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Appendix F:

Hydrology and Water Quality Supporting Information

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F.1 - Stormwater Control Plan

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STORMWATER CONTROL PLAN
FOR A REGULATED PROJECT

ZINFANDEL SUBDIVISION
1583 EL CENTRO AVENUE
NAPA, CALIFORNIA 94558

Prepared for:

Trinity Project, LLC

REGISTERED PROFESSIONAL ENGINEER
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CIVIL ENGINEER
STATE OF CALIFORNIA

Project #4117017.0

September 15, 2023

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TABLES

FIGURES

ATTACHMENTS

I. Project Data

Table 1. Project Data Form

Project Name/Number	Zinfandel Subdivision / PL19-0016 / 4117017.0
Application Submittal Date	
Project Location	1583 El Centro Avenue Napa, California 94558 APN: Pending, Adjusted Parcel 2 per 2019-0016141
Project Phase No.	Not Applicable
Project Type and Description	Construction of a 51-lot single family residential subdivision including streets, driveways, utilities bioretention facilities and detention ponds.
Total Project Site Area	9.7 acres
Total New and Replaced Impervious Surface Area	199,285 sq. ft (including El Centro Avenue half street frontage & Lassen Street frontage)
Total Pre-Project Impervious Surface Area	26,197 sq. ft (including El Centro Avenue half street frontage & Lassen Street frontage)
Total Post-Project Impervious Surface Area	199,285 sq. ft (including El Centro Avenue half street frontage and Lassen Street frontage)

II. Setting

II.A. Project Location and Description

This project involves the demolition of an existing residential house and barn with asphalt driveway. The site will be developed to a 51-lot single family residential subdivision with public roads. This development is located at 1583 El Centro Avenue in Napa, California as shown in Figure 1 below.

Figure 1. Vicinity Map

The proposed use is consistent with the current RS 4 zoning. The project will include the construction of 51 residential houses, connecting public roads and installation of new public utilities along with stormwater quality control bioretention and detention facilities.

Refer to Attachment 2 for the overall scope of the project.

II.B. Existing Site Features and Conditions

The project site is irregular in shape and is generally flat. The site is currently used as vineyards with a residential house that fronts El Centro Avenue. The site is bounded by El Centro Avenue to the north and residential developments with public roads to the east, west and south. See Figure 2 below for existing site conditions.

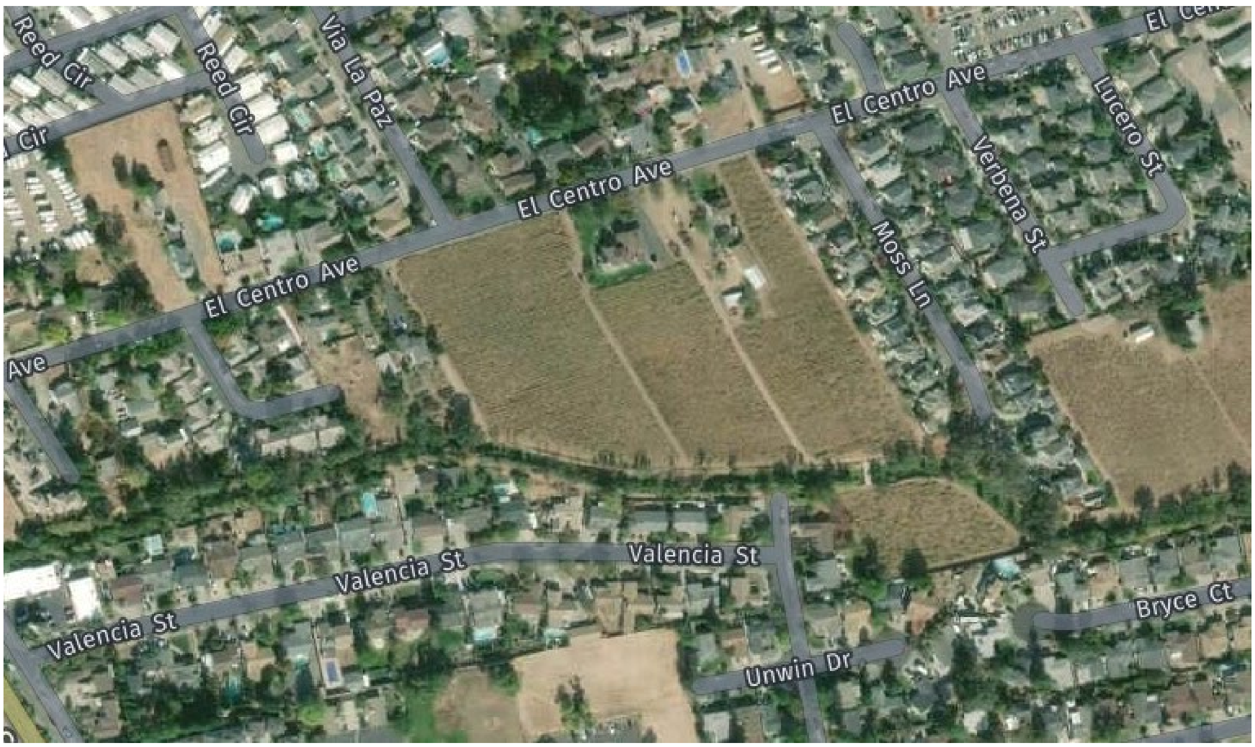


Figure 2. Existing Site Conditions

Mapping by the U.S. Conservation Service has classified soil over this project area as Clear Lake Clay (116) which is of the Hydraulic Soil Group D and Haire Loam (145) which is of the Hydraulic Soil Group D. Refer to Attachment 1 for Soils Map. Natural drainage from these parcels generally flows towards Salvador Channel. Stormwater is ultimately conveyed to the Napa River.

II.C. Opportunities and Constraints for Stormwater Control

Stormwater treatment facilities have been integrated into the planning, design, construction, operation, and maintenance of the proposed development. The following potential opportunities and constraints were considered in determining the best stormwater control design for this development.

Opportunities for this site are the availability of landscaped areas in the front and rear yards. Landscape areas on the parcels along Salvador Channel will be used as self-treating management areas since these parcels will be predominantly pervious areas. Bioretention facilities will be installed to treat stormwater runoff prior to discharge from the site. Runoff will be conveyed to the bioretention facilities from roof downspouts and surface flows from the streets. Once in the bioretention basin, runoff will be treated via infiltration together with the pollutant retention capabilities of the plants in the facilities. These bioretention facilities will also be used for detention such that the proposed post-developed flow discharge from the development will be maintained at, or below pre-developed levels that will outfall to Salvador Channel. See Attachment 2 for locations of bioretention facilities.

Constraints will be the excavation of approximately 5,000 CY terrace along Salvador Channel to widen the channel laterally to mitigate development fill in the flood plain. In order to reduce the flood hazard to the development and other neighbors downstream, vegetation and native trees will be planted along this terrace to help prevent the land from eroding downstream. Additional channel restoration mitigation measures and plans approved by the City will be implemented to help reduce potential flood hazard.

III. Low Impact Development Design Strategies

III.A. Optimization of Site Layout

1. Limitation of development envelope
   The development of the houses will occur within the building setback lines per Section 17.08.030 of the City of Napa Municipal Code.
2. Preservation of natural drainage features
   Natural drainage consists of sheet flow over the ground surface that concentrates in man-made surface drainage elements such as ditches, gutters and onsite storm drain pipes. See constraints on Section II.C above.
3. Setbacks from creeks, wetlands, and riparian habitats
   Riparian setback from Salvador Channel to the maximum degree possible and at minimum as required by local ordinances.
4. Minimization of imperviousness
   Landscaping will be used in the front and rear yards. Impervious areas will be minimized to the maximum extent practicable.
5. Use of drainage as a design element
   Bioretention facilities are incorporated into the aesthetic landscape design of the site. Grading and storm drain locations have been designed to direct runoff to bioretention facilities.

III.B. Use of Permeable Pavements

Permeable pavements are not in the scope of this project.

III.C. Dispersal of Runoff to Pervious Areas

Stormwater runoff will be directed to landscaped areas.

