45 degrees below the edge of an adjacent existing building or settlement sensitive structure should be properly shored to maintain support of such adjacent elements. Excavations and shoring systems should meet the minimum requirements given in the most current State of California Occupational Safety and Health Standards.

Details regarding slope related cuts and fills are presented on Figure 7, Slope Buttress or Replacement Fill Detail, Figure 8, Benching Detail, and Figure 9, Side Hill Cut Pad Detail. Buttress fills should include a subdrain as outlined in the details.

Slot Cuts

Where space is not available for open cuts of deeper removals along property lines and adjacent to existing improvements, shoring or slot cuts will be required. Recommendations for shoring are provided in the "Retaining Structures and Shoring" section of the report. Removals that will undermine existing adjacent pavements or hardscape may utilize "ABC" slot cuts to depths not greater than 12 feet. "ABC" slot cuts adjacent to the public right-of-way where traffic loads will be present should account for a live-load traffic surcharge. Slot cut recommendations for unsurcharged and surcharged conditions are provided below:

- Unsurcharged slots up to 8 feet in height should not be wider than 8 feet.
- Unsurcharged slots up to 12 feet in height should not be wider than 6 feet.
- Surcharged slots (subject to 250 psf traffic loads) up to 8 feet in height should not be wider than 6 feet.
- Surcharged slots (subject to 250 psf traffic loads) up to 12 feet in height should not be wider than 4 feet.

The slot cuts should be backfilled immediately to finished grade prior to excavation of the adjacent four slots (two on each side of the excavated slot). Although not anticipated, the allowable width of the slots adjacent to existing structures that will surcharge the excavation can be provided when additional details of the surcharge loads are provided. A test slot should be performed prior to production slots to confirm the stability of the planned cuts.

Rippability

Based on our site-specific explorations, we anticipate that the on-site materials can generally be excavated using conventional grading equipment. We did encounter localized hard, cemented layers in some of our borings in the upper portion of the site, but these materials were outside of the proposed limits of the new building and parking areas. If plans change to include the southern portion of the overall site, these materials will likely require special excavation equipment and processes, such as heavy ripping, and should be considered high cost/low production material to excavate.

4.4.3 Subgrade Preparation

Prior to placing fills, the subgrade should be scarified to a depth of 12 inches, moisture-conditioned, and compacted to at least 90 percent (95 percent for granular soils) of the maximum dry density in accordance with ASTM D1557 and to a firm and unyielding condition.

Localized areas of very moist soils (4 to 8 percent above the optimum moisture content) were encountered within the near surface soils in our Borings B-1, B-5, B-101 and B-102. Such materials are anticipated to yield under heavy rubber-tired equipment and will likely require stabilization to support compaction efforts. For planning purposes, we anticipate about one-third of the exposed subgrade areas in the planned pavement areas in the lower site elevations will require stabilization. Such measures may consist of mechanically drying the wet soils and compacting using steel-wheel or track equipment, or the placement of 12 inches of crushed miscellaneous base prior to construction of the planned pavement section or placement of compacted fill soils. A thicker layer of aggregate base or placement of a suitable geogrid material may be required if the exposed subgrade is disturbed to the point of yielding (i.e. “pumping”). Where wet soils are encountered, subgrade processing will be evaluated in the field by GPI.

4.4.4 Material for Fill

In general, the on-site soils are suitable for use as compacted fill. However, the on-site clays should not be used as retaining wall backfill or within the upper 2 feet of concrete pedestrian hardscape areas. Additionally, drying and mixing of the on-site materials will be required to obtain near optimum moisture conditions prior to placement and compaction. Given the clayey nature of the soils and presence of siltstone and cemented sandstone, processing times will likely take longer than would with more granular soils. In addition, grading during the rainy season will likely result in more challenges with moisture conditioning the on-site soils.

Provided it is acceptable to the reviewing governmental agencies and owner, crushed, inert demolition debris, such as concrete and asphalt, may be used in fills provided it is crushed to a well graded mixture with maximum particle size of 1½ inches and blended with the on-site soils.

Imported fill material should be predominately granular (contain less than 40 percent fines-portion passing the No. 200 sieve) and non-expansive (an Expansion Index of 20 or less). The import soils should contain sufficient fines/binder to be stable in open excavations, as well as be able to support construction equipment without rutting. GPI should be provided with a sample (at least 50 pounds) and notified of the location of soils proposed for import at least 72 hours in advance of importing. Each proposed import source should be sampled, tested and accepted for use prior to delivery of the soils to the site. Soils imported prior to acceptance by GPI may be rejected if not suitable.

Both imported and existing on-site soils to be used as fill should be free of deleterious debris and particles larger than 6 inches in diameter. Within the footing depths for the building pad and retaining walls, soil particles should be 3 inches in diameter or less, precluding the placement of cobble-sized particles.

In backfill areas where mechanical compaction of soil backfill is impractical due to space constraints, sand-cement slurry may be substituted for compacted backfill. The slurry should contain two sacks of cement per cubic yard and have a maximum slump of 5 inches.

If open-graded rock is used as backfill, the material should be placed in lifts and mechanically densified. Open-graded rock should be separated from the on-site soils by a suitable filter fabric (Mirafi 140N or equivalent).

4.4.5 Placement and Compaction of Fills

Fill soils should be placed in horizontal lifts, moisture-conditioned, and mechanically compacted to at least 90 percent (siltstone, silts and clays) of the maximum dry density (ASTM D1557). Fills placed at depths greater than 10 feet below finished grades should be compacted to at least 92 percent of the maximum dry density. Additionally, granular soils (sandstone, sands, silty sands, and clayey sands) should be compacted to at least 95 percent. The optimum lift thickness will depend on the compaction equipment used and can best be determined in the field. The following uncompacted lift thickness can be used as preliminary guidelines.

Plate compactors, track equipment	4-6 inches
Small vibratory or static rollers	6-8 inches
Scrapers, heavy loaders, large vibratory rollers	8-12 inches

The maximum lift thickness should not be greater than 12 inches. Each lift should be thoroughly compacted and accepted prior to placement of the next lift of fill.

The moisture content of the clayey fill materials should be within 2 to 4 percent over the optimum moisture content (pumping imminent) to readily achieve the required degree of compaction and reduce the potential for moisture related swell or subsidence. For the sandy fill materials, the moisture content should be within 0 to 2 percent over the optimum moisture content. The moisture content of existing near surface soils, in general, is variable and will require both drying and moistening prior to compaction. Earthwork contractors should include moisture conditioning of the existing soils prior to recompaction in the bids.

During backfill of excavations, the fill should be properly benched into the construction slopes as it is placed in lifts.

4.4.6 Shrinkage and Subsidence

Shrinkage is the loss of soil volume caused by compaction of fills to a higher density than before grading. Subsidence is the settlement of in-place subgrade soils caused by loads generated by large earthmoving equipment. For earthwork volume estimating purposes, an average shrinkage value of 20 to 25 percent and subsidence of 0.1 feet may be assumed for the existing fill and colluvium/alluvium soils. Significant shrinkage or subsidence is not anticipated for the bedrock materials. These values are estimates only and exclude losses due to removal of vegetation, debris, or existing underground structures. Actual shrinkage and subsidence will depend on the types of earthmoving equipment used and should be determined during grading.

4.4.7 Trench and Retaining Wall Backfills

Utility trench and wall backfill should be mechanically compacted in lifts. Wall backfill should consist of the on-site or imported silty sands or sands. As detailed previously, some moisture conditioning of the on-site soils should be anticipated prior to replacement as trench backfill. Lift thickness should not exceed those values given in the "Compacted Fill" section of this report. Jetting or flooding of backfill materials should not be permitted. A representative of GPI should observe and test trench and wall backfill as they are placed.

4.4.8 Observation and Testing

A representative of GPI should observe excavations, subgrade preparation and fill placement activities. Sufficient in-place field density tests should be performed during fill placement to evaluate the overall compaction of the soils. Soils that do not meet minimum compaction requirements should be reworked and tested prior to placement of additional fill.

Our geologist should observe the conditions exposed during the mass excavation. If conditions are different than expected, alternate recommendations, based on the actual conditions encountered, may be required.

4.5 FOUNDATIONS

4.5.1 General

Because the proposed building will be partially subterranean and extending into the bedrock in the southern subterranean portion while being underlain by up to about 20 feet of properly compacted fill in the northern at-grade portion, we evaluated several foundation alternatives.

Included in these alternatives was the use of deep foundations or ground improvement methods such as rammed aggregate piers for the northern at-grade portion, or performing remedial grading and supporting the building on a more uniform mat foundation across both the northern and southern portions. As a result of our evaluation, we recommend performing the remedial earthwork (overexcavation and recompaction of the undocumented fill and a portion of the natural soils and bedrock) outlined previously in the Earthwork section of the report and supporting the building on a mat foundation.

Retaining walls and minor structures may be supported on conventional isolated and/or continuous shallow footings, provided the subsurface soils are prepared in accordance with the recommendations given in this report.

New footings should be located beyond a 1:1 plane drawn upward from the base of new or existing retaining walls, or the wall should be designed for the additional surcharge load.

4.5.2 Mat Foundation - Building

Allowable Bearing Capacity and Elastic Modulus

The allowable bearing pressure for a mat foundation is generally not the governing geotechnical design issues as compared to the anticipated settlement. At this preliminary stage, estimate static mat foundation pressures for the proposed dealership building are not yet available. GPI should be provided with a detailed plot of the anticipated mat bearing pressures for our review when those plans are available.

For elastic design of the mat foundation, a preliminary modulus of subgrade reaction (k-value) of 120 pounds per cubic inch (pounds per square inch per inch of deflection. This value is for a 1-foot by 1-foot square loaded area and should be adjusted for the area of the mat foundation using appropriate elastic theory. Using generally accepted methods and our site-specific consolidation test results, we recommend using a value of approximately 30 pci for the adjusted k-value in designing the mat foundation. As previously discussed, we should be provided with the anticipated mat pressures when they are developed so that we can review and confirm the recommendations provided as well as provide an estimate for the anticipated maximum settlements for the mat foundations.

The allowable soil bearing pressure will be significantly greater than the average bearing pressures required for the mat foundation discussed above. For preliminary design purposes, an average allowable bearing pressure of 2,500 pounds per square foot (psf) may be used. At localized areas of the mat, such as columns and point-of-load applications along exterior walls, a static allowable bearing pressure of up to 4,000 psf may be used. These allowable bearing pressures are for dead-plus-live loads and may be increased one-third for short-term, transient, wind and seismic loading.

Settlement

Based on the load information assumed for the building (column loads on the order of 400 kips and wall loads on the order of 12 kips per lineal foot corresponding to mat pressures on the order of 2,500 psf), the total static foundation settlement for the mat foundation is estimated to range from ¼ to 1 inch with maximum differential settlements expected to be on the order of ¾-inch.

4.5.3 Shallow Footings – Retaining Walls and Minor Structures

Allowable Bearing Capacity and Footing Depths

Based on the shear strength and elastic settlement characteristics of the recompacted on-site soils (new fills), a static allowable bearing pressure of up to 3,000 pounds per square foot (psf) may be used for retaining wall and minor structure footings supported on properly compacted fill. Based on the shear strength and elastic settlement characteristics of the undisturbed on-site bedrock, a static allowable bearing pressure of up to 6,000 pounds per square foot (psf) may be used for continuous footings that extend at least 1 foot into the undisturbed bedrock (wall footings should be supported entirely in properly compacted fill if it cannot be supported entirely in the undisturbed bedrock).

These bearing pressures are for dead load-plus-live loads, and may be increased one-third for short-term, transient, wind and seismic loading. The actual bearing pressure used may be based on economics and structural loads and will determine the minimum width for footings as discussed below. The maximum edge pressures induced by eccentric loading or overturning moments should not be allowed to exceed these recommended values.

The following minimum footing widths and embedments are recommended for the corresponding allowable bearing pressure in the properly compacted fill.

Footings on Properly Compacted Fill

Static Bearing Pressure (psf)	Minimum Footing Width (inches)	Minimum Footing* Embedment (inches)
3,000	36	24
2,500	24	24
2,000	24	18
1,500	18	18

* Refers to minimum depth below lowest adjacent grade.

Minimum footing widths and depths of 18 inches should be used even if the actual bearing pressure is less than 1,500 psf for footings supported in properly compacted fill.

The following minimum footing widths and embedments are recommended for the corresponding allowable bearing pressure in the undisturbed bedrock.

Footings on Undisturbed Bedrock

Static Bearing Pressure (psf)	Minimum Footing Width (inches)	Minimum Footing* Embedment (inches)
6,000	48	24
5,000	36	24
4,000	24	24
2,500	18	18

* Refers to minimum depth below lowest adjacent grade.

Minimum footing widths and depths of 18 inches should be used even if the actual bearing pressure is less than 2,500 psf for footings supported in the undisturbed bedrock.

Settlement

Based on the allowable bearing capacities previously presented for shallow footings supporting retaining walls and minor structures, the total static foundation settlement is anticipated to be within tolerable limits. We can provide more detailed settlement estimates for footings when further details on the loads are provided for walls and minor structures.

4.5.4 Lateral Load Resistance

Soil resistance to lateral loads will be provided by a combination of frictional resistance between the bottom of the mat foundation or footings and underlying soils and by passive soil pressures acting against the embedded sides of the footings. For frictional resistance, a coefficient of friction of 0.35 may be used for design of foundations for structures supported in properly compacted fills, such as the building mat foundation. A coefficient of friction of 0.40 may be used for design of retaining wall footings supported on the undisturbed bedrock. In addition, an allowable lateral bearing pressure equal to an equivalent fluid weight of 250 pounds per cubic foot may be used for the mat foundation or footings are poured tight against compacted fill soils. An allowable lateral bearing pressure equal to an equivalent fluid weight of 400 pounds per cubic foot may be used for retaining wall footings that are poured tight against the undisturbed bedrock. These values (friction and lateral bearing) may be used in combination without reduction.

Footings adjacent to descending slopes should be deepened to allow for a lateral distance of at least one-half of the slope height, but not less than 15 feet, between the base of the footing and the face of the slope. We should be provided with the foundation and grading plans to review the footing conditions relative to the proposed adjacent grades prior to bidding the project.

4.5.5 Foundation Concrete

Laboratory testing by HDR (Appendix B) on a selected sample indicates that the near surface soils exhibit a soluble sulfate content between 65 and 1,750 mg/kg. For the 2022 CBC, foundation concrete should conform to the requirements outlined in ACI 318, Section 4.3 for Category S2 levels of soluble sulfate exposure from the on-site soils. Chloride levels in the sample of the upper soils tested were found to be between 11 and 438 mg/kg, and we recommend a Category C1 be used for design.

4.5.6 Footing Excavation Observation

Prior to placement of concrete and steel, a representative of GPI should observe and approve the subgrade for the mat foundation and footing excavations.

4.6 RETAINING STRUCTURES AND SHORING

At the time of this report, plans for temporary shoring and permanent retaining walls were not finalized. However, the southern, eastern, and western building walls will require retaining walls, and if deeper removals are chosen over deep foundations, temporary shoring will likely be required along the north, east, and western portions of the site due to the relatively close proximity to the property lines. Retaining structures may include subterranean building walls and stand-alone retaining walls. The following recommendations are provided for temporary shoring and conventional retaining walls. We recommend that conventionally backfilled walls be backfilled with sandy (granular) soils. Sufficient granular soils may not be readily available on-site to backfill retaining walls and cap the floor slab and pedestrian hardscape subgrade. Because of the preliminary timing of this report and complexity of the site and project configuration, we should be provided with the design plans for shoring and retaining systems prior to finalizing to confirm suitable geotechnical design parameters have been used.

4.6.1 Lateral Earth Pressures

We recommend cantilevered shoring or retaining walls be designed using a triangular lateral earth pressure distribution. For braced or tied-back shoring or retaining walls, we recommend a trapezoidal lateral earth pressure distribution be used in design, as shown on Figure 10, Trapezoidal Earth Pressure Distribution. In addition to the static lateral earth pressure, retaining walls should be designed to resist short term seismic lateral earth pressures. We recommend seismic earth pressures be taken as an inverse triangular distribution.

The magnitude of lateral earth pressures will depend on the direction of the shoring or retaining wall, as well as the condition of backfill (level or sloping). The following earth pressures are recommended for the design of shoring and retaining walls, and assume fully drained conditions:

Direction Facing*	Bedding Condition	Backfill Condition	Cantilever (pcf)	Braced/Tie-Back (psf)	Seismic (psf)
North	Adverse	Level	58	40	15H
North	Adverse	2:1 (h:v)	87	61	25H
West	Favorable	Level	32	22	15H
West	Favorable	2:1 (h:v)	48	33	25H
East	Favorable	Level	32	22	15H
East	Favorable	2:1 (h:v)	48	33	25H
South	Favorable	Level	32	22	15H
South	Favorable	2:1 (h:v)	48	33	25H

* A north-facing wall is located at the south side of the building.

The coefficient H is the height of wall in feet. When the plans for the retaining walls and shoring are further developed, we can evaluate the specific configurations to further refine the above values.

In addition to the recommended earth pressure, the upper 10 feet of the shoring adjacent to roads should be designed to resist a uniform lateral pressure of 100 pounds per square foot, acting as a result of an assumed 300 pound per square foot surcharge behind the shoring due to normal automobile traffic. If traffic is kept at least 10 feet from the shoring, the traffic surcharge may be neglected.

Walls subject to surcharge loads should be designed for an additional uniform lateral pressure equal to one-third and one-half the anticipated surcharge pressure for unrestrained and restrained walls, respectively. We can provide more specific lateral earth pressures resulting from surcharge loads when further details on the surcharge load are available.

4.6.2 Conventional Retaining Walls

The recommended lateral earth pressures are based on the assumption that the supported earth will be fully drained, preventing the build-up of hydro-static pressures. For traditional backfilled retaining walls, a drain consisting of perforated pipe surrounded by ¾ inch gravel and wrapped in filter fabric should be used. As a minimum, one cubic foot of rock should be used for each lineal foot of drain. The fabric (non-woven filter fabric, Mirafi 140N or equivalent) should be lapped at the top.

For retaining walls taller than 15 feet, the drain at the base of the wall should be supplemented by chimney drains extending up the back side of the wall. The chimney drain can consist of a prefabricated product, such as Miradrain, or a layer of gravel immediately behind the wall at least 12 inches wide and separated from the backfill soil with a suitable filter fabric. The chimney drain should be terminated about 2 feet from the finished grade, with the upper backfill consisting of the on-site soils.

The Structural Engineer should specify the use of select, granular wall backfill on the plans for conventional retaining walls. The select fill should extend behind the wall a distance laterally of one-third the wall height or to the back of the retaining wall footing, whichever is less. Wall footings should be designed as discussed in the "Foundations" section.

4.6.3 Soldier Pile Shoring and Tie-Backs

For the deep cuts planned at the south, east, and west sides of the planned dealership building (and the north side if deep foundations are not used), there may not be sufficient space for a sloped embankment. Potential methods for retaining the cut would be to install temporary shoring in front of a conventional wall, or use permanent tied-back soldier pile walls. The temporary shoring or soldier pile wall would consist of steel soldier piles placed in drilled holes, backfilled with concrete, and tied-back with earth anchors.

For temporary shoring and permanent soldier pile walls, the lateral earth pressures previously provided in Section 4.7.1 may be used for design.

<u>Soldier Piles</u>

For design of soldier piles spaced at least two diameters on centers for a permanent soldier pile wall, the allowable lateral bearing value (passive value) of the bedrock extending below the excavation may be taken to be 800 pounds per square foot at the excavated surface, up to a maximum of 8,000 psf. The allowable lateral bearing value (passive value) of the colluvium/alluvium extending below the excavation may be taken to be 500 pounds per square foot at the excavated surface, up to a maximum of 5,000 psf. To develop the full lateral value, provisions should be made to assure firm contact between the soldier piles and the undisturbed soils. The concrete placed in the soldier pile excavation below the excavated level may be a lean mix, but it should be of adequate strength to transfer the imposed loads to the surrounding soils.

Difficult drilling and refusal was locally encountered in our explorations into the bedrock materials. The shoring contractor should evaluate the potential drilling conditions when planning the installation methods. Caving was not encountered during our explorations.

The frictional resistance between the soldier piles and the retained earth may be used in resisting the downward component of the anchor load. The coefficient of friction between the soldier pile and the retained earth may be taken as 0.35. This value is based on the assumption ,that uniform full bearing will be developed between the steel soldier beam and the lean-mix concrete and between the lean mix concrete and the retained earth. In addition, provided the portion of the soldier piles below the excavated level is backfilled with structural concrete, the soldier piles below the excavated level may be used to resist downward loads. The frictional resistance between the concrete soldier piles and the soils below the excavated level may be taken as equal to 450 pounds per square foot.

For permanent tie-back walls, soldier piles should be protected from corrosion caused by contact with the on-site materials. Recommendations may be required from a corrosion engineer, such as HDR, who performed the corrosivity testing presented in Appendix B.

Lagging

Continuous lagging will be required between the soldier piles. The lagging could consist of timber or wire mesh and shotcrete/gunite. Careful installation of timber lagging will be necessary to achieve bearing against the retained earth. In areas where caving occurs, backfill of the lagging with clean sand fill or grout will be required. The soldier piles should be designed for the full anticipated lateral pressure. However, the pressure on the lagging will be less because of arching of the soils between piles. We recommend that the lagging be designed for the recommended earth pressure but limited to a maximum value of 400 pounds per square foot.

The temporary vertical lifts for the cuts between soldier piles to install the lagging should be limited to 5 feet. Shotcrete/gunite lagging should be properly cured prior to allowing excavation for the subsequent lift.

Tie-Back Anchors

Tie-back friction anchors may be used to resist lateral loads. For design purposes, it may be assumed that the active wedge adjacent to the shoring or wall is defined by a plane drawn at 35 degrees from the vertical through the bottom of the excavation for cuts facing east, west, and south (favorable bedding). For north facing cuts (adverse bedding), it may be assumed that the active wedge adjacent to the shoring or wall is defined by a plane drawn at 72 degrees from the vertical through the bottom of the excavation. The anchors should extend at least 25 feet beyond the potential active wedge and to a greater length if necessary to develop the desired capacities.

The capacities of anchors should be determined by testing of the initial anchors as outlined in a following paragraph. For design purposes, it may be estimated that drilled friction anchors will develop an average friction value of 800 pounds per square foot. Higher friction values may be feasible depending on the layout of the anchors. If two rows of tie-backs are planned, a higher friction value may be used, and can be provided if requested. Only the frictional resistance developed beyond the active wedge would be effective in resisting lateral loads. If post-grouted tie-back anchors are used, a preliminary average friction value of 2,000 psf may be assumed for planning. A higher value may be assumed if confirmed by field load testing. If the anchors are spaced at least 6 feet on-centers, no group action reduction in the capacity of the anchors need be considered.

The anchors may be installed at angles of 15 to 40 degrees below the horizontal. Care should be taken to confirm that the new anchors do not conflict with the utility lines upslope from the development. Caving of the anchor holes should be prevented with the installation method selected. The anchors should be filled with concrete placed by pumping from the tip out, and the concrete should extend from the tip of the anchor to the active wedge. The annular space around conventional anchors within the active wedge should not be backfilled until after testing has been completed. If caving is encountered, the void may be filled with wet sand. The anchor may be filled with concrete to the surface of the shoring for post-grouted anchors that are 8 inches in diameter or less.

For permanent tie-backs, the anchors should be protected from corrosion by epoxy coating or an equivalent method. The actual method used should be developed by a corrosion protection consultant, such as HDR.

Tie-Back Anchor Testing

For temporary shoring, GPI should select at least one of the initial anchors for a 24-hour hour, 200 percent test and three additional anchors for quick 200 percent tests. For a permanent tie-back wall, each of the permanent anchors should be proof-tested to 200 percent of the design load using a quick test, with four anchors selected for 24-hour tests. The purpose of the 200 percent test is to verify the friction value assumed in design. The anchors should be tested to develop twice the assumed friction value capacity. Where satisfactory tests are not achieved on the initial anchors, the anchor diameter and/or length should be increased until satisfactory test results are obtained. When the extent of the shoring program is known, we should review the recommended test program and make modifications as necessary.

For the 200 percent tests, the 200 percent test load should be maintained for 24 hours (24-hour test) and 1 hour (quick tests). The total deflection of the anchor during the test should not exceed 12 inches. The deflection after the 200 percent test load has been applied should not exceed 0.50 inch during any 1 hour period.

For temporary anchors, the remaining anchors should be pretested to at least 150 percent of the design load. The total deflection during the test should not exceed 12 inches between the anchor and the soldier pile. The rate of creep under the 150 percent load should not exceed 0.25 inch over a one-hour period and 0.1 inch over any 15-minute period within the one hour for the anchor to be approved for the design loading.

After a satisfactory test, each production anchor should be locked-off at the design load. The locked-off load should be verified by rechecking the load in the anchor. If the locked-off load varies by more than 10 percent from the design load, the load should be reset until the target load is achieved.

Deflection

It is difficult to accurately predict the amount of deflection of the shored embankment. It should be realized, however, that some deflection will occur. If it is desired to reduce the deflection, such as immediately adjacent to existing settlement sensitive improvements, the wall should be designed for higher lateral earth pressures.

With the relatively high tie-back anchor loads anticipated, the downward component of the anchor load will impose significant axial loads on the soldier piles. The frictional resistance of the soldier pile should be confirmed versus the downward component of the anchor load to evaluate the adequacy of the soldier pile embedment.

Monitoring

We recommend performing a detailed survey of the improvements supported above the planned cut prior to and during the shoring or wall installation. The survey should include topographic data and a video account of the condition of the existing improvements, including cracks or signs of distress. During construction, the monitoring should consist of periodic surveying of the lateral and vertical locations of the tops of the soldier piles. We can discuss the scope of the monitoring with the design team and owner when the design of the shoring or wall system has been finalized.

Drainage

The permanent walls should be drained full-height using a suitable drainage composite. If shoring is used, the drainage composite should be placed between the soldier piles prior to applying the shotcrete surface to allow for groundwater seepage within the height of the cut to be collected and discharged without building up hydrostatic pressures behind the wall. We recommend that the continuous drainage panels be installed at the same spacing as the soldier piles.

4.7 SLOPES

Based on the available information, cut and fill slopes are planned as part of the project. Permanent slopes at the site should be constructed at an inclination of 2:1 (horizontal to vertical) or flatter. We recommend fill slopes be overbuilt by at least 3 feet during rough grading and trimmed back to a hard and unyielding surface. Slope rolling to achieve a finished compacted surface should not be performed. A keyway at least 3 feet deep and equal in width to one-half the slope height, but not less than 15 feet, should be constructed prior to filling the slope. See Figures 7 and 8 for further details.

Setbacks of structures from the top and toe of the planned slopes should be maintained as directed by the regulatory agency.

Drainage devices and erosion control measures should be installed as required by the governing agencies.

4.8 DRAINAGE

Positive surface gradients should be provided adjacent to structures so as to direct surface water run-off and roof drainage away from foundations and slabs toward suitable discharge facilities. Long-term ponding of surface water should not be allowed on pavements or adjacent to structures. Subsurface drainage should be provided on walls below grade as discussed in a previous section.

4.9 EXTERIOR CONCRETE AND MASONRY FLATWORK

Exterior pedestrian concrete and masonry flatwork should be supported on a layer of non-expansive compacted fill if differential heave is not tolerable. The use of clayey soils within 2 feet of floor slab and concrete hardscape subgrade is not recommended. Prior to placement of concrete, the subgrade should be prepared as recommended in “Subgrade Preparation” section. The moisture content of subgrade soils should be maintained above the optimum moisture content and confirmed by a representative of GPI prior to covering. Subgrade soils allowed to dry out will require moisture conditioning, including the potential for additional processing.

4.10 PAVED AREAS

Testing on a sample of the upper soils in the planned parking area resulted in an R-value of 7. Preliminary pavement design has been based on an R-value of 5. These recommendations are based on the assumption that the pavement subgrades will consist of the existing clayey on-site soils. The following pavement sections are recommended for planning purposes only.

TRAFFIC INDEX	SECTION THICKNESS (inches)	
	ASPHALTIC CONCRETE	**AGGREGATE BASE COURSE**
4	3	7
5	3	10
6	4	12
	Portland Cement Concrete	**Aggregate Base Course**
4	6.0	4
5	6.5	4
6	7.0	4

The pavement subgrade underlying the aggregate base or concrete should be properly prepared and compacted in accordance with the recommendations outlined under "Subgrade Preparation".

The concrete used for paving should have a modulus of rupture of at least 550 psi (equivalent to an approximate compressive strength of 3,700 psi) at the time the pavement is subjected to truck traffic. Where new pavements will be constructed directly adjacent to and entering the proposed building, we recommend dowels be placed to reduce the potential for differential settlement at the transition between the building and the adjacent pavements.

The pavement base course should be compacted to at least 95 percent of maximum density (ASTM D-1557). Aggregate base should conform to the requirements of Section 26 of the California Department of Transportation Standard Specifications for Class II aggregate base (three-quarter inch maximum) or Section 200-2 of the Standard Specifications for Public Works Construction (Green Book) for untreated base materials, except processed miscellaneous base.

The above recommendations assume that the base course and compacted subgrade will be properly drained. The design of paved areas should incorporate measures to prevent moisture build-up within the base course which can otherwise lead to premature pavement failure. For example, curbing adjacent to landscaped areas should be deep enough to act as a barrier to infiltration of irrigation water into the adjacent base course.

4.11 STORMWATER INFILTRATION

To evaluate the infiltration characteristics of the near surface soils, we performed two field infiltration tests in accordance with methods established by the County of Los Angeles (County, 2014). Infiltration testing was performed at depths of about 10 feet below the existing ground surface at the general locations proposed for underground detention basins as provided by the project Civil Engineer.

The tests were performed in shallow borings drilled with an 8-inch hollow stem auger. The test well was constructed in the boring using a 2-inch diameter slotted well casing. The annular space between the perforated casing and the borehole was filled with No. 3 well sand.

Prior to running the tests, the soils adjacent to the well were soaked with water a minimum of five times the diameter of the well above the bottom of the boring.

The well was filled with a minimum of 18 inches of water at the initiation of the test. After confirming the initial infiltration rate, we performed the infiltration testing by taking water level measurements every 30 minutes. The infiltration rate was calculated using the lowest infiltration rate during the testing.

The adjusted infiltration rate was determined by dividing the preadjusted infiltration rate by a reduction factor that is dependent on the initial water depth and the diameter of the boring. The results of the tests indicated infiltration rates of approximately 0.0 and 0.1 inches per hour (nearly no infiltration). Detailed results of the testing are presented in Tables 1.1 and 1.2, Borehole Infiltration Test Results.

Grading at the location of the proposed underground basin is expected to be minimal and densification of the on-site soil due to grading activities is anticipated to have little effect on the permeability of the soils at the proposed depth of infiltration. The Civil Engineer should evaluate feasibility of groundwater infiltration using the infiltration rates provided and suitable factors of safety.

4.12 GEOTECHNICAL OBSERVATION AND TESTING

We recommend that a representative of GPI observe earthwork during construction to confirm that the recommendations provided in our report are applicable during construction. The earthwork activities include grading, compaction of fills, subgrade preparation, pavement construction and foundation excavations. If conditions are different than expected, we should be afforded the opportunity to provide an alternate recommendation based on the actual conditions encountered.

5.0 LIMITATIONS

The report, exploration logs, and other materials resulting from GPI's efforts were prepared exclusively for use by Hello Auto Group and their consultants in designing the proposed development. The report is not intended to be suitable for reuse on extensions or modifications of the project or for use on any project other than the currently proposed development as it may not contain sufficient or appropriate information for such uses. If this report or portions of this report are provided to contractors or included in specifications, it should be understood that they are provided for information only.

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

Furthermore, our recommendations were developed with the assumption that a proper level of field observations and construction review will be provided by GPI during grading, excavation, and foundation construction. If field conditions during construction appear to be different than is indicated in this report, we will need to assess the impact of such conditions on our recommendations.

If construction phase geotechnical services are provided by others, the use of this report and its contents will be solely at their risk. In addition, the firm will need to accept full responsibility for geotechnical aspects of the project, including the recommendations contained herein.

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

Respectfully submitted,
Geotechnical Professionals Inc.

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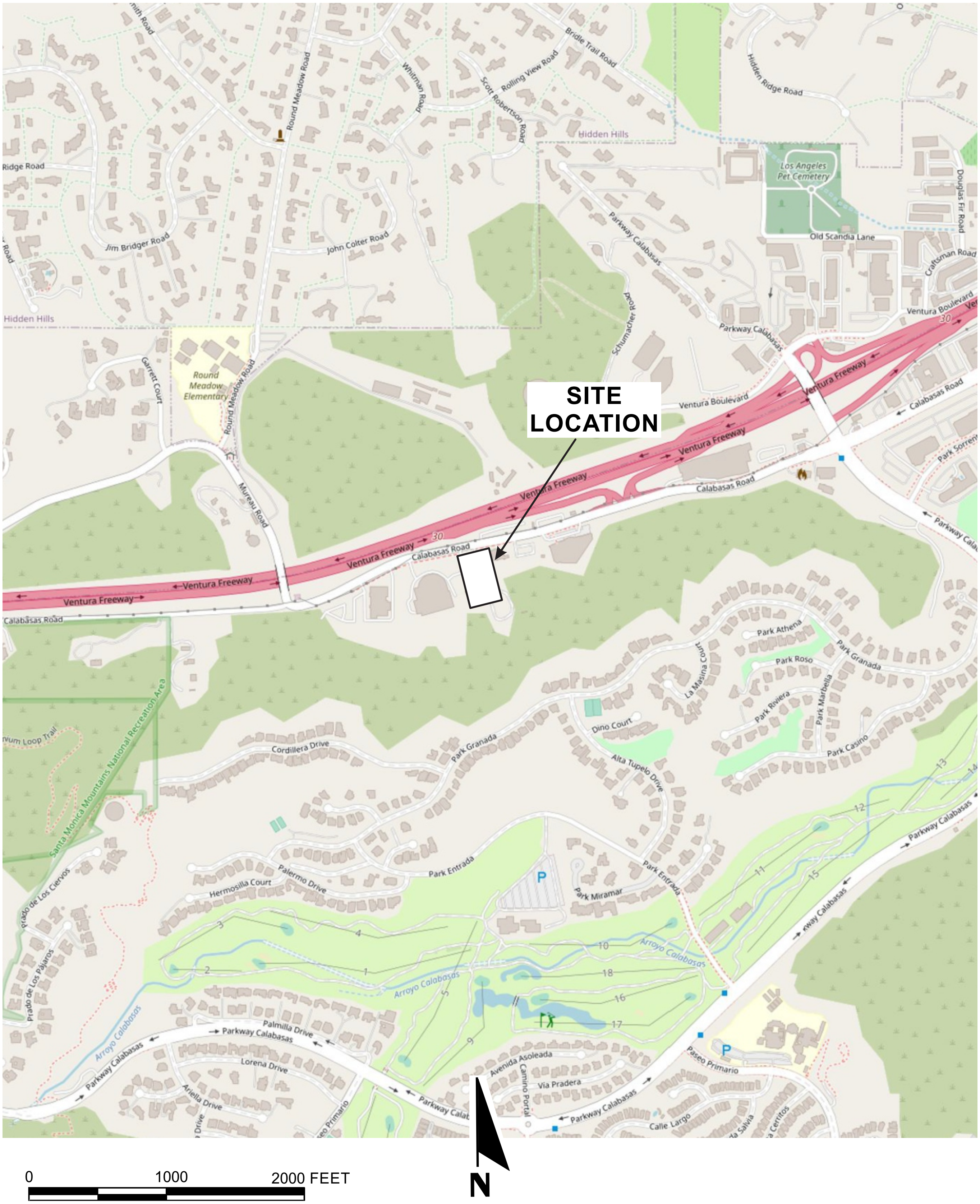


BASE PLAN REPRODUCED FROM CALTOPO © 2021

GEOTECHNICAL PROFESSIONALS, INC.

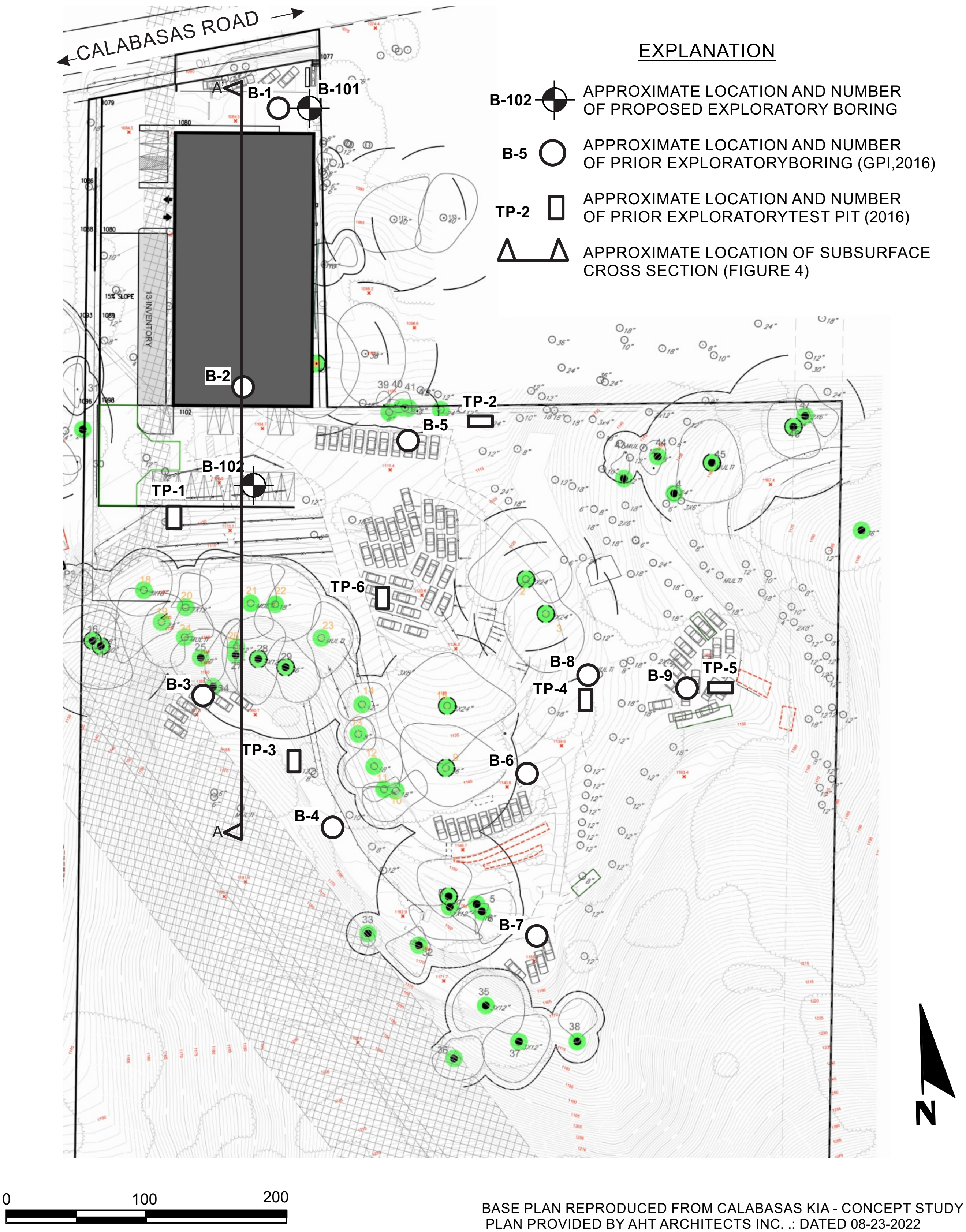
KIA CALABASAS

GPI PROJECT NO.: 3162.I

SCALE: 1" = 1000'

SITE LOCATION MAP

FIGURE 1



BASE PLAN REPRODUCED FROM CALABASAS KIA - CONCEPT STUDY PLAN PROVIDED BY AHT ARCHITECTS INC. .: DATED 08-23-2022

GPI GEOTECHNICAL PROFESSIONALS, INC.

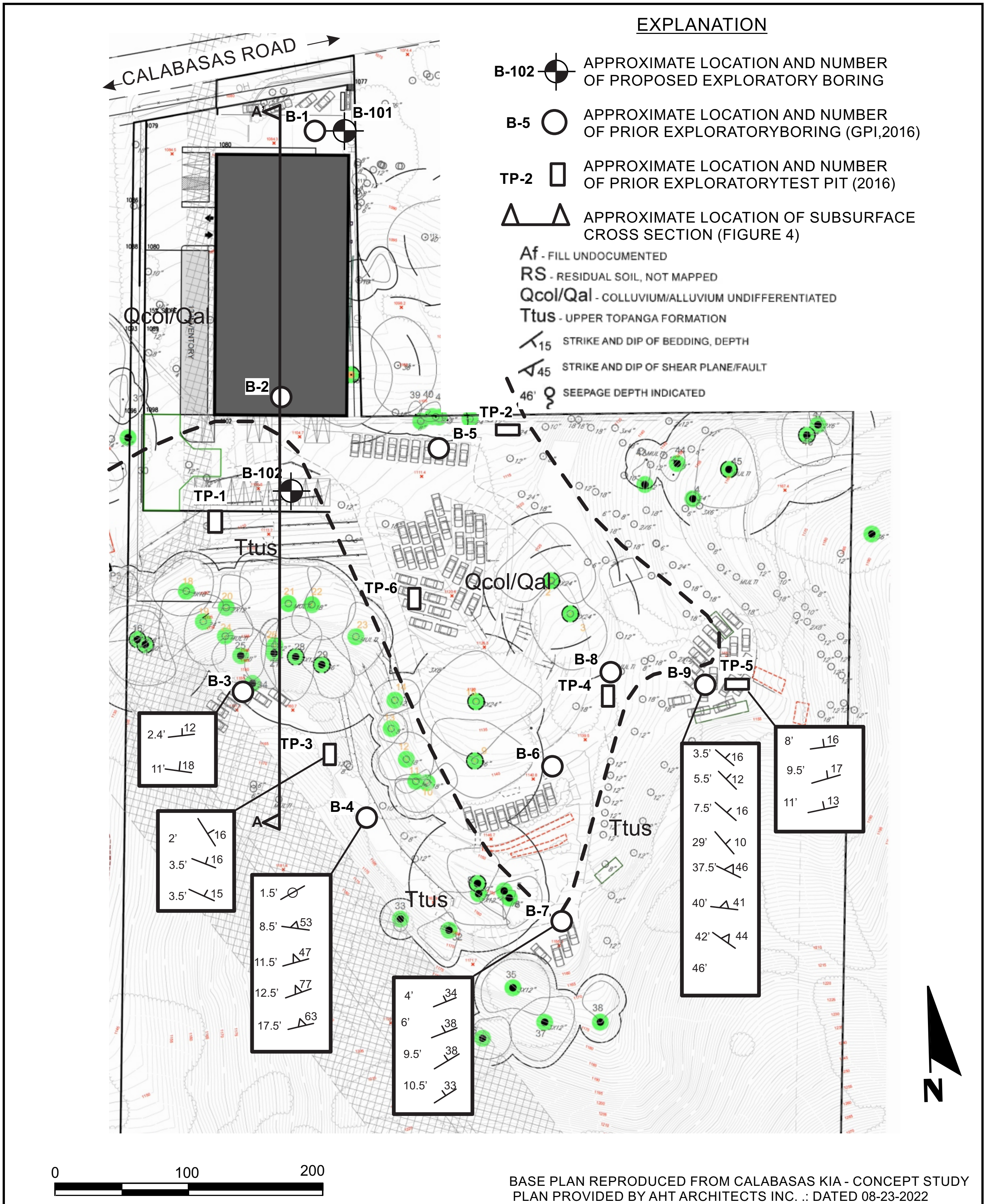
KIA CALABASAS

GPI PROJECT NO.: 3162.I

SCALE: 1" = 100'

SITE PLAN

FIGURE 2



GPI GEOTECHNICAL PROFESSIONALS, INC.