III.D. Stormwater Control Measures

Runoff from the project site, including roof and paved areas, will be routed to four bioretention facilities (see Attachment 2). BRF #1 and #2 will also function as stormwater detention basins. All facilities are designed and will be constructed to the criteria in the BASMAA Post-Construction Manual (January 2019), including the following features (see Figure 3):

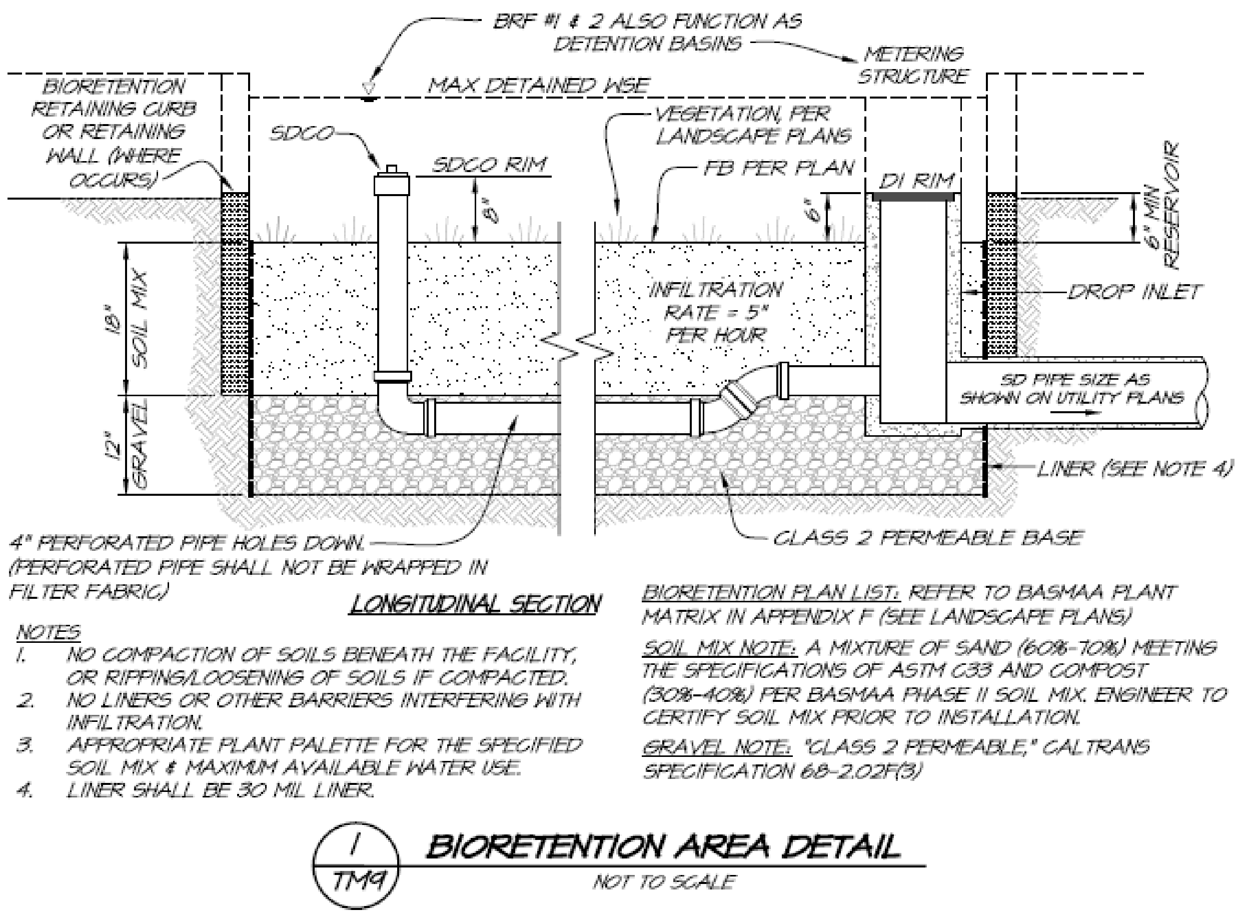


Figure 3. Bioretention Cross Section

- Surrounded by a concrete curb. Where adjacent to pavement, curbs will be thickened and an impermeable vertical cutoff wall will be included.
- Each layer built flat, level, and to elevations specified in the plans:
  - Bottom of Gravel Layer (BGL)
  - Top of Gravel Layer (TGL)
  - Top of Soil Layer (TSL)
  - Overflow Grate
  - Facility Rim
- 12 inches of Class 2 permeable, Caltrans specification 68-2.02F (3).

- 18 inches sand/compost mix meeting BASMAA specifications.
- 4-inch diameter PVC SDR 35 perforated pipe underdrain, installed with the invert at the top of the Class 2 permeable layer with holes facing down, and connected to the overflow structure at that same elevation.
- 6-inch-deep reservoir between top of soil elevation and overflow grate elevation.
- Concrete drop inlet with frame overflow structure, with grate set to specified elevation, connected to the on-site storm drain system.
- Vertical cutoff walls to protect adjacent pavement.
- Plantings selected for water conservation.
- Irrigation system on a separate zone, with drip emitters and "smart" irrigation controllers.
- Sign identifying the facility as a stormwater treatment facility.

Areas on the site which do not drain to a bioretention facility are the following (see Attachment 2 for reference):

- DMA 5 – The west portion of the private driveway along the Lassen Street frontage, totaling 700 square feet. Grading in this area must conform with existing street elevations. As a result, stormwater runoff from this DMA leaves the site untreated.
- DMA 6 – The southern flood terrace and maintenance path near lots 50-51, totaling 13,216 square feet. This DMA is considered as <u>self-treating area</u> (See Section 4.1 for BASMAA requirements for self-treating areas).
- DMA 7 – The northern flood terrace and access road near lots 2-19, totaling 45,697 square feet. This DMA is considered as <u>self-treating area</u> (See Section 4.1 for BASMAA requirements for self-treating areas).
- DMA 8 – The north portion of Lot 1, totaling 1,445 square feet. This DMA is considered as <u>self-treating area</u> (See Section 4.1 for BASMAA requirements for self-treating areas).
- DMA 9 – The north half street area of El Centro Avenue along Lot 1, totaling 3,734 square feet. Grading in this areas must conform with existing street elevations. As a result, stormwater runoff from this DMA leaves the site untreated.

The bioretention facilities that will collect and treat onsite stormwater will also function as Multi-Benefit Trash Treatment Systems in accordance with the State Water Board standards. They are designed to trap trash particles that are 5-mm and greater for the peak flow rate generated by the 1-year, 1-hour storm event from each drainage management area. The bioretention facilities will provide a 6" ponding reservoir per BASMAA requirements, which is sufficient depth such that the 1-year, 1-hour storm event will not reach the overflow elevations. Thus, all trash is captured at the surface of each bioretention facility. The overflow inlets have a grated lid for larger storm events.

IV. Documentation of Drainage Design

IV.A. Descriptions of Each Drainage Management Areas

IV.A.1. Drainage Management Areas

Table 2. Drainage Management Areas (DMAs) as shown on Attachment 2.

DMA Name	DMA perv (Pervious Area, square feet)	DMA imp (Impervious Area, square feet)	Pervious Pavers Area (square feet)	Total Area (square feet)	Bioretention Facility Name
1	129,479	161,020	--	298,293	BRF #1
2	13,038	13,866	--	27,627	BRF #2
3	8,587	14,637	--	23,876	BRF #3
4	1,713	4,400	--	6,306	BRF #4
5	54	646	--	700	Untreated
6	13,216	0	--	13,216	Self-Treating
7	44,209	1,488	--	45,697	Self-Treating
8	1,445	0	--	1,445	Self-Treating
9	506	3,228	--	3,734	Untreated

IV.A.2. Drainage Management Area Descriptions

DMA 1: Totaling 298,293 square feet, this DMA consists of Lots 2 to 19, 20 to 26, 29 to 46, 49, and portions of Lots 1, 27 to 28, 47, 48, and parcel A. It also includes Clementina Circle, a small portion of street of El Centro Avenue intersecting Clementina Circle along the project frontage. Runoff from the roof will drain out from downspouts to splash boxes that flows towards the street via landscape areas then along the street gutter toward the street catch basins then to a storm drain pipe that outfalls to BRF #1. This bioretention facility has a total treatment area of 7,794 square feet and will also function as a stormwater detention basin.

DMA 2: Totaling 27, 627 square feet, this DMA consists of Lots 50 to 51 and a large portion of the private driveway and parcel C. Runoff from the roof will drain out from downspouts to splash boxes that flows towards the street via landscape areas then along the driveway gutter toward the curb opening inlet adjacent to BRF #2. This bioretention facility has a total treatment area of 723 square feet and will also function as a stormwater detention basin.

DMA 3: Totaling 23,876 square feet, this DMA consists of portions of Lots 28, 47, 48 and APN 036-361-043 together with the half street frontage portion of El Centro Avenue along these areas. Runoff from the roof will drain out from downspouts to splash boxes that flows towards the street via landscape areas then along the street gutter toward the curb opening inlet adjacent to BRF #3. This bioretention facility has a total treatment area of 652 square feet.

DMA 4: Totaling 6,306 square feet, this DMA consists of a portion of Lot 27 together with the half street frontage portion of El Centro Avenue along this area. Runoff from the roof will drain from downspouts to splash boxes that flow toward the street via landscape areas then along the street gutter toward the curb opening inlet adjacent to BRF #4. This bioretention facility has a total treatment area of 193 square feet.