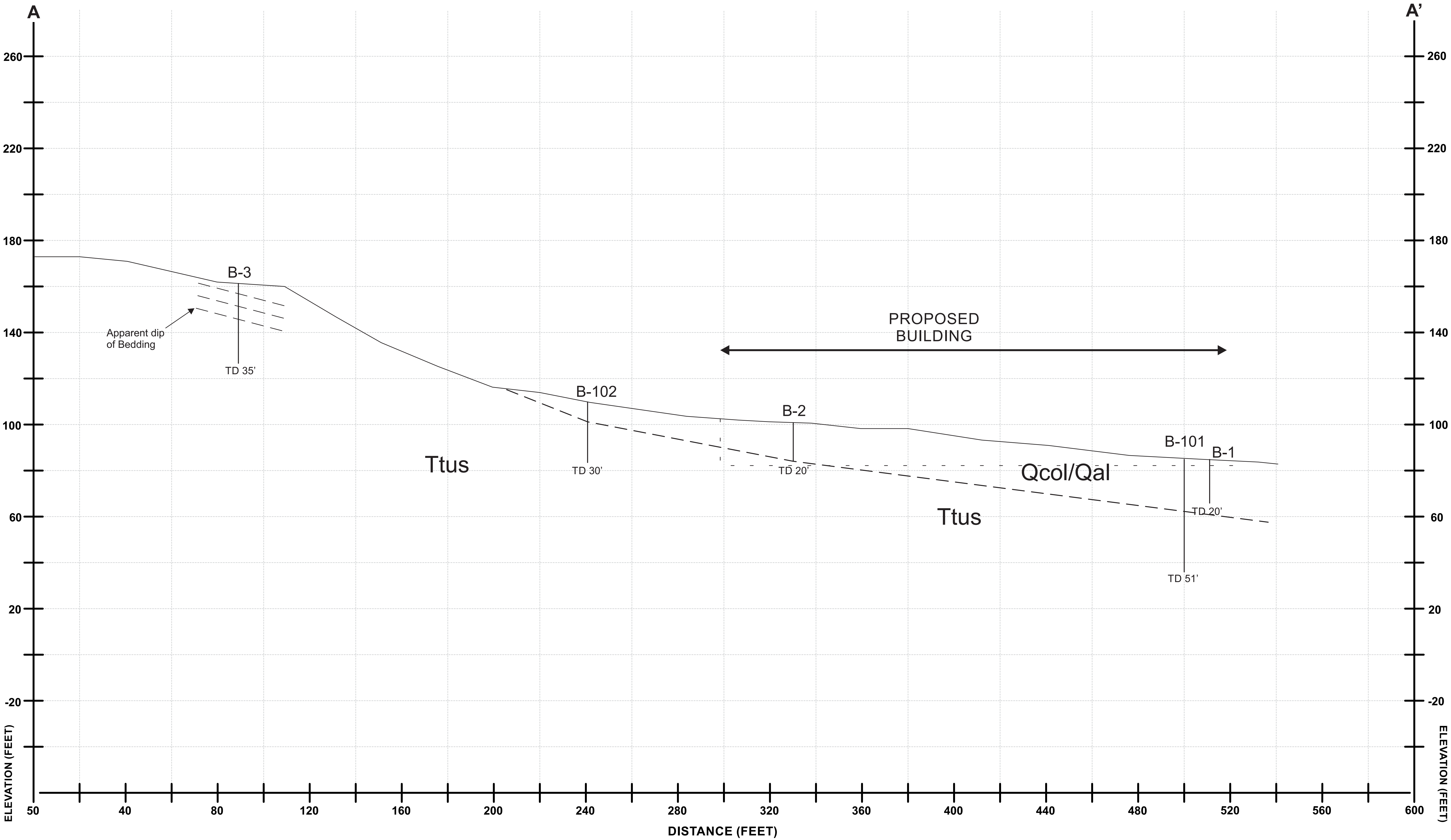
KIA CALABASAS

GPI PROJECT NO.: 3162.I

SCALE: 1" = 100'

GEOLOGIC SITE PLAN

FIGURE 3



Note: This section is based upon information obtained at borings and CPTs obtained during geotechnical investigation. The section is based upon limited geotechnical data and localized variations should be anticipated. This section is intended for descriptive purposes only.

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KIA CALABASAS

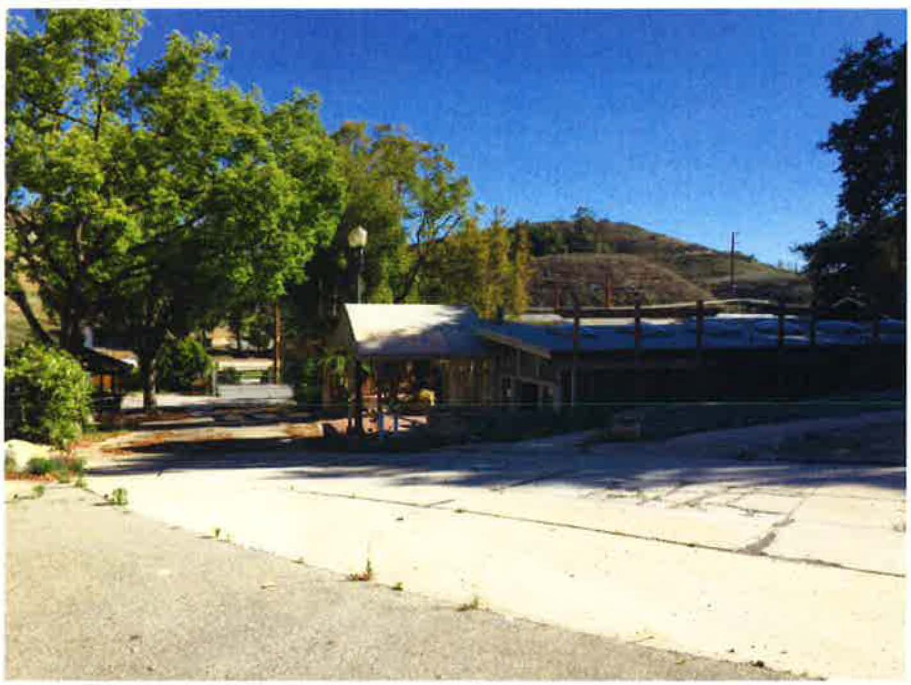
GPI PROJECT NO.: 3162.I

SCALE AS SHOWN

SUBSURFACE CROSS SECTION A-A'

FIGURE 4

AT BORING B-5 LOOKING SOUTH TOWARDS BORING B-8



NEAR BORING B-2 LOOKING NORTH TOWARDS CALABASAS ROAD

SITE PHOTOGRAPHS TAKEN APRIL 17, 2016

GPI GEOTECHNICAL PROFESSIONALS, INC.

KIA CALABASAS

GPI PROJECT NO. 3162.1 | NTS

SITE PHOTOGRAPHS

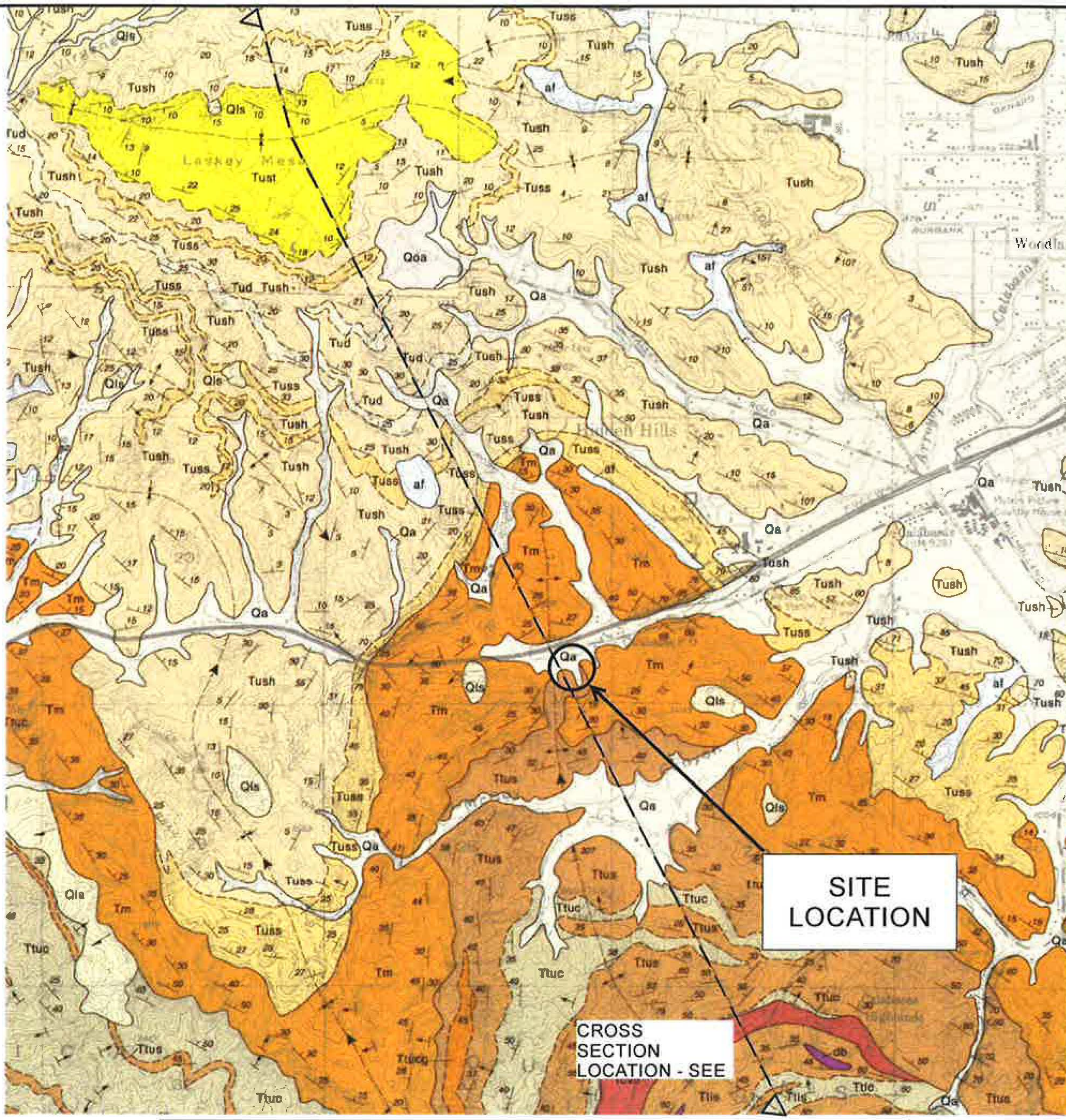
FIGURE 5.1

NEAR BORING B-4 LOOKING WEST TOWARDS BORING B-3

AT BORING B-9 LOOKING WEST TOWARDS BORING B-6

SITE PHOTOGRAPHS TAKEN APRIL 17, 2016

GPI GEOTECHNICAL PROFESSIONALS, INC.		**SITE PHOTOGRAPHS**
KIA CALABASAS		
GPI PROJECT NO. 3162.1	NTS	FIGURE 5.2



LEGEND

Tm Tmss Tmcg

MONTEREY FORMATION

(Lower part Modelo Formatin of Hoots 1931; Soper 1938; Durrell 1954; A.E.G. maps 1982; Modelo Formation of Yerkes and Campbell 1979; Weber 1984; Modelo-Monterey and lower Monterey Formation of Truex and Hall 1969, Truex 1976; ; equivalent to Monterey Formation of Dibblee 1989, in Ventura basin)

Marine, biogenic and clastic; middle and late Miocene age (late (?) Luisian and Mohnian Stages)

Tm *Gray-brown, white weathering siliceous shale, thin bedded, moderately hard with platy fracture; includes soft fissile diatomaceous shale, hard, brittle, cherty shale, and few layers of hard, yellow-weathering calcareous concretions or lenses*

Tmss *Light gray to tan, semi-friable bedded sandstone*

Tmcg *Grey cobble conglomerate of mostly granitic detritus in sandstone matrix*

af Qa

SURFICIAL SEDIMENTS

af *Artificial cut and fill*

Qa *Alluvium: gravel, sand and clay of valley areas, includes gravel of stream channels gravel and sand of alluvial fans, and slope wash; undissected to slightly dissected*

Ttuc Ttus Ttucg

UPPER TOPANGA FORMATION

(Of Durrell 1954; Topanga Formation of Soper 1938; Truex and Hall 1969; Truex 1976; Weber 1984; Calabasas Formation of Yerkes and Campbell 1979, 1980)

Marine clastic; middle Miocene age (Luisian Stage)

Ttuc *Gray claystone, bedded; crumbly with ellipsoidal fracture*

Ttus *Light gray sandstone, semi friable, thick bedded*

Ttucg *Gray conglomerate of cobbles of granitic rocks, sandstone, and volcanic rocks in sandstone matrix*

N

0 3000 6000 FEET

BASE MAP REPRODUCED FROM THE GEOLOGIC MAP OF THE CALABASAS QUADRANGLE PREPARED BY THE DIBBLEE GEOLOGICAL FOUNDATION: DATED 1992

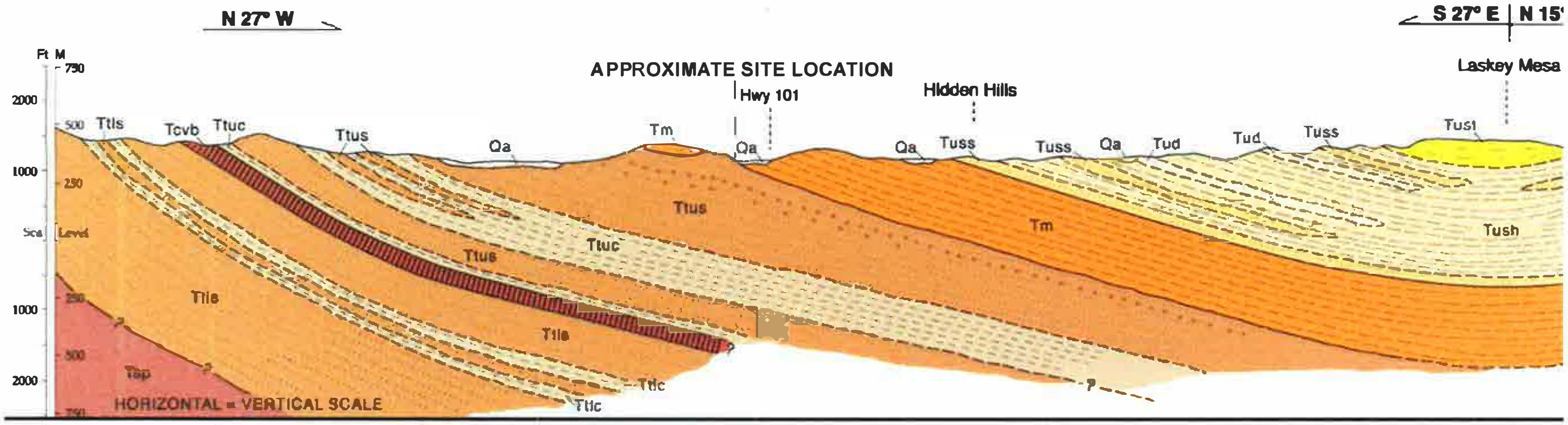
GPI GEOTECHNICAL PROFESSIONALS, INC.

KIA CALABASAS

GPI PROJECT NO.:3162.I	SCALE: 1" = 3000'

REGIONAL GEOLOGIC MAP

FIGURE 6.1



SEE FIGURE 4.1 FOR CROSS SECTION LOCATION

LEGEND

MONTEREY FORMATION

(Lower part Modelo Formation of Hoots 1931; Soper 1938; Durrell 1954; A.L.C. maps 1982; Modelo Formation of Yerkes and Campbell 1979; Weber 1984; Modelo-Monterey and lower Monterey Formation of Truex and Hall 1969, Truex 1976; equivalent to Monterey Formation of Dibblee 1989, in Ventura basin)

Marine, biogenic and clastic; middle and late Miocene age (late (?) Luisian and Mohnian Stages)

Tm Gray-brown, white weathering siliceous shale, thin bedded, moderately hard with platy fracture; includes soft fissile diatomaceous shale, hard, brittle, cherty shale, and few layers of hard, yellow-weathering calcareous concretions or lenses

Tmss Light gray to tan, semi-friable bedded sandstone

Tmcg Gray cobble conglomerate of mostly granitic detritus in sandstone matrix

SURFICIAL SEDIMENTS

af Artificial cut and fill

Qa Alluvium: gravel, sand and clay of valley areas, includes gravel of stream channels gravel and sand of alluvial fans, and slope wash; undissected to slightly dissected

UPPER TOPANGA FORMATION

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Ttuc Gray claystone, bedded; crumbly with ellipsoidal fracture

Ttus Light gray sandstone, semi-friable, thick bedded

Ttucg Gray conglomerate of cobbles of granitic rocks, sandstone, and volcanic rocks in sandstone matrix

0 2500 5000 FEET

BASE MAP REPRODUCED FROM THE GEOLOGIC MAP OF THE CALABASAS QUADRANGLE PREPARED BY THE DIBBLEE GEOLOGICAL FOUNDATION: DATED 1992

GEOTECHNICAL PROFESSIONALS, INC.

KIA CALABASAS

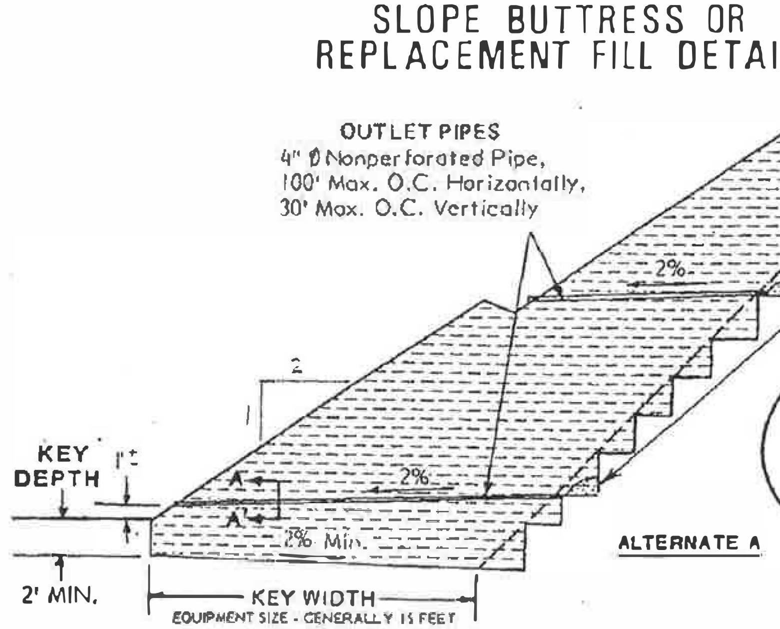
GPI PROJECT NO.: 3162.I

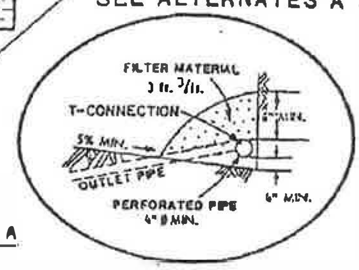
SCALE: 1" = 2500'

REGIONAL GEOLOGIC MAP CROSS SECTION

FIGURE 6.2

SLOPE BUTTRESS OR REPLACEMENT FILL DETAIL



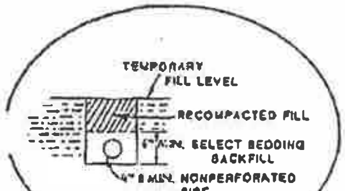


ALTERNATE A

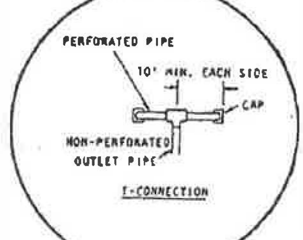
FILTER MATERIAL

Filter material shall be Class 2 permeable material per State of California Standard Specifications, or approved alternate. Class 2 grading as follows:

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

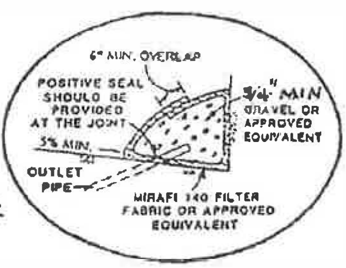


DETAIL A-A'



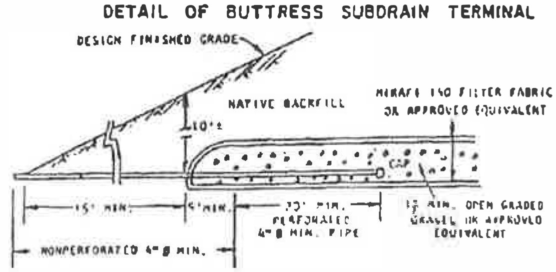
NOTES:

- Fill blanket, back cut, key width and key depth are subject to field change, per report/plans.
- Key heel subdrain, blanket drain, or vertical drain may be required at the discretion of the geotechnical consultant.
- SUBDRAIN INSTALLATION - Subdrain pipe shall be installed with perforations down or, at locations designated by the geotechnical consultant, shall be nonperforated pipe.
- SUBDRAIN TYPE - Subdrain type shall be ASTM C508 Asbestos Cement Pipe (ACP) or ASTM D2751, SDR 23.5 or ASTM D1527, Schedule 40 Acrylonitrile Butadiene Styrene (ABS) or ASTM D3034 SDR 23.5 or ASTM D1785, Schedule 40 Polyvinyl Chloride Plastic (PVC) pipe or approved equivalent.



ALTERNATE B

DETAIL OF BUTTRESS SUBDRAIN TERMINAL



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KIA CALABASAS

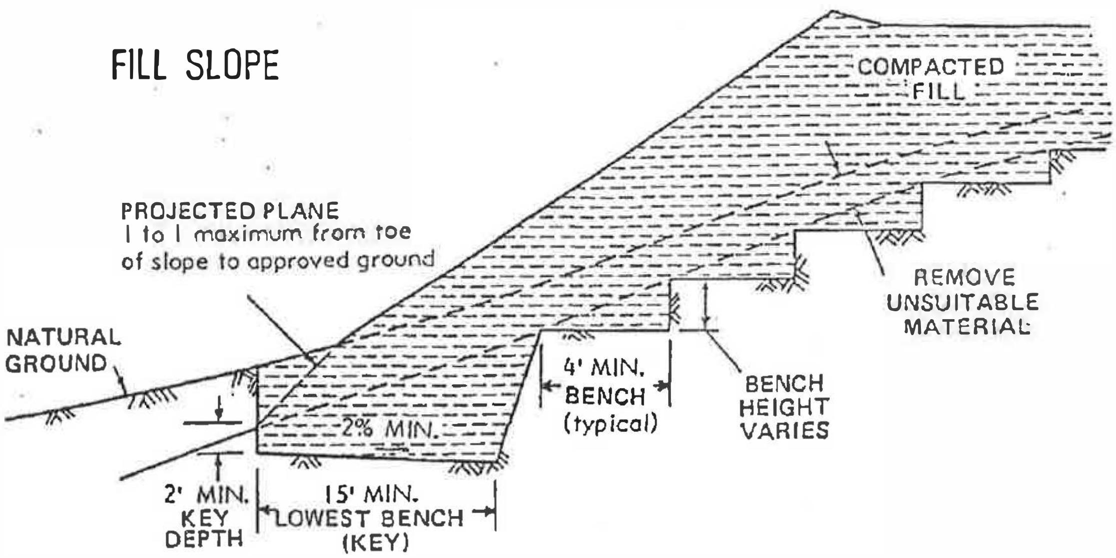
GPI PROJECT NO.: 3162.I | NO SCALE

SLOPE BUTTRESS OR REPLACEMENT FILL DETAIL

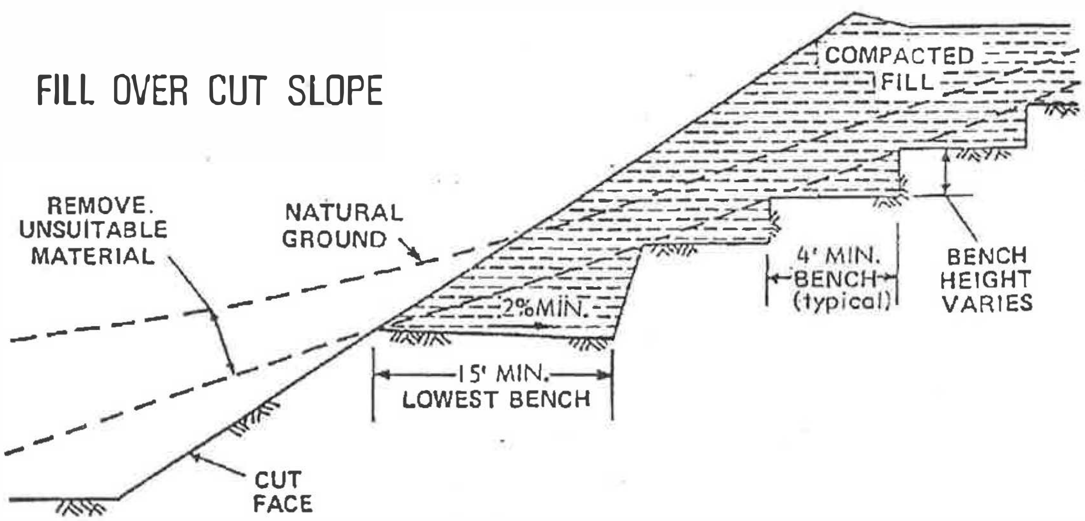
FIGURE 7

BENCHING DETAILS

FILL SLOPE



FILL OVER CUT SLOPE



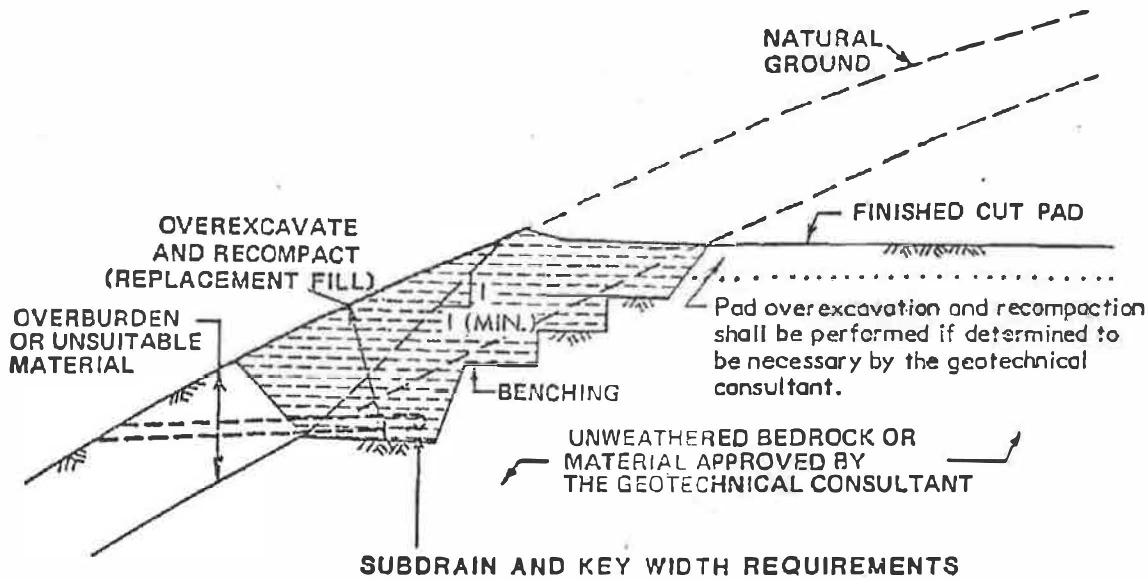
NOTES:

LOWEST BENCH: Depth and width subject to field change based on consultant's inspection.

SUBDRAINAGE: Back drains may be required at the discretion of the geotechnical consultant.

GEOTECHNICAL PROFESSIONALS, INC.		BENCHING DETAIL
KIA CALABASAS		
GPI PROJECT NO.: 3162.I	NO SCALE	FIGURE 8

SIDE HILL
CUT PAD DETAIL



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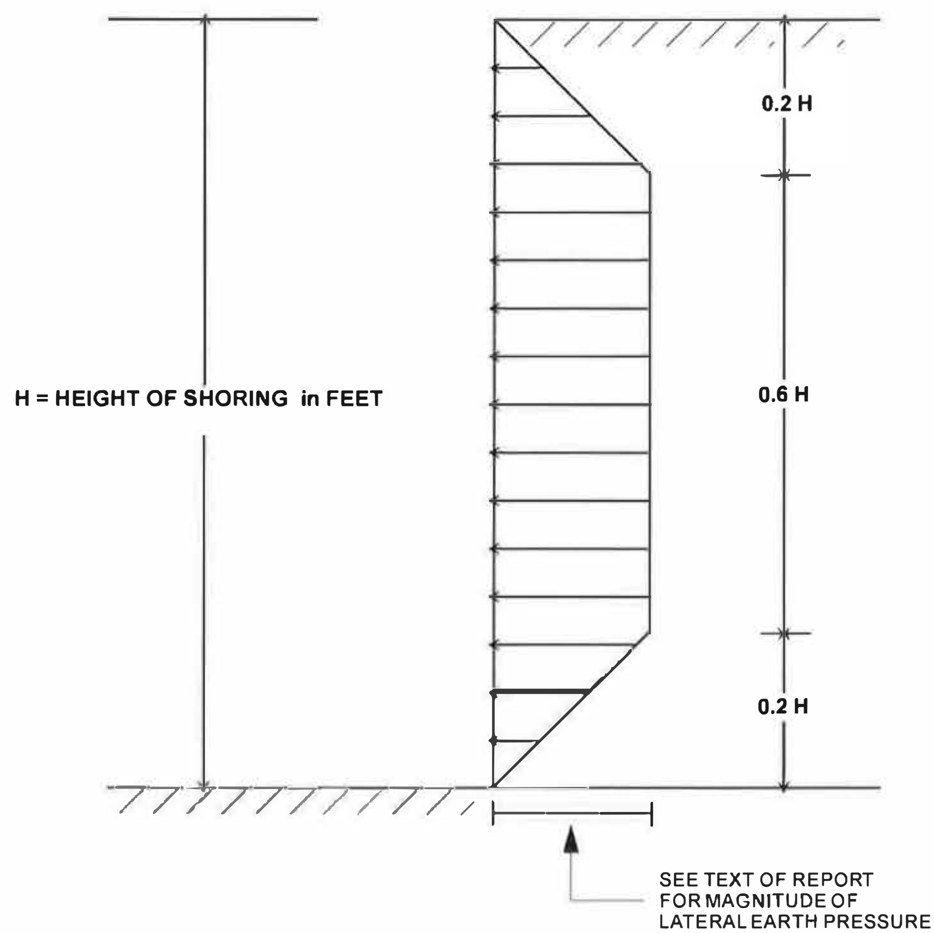
KIA CALABASAS

GPI PROJECT NO.: 3162.I | NO SCALE

SIDE HILL CUT PAD DETAIL

FIGURE 9



It should be noted that the provided lateral earth pressues do not include hydrostatic pressures

GEOTECHNICAL PROFESSIONALS, INC.

KIA CALABASAS

GPI PROJECT NO.: 3162.I

NO SCALE

LATERAL EARTH PRESSURES FOR TIE-BACK SHORING

FIGURE 10

TABLE 1.1

BOREHOLE INFILTRATION TEST RESULTS
Los Angeles County Method (GS200.1, 06/01/11)

Project No. 2730.I
Client: Calabasas Nissan
By AS

Date: 9/13/16
Test Date 5/20/16

NOTE: Slowest or average rate from percolation testing used to calculate infiltration rate

Test Well/ Adj. Boring	Test Duration (min) Δt	Depth of Well (ft)	Depth to Water Initial* (ft)	Depth to Water Final (ft)	Initial Water Depth (ft) d_1	Water Level Drop (ft) Δd	Hole Diameter (inches) *DIA*	Preadjusted Percolation Rate (in/hr)	Reduction Factor R_f**	Infiltration Rate (in/hr) I_t
P-1/B-1	30	9.95	7.82	7.82	2.13	0.00	8	0.0	7.4	0.0
P-1/B-1	30	9.95	7.82	7.82	2.13	0.00	8	0.0	7.4	0.0
P-1/B-1	30	9.95	7.82	7.82	2.13	0.00	8	0.0	7.4	0.0
P-1/B-1	30	9.95	7.82	7.82	2.13	0.00	8	0.0	7.4	0.0
P-1/B-1	30	9.95	7.82	7.82	2.13	0.00	8	0.0	7.4	0.0
P-1/B-1	30	9.95	7.82	7.82	2.13	0.00	8	0.0	7.4	0.0
P-1/B-1	30	9.95	7.82	7.82	2.13	0.00	8	0.0	7.4	0.0
P-1/B-1	30	9.95	7.82	7.82	2.13	0.00	8	0.0	7.4	0.0

* Test well pipe may be higher than ground surface

** $R_{f=(}(2d_1-\Delta d)/DIA)+1$

TABLE 1.2

BOREHOLE INFILTRATION TEST RESULTS
Los Angeles County Method (GS200.1, 06/01/11)

Project No. 2730.I
Client: Calabasas Nissan
By AS

Date: 9/13/16
Test Date 5/20/16

NOTE: Slowest or average rate from percolation testing used to calculate infiltration rate

Test Well/ Adj. Boring	Test Duration (min) Δt	Depth of Well (ft)	Depth to Water Initial* (ft)	Depth to Water Final (ft)	Initial Water Depth (ft) d_1	Water Level Drop (ft) Δd	Hole Diameter (inches) *DIA*	Preadjusted Percolation Rate (in/hr)	Reduction Factor R_f**	Infiltration Rate (in/hr) I_t
P-2/B-2	30	9.42	7.42	7.48	2.00	0.05	8	1.3	6.9	0.2
P-2/B-2	30	9.42	7.42	7.48	2.00	0.06	8	1.4	6.9	0.2
P-2/B-2	30	9.42	7.42	7.47	2.00	0.05	8	1.2	6.9	0.2
P-2/B-2	30	9.42	7.42	7.46	2.00	0.04	8	0.8	6.9	0.1
P-2/B-2	30	9.42	7.40	7.44	2.02	0.04	8	1.0	7.0	0.1
P-2/B-2	30	9.42	7.40	7.44	2.02	0.04	8	0.8	7.0	0.1
P-2/B-2	30	9.42	7.40	7.44	2.02	0.04	8	0.8	7.0	0.1
P-2/B-2	30	9.42	7.40	7.44	2.02	0.04	8	1.0	7.0	0.1

* Test well pipe may be higher than ground surface

** $R_{f}=((2d_1-\Delta d)/DIA)+1$

APPENDIX A

APPENDIX A

EXPLORATORY BORINGS AND TEST PITS

The subsurface conditions at the site were investigated by drilling and sampling a total of 11 exploratory borings and 6 test pits. Two of the hollow stem auger borings (B-101 and B-102) were performed as part of our current field investigation with the remainder being performed as part of our 2016 investigation. The borings were advanced to depths between 20 and 51 feet below the existing ground surface and the test pits were advanced to depths ranging from 5 to 13.5 feet. Four of the borings were terminated prior to the planned depth because of refusal in the dense bedrock. The exploration locations are shown on the Site Plan, Figure 2.

The borings were drilled using bucket EZ-bore auger, limited access auger rig, and truck-mounted hollow-stem auger equipment. Borings B-7 and B-9 were drilled using a large diameter bucket auger. Relatively undisturbed samples were obtained using a brass-ring lined sampler (ASTM D3550). The drive sampler has an inside diameter of 2.42 inches and an outside diameter of 3.25 inches. The brass rings have an inside diameter of 2.42 inches and a height of 1-inch. In addition, relatively disturbed bulk samples were obtained at various depths. The drive sampler is driven into the soil using a drop of 12 inches with a driving weight of the Kelly bar as shown.

RIG TYPE	DEPTH (ft)	KELLY BAR DRIVING WEIGHT (lbs)
Bucket Auger 24” (EZ-Bore)	0-29 30-57 58-85 86 – deeper	3615 2395 1310 450
Bucket Auger 24”	0-24 25 – deeper	1590 825

The number of blows needed to drive the sampler was recorded as the penetration resistance. It should be noted that the number of blows, in this case, is much lower than the standard penetration resistance because of the greater driving weight. At select locations bulk samples were taken from the auger’s cuttings and placed in sealed containers.

Borings B-1, B-2, B-5, B-6, B-8, B-101, and B-102 were drilled using the hollow-stem auger. Relatively undisturbed samples were obtained using a brass ring-lined sampler driven into the soil by a 140-pound “free-fall” hammer dropping 30 inches. The number of blows needed to drive the sampler into the soil was recorded as the penetration resistance. Due to the use of a “free-fall” hammer (rather than a hammer attached to a rope), the blow-counts recorded with the (D) sampler are approximately equal to the Standard Penetration Test blow-count (N_{60}). Drives less than 12 inches are denoted with the blow count per length of drive.

At selected locations, disturbed samples were obtained using a split-spoon sampler by means of the Standard Penetration Test (SPT, ASTM D 6066). The spoon sampler was driven into the soil by a 140-pound hammer dropping 30 inches. After an initial seating drive of 6 inches, the number of blows needed to drive the sampler into the soil a depth of 12 inches was recorded as the penetration resistance. These values are the raw uncorrected blowcounts.

Borings B-3 and B-4 were drilled using the limited access spiral auger rig. Relatively undisturbed samples were obtained using a brass-ring lined sampler (ASTM D3550). The drive sampler has an inside diameter of 2.42 inches and an outside diameter of 3.25 inches. The brass rings have an inside diameter of 2.42 inches and a height of 1-inch. In addition, relatively disturbed bulk samples were obtained at various depths. The drive sampler is driven into the soil using a drop of 12 inches with a driving weight of the Kelly bar. The number of blows needed to drive the sampler was recorded as the penetration resistance. It should be noted that the number of blows, in this case, is much lower than the standard penetration resistance because of the greater driving weight. At select locations bulk samples were taken from the auger's cuttings and placed in sealed containers.

The test pits were performed using a backhoe with a 24-inch wide bucket. The test pits were excavated to depths of 5 to 16 feet to expose the subsurface materials for observation and sampling by our Certified Engineering Geologist. Upon completion, the test pits were backfilled with the excavated materials.

The field exploration for the investigation was performed under the continuous technical supervision of GPI's representative, who visually inspected the site, maintained detailed logs of the borings, classified the soils encountered, and obtained relatively undisturbed samples for examination and laboratory testing. The soils encountered in the boring were classified in the field and through further examination in the laboratory in accordance with the Unified Soils Classification System. Detailed logs of the borings and test pits are presented in Figures A-1 through A-17 in this appendix. Borings B-3, B-4, B-7 and B-9, as well as the six test pits, were down-hole logged by our Engineering Geologist to evaluate bedding conditions.

The boring locations were laid out in the field by measuring from existing features at the site. Upon completion, the borings were backfilled with the excavated soil cuttings. The ground surface elevation at the boring location was estimated from Google Earth, and the concept study plan from AHT Architects, and should be considered approximate.

MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
					This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	
			B	0	5-INCH AC	
					Fill: **CLAY (CL)** dark brown, very moist, very stiff, with sand, with siltstone gravel fragments	
18.5		39	D			1085
18.5	93	27	D	5	Natural COLLUVIUM (Qc): **CLAY (CL)** dark brown, very moist, very stiff, with sand, with siltstone fragments, abundant white caliche	
20.8	97	27	D		@ 7 feet, trace siltstone gravel fragments	1080
13.6	98	24	D	10	**SANDY CLAY (CL)** dark brown, moist, very stiff, trace siltstone gravel, abundant caliche, minor porosity	1075
25.4	90	47	D	15	@ 15 feet, wet, hard, with gravel	1070
15.8	105	50/5"	D	20	@ 20 feet, moist	
8.9	111				UPPER TOPANGA FORMATION (Ttus): **SANDSTONE** brown, moist, very dense, with gravel, weathered	1065
19.4		43	S	25	**WEATHERED SILTSTONE** light brown, wet, medium dense	1060
15.8	80	50/5"	D	30	@ 30 feet, brown, moist, hard	1055
12.5		68	S	35	**SANDSTONE** grey, very moist, very dense, mottled with orange-brown iron oxide	1050

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED:
11-10-22

EQUIPMENT USED:
8 " HOLLOW STEM AUGER

GROUNDWATER LEVEL (ft):
NOT ENCOUNTERED

GPI

PROJECT NO.: 3162.I

KIA CALABASAS

LOG OF BORING NO. B-101

FIGURE A-1

	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	ELEVATION (FEET)
	15.5	106	50/3"	D	40	@ 40 feet, wet	
							1045
	14.3		50/6"	S	45	**SILTSTONE** grey, moist, hard	
							1040
	12.0	120	82	D	50	**SANDSTONE** grey, wet, very dense, trace orange-brown	
						Total Depth 51 feet	

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED:
11-10-22

EQUIPMENT USED:
8 " HOLLOW STEM AUGER

GROUNDWATER LEVEL (ft):
NOT ENCOUNTERED

GPI

PROJECT NO.: 3162.I
KIA CALABASAS

LOG OF BORING NO. B-101

FIGURE A-1

	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	*DESCRIPTION OF SUBSURFACE MATERIALS* This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	ELEVATION (FEET)
				B	0	4-INCH AC OVER 5-INCH BASE	1105
						Fill: **SANDY SILT (ML)** brown, moist, stiff	
	12.0	89	13	D		Natural COLLUVIUM (Qc): **SILTY SAND (SM)** brown, moist, loose	
	9.3	102	37	D	5	**SANDY SILT (ML)** brown, slightly moist, very stiff	1100
	6.5	110	51	D		@ 7 feet, hard	
	21.4	99	41	D	10	UPPER TOPANGA FORMATION (Ttus): **SILTSTONE** light brown, very moist, very stiff, weathered	1095
	6.0	114	50/4"	D	15	**SANDSTONE** light grey, slightly moist, very dense	1090
	14.3	115	50/3"	D	20	**SILTSTONE** grey, moist, hard	1085
	7.9		50/4"	S	25	**SANDSTONE** brown, moist, very dense	1080
	8.6	102	50/4"	D	30		1075
						Total Depth 31 feet	

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED:
11-10-22

EQUIPMENT USED:
8 " HOLLOW STEM AUGER

GROUNDWATER LEVEL (ft):
NOT ENCOUNTERED

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PROJECT NO.: 3162.I

KIA CALABASAS

LOG OF BORING NO. B-102

FIGURE A-2

MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	*DESCRIPTION OF SUBSURFACE MATERIALS* This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	ELEVATION (FEET)
				0	3" PCC over 3" AB	
					Fill: **SANDY CLAY (CL)** dark brown, very moist, hard	
21.6	101	18	D			
			B			
						80
21.6	97	26	D	5	Natural: COLLUVIUM (Qc):**SANDY CLAY (CL)** dark brown, very moist, very stiff	
19.9	100	23	D		Below 7 feet, hard	
						75
19.8	102	30	D	10		
18.7		19	S			
30.1		20	S		Below 13 feet, wet	
						70
27.2		18	S	15		
37.8		25	S			65
				20	Total Depth 20 feet Well installed to depth of 10 feet	

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED:
5-18-16

EQUIPMENT USED:
8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft):
Not Encountered

PROJECT NO.: 3162.I

KIA CALABASAS

LOG OF BORING NO. B-1

FIGURE A-3

MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	*DESCRIPTION OF SUBSURFACE MATERIALS* This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	ELEVATION (FEET)
				0	Fill: **SANDY CLAY (CL)** brown, moist, stiff	
9.9	87	12	D			
			B			100
				5		
14.2	92	16	D		@ 5 feet, hard	
					Natural: COLLUVIUM (Qc): **SILTY SAND (SM)** brown, slightly moist, medium dense	
9.1	92	16	D			
						95
				10		
9.7	103	45	D			
					UPPER TOPANGA FORMATION (Ttus):**SANDSTONE** light brown, slightly moist, very dense	
8.1		53	S			
5.2		50/5"	S			
						90
				15		
5.8		50/5"	S			
6.3		50/5"	S			85
				20	Total depth 20 feet Well installed to depth of 9.5 feet	

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED:
5-18-16

EQUIPMENT USED:
8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft):
Not Encountered