DMA 5: The west portion of the private driveway along the Lassen Street frontage, totaling 700 square feet, a small portion of parcel C. Grading in this area must conform with existing street elevations. As a result, stormwater runoff from this DMA leaves the site untreated.

DMA 6: The southern flood terrace and maintenance path near Lots 50 to 51, totaling 13,216 square feet, a portion of parcel C. This DMA is considered as self-treating area meeting the following BASMAA requirements: 1) There are no impervious areas or very small impervious area (5% or less) relative to the receiving pervious area; and, 2) Slopes are gentle enough to ensure runoff will be absorbed into the vegetation and soil.

DMA 7: The northern flood terrace and access road near Lots 2 to 19, totaling 45,697 square feet. This DMA is considered self-treating area meeting the following BASMAA requirements: 1) There are no impervious areas or very small impervious area (5% or less) relative to the receiving pervious area; and, 2) Slopes are gentle enough to ensure runoff will be absorbed into the vegetation and soil.

DMA 8: The north portion of Lot 1, totaling 1,445 square feet. This DMA is considered self-treating area meeting the following BASMAA requirements: 1) There are no impervious areas or very small impervious area (5% or less) relative to the receiving pervious area; and, 2) Slopes are gentle enough to ensure runoff will be absorbed into the vegetation and soil.

DMA 9: The north half street area of El Centro Avenue along Lot 1, totaling 3,734 square feet. Grading in these areas must conform with existing street elevations. As a result, stormwater runoff from this DMA leaves the site untreated.

IV.B. Tabulation and Sizing Calculations

Refer to Attachment 3 for Provision E.12 Sizing Calculator Spreadsheet.

V. Source Control Measures

V.A. Site activities and potential sources of pollutants

On-site activities that could potentially produce stormwater pollutants include:

- On-site storm drains
- Interior floor drains
- Pest control
- Landscaping
- Refuse areas
- Fire sprinkler test water
- Miscellaneous drain water
- Streets and sidewalks

V.B. Potential Pollutant Sources and Source Control Measures

The site activities and potential sources of pollutants for the Zinfandel Subdivision project are listed in Table 3, below.

Table 3. Potential Pollutant Sources and Source Control Measures

Potential Sources of Runoff Pollutants	Permanent Source Control BMPs	Operational Source Control BMPs
A. On-site storm drain inlets (unauthorized non-stormwater discharges and accidental spills or leaks)	☐ Mark all inlets with the words "No Dumping! Flows to River" or similar.	☐ Maintain and periodically repaint or replace inlet markings. ☐ Provide stormwater pollution prevention information to new site owners, lessees, or operators. ☐ See applicable operational BMPs in Fact Sheet SC-74, "Drainage System Maintenance."
B. Interior floor drains and elevator shaft sump pumps	☐ Interior floor drains and elevator shaft sump pumps will be plumbed to the sanitary sewer.	☐ Inspect and maintain drains to prevent blockages and overflow.
D_1. Need for future indoor & structural pest control	☐ Building design shall incorporate features that discourage entry of pests.	☐ Provide Integrated Pest Management information to owners, lessees, and operators.
D_2. Landscape / outdoor pesticide use / building and grounds maintenance	Final landscape plans will accomplish all of the following: ☐ Preserve existing native trees, shrubs, and ground cover to the maximum extent possible. ☐ Minimize irrigation and runoff, to promote surface infiltration where appropriate, and to minimize the use of fertilizers and pesticides that can contribute to stormwater pollution. ☐ Where landscaped areas are used to retain or detain stormwater, specify plants that are tolerant of saturated soil conditions. ☐ Use pest-resistant plants, especially adjacent to hardscape. ☐ To insure successful establishment, select plants appropriate to site soils, slopes, climate, sun, wind, rain, land use, air movement, ecological consistency, and plant interactions.	☐ Maintain landscaping using minimum or no pesticides. ☐ See applicable operational BMPs in Fact Sheet SC-41, "Building and Grounds Maintenance." ☐ Provide IPM information to new owners, lessees and operators.
G. Refuse areas	☐ Refuse areas shall be paved with an impervious surface, designed not to allow run-on from adjoining areas, and screened to prevent off-site transport of trash. ☐ Refuse areas shall contain a roof to minimize direct precipitation. ☐ No drain connections shall be made to the Refuse area.	☐ Provide adequate number of receptacles. ☐ Inspect receptacles regularly; repair or replace leaky receptacles. ☐ Keep receptacles covered. ☐ Prohibit/prevent dumping of liquid or hazardous wastes. ☐ Post "no hazardous materials" signs. ☐ Inspect and pick up litter daily and clean up spills immediately.

Potential Sources of Runoff Pollutants	Permanent Source Control BMPs	Operational Source Control BMPs
		☐ Keep spill control materials available on-site. ☐ Clean by dry-sweeping only, or with wet/dry vacuum. ☐ See Fact Sheet SC-34, "Waste Handling and Disposal"
N. Fire sprinkler test water	☐ Fire sprinkler test water shall be discharged to the sanitary sewer.	☐ See the note in Fact Sheet SC-41, "Building and Grounds Maintenance"
O. Miscellaneous drain or wash water or other sources • Boiler drain lines • Condensate drain lines • Rooftop equipment • Drainage sumps • Roofing, gutters, and trim • Other sources	☐ Boiler drain lines shall be directly or indirectly connected to the sanitary sewer system and may not discharge to the storm drain. ☐ Condensate drain lines may discharge to landscaped areas if the flow is small enough that runoff will not occur. Condensate drain lines may not discharge to the storm drain system. ☐ Rooftop equipment with potential to produce pollutants shall be roofed and/or have secondary containment. ☐ Any drainage sumps on-site shall feature a sediment sump to reduce the quantity of sediment in pumped water.	If architectural copper is used, implement the following BMPs for management of rinse water during installation: ☐ If possible, purchase copper materials that have been pre-patinated at the factory. ☐ If patination is done on-site, prevent rinse water from entering storm drains by discharging to landscaping or by collecting in a tank and hauling off-site. ☐ Consider coating the copper materials with an impervious coating that prevents further corrosion and runoff. ☐ Implement the following BMPs during routine maintenance: ☐ Prevent rinse water from entering storm drains by discharging to landscaping or by collecting in a tank and hauling off-site.
P. Plazas, sidewalks, and parking lots		☐ Sweep plazas, sidewalks, and parking lots regularly to prevent accumulation of litter and debris. Collect debris from pressure washing to prevent entry into the storm drain system. Collect wash water containing any cleaning agent or degreaser and discharge to the sanitary sewer not to a storm drain.

VI. Stormwater Facility Maintenance

VI.A. Ownership and Responsibility for Maintenance in Perpetuity

Maintenance of stormwater facilities will be the responsibility of the property owner and will be performed by the owner's contractors or employees as part of routine maintenance of buildings, grounds and landscaping. The applicant will review the Post-Construction BMP Maintenance Agreement with the City of Napa regarding the maintenance of the stormwater facilities and commit to execute any necessary agreements prior to completion of construction. Applicant accepts responsibility for interim operation and maintenance of stormwater treatment and flow-control facilities until such time as this responsibility is formally transferred to a subsequent owner.

VI.B. Summary of Maintenance Requirements for Each Stormwater Facility

The bioretention/detention facilities will be maintained on the following schedule at a minimum. Details of maintenance responsibility and procedures will be included in an Operation and Maintenance Plan to be submitted for approval prior to the completion of construction.

At no time will synthetic pesticides or fertilizers be applied, nor will any soil amendments, other than aged compost mulch or sand/compost mix, be introduced.

Daily: The facilities will be examined for visible trash during regular policing of the site, and trash will be removed.

After Significant Rain Events: A significant rain event is one that produces approximately a half-inch or more rainfall in a 24-hour period. Within 24 hours after each such event, the following will be conducted:

- The surface of the facility will be observed to confirm there is no excessive ponding. All facilities are designed to pond up to a 6" reservoir for stormwater treatment, and BRF #1 & #2 are designed to further detain up to a 24-hour, 100-year rainfall event.
- Inlets will be inspected, and any accumulations of trash or debris will be removed.
- The surface of the mulch layer will be inspected for movement of material. Mulch will be replaced and raked smooth if needed.
- At BRF #1 & #2, the metering structure and orifice will be inspected, and any accumulations of debris or sediment will be removed.

Prior to the Start of the Rainy Season: In September of each year, the facility will be inspected to confirm there is no accumulation of debris that would block flow, and that growth and spread of plantings does not block inlets or the movement of runoff across the surface of the facility. At BRF #1 & #2, the metering structure and orifice will be inspected, and any accumulations of debris or sediment will be removed.

Annual Landscape Maintenance: In December – February of each year, vegetation will be cut back as needed, debris removed, and plants and mulch replaced as needed. The concrete work will be inspected for damage. The elevation of the top of soil and mulch layer will be confirmed to be consistent with the 6-inch reservoir depth.

VII. Construction Plan E.12 Checklist

Table 4. Construction Plan E.12 Checklist

Stormwater Control Plan Page #	Source Control or Treatment Control Measure	See Plan
1	Bioretention Facilities	SCP Site Plan in Attachment 2

VIII. Certifications

The preliminary design of stormwater treatment facilities and other stormwater pollution control measures in this plan are in accordance with the current edition of the BASMAA Post-Construction Manual, dated January 2019.

____

Preparer
Derek Dittman, PE

ATTACHMENT 1

SOIL CLASSIFICATION

122° 19' 2" W   122° 18' 43" W

38° 20' 10" N   38° 20' 10" N

38° 20' 0" N   38° 20' 0" N

122° 19' 2" W   122° 18' 43" W

Map Scale: 1:2,120 if printed on A landscape (11" x 8.5") sheet.

N

Meters: 0, 30, 60, 120, 180

Feet: 0, 100, 200, 400, 600

Map projection: Web Mercator Corner coordinates: WGS84

MAP LEGEND

Area of Interest (AOI)
- Area of Interest (AOI)

Soils

Soil Rating Polygons
- A
- A/D
- B
- B/D
- C
- C/D
- D
- Not rated or not available

Soil Rating Lines
- A
- A/D
- B
- B/D
- C
- C/D
- D
- Not rated or not available

Soil Rating Points
- A
- A/D
- B
- B/D
- C
- C/D
- D
- Not rated or not available

Water Features
- Streams and Canals

Transportation
- Rails
- Interstate Highways
- US Routes
- Major Roads
- Local Roads

Background
- Aerial Photography

MAP INFORMATION

The soil surveys that comprise your AOI were mapped at 1:24,000.

Warning: Soil Map may not be valid at this scale.

Enlargement of maps beyond the scale of mapping can cause misunderstanding of the detail of mapping and accuracy of soil line placement. The maps do not show the small areas of contrasting soils that could have been shown at a more detailed scale.

Please rely on the bar scale on each map sheet for map measurements.

Source of Map: Natural Resources Conservation Service
Web Soil Survey URL:
Coordinate System: Web Mercator (EPSG:3857)

Maps from the Web Soil Survey are based on the Web Mercator projection, which preserves direction and shape but distorts distance and area. A projection that preserves area, such as the Albers equal-area conic projection, should be used if more accurate calculations of distance or area are required.

This product is generated from the USDA-NRCS certified data as of the version date(s) listed below.

Soil Survey Area: Napa County, California
Survey Area Data: Version 10, Sep 25, 2017

Soil map units are labeled (as space allows) for map scales 1:50,000 or larger.

Date(s) aerial images were photographed: Apr 17, 2015—Oct 18, 2016

The orthophoto or other base map on which the soil lines were compiled and digitized probably differs from the background imagery displayed on these maps. As a result, some minor shifting of map unit boundaries may be evident.

Hydrologic Soil Group

Map unit symbol	Map unit name	Rating	Acres in AOI	Percent of AOI
116	Clear Lake clay, drained, 0 to 2 percent slopes, MLRA 14	D	1.2	11.9%
145	Haire loam, 0 to 2 percent slopes	D	9.2	88.1%
Totals for Area of Interest			**10.5**	**100.0%**

Description

Hydrologic soil groups are based on estimates of runoff potential. Soils are assigned to one of four groups according to the rate of water infiltration when the soils are not protected by vegetation, are thoroughly wet, and receive precipitation from long-duration storms.

The soils in the United States are assigned to four groups (A, B, C, and D) and three dual classes (A/D, B/D, and C/D). The groups are defined as follows:

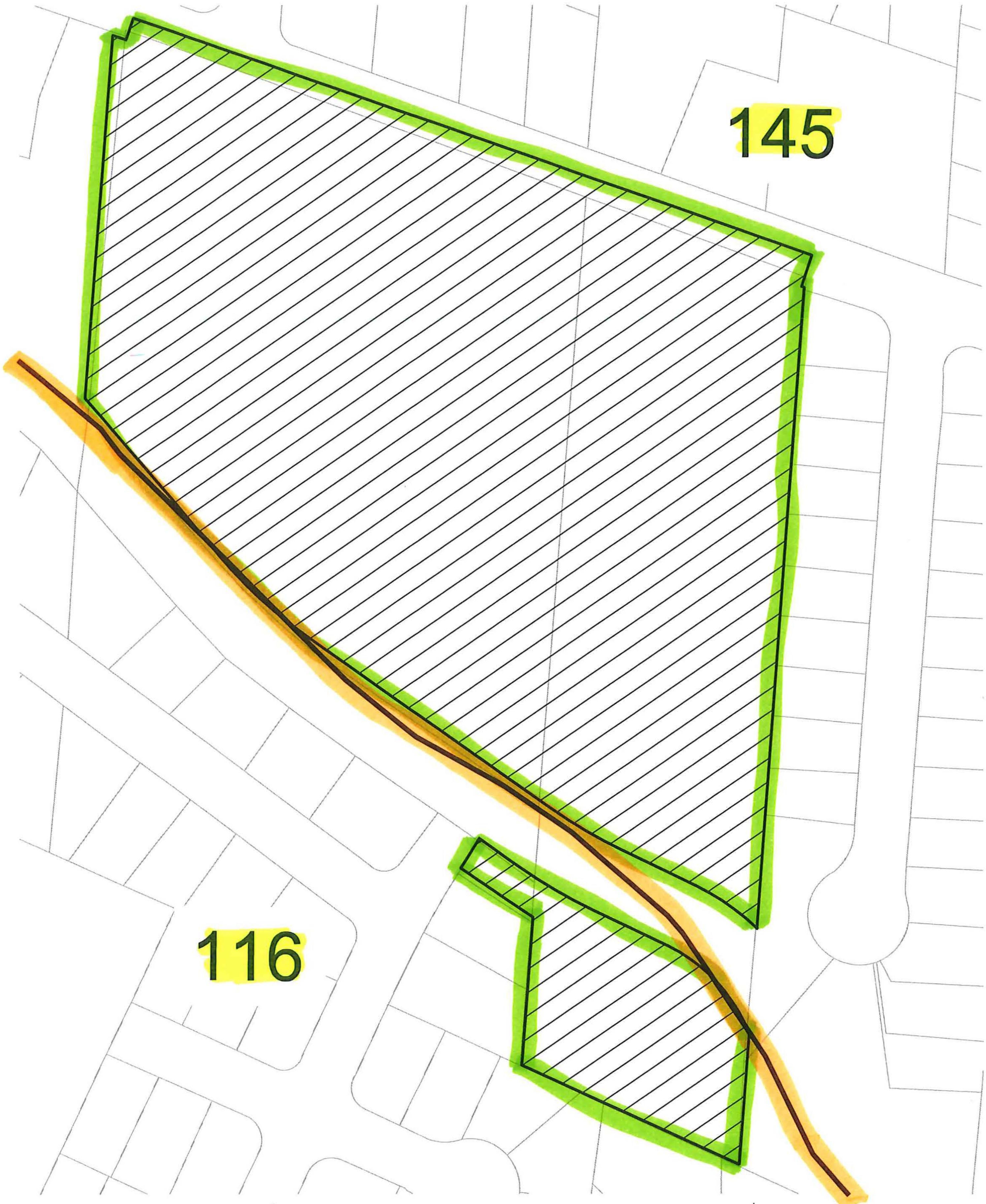
Group A. Soils having a high infiltration rate (low runoff potential) when thoroughly wet. These consist mainly of deep, well drained to excessively drained sands or gravelly sands. These soils have a high rate of water transmission.

Group B. Soils having a moderate infiltration rate when thoroughly wet. These consist chiefly of moderately deep or deep, moderately well drained or well drained soils that have moderately fine texture to moderately coarse texture. These soils have a moderate rate of water transmission.

Group C. Soils having a slow infiltration rate when thoroughly wet. These consist chiefly of soils having a layer that impedes the downward movement of water or soils of moderately fine texture or fine texture. These soils have a slow rate of water transmission.

Group D. Soils having a very slow infiltration rate (high runoff potential) when thoroughly wet. These consist chiefly of clays that have a high shrink-swell potential, soils that have a high water table, soils that have a claypan or clay layer at or near the surface, and soils that are shallow over nearly impervious material. These soils have a very slow rate of water transmission.