GPI

PROJECT NO.: 3162.I

KIA CALABASAS

LOG OF BORING NO. B-2

FIGURE A-4

MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	*DESCRIPTION OF SUBSURFACE MATERIALS* This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	ELEVATION (FEET)
				0	Fill: **SILT (ML)** brown, loose, dry	160
14.7			B		Natural: UPPER TOPANGA FORMATION (Ttus): **SILTSTONE** yellow brown and olive grey, moist @ 2.4 feet, B: N82E, 12NW, continuous 1-2" thick tuffaceous bed Below 4 feet, very hard	
12.0			B	5		155
11.8			B			
15.8			B	10	@ 10 feet, very moist @ 11 feet, B: N84W, 18NE	150
21.5			B		@ 13 feet, wet	
2.8			B	15	**CONGLOMERATE** hard, dry, cemented gravel bed, well rounded, fine to medium	145
					SANDSTONE hard, grey, dry, fine to medium grained, massive throughout, moderately cemented	
2.3			B	20		140
6.7			B	25	Below 25 feet, slightly moist 26 to 28.5 feet, orange brown, oxidized zone	135
6.7			B	30		130
8.1			B	35	Total Depth 35 feet Refusal on hard, cemented layer	

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED: 5-24-16
EQUIPMENT USED: 24 " Limited Access Auger
GROUNDWATER LEVEL (ft): Not Encountered

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PROJECT NO.: 3162.I
KIA CALABASAS

LOG OF BORING NO. B-3

MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
			B	0	Fill: **GRAVEL (GP)** with scattered organic debris	160
3.2	96	42/12"	D		Natural: UPPER TOPANGA FORMATION (Ttus):**SANDSTONE** orange brown, dry, highly oxidized, fine grained, massive, caliche in thin veinlets, fracture fillings	
3.0			B	5	Below 1.5 feet, J: N60E, 90deg, grey, fractures iron oxide stained and filled with caliche, joints 2-4" spacing	155
3.0			B		@ 8.5 feet, FZ: N81W, 53NE, steeply dipping fault/shear zone with sandstone above and gray siltstone below	
14.8			B	10	**SILTSTONE** grey, moist, massive, zone of shearing	150
					@ 11.5 feet, S: N73E, 47NW	
19.1	109	20/11"	D		@ 12.5 feet, N70E, 77NW Below 13 feet, wet	
20.3	109	26/12"	D	15		145
					@ 17.5 feet, S: N72E, 63NW, slicken sides sub horizontal	
18.7	109	20/12"	D	20		140

Total Depth 22 feet
Refusal on hard, cemented layer

This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED:
5-24-16

EQUIPMENT USED:
24 " Limited Access Auger

GROUNDWATER LEVEL (ft):
Not Encountered

GPI

PROJECT NO.: 3162.I

KIA CALABASAS

LOG OF BORING NO. B-4

FIGURE A-6

MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
					This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	
			B	0	2" AC over 4" AB	110
25.4	70	13	D		Fill: **SANDY CLAY (CH)** brown with white, very moist, stiff, caliche	
19.7	67	19	D	5		105
19.5	84	32	D		Natural: COLLUVIUM (Qc):**SANDY CLAY (CL)** greyish brown with white, wet, very stiff, caliche shale fragments	
19.4	94	46	D	10	@ 10 feet, slightly moist, hard	100
27.7	84	46	D			
				15		95
35.9	77	34	D		UPPER TOPANGA FORMATION (Ttus):**SILTSTONE** white with brown streaks, wet, very stiff, caliche	
21.3	98	48	D	20	@ 20 feet, hard	90
15.2	115	97/11"	D	25	@ 25 feet, greyish brown, very moist, hard, trace caliche	85
15.6	111	50/5"	D	30		80
					Total depth of 31 feet	

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED:
5-19-16

EQUIPMENT USED:
8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft):
Not Encountered

GPI

PROJECT NO.: 3162.I

KIA CALABASAS

LOG OF BORING NO. B-5

FIGURE A-7

	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
						This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	
					0	3" AC over 4" AB	
	14.7	79	10	D		Fill: **SANDY CLAY (CH)** dark brown, slightly moist, firm, with roots, trace gravel	135
	12.1	78	20	D	5	Natural: COLLUVIUM (Qc): **SILT (ML)** dark brown with white, moist, stiff, porosity, trace sand, caliche	
	9.6	85	24	D		@ 7 feet, very stiff	130
	11.2	95	80/10"	D	10	UPPER TOPANGA FORMATION (Ttus):**SILTSTONE** light brown with white, moist, hard	
	10.3	104	90	D			125
					15	@ 15 feet light brown, trace sand	
	9.1	107	50/6"	D		@ 16 feet, slightly moist	120
	7.8	92	50/3"	D	20	**SANDSTONE** light brown, moist, hard	115
	9.1 5.8	99	50/1"	D	25		110
	8.8	105	50/3"	D	30	@ 30 feet, trace gravel	105
	18.1	118	50/3"	D	35	**SANDY SILTSTONE** light brown, wet, hard	100
	7.2	124	50/4"	D		Total depth of 40 feet	

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED:
5-19-16

EQUIPMENT USED:
8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft):
34

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PROJECT NO.: 3162.I

KIA CALABASAS

LOG OF BORING NO. B-6

FIGURE A-8

	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
						This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	
					0	Fill:**SILTY CLAY** light brown, dry	
						Natural: COLLUVIUM (Qc):**SILTY CLAY (CL)** dark brown, moist, stiff, colluvium, with sparse shale fragments,	150
	6.2	114	5/9"	D	5	UPPER TOPANGA FORMATION (Ttus):**SILTSTONE** grey with whitish caliche, massive-poorly bedded, weathered, hard, B:N70E, 34NW	
	16.4	108	5/12"	D		**SANDSTONE** light brownish grey, slightly moist, fine grained, massive, very tight, no visible fractures	145
	16.5	108	5/12"	D	10	**SILTSTONE** grey, very moist, very hard, poorly bedded B: N66E, 38NW @ 9.5 feet B: N61E, 38NW @ 10.5 feet, B: N56E, 33NW	
	12.7	107	5/6"	D		**SANDSTONE** yellow brown, very moist, hard, massive	140
	12.3	115	5/8"	D	15		
						Total depth of 18 feet Refusal on hard, cemented layer	135

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED:
5-18-16

EQUIPMENT USED:
28 " EZ-Bore

GROUNDWATER LEVEL (ft):
Not Encountered

GPI

PROJECT NO.: 3162.I

KIA CALABASAS

LOG OF BORING NO. B-7

FIGURE A-9

MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	*DESCRIPTION OF SUBSURFACE MATERIALS* This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	ELEVATION (FEET)
			B	0	3" AC over 4" AB	
11.0	85	21	D		Natural: COLLUVIUM (Qc):**SANDY CLAY (CL)** dark brown with white, slightly moist, stiff, porosity	135
14.8	100	57	D	5	UPPER TOPANGA FORMATION (Ttus):**SILTSTONE** light brown with white, moist, hard, weathered, caliche	
6.2	97	34	D		@ 7 feet, slightly moist, very stiff, with sand	130
5.9	98	84/12"	D	10	Below 10 feet, hard, dry	
4.8	86	67/10"	D		Below 13 feet, light yellow brown, no caliche	125
				15		
4.8	94	68/10"	D			120
24.1	97	50	D	20	@ 20 feet, wet	115
11.2	107	50/5"	D	25	**SILTSTONE** light brownish yellow, slightly moist, hard	110
10.5	95	50/3"	D	30	**SANDSTONE** light brown, very moist, hard	
					Total Depth 31 feet	

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED:
5-19-16

EQUIPMENT USED:
8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft):
31

GPI

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KIA CALABASAS

LOG OF BORING NO. B-8

FIGURE A-10

MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
				0	3" AC over 5" AB	
					UPPER TOPANGA FORMATION (Ttus): **SILTSTONE** mottled grey and brown, moist, hard, massive, vaguely bedded, with whitish caliche on fracture surfaces	
10.9	84	5/6"	D			
					@ 3.5 feet, B: N52W, 16NE	145
8.3	86	5/6"	D	5		
					@ 5.5 feet, B: N53W, 12NE	
13.5	100	4/6"	D		@ 7.5 feet, B: N53W, 13NE	140
15.8	100	5/6"	D	10		
					SANDSTONE grey, dry, fine to medium grained, massive, poorly cemented, easily friable, micaceous	
4.2	99	4/6"	D		Below 13.5 feet, yellow brown, tight, unfractured, trace rounded gravel	135
				15		
0.8	234	5/10"	D			
						130
6.3	83	4/5"	D	20	**CONGLOMERATE** slightly moist, hard, cemented, massive, well rounded with cobble	
					SANDSTONE yellow brown, moist, tight, fine grained, massive, unfractured	125
6.6	89	5/6"	D	25		
						120
					CONGLOMERATE cemented bed, clasts rounded, undulatory channel into sandstone below Generalized Bedding: N50W, 10NE	
15.6	110	5/6"	D	30		
					SANDSTONE grey, wet, fine grained, massive, tight	115
14.3	116	9/12"	D	35	From 35 to 36.5 feet, carbonate cemented zone, very hard, irregular	
					@ 37.5 feet, 1/4-inch thick clayey shear zone, irregular S: N65W, 46NE	110

This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED: 5-18-16

EQUIPMENT USED: 28 " EZ-Bore

GROUNDWATER LEVEL (ft): 46

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KIA CALABASAS

LOG OF BORING NO. B-9

FIGURE A-11

	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	ELEVATION (FEET)
	15.0	119	11/12"	D	40	**SANDSTONE** grey, wet, fine grained, massive, tight @ 40 feet, S: N51W, 41NE, sheared clayey zone, grey @ 42 feet, S: N55W, 44NE, oxidation along shear surface, continuous, thin clay shear	105
	12.2	111	10/6"	D	45		100
						Total Depth 49 feet	

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED:
5-18-16

EQUIPMENT USED:
28 " EZ-Bore

GROUNDWATER LEVEL (ft):
46

GPI

PROJECT NO.: 3162.I

KIA CALABASAS

LOG OF BORING NO. B-9

FIGURE A-11

PROBE (in)	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
					0	Fill: **SILTY SAND (SM)** yellow brown, slightly moist, with rootlets, 3/4" PVC water lines, porous	
						Natural: COLLUVIUM (Qc) **SILTY SAND (SM)** orange brown, slightly moist, porous, with roots to 1/2-inch diameter Gradational contact with sandstone below	110
					5	UPPER TOPANGA FORMATION (Ttus):**SANDSTONE** yellow and orange brown, fine grained, irregular discontinuous pebble and cobble beds, slightly moist, very dense	
						Total Depth 6 feet	

DESCRIPTION OF SUBSURFACE MATERIALS: This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED:
5-24-16
EQUIPMENT USED:
24" Backhoe
GROUNDWATER LEVEL (ft):
Not Encountered

GPI

PROJECT NO.: 3162.I
KIA CALABASAS

LOG OF BORING NO. TP-1

FIGURE A-12

PROBE (in)	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
					0	Fill: **SILTY CLAY (CH)** brown, slightly moist to moist, firm to stiff, with sand and whitish shale fragments, trace porosity with caliche as small irregular masses, tubules	110
					5		105
					10	Natural: COLLUVIUM (Qc): **SILTY CLAY (CL)** dark brown, moist, stiff, with shale fragments, calcium abundant as irregular tubules, few roots	100
						Total Depth 13.5 feet	

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED:
5-24-16
EQUIPMENT USED:
24" Backhoe
GROUNDWATER LEVEL (ft):
Not Encountered

GPI

PROJECT NO.: 3162.I
KIA CALABASAS

LOG OF BORING NO. TP-2

FIGURE A-13

PROBE (in)	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
					0	Natural: Residual: **CLAYEY SILT (ML)** brown, dry to slightly moist, loose, with shale fragments, crumbly with roots to 1-inch diameter	160
						UPPER TOPANGA FORMATION (Ttus):**SILTSTONE** yellow brown and grey, hard, massive, vaguely bedded, moderately fractured @ 2 feet, B: N36W, 16NE @ 3.5 feet, B: N71W, 15NE, white, weathered 1 to 2-inch thick tuff bed, continuous B: N66W, 15NE	
					5	Total Depth 5 feet	

This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED: 5-24-16
EQUIPMENT USED: 24" Backhoe
GROUNDWATER LEVEL (ft): Not Encountered

GPI

PROJECT NO.: 3162.I
KIA CALABASAS

LOG OF BORING NO. TP-3

FIGURE A-14

PROBE (in)	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
					0	Natural: COLLUVIUM (Qc): **SILTY CLAY (CL)** dark brown, slightly moist, firm to stiff, porous, few roots, with shale fragments Below 1.5 feet, abundant white caliche in thin veinlets	135
					5	UPPER TOPANGA FORMATION (Ttus): **SILTSTONE** olive grey, very tight, highly weathered into 1/2 to 1-inch diameter pieces with caliche, no continuous structure, massive Below 9 feet, less weathered, massive siltstone	130
					10	Total Depth 11 feet	

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED: 5-24-16
EQUIPMENT USED: 24" Backhoe
GROUNDWATER LEVEL (ft): Not Encountered

GPI

PROJECT NO.: 3162.I
KIA CALABASAS

LOG OF BORING NO. TP-4

FIGURE A-15

PROBE (in)	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
					0	Natural: Residual: **SILTY CLAY (CL)** dark brown, dry, many roots to 1/2-inch diameter	
						UPPER TOPANGA FORMATION (Ttus):**SILTSTONE** yellow brown, hard, poorly bedded	145
					5		
							140
					10	@ 9.5 feet, 6-inch thick orange weathering carbonate bed, overall slightly fractured, very tight @ 11 feet, B: N76E, 13NW	
							135
					15	**SANDSTONE** grey, slightly moist, easily friable, massive	
						Total Depth 16 feet	

DESCRIPTION OF SUBSURFACE MATERIALS

This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED: 5-24-16
EQUIPMENT USED: 24" Backhoe
GROUNDWATER LEVEL (ft): Not Encountered

GPI

PROJECT NO.: 3162.I
KIA CALABASAS

LOG OF BORING NO. TP-5

FIGURE A-16

PROBE (in)	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
					0	Natural: COLLUVIUM (Qc): **SILTY CLAY (CL)** dark brown, slightly moist, firm, very porous, trace shale, roots and rootlets to 1/2-inch diameter	
					5	**SANDY SILT (ML)** yellow brown, slightly moist, very porous, with shale fragments	120
					10		115
						UPPER TOPANGA FORMATION (Ttus):**SANDSTONE** yellow brown, fine grained, massive, slightly moist	
						Total Depth 11.5 feet	

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED: 5-24-16
EQUIPMENT USED: 24" Backhoe
GROUNDWATER LEVEL (ft): Not Encountered

GPI

PROJECT NO.: 3162.I
KIA CALABASAS

LOG OF BORING NO. TP-6

FIGURE A-17

APPENDIX B

APPENDIX B

LABORATORY TESTS

INTRODUCTION

Representative undisturbed soil samples and bulk samples were carefully packaged in the field and sealed to prevent moisture loss. The samples were then transported to our Cypress office for examination and testing assignments. Laboratory tests were performed on selected representative samples as an aid in classifying the soils and to evaluate the physical properties of the soils affecting foundation design and construction procedures. Detailed descriptions of the laboratory tests are presented below under the appropriate test headings. Test results are presented in the figures that follow.

MOISTURE CONTENT AND DRY DENSITY

Moisture content and dry density were determined from a number of the ring samples. The samples were first trimmed to obtain volume and wet weight and then dried in accordance with ASTM D2216. After drying, the weight of each sample was measured, and moisture content and dry density were calculated. Moisture content and dry density values are presented on the boring logs in Appendix A.

GRAIN SIZE DISTRIBUTION

Selected soil samples were dried, weighed, soaked in water until individual soil particles were separated, and then washed on the No. 200 sieve. That portion of the material retained on the No. 200 sieve was oven-dried and weighed to determine the percentage of the material passing the No. 200 sieve. (ASTM D1140) The percentages passing the No. 200 sieve are tabulated below.

BORING NO.	DEPTH (ft)	SOIL DESCRIPTION	PERCENT PASSING No. 200 SIEVE
B-1	11	Sandy Clay (CL)	59
B-2	11	Silty Sand (SM)	32
B-101	0-5	Clay (CL) with Sand	78
B-101	10	Sandy Clay (CL)	51
B-102	5	Sandy Silt (ML)	50
B-102	15.5	Siltstone	50

ATTERBERG LIMITS

Liquid and plastic limits were determined for select samples in accordance with ASTM D4318. The results of the Atterberg Limits tests are presented in Figure B-1.

CONSOLIDATION

One-dimensional consolidation testing was performed on undisturbed samples in accordance with ASTM D 2435. After trimming the ends, the samples were placed in the consolidometer and loaded to 0.25 and 0.4 ksf. Thereafter, the samples were incrementally loaded to a maximum load of 25.6 and 32 ksf. The samples were inundated at 1 and 1.6 ksf. Sample deformation was measured to 0.0001 inch. Rebound behavior was investigated by unloading the samples back to 0.25 and 0.4 ksf. Results of the consolidation tests, in the form of percent consolidation versus log pressure, are presented in Figures B-2 to B-5.

DIRECT SHEAR

Direct shear tests were performed on undisturbed samples in accordance with ASTM D3080. The samples were placed in the shear machine, and a normal load comparable to the in-situ overburden stress was applied. The samples were inundated, allowed to consolidate, and then were sheared to failure. The tests were repeated on additional test specimens under increased normal loads. Shear stress and sample deformation were monitored throughout the test. For samples B-4 @ 20-feet, and B-6 @ 13-feet multiple passes were made on the specimens at a strain rate of 0.025, and 0.04 inches per minute respectively. Shear stress and sample deformation were monitored throughout the tests. The results of the direct shear tests are presented in Figures B-6 to B-13.

COMPACTION TEST

A maximum dry density/optimum moisture test was performed in accordance with ASTM D 1557 on a representative bulk sample of the site soils. The test results are as follows:

BORING NO.	DEPTH (ft)	SOIL DESCRIPTION	MAXIMUM DRY DENSITY (pcf)	OPTIMUM MOISTURE (%)
B-5	0-5	Sandy Clay (CH)	108	17.0
B-101	0 – 5	Clay (CL) with Sand	108	18

R-VALUE

Suitability of the near-surface soils for pavement support was evaluated by conducting an R-Value test. The test was performed in accordance with ASTM D 2844 by Geologic Associates under subcontract to GPI. The result of the test is as follows:

BORING NO.	DEPTH (ft.)	SOIL DESCRIPTION	R-VALUE BY EXPANSION
B-1	0-5	Sandy Clay (CL)	7

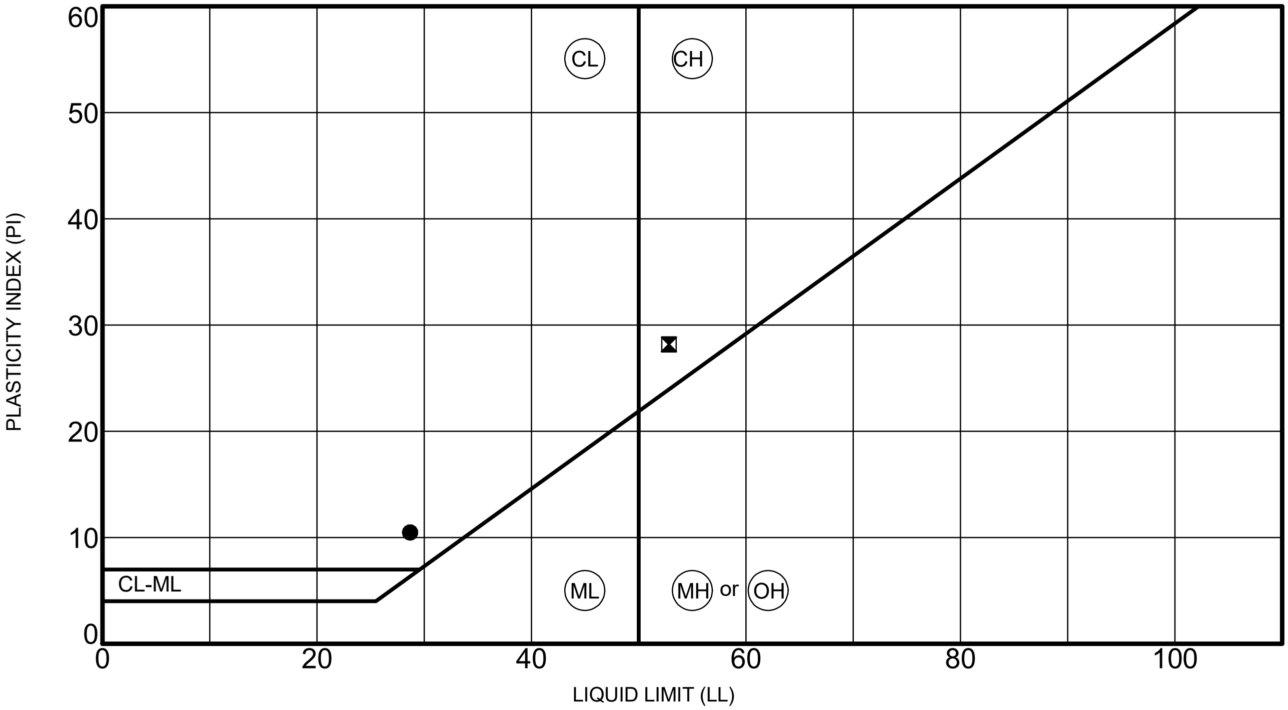
EXPANSION INDEX

An expansion index test was performed on a bulk sample. The test was performed in accordance with ASTM D4829, to assess the expansion potential of on-site soils. The results of the test are summarized below:

BORING NO.	DEPTH (ft)	SOIL DESCRIPTION	EXPANSION INDEX
B-1	0-5	Sandy Clay (CL)	7
B-101	0 – 5	Clay (CL) with Sand	65

CORROSIVITY

Soil corrosivity testing was performed by HDR a soil samples provided by GPI. The test results are summarized in Table 1 of this Appendix.