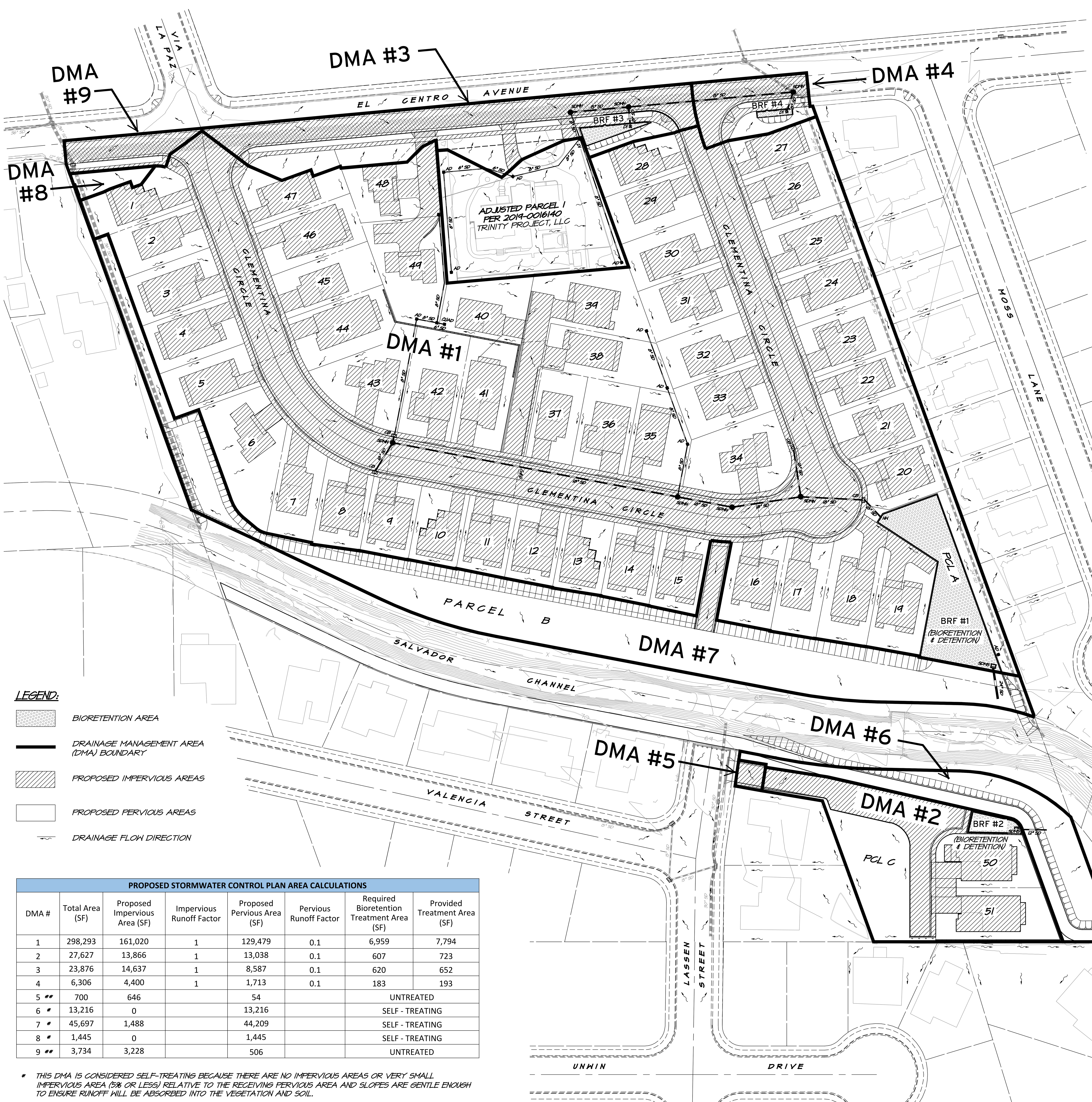
If a soil is assigned to a dual hydrologic group (A/D, B/D, or C/D), the first letter is for drained areas and the second is for undrained areas. Only the soils that in their natural condition are in group D are assigned to dual classes.



SCS SOIL CLASSIFICATION EXHIBIT
FOR PRELIMINARY
DETENTION SYSTEM CALCULATION

ATTACHMENT 2

STORMWATER CONTROL PLAN (SHEET TM9)



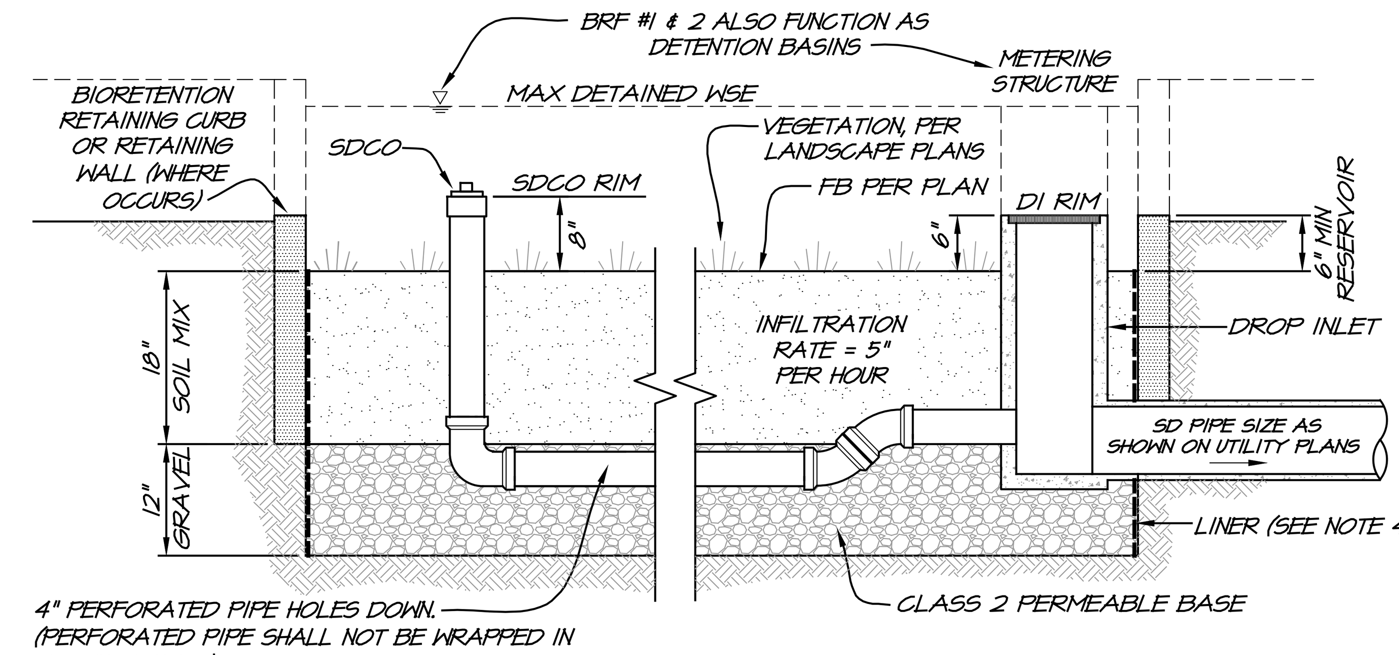
PROPOSED STORMWATER CONTROL PLAN AREA CALCULATIONS							
DMA #	Total Area (SF)	Proposed Impervious Area (SF)	Impervious Runoff Factor	Proposed Pervious Area (SF)	Pervious Runoff Factor	Required Bioretention Treatment Area (SF)	Provided Treatment Area (SF)
1	298,293	161,020	1	129,479	0.1	6,959	7,794
2	27,627	13,866	1	13,038	0.1	607	723
3	23,876	14,637	1	8,587	0.1	620	652
4	6,306	4,400	1	1,713	0.1	183	193
5 **	700	646		54		UNTREATED	
6 *	13,216	0		13,216		SELF - TREATING	
7 *	45,697	1,488		44,209		SELF - TREATING	
8 *	1,445	0		1,445		SELF - TREATING	
9 **	3,734	3,228		506		UNTREATED	

* THIS DMA IS CONSIDERED SELF-TREATING BECAUSE THERE ARE NO IMPERVIOUS AREAS OR VERY SMALL IMPERVIOUS AREA (5% OR LESS) RELATIVE TO THE RECEIVING PERVIOUS AREA AND SLOPES ARE GENTLE ENOUGH TO ENSURE RUNOFF WILL BE ABSORBED INTO THE VEGETATION AND SOIL.