	SAMPLE LOCATION		LL	PL	PI	Fines, %	Classification
●	B-2	0.0	29	18	10		CLAY (CL)
⧗	B-5	0.0	53	25	28		SANDY CLAY (CH)

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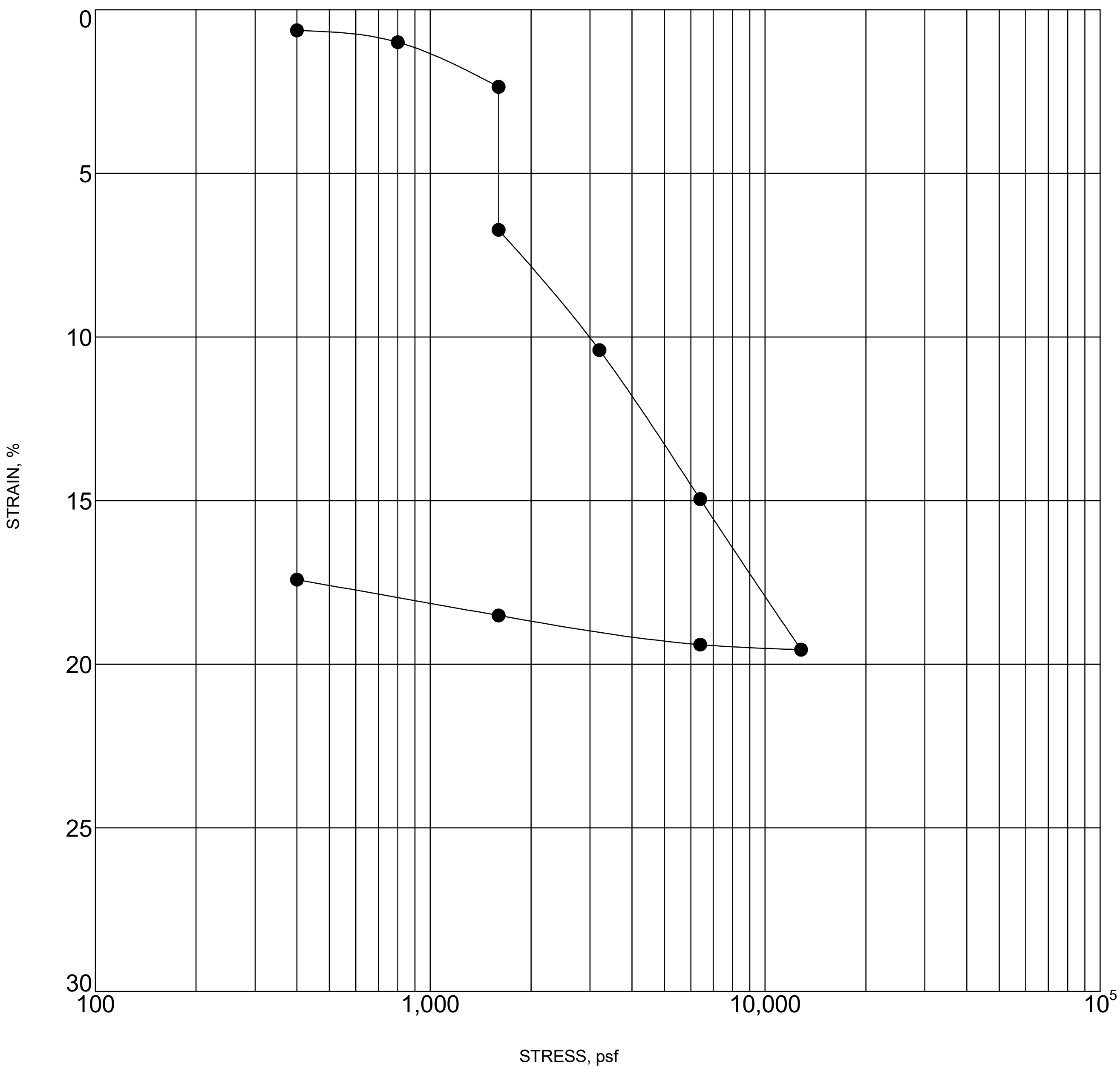
ATTERBERG LIMITS TEST RESULTS

FIGURE B-1



Sample inundated at 1600 psf

	Sample Location		Classification	DD,pcf	MC,%
●	B-5	5.0	FILL - SANDY CLAY (CH)	67	19.7

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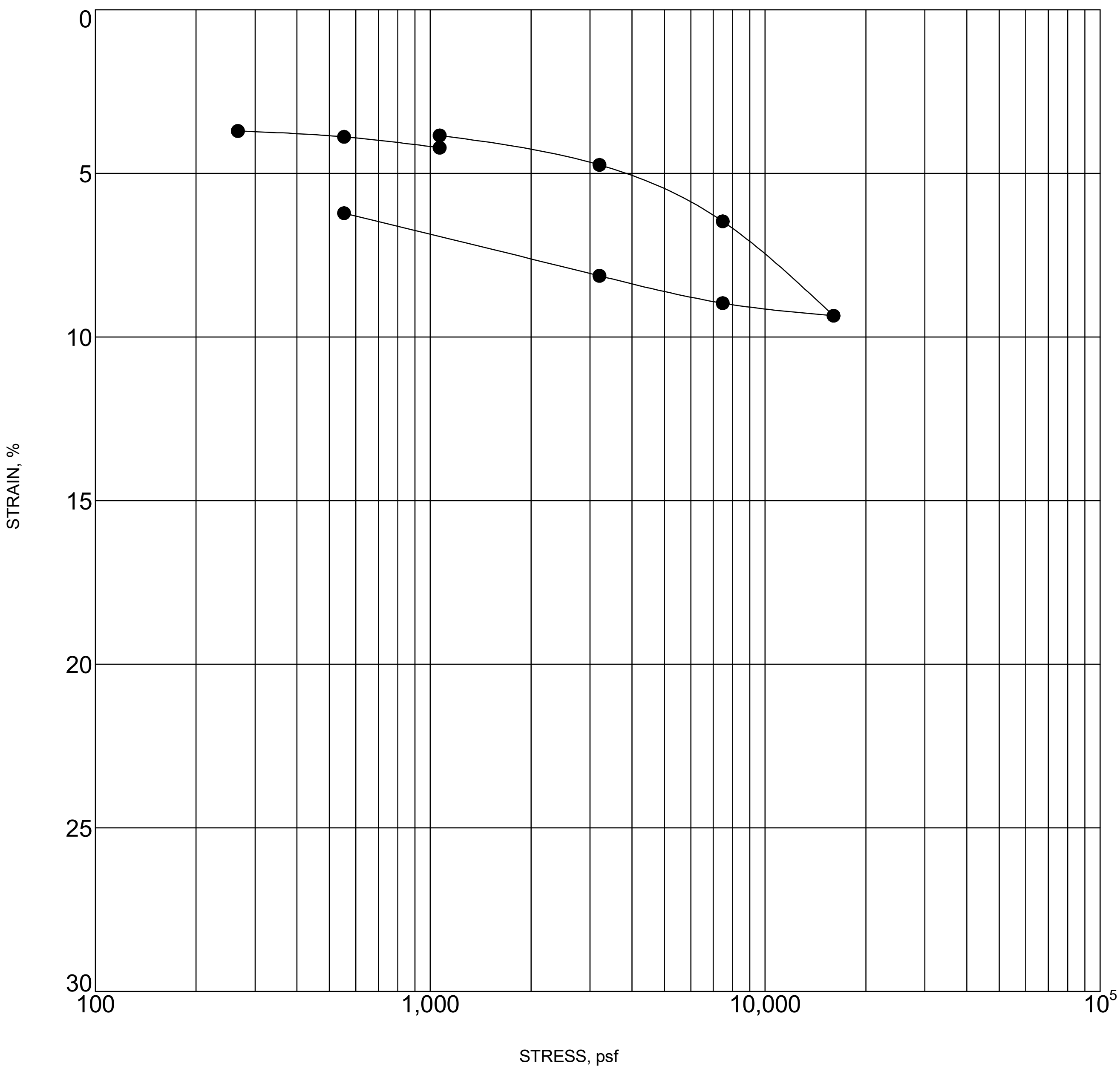
CONSOLIDATION TEST RESULTS

FIGURE B-2



Sample inundated at 1000 psf

	Sample Location		Classification	DD,pcf	MC,%
●	B-5	13.0	SANDY CLAY (CL)	84	27.7

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PROJECT NO.: 3162.I

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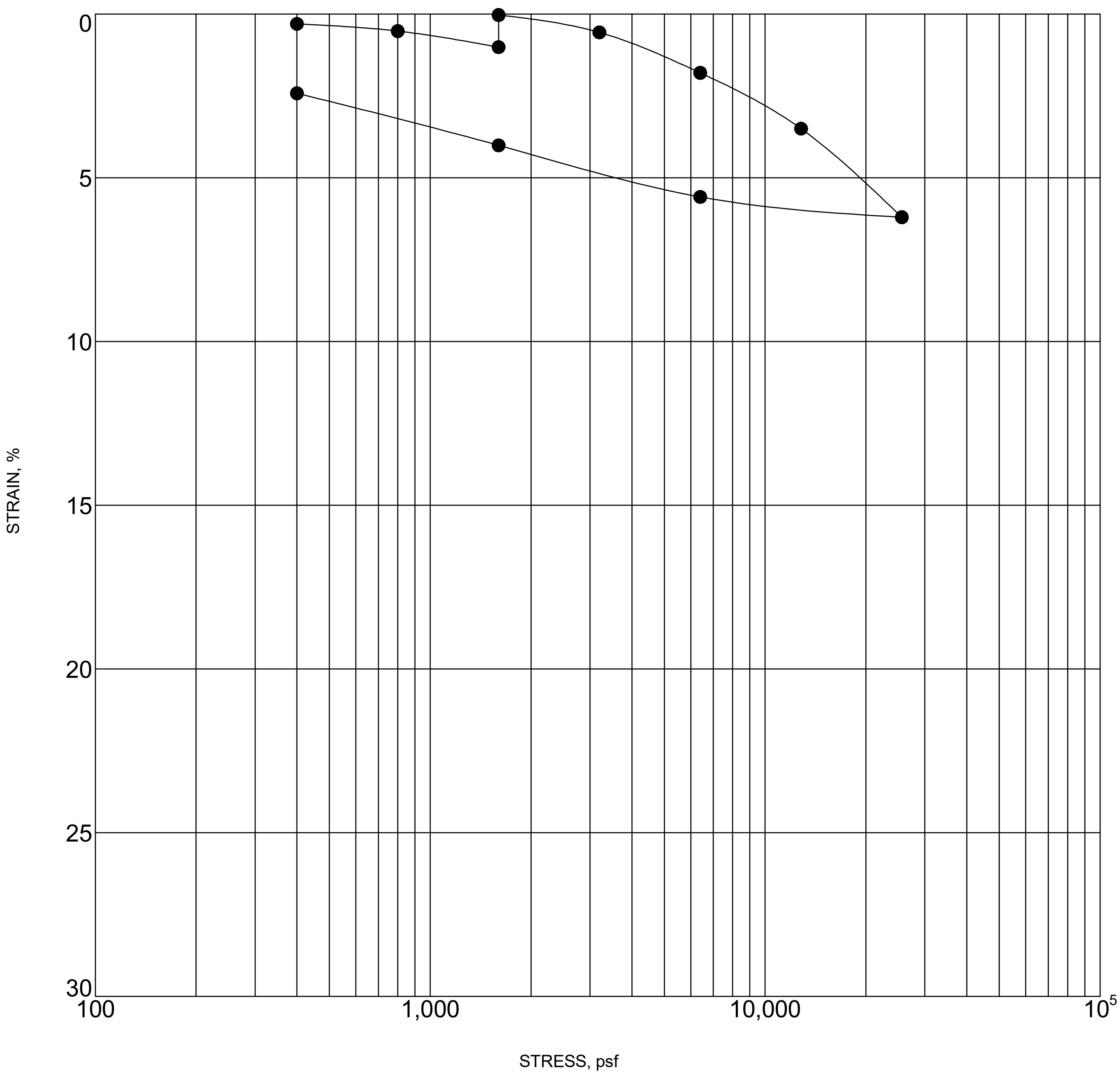
CONSOLIDATION TEST RESULTS

FIGURE B-3



Sample inundated at 1600 psf

	Sample Location		Classification	DD,pcf	MC,%
●	B-101	5.0	CLAY (CL)	93	18.5

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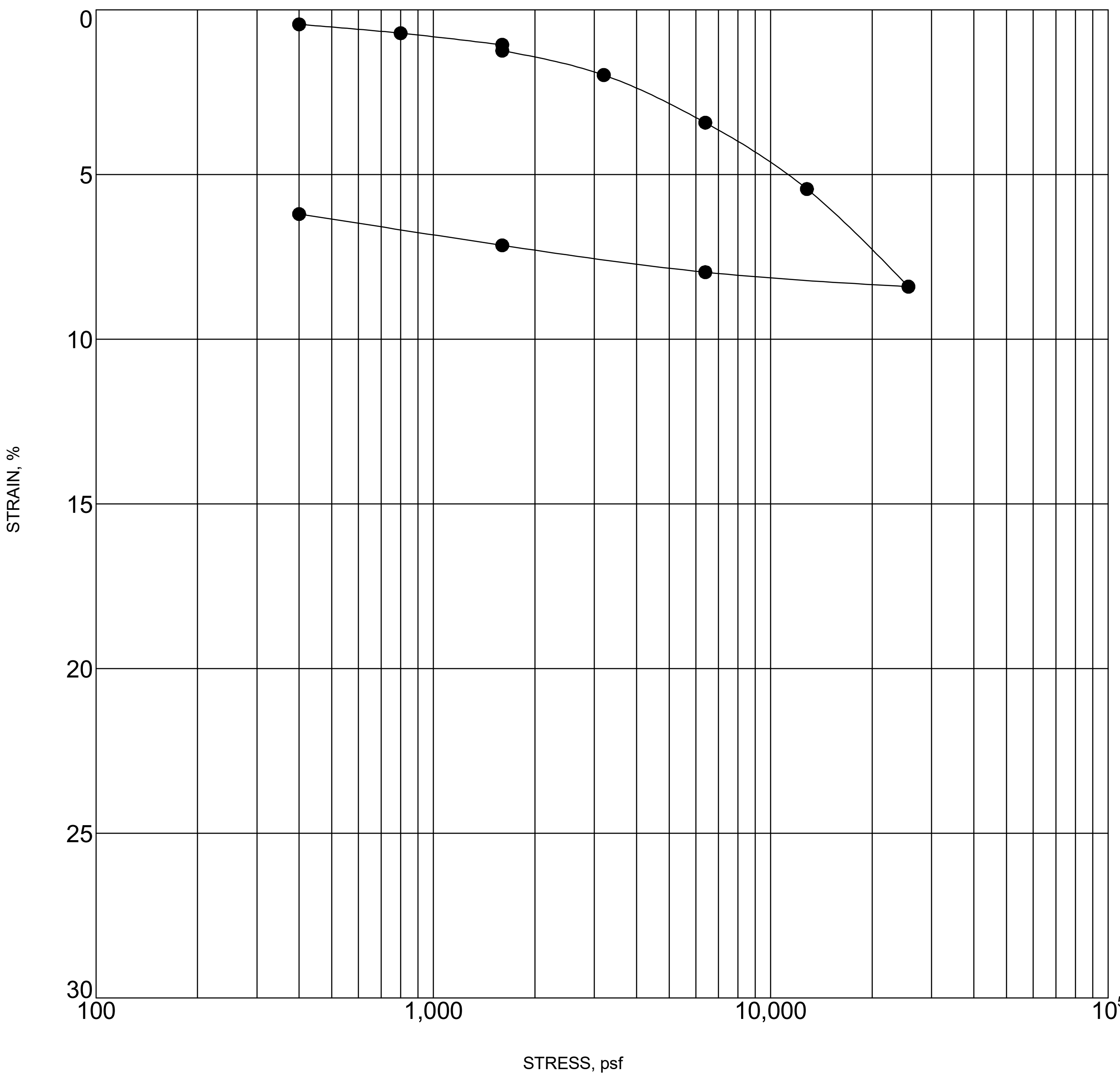
CONSOLIDATION TEST RESULTS

FIGURE B-4



Sample inundated at 1600 psf

	Sample Location		Classification	DD,pcf	MC,%
●	B-101	10.0	SANDY CLAY (CL)	98	13.6

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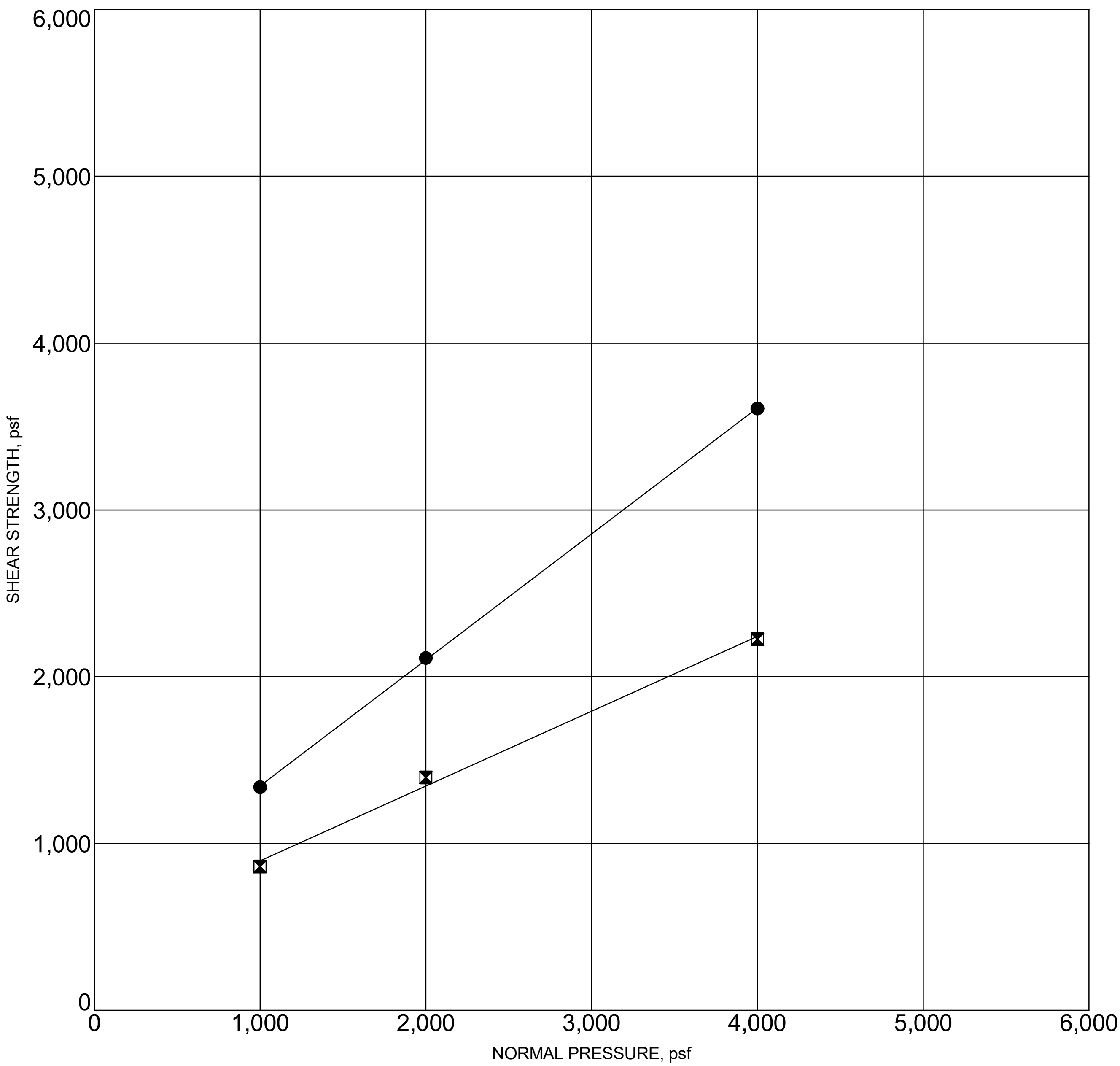
CONSOLIDATION TEST RESULTS

FIGURE B-5



● PEAK STRENGTH
Friction Angle= 37 degrees
Cohesion= 590 psf

⧗ ULTIMATE STRENGTH
Friction Angle= 24 degrees
Cohesion= 448 psf

Sample Location		Classification	DD,pcf	MC,%
B-4	13.0	SILTSTONE	109	19.1

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PROJECT NO.: 3162.I

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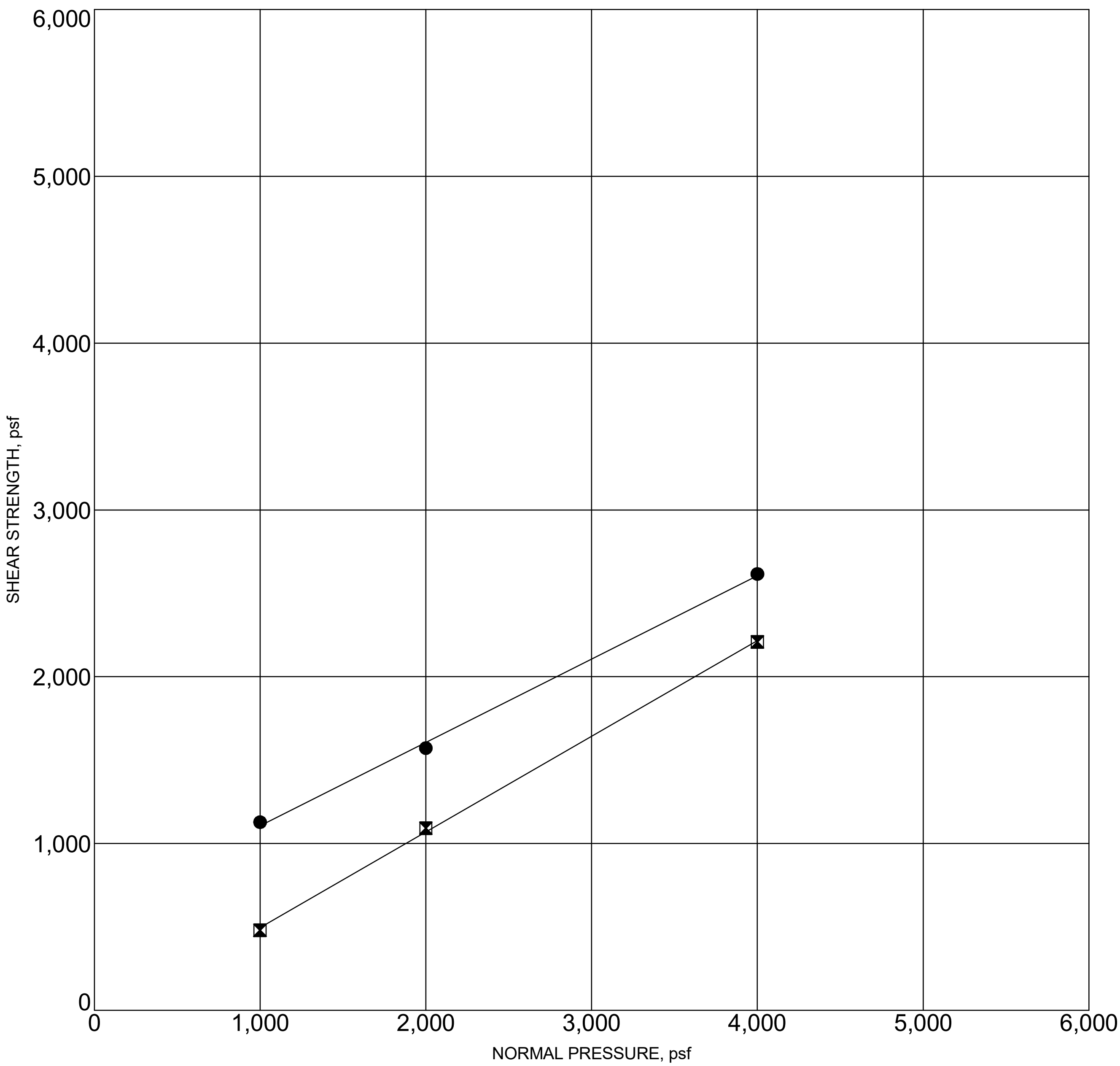
DIRECT SHEAR TEST RESULTS

FIGURE B-6



● **PEAK STRENGTH**
Friction Angle= 27 degrees
Cohesion= 606 psf

ULTIMATE STRENGTH
Friction Angle= 29 degrees
Cohesion= 0 psf

Sample Location		Classification	DD,pcf	MC,%
B-4	20.0	SILTSTONE	109	18.7

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PROJECT NO.: 3162.I

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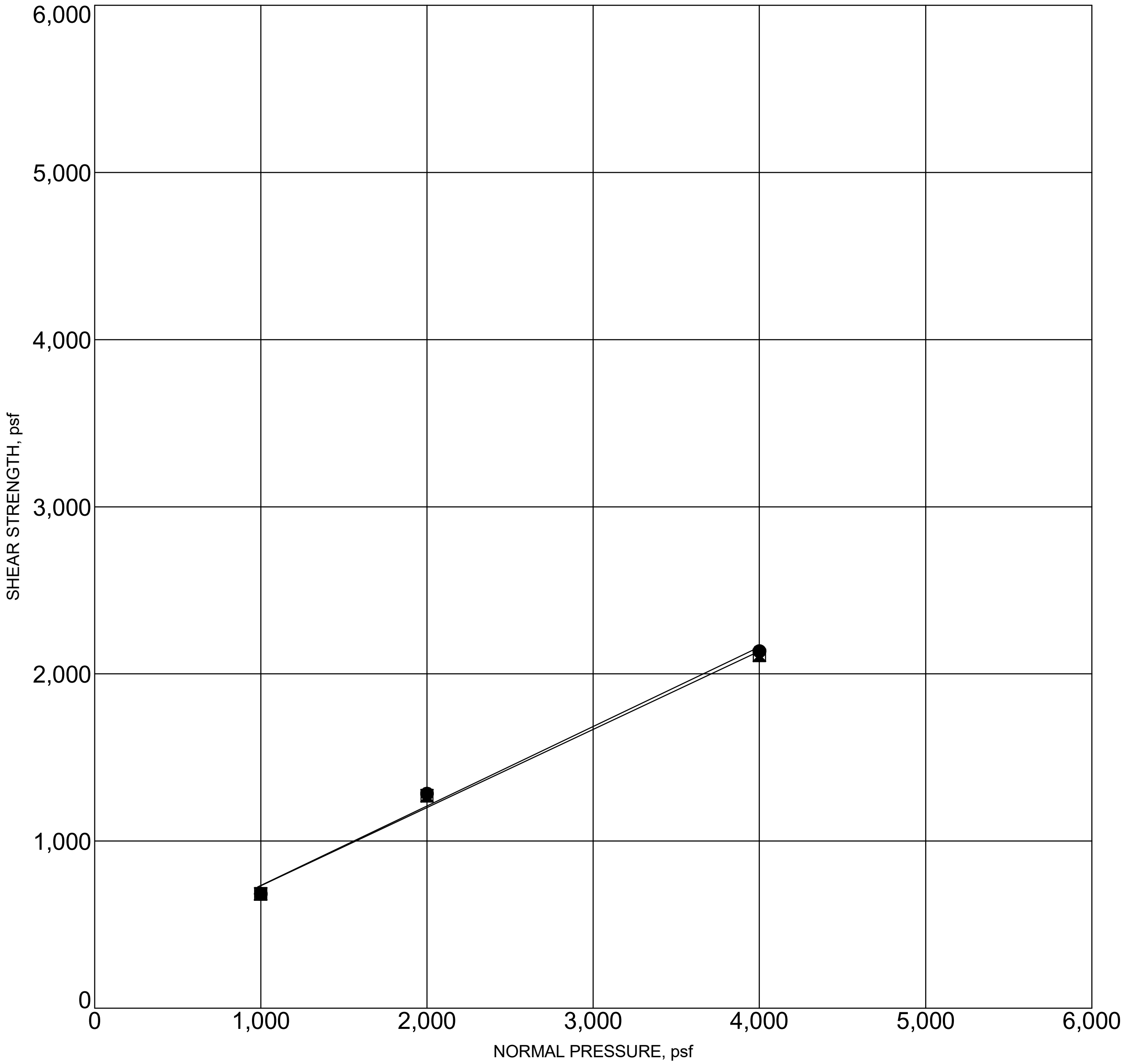
DIRECT SHEAR TEST RESULTS

FIGURE B-7



● **PEAK STRENGTH**
Friction Angle= 25 degrees
Cohesion= 258 psf

ULTIMATE STRENGTH
Friction Angle= 25 degrees
Cohesion= 264 psf

Sample Location		Classification	DD,pcf	MC,%
B-5	7.0	SANDY CLAY (CL)	84	19.5

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PROJECT NO.: 3162.I

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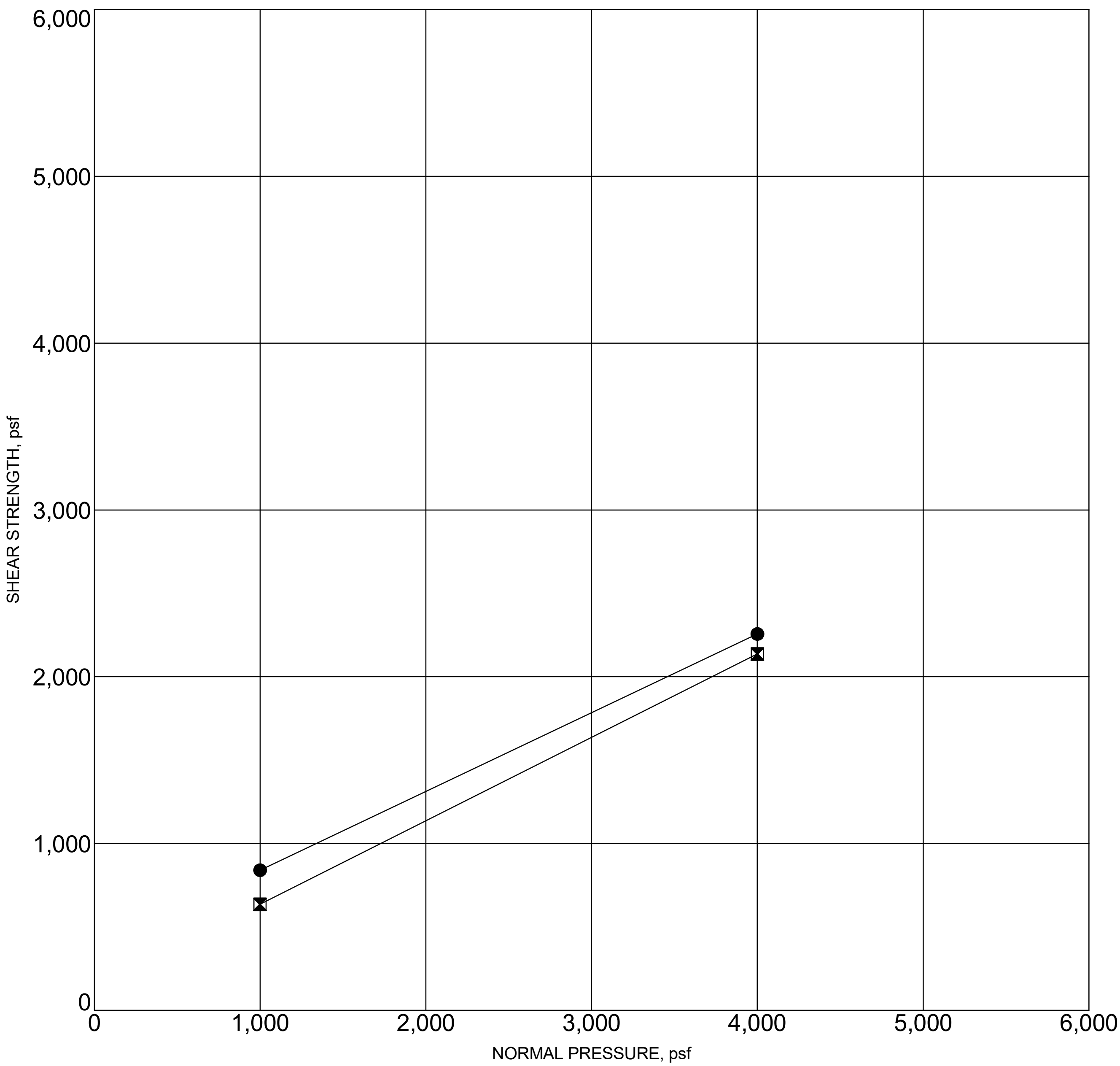
DIRECT SHEAR TEST RESULTS

FIGURE B-8



● **PEAK STRENGTH**
Friction Angle= 25 degrees
Cohesion= 368 psf

⧗ **ULTIMATE STRENGTH**
Friction Angle= 27 degrees
Cohesion= 136 psf

Sample Location		Classification	DD,pcf	MC,%
B-6	13.0	SILTSTONE	104	10.3

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PROJECT NO.: 3162.I

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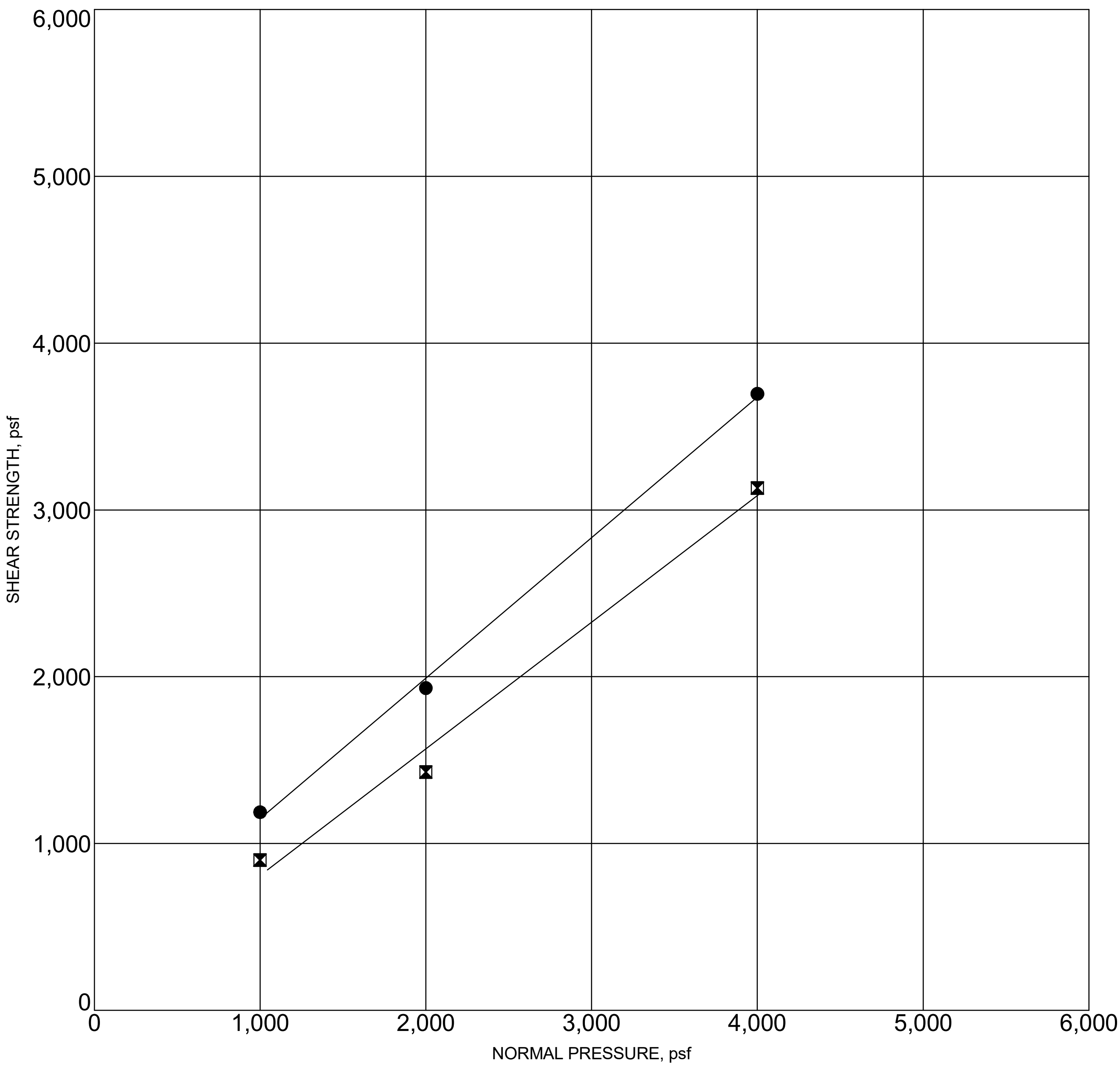
DIRECT SHEAR TEST RESULTS

FIGURE B-9



● **PEAK STRENGTH**
Friction Angle= 40 degrees
Cohesion= 306 psf

⧗ **ULTIMATE STRENGTH**
Friction Angle= 37 degrees
Cohesion= 48 psf

Sample Location		Classification	DD,pcf	MC,%
B-8	10.0	SILTSTONE	98	5.9

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PROJECT NO.: 3162.I

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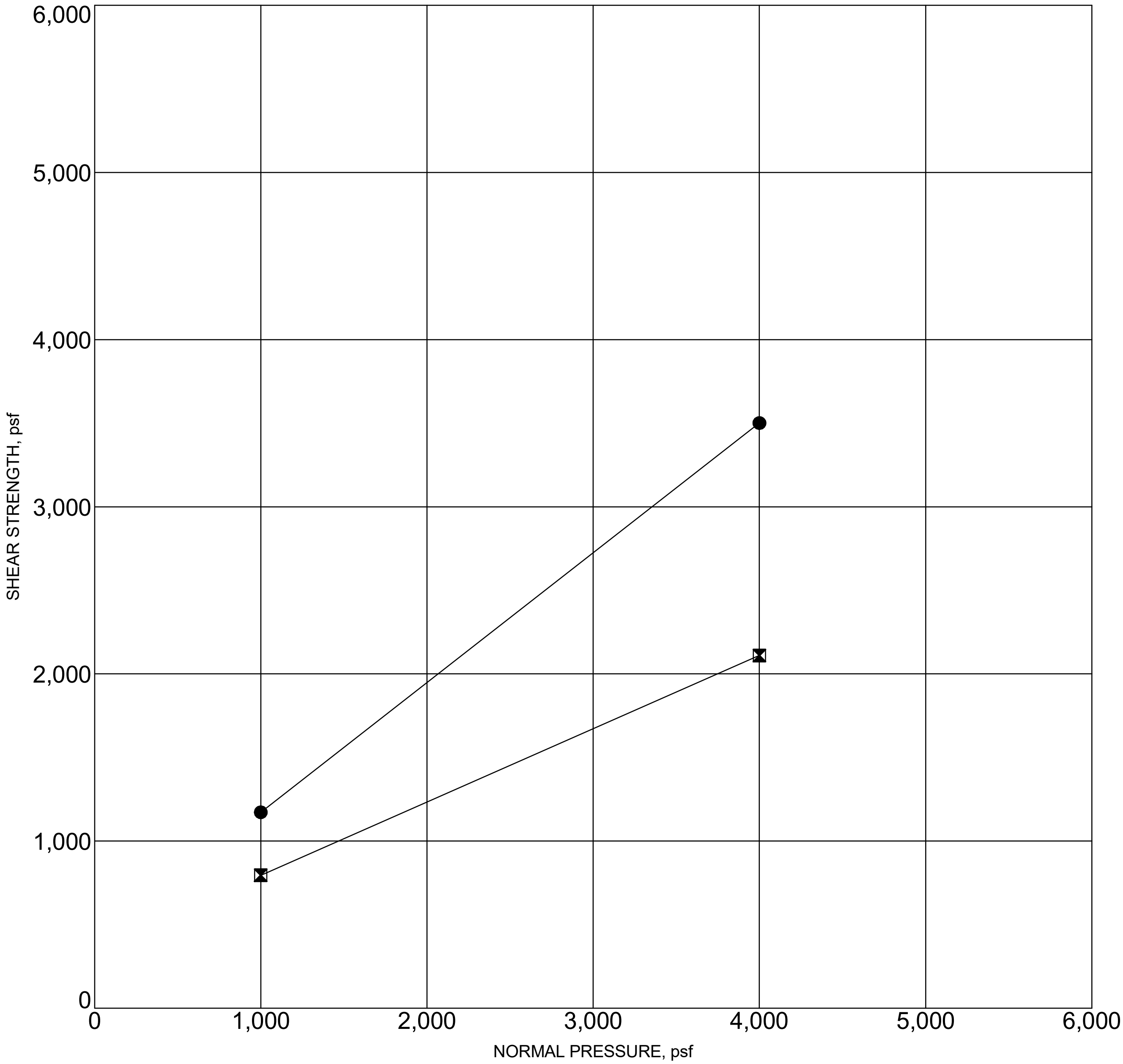
DIRECT SHEAR TEST RESULTS

FIGURE B-10



● **PEAK STRENGTH**
Friction Angle= 38 degrees
Cohesion= 396 psf

⧗ **ULTIMATE STRENGTH**
Friction Angle= 24 degrees
Cohesion= 357 psf

Sample Location		Classification	DD,pcf	MC,%
B-9	7.0	SILTSTONE	100	13.5

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PROJECT NO.: 3162.I

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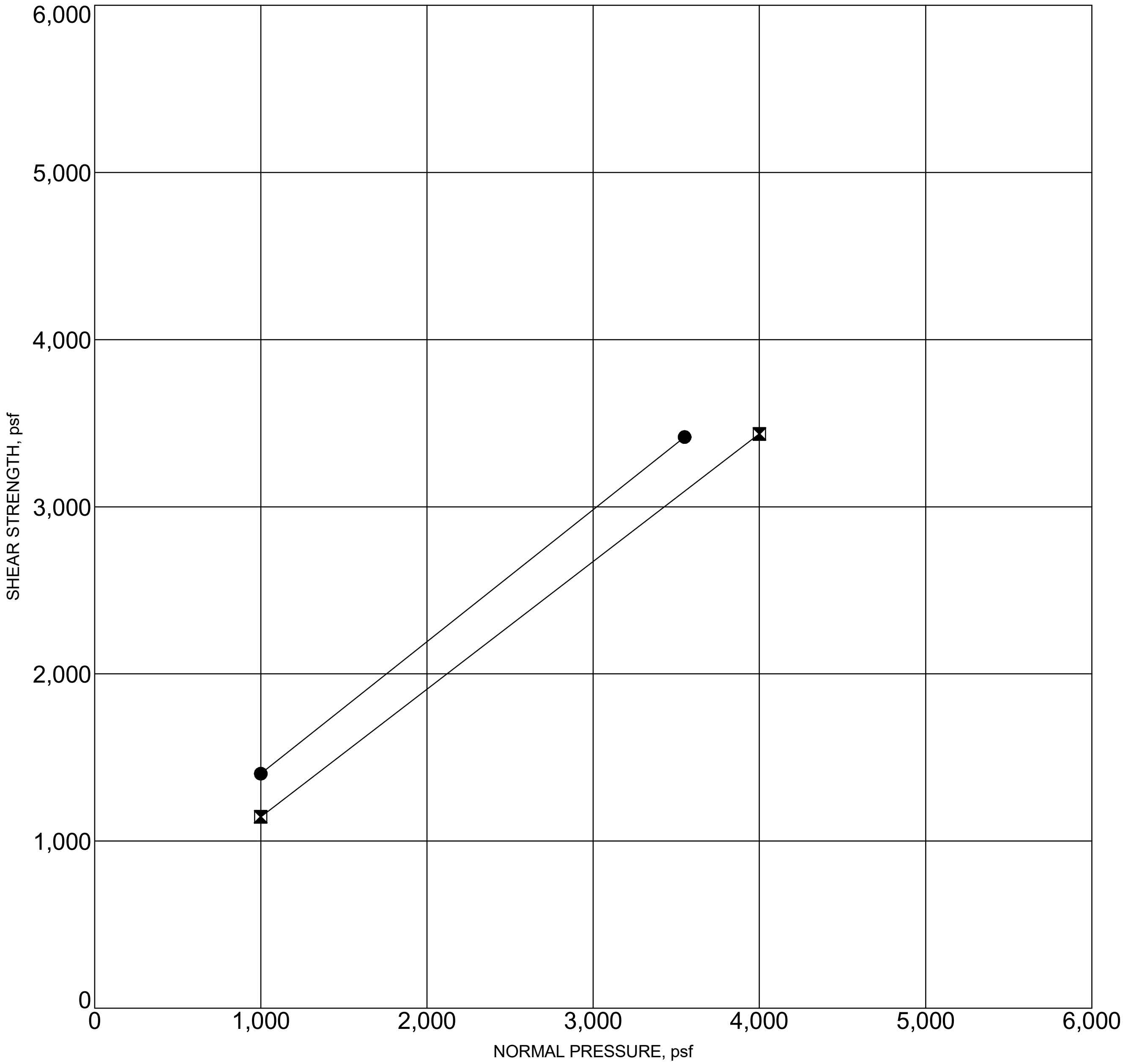
DIRECT SHEAR TEST RESULTS