** STORMWATER RUNOFF FROM THIS DMA LEAVES THE SITE UNTREATED.

PROPOSED POST CONSTRUCTION BMP's

BMP (BEST MANAGEMENT PRACTICES)	TREATMENT AREA
1. BIORETENTION AREAS, 4 EACH (SEE DETAIL 1/TM9 IN THIS SHEET)	SEE MAP FOR DMA
2. 'NO DUMPING, DRAINS TO RIVER' STAMPING ON ALL INLETS (SEE DETAIL 2/TM9 IN THIS SHEET)	ENTIRE PROJECT
3. 'WATER QUALITY FEATURE ON SITE' STAMPING ON ALL BIORETENTION INLETS (SEE DETAIL 3/TM9 IN THIS SHEET)	ENTIRE PROJECT
4. SPLASH BLOCKS ON ALL DOWNSPOUTS (SEE DETAIL 4/TM9 IN THIS SHEET)	ENTIRE PROJECT
5. HOMEOWNER EDUCATION: COPY OF SCP AND MAINTENANCE AGREEMENT TO BE PROVIDED TO OWNER	ENTIRE PROJECT



LONGITUDINAL SECTION

NOTES

1. NO COMPACTION OF SOILS BENEATH THE FACILITY, OR RIPPING/LOOSENING OF SOILS IF COMPACTED.
2. NO LINERS OR OTHER BARRIERS INTERFERING WITH INFILTRATION.
3. APPROPRIATE PLANT PALETTE FOR THE SPECIFIED SOIL MIX & MAXIMUM AVAILABLE WATER USE.
4. LINER SHALL BE 30 MIL LINER.

BIORETENTION PLAN LIST: REFER TO BASMAA PLANT MATRIX IN APPENDIX F (SEE LANDSCAPE PLANS)

SOIL MIX NOTE: A MIXTURE OF SAND (60%-70%) MEETING THE SPECIFICATIONS OF ASTM C33 AND COMPOST (30%-40%) PER BASMAA PHASE II SOIL MIX. ENGINEER TO CERTIFY SOIL MIX PRIOR TO INSTALLATION.

GRAVEL NOTE: "CLASS 2 PERMEABLE," CALTRANS SPECIFICATION 68-2.02F(3)

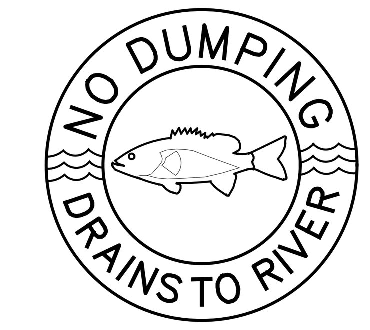
1/TM9 BIORETENTION AREA DETAIL

NOT TO SCALE

GRAPHIC SCALE

50 0 50 100

(IN FEET)
1 inch = 50 FT



2/TM9 DROP INLET MARKER

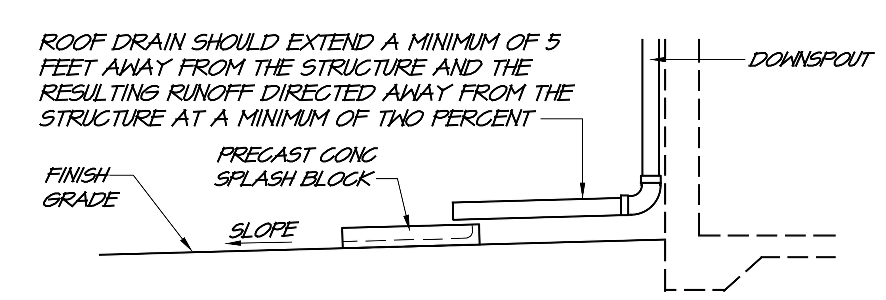
NOT TO SCALE

NOTES

1. EACH BIORETENTION FACILITY WITHIN THE CITY LIMITS SHALL BE MARKED WITH A "WATER QUALITY FEATURE ON SITE - DO NOT ALTER OR REMOVE" MARKER.
2. CONTRACTOR IS TO PURCHASE THESE MARKERS FROM THE PUBLIC WORKS DEPARTMENT LOCATED AT 1600 FIRST STREET.

3/TM9 BIORETENTION FACILITY MARKER

NOT TO SCALE



4/TM9 SPLASH BLOCK DETAIL

NOT TO SCALE

NO.	DATE	REVISIONS	BY	APPD

RSA+ | CONSULTING CIVIL ENGINEERS + SURVEYORS + est. 1980
1515 FOURTH STREET
NAPA, CALIF. 94559
OFFICE|707|252.3301
+ www.RSAcivil.com +

ZINFANDEL SUBDIVISION
STORMWATER CONTROL PLAN
NAPA CALIFORNIA

REGISTERED PROFESSIONAL ENGINEER DEREK MASON DITTMAN No. 90885 Exp. 03.31.2024 CIVIL ENGINEER STATE OF CALIFORNIA

DATE	JUNE 9, 2023
DRAWN	DEL
DESIGNED	RAY
CHECKED	DMD
JOB NO.	4117017.0

SHEET NO.
TM9
10 OF 11 SHEETS

PRELIMINARY - NOT FOR CONSTRUCTION

ATTACHMENT 3

PROVISION E.12 SIZING CALCULATOR SPREADSHEET

Provision E.12 Sizing Calculator

See the instructions and the BASMAA Post-Construction Manual

Step 1: Enter Total Site Area	**Step 2:** List names of all DMAs and square footage of each	**Step 3:** If DMA is "Self-Treating" or "Self-Retaining," **copy** square footage to appropriate column		**Step 4:** If the DMA is "Drains to Self Retaining" or "Drains to Bioretention" enter runoff factor from Table 4-1		**Step 6:** For "Drains to Self-Retaining" DMAs, enter the **name of receiving DMA**	**Step 5:** Slide (move) number from this column to correct column (F or H-Q)							
Total Site Area:	420,894						**BIORETENTION FACILITIES**							
DMA Names	**Square Feet**	**Self-Treating**	**Self-Retaining**	**Runoff Factor**	**Untreated**	**Name of Receiving DMA**	**BRF #1**	**BRF #2**	**BRF #3**	**BRF #4**				
DMA-1$_{perv}$	129,479			0.1			12,948							
DMA-1$_{imp}$	161,020			1			161,020							
DMA-2$_{perv}$	13,038			0.1				1,304						
DMA-2$_{imp}$	13,866			1				13,866						
DMA-3$_{perv}$	8,587			0.1					859					
DMA-3$_{imp}$	14,637			1					14,637					
DMA-4$_{perv}$	1,713			0.1						171				
DMA-4$_{imp}$	4,400			1						4,400				
DMA-5$_{perv}$	54				54									
DMA-5$_{imp}$	646				646									
DMA-6$_{perv}$	13,216	13,216												
DMA-6$_{imp}$	0	0												
DMA-7$_{perv}$	44,209	44,209												
DMA-7$_{imp}$	1,488	1,488												
DMA-8$_{perv}$	1,445	1,445												
DMA-8$_{imp}$	0	0												
DMA-9$_{perv}$	506				506									
DMA-9$_{imp}$	3,228				3,228									
Total DMAs	**411,532**	**60,358**	**0**		**4,434**		**173,968**	**15,170**	**15,496**	**4,571**	**0**	**0**	**0**	**0**
						Sizing Factor	**0.04**	**0.04**	**0.04**	**0.04**	**0.04**	**0.04**	**0.04**	**0.04**
						Minimum Size	**6,959**	**607**	**620**	**183**	**0**	**0**	**0**	**0**
Total Facilities	9,362	**Step 7:** Enter Facilty Footprints				**Footprint on Exhibit**	7,794	723	652	193	0	0	0	0
DMAs + Facilities	**420,894**						**OK**	**OK**	**OK**	**OK**	**OK**	**OK**	**OK**	**OK**
	OK													

Step 8: Iterate sizes of facility footprints and DMAs until all footprints are at least the minimum **AND** DMAs + Facilities equals Total Site Area

Step 9: Check to make sure Areas Draining to each Receiving Self-Retaining Area do not exceed maximum 2:1 ratio.

Step 10: Check results on this spreadsheet are consistent with what is shown on the SCP Exhibit.

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F.2 - Preliminary Detention Calculations

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PRELIMINARY DETENTION CALCULATIONS

ZINFANDEL SUBDIVISION
1583 EL CENTRO AVENUE
NAPA, CALIFORNIA 94558

Trinity Project, LLC

REGISTERED PROFESSIONAL ENGINEER
DEREK MASON DITTMAN
No. 90885
Exp. 03.31.2024
CIVIL ENGINEER
STATE OF CALIFORNIA

Project #4117017.0

September 15, 2023

Table of Contents

INTRODUCTION

In order to satisfy the City of Napa Drainage Design Standard Section 2.10.02, which states that projects must provide detention of stormwater such that peak flows do not exceed predeveloped runoff rates, the TR-55 method was used to demonstrate the peak runoff rates of the site in both the pre- and post-developed conditions. The calculations were then used to determine the on-site storage volumes necessary to limit post-development rates below the pre-developed conditions. Because the project site is located within the Salvador Basin and proposes more than 4 residential units, it is required to detain up to the 100-year design storm. Based on these calculations, as summarized in the Conclusion in Appendix C, the site has adequate storage capacity in the bioretention/detention facilities to detain the post-development peak flows as required.

The method used for this calculation is hydrograph analysis. The unit hydrograph rainfall distribution for the City of Napa falls under Type IA-distribution. The SCS hydrograph analysis is based on the National Resources Conservation Service Technical Release 55 for Urban Hydrology for Small Watersheds (TR-55) method (refer to Appendix B for Hydrograph Calculation Parameters).

There are two watersheds considered in this calculation. The larger Watershed #1 consists of the northern portion of the site between El Centro Avenue and Salvador Creek, while the smaller Watershed #2 consists of the remaining portion of the site south of Salvador Creek.

EXISTING CONDITIONS

The entire site, including Watersheds #1 & #2, currently drains to Salvador Creek at the eastern limit of the project. An exhibit showing the existing watersheds and time of concentration flow summary can be found in Appendix A.

PROPOSED CONDITIONS

Combination bioretention/detention facilities will be provided to detain runoff and mitigate peak flows. Portions of the developed site are not feasible to be captured and detained, including the new frontage along El Centro Avenue and the terraces along Salvador Creek. Therefore, both Watersheds #1 & #2 have portions that will be detained and portions that will not be detained. Total post-development flow was calculated by summing the detained and undetained portions for comparison with pre-development conditions using the terminus at Salvador Creek.

Refer to Appendix A for Watershed Exhibits for the proposed detained and undetained watersheds with areas. The proposed runoff for the 100-year storm is shown in the Conclusion (refer to Appendix C for Detention Calculation using Hydraflow Hydrographs Extension).

CONCLUSION

These calculations identify and describe the impacts of the proposed Zinfandel Subdivision on the hydrologic characteristics of the site and quantify the necessary storage requirement for the detention facility. The storm drain system of Zinfandel Subdivision is designed such that the proposed post-developed flow discharge from the development will not exceed pre-developed levels in accordance with the City of Napa Drainage Standards.

Summary of hydrologic analysis:

FOR DETENTION BASIN #1

100-year Pre & Post Developed Flow Discharge

Pre-developed peak run-off = **8.663 cfs**

Post-developed (Undetained) peak run-off = 2.698 cfs
Post-developed (Detained) peak run-off = 5.912 cfs
Post-developed flow discharge = 8.61 cfs

Results
100-year: **8.61 cfs** (Post-developed) ≤ **8.663 cfs** (Pre-developed) √

Detention Volume Requirement

Detention volume required = 16,710 ft^3 or 0.3836 ac-ft
Detention volume provided * = 21,069 ft^3 or 0.4837 ac-ft

Results
Detention: **16,710 ft^3** (required) ≤ **21,069 ft^3** (provided) √

Orifice Requirement

The routing and detention are accomplished by a broad crested orifice in the metering structure within the bioretention and detention basin.

The required orifice dimensions are: 18 inches long & 9.2 inches high.

FOR DETENTION BASIN #2

100-year Pre & Post Developed Flow Discharge

Pre-developed peak run-off = **1.035 cfs**

Post-developed (Undetained) peak run-off = 0.411 cfs
Post-developed (Detained) peak run-off = 0.563 cfs
Post-developed flow discharge = 0.974 cfs

Results
100-year: **0.974 cfs** (Post-developed) ≤ **1.035 cfs** (Pre-developed) √

Detention Volume Requirement

Detention volume required = 1,740 ft^3 or 0.0399 ac-ft
Detention volume provided * = 2,024 ft^3 or 0.0465 ac-ft

Results
Detention: **1,740 ft^3** (required) ≤ **2,024 ft^3** (provided) √

Orifice Requirement

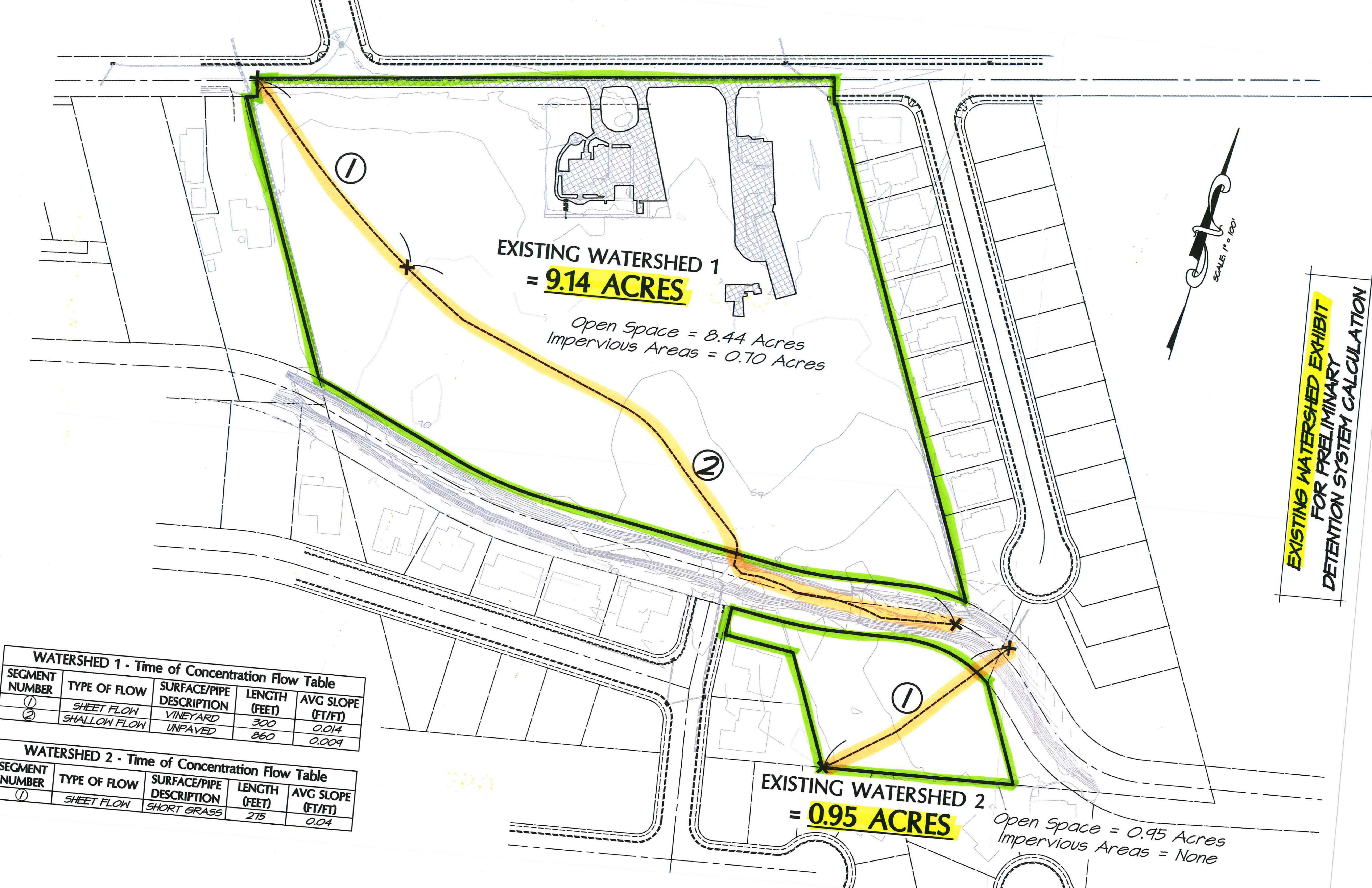
The routing and detention are accomplished by a broad crested orifice in the metering structure within the bioretention and detention basin.

The required orifice dimensions are: 8 inches long & 2 inches high.

Appendix A

Watershed Exhibits

EXISTING WATERSHED EXHIBIT FOR PRELIMINARY DETENTION SYSTEM CALCULATION



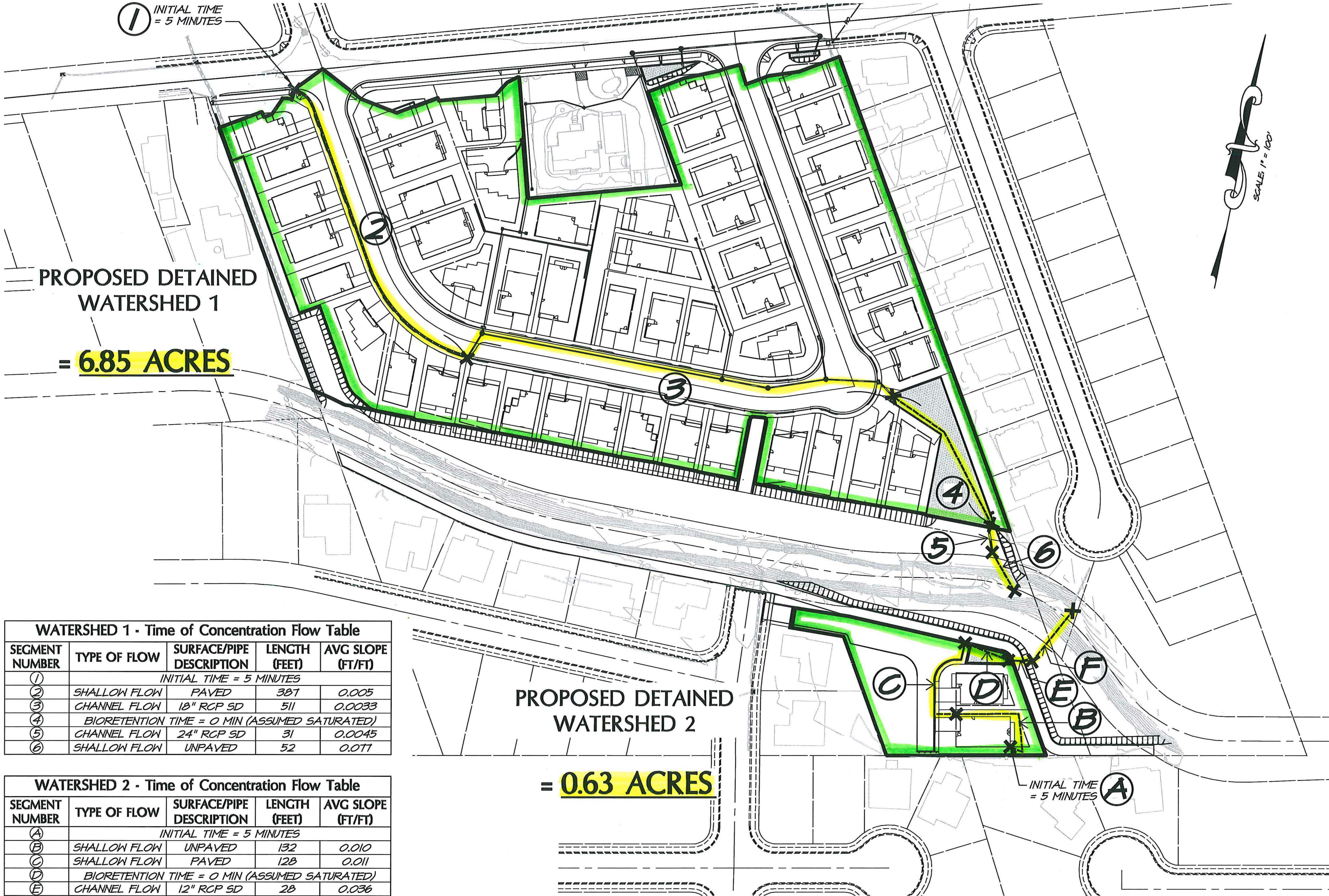
WATERSHED 1 - Time of Concentration Flow Table

SEGMENT NUMBER	TYPE OF FLOW	SURFACE/PIPE DESCRIPTION	LENGTH (FEET)	AVG SLOPE (FT/FT)
1	SHEET FLOW	VINEYARD	300	0.014
2	SHALLOW FLOW	UNPAVED	860	0.009

WATERSHED 2 - Time of Concentration Flow Table

SEGMENT NUMBER	TYPE OF FLOW	SURFACE/PIPE DESCRIPTION	LENGTH (FEET)	AVG SLOPE (FT/FT)
1	SHEET FLOW	SHORT GRASS	275	0.04

PROPOSED DETAINED WATERSHED EXHIBIT FOR PRELIMINARY DETENTION SYSTEM CALCULATION

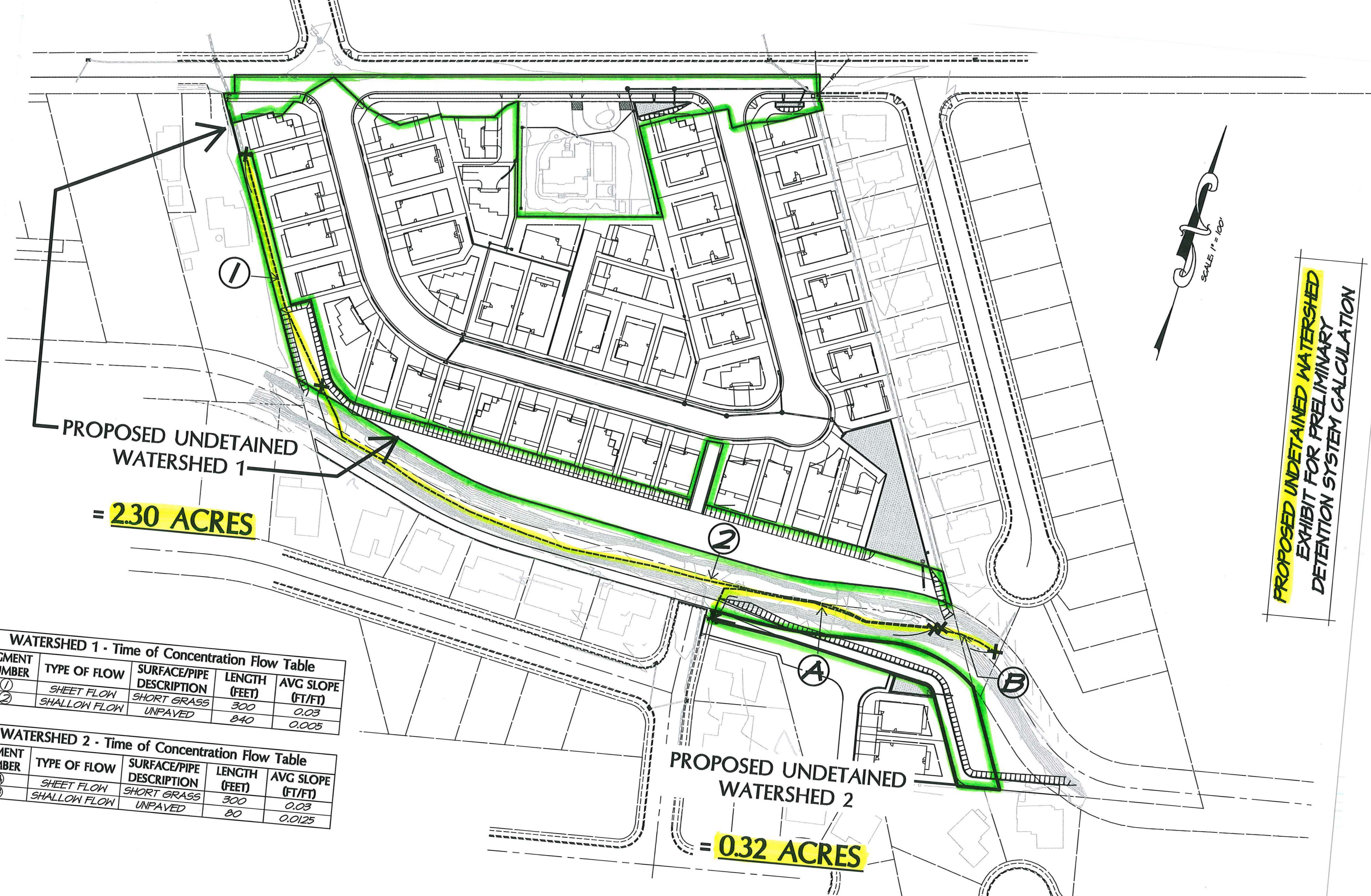


WATERSHED 1 - Time of Concentration Flow Table

SEGMENT NUMBER	TYPE OF FLOW	SURFACE/PIPE DESCRIPTION	LENGTH (FEET)	AVG SLOPE (FT/FT)
1	INITIAL TIME = 5 MINUTES			
2	SHALLOW FLOW	PAVED	387	0.005
3	CHANNEL FLOW	18" RCP SD	511	0.0033
4	BIORETENTION TIME = 0 MIN (ASSUMED SATURATED)			
5	CHANNEL FLOW	24" RCP SD	31	0.0045
6	SHALLOW FLOW	UNPAVED	52	0.077

WATERSHED 2 - Time of Concentration Flow Table

SEGMENT NUMBER	TYPE OF FLOW	SURFACE/PIPE DESCRIPTION	LENGTH (FEET)	AVG SLOPE (FT/FT)
A	INITIAL TIME = 