FIGURE B-11



● **PEAK STRENGTH**
Friction Angle= 38 degrees
Cohesion= 613 psf

ULTIMATE STRENGTH
Friction Angle= 37 degrees
Cohesion= 381 psf

Sample Location		Classification	DD,pcf	MC,%
B-9	13.0	SANDSTONE	99	4.2

PROJECT: KIA CALABASAS

PROJECT NO.: 3162.I

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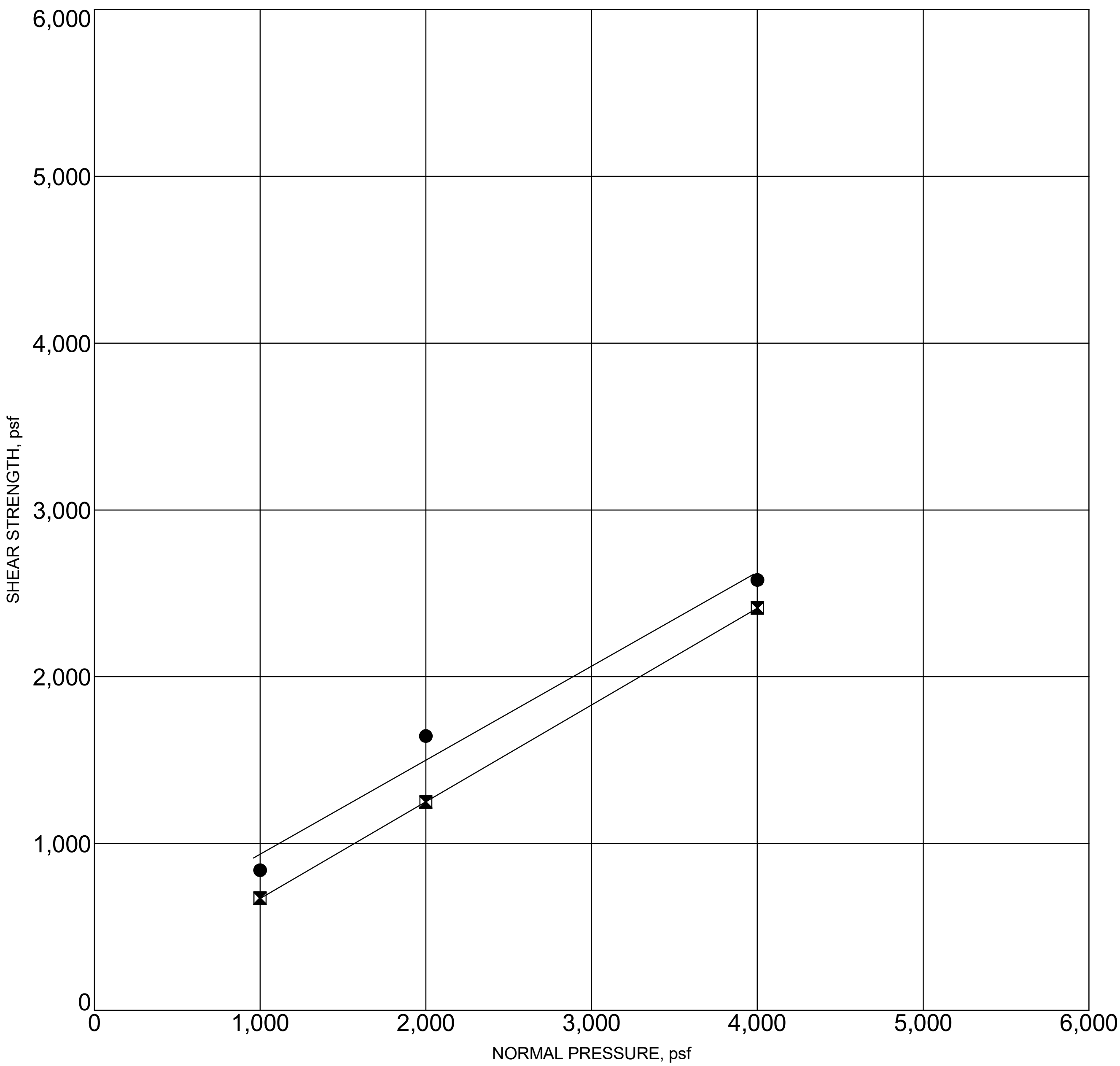
DIRECT SHEAR TEST RESULTS

FIGURE B-12



● **PEAK STRENGTH**
Friction Angle= 29 degrees
Cohesion= 372 psf

⧗ **ULTIMATE STRENGTH**
Friction Angle= 30 degrees
Cohesion= 90 psf

Sample Location		Classification	DD,pcf	MC,%
B-101	7.0	CLAY (CL)	97	20.8

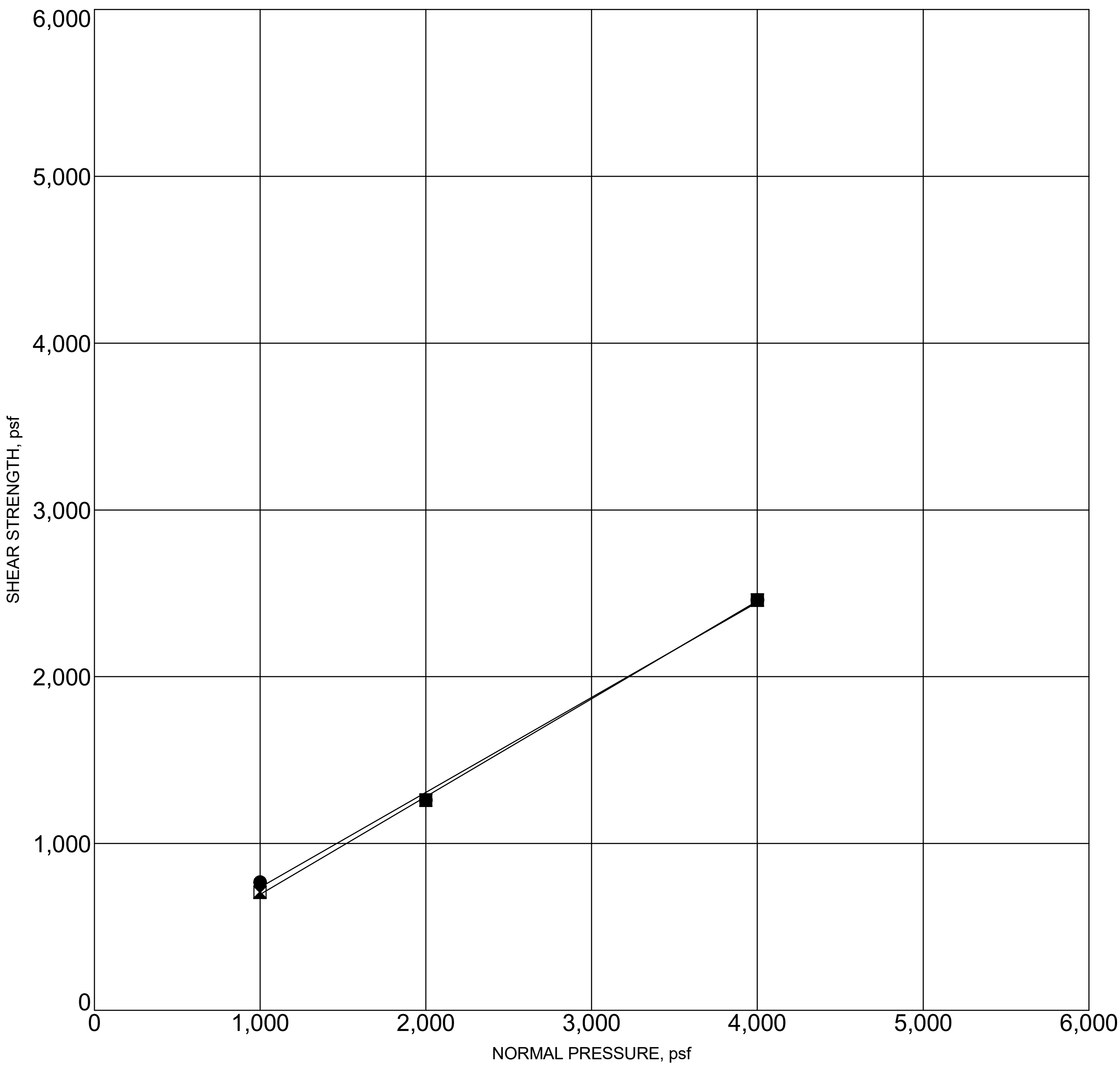
PROJECT: KIA CALABASAS

PROJECT NO.: 3162.I

GPI

DIRECT SHEAR TEST RESULTS

FIGURE B-13



● **PEAK STRENGTH**
Friction Angle= 30 degrees
Cohesion= 168 psf

⧗ **ULTIMATE STRENGTH**
Friction Angle= 30 degrees
Cohesion= 108 psf

Sample Location		Classification	DD,pcf	MC,%
B-102	5.0	SANDY SILT (ML)	102	9.3

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PROJECT NO.: 3162.I

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DIRECT SHEAR TEST RESULTS

FIGURE B-14

HDR

Table 1 - Laboratory Tests on Soil Samples

Geotechnical Professionals, Inc.
Calabasas
Your #2730.I, HDR Lab #16-0402LAB
31-May-16

Sample ID

			B-2 @ 0-5'	B-8 @ 0-5'
Resistivity		**Units**		
as-received		ohm-cm	12,000	52,000
saturated		ohm-cm	440	1,040
pH			6.7	7.0
Electrical Conductivity		mS/cm	1.51	0.29
Chemical Analyses				
Cations				
calcium	Ca^{2+}	mg/kg	780	146
magnesium	Mg^{2+}	mg/kg	107	7.3
sodium	Na^{1+}	mg/kg	323	27
potassium	K^{1+}	mg/kg	150	57
Anions				
carbonate	CO_3^{2-}	mg/kg	ND	ND
bicarbonate	HCO_3^{1-}	mg/kg	174	250
fluoride	F^{1-}	mg/kg	2.0	1.4
chloride	Cl^{1-}	mg/kg	438	14
sulfate	SO_4^{2-}	mg/kg	1,750	65
phosphate	PO_4^{3-}	mg/kg	3.5	ND
Other Tests				
ammonium	NH_4^{1+}	mg/kg	ND	ND
nitrate	NO_3^{1-}	mg/kg	2,240	616
sulfide	S^{2-}	qual	na	na
Redox		mV	na	na

Resistivity per ASTM G187, Cations per ASTM D6919, Anions per ASTM D4327, and Alkalinity per APHA 2320-B.

Electrical conductivity in millisiemens/cm and chemical analyses were made on a 1:5 soil-to-water extract.

mg/kg = milligrams per kilogram (parts per million) of dry soil.

Redox = oxidation-reduction potential in millivolts

ND = not detected

na = not analyzed

HDR

Table 1 - Laboratory Tests on Soil Samples

Geotechnical Professionals, Inc.
KIA Calabasas
Your #3162.I, HDR Lab #22-1094LAB
21-Nov-22

Sample ID			B-101 @ 0-5'
Resistivity		**Units**	
as-received		ohm-cm	4,400
saturated		ohm-cm	1,000
pH			7.9
Electrical Conductivity		mS/cm	0.16
Chemical Analyses			
Cations			
calcium	Ca^{2+}	mg/kg	69
magnesium	Mg^{2+}	mg/kg	ND
sodium	Na^{1+}	mg/kg	65
potassium	K^{1+}	mg/kg	13
ammonium	NH_4^{1+}	mg/kg	ND
Anions			
carbonate	CO_3^{2-}	mg/kg	ND
bicarbonate	HCO_3^{1-}	mg/kg	305
fluoride	F^{1-}	mg/kg	3.1
chloride	Cl^{1-}	mg/kg	11
sulfate	SO_4^{2-}	mg/kg	76
nitrate	NO_3^{1-}	mg/kg	1.3
phosphate	PO_4^{3-}	mg/kg	ND
Other Tests			
sulfide	S^{2-}	qual	na
Redox		mV	na

Resistivity per ASTM G187, pH per ASTM G51, Cations per ASTM D6919, Anions per ASTM D4327, and Alkalinity per APHA 2320-B.

Electrical conductivity in millisiemens/cm and chemical analyses were made on a 1:5 soil-to-water extract.

mg/kg = milligrams per kilogram (parts per million) of dry soil.

Redox = oxidation-reduction potential in millivolts

ND = not detected

na = not analyzed

Appendix J:

Geotechnical Reports

FOR REFERENCE

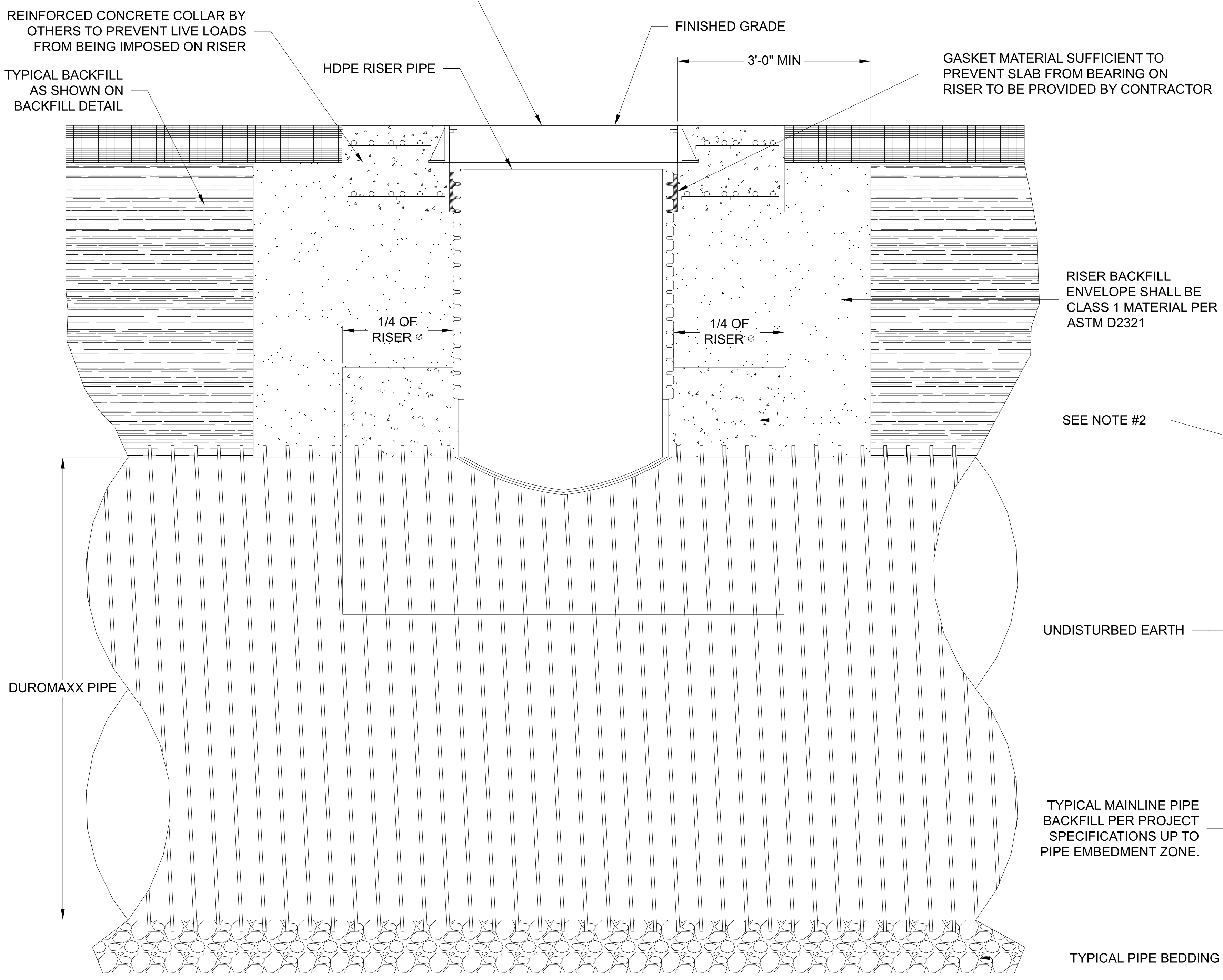
NOTES:

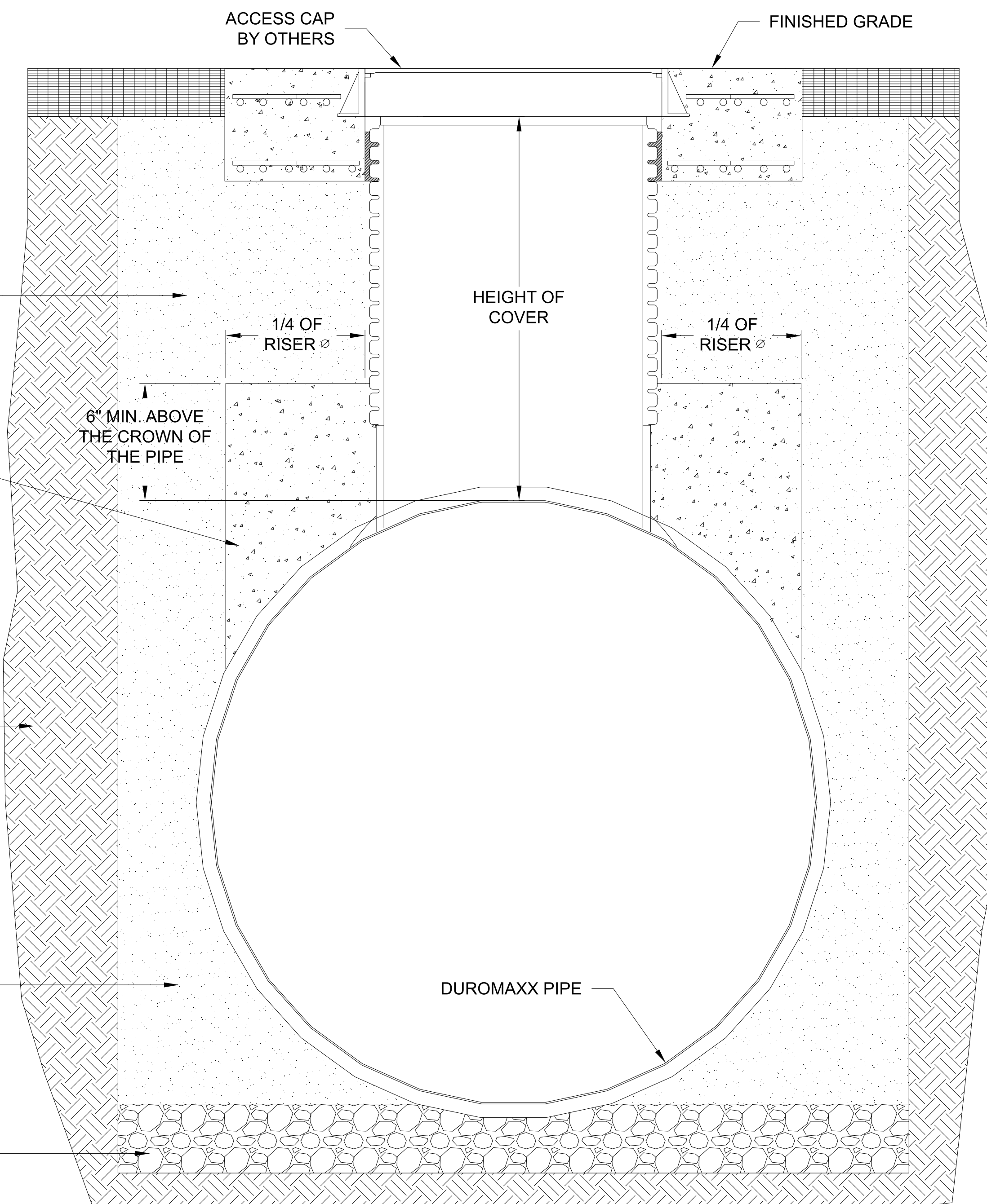
1) RECOMMENDED 30" DIAMETER MIN. RISER.

2) STABILIZED BACKFILL MATERIAL SUCH AS CLSM OR CEMENT STABILIZED SAND SHALL HAVE A MINIMUM COMPRESSIVE STRENGTH OF 750 PSI (SUPPLIED AND INSTALLED BY OTHERS).

MAX RISER ⌀ THAT DOES NOT REQUIRE CONCRETE REINFORCEMENT FOR COVER HEIGHTS OF ≤10'	
MAINLINE DIAMETER	RISER (MAX ⌀)
42"	42"
48" TO 120"	36"

FOR MORE DETAILED INFORMATION CONSULT FITTING STUB REINFORCEMENT TABLE ON DETAIL #300. IF REINFORCEMENT IS NOT REQUIRED, USE RISER DETAIL #401.





DUROMAXX ACCESS RISER & REINFORCEMENT INSTALLATION DETAIL (<10'-0" HEIGHT OF COVER)

NOT TO SCALE

MARK	DATE	REVISION DESCRIPTION	BY

CONTECH ENGINEERED SOLUTIONS LLC
www.ContechES.com
9025 Centre Pointe Dr., Suite 400, West Chester, OH 45069
800-338-1122 513-645-7000 513-645-7993 FAX

DuroMaxx
STEEL REINFORCED PE TECHNOLOGY

CONTECH
DYODS
DRAWING

DYO28355 Kia
SWQDv
Calabasas, CA
DETENTION SYSTEM

PROJECT No.: 18982	SEQ. No.: 28355	DATE: 3/13/2023
DESIGNED: DYO		DRAWN: DYO
CHECKED: DYO		APPROVED: DYO
SHEET NO.:	D4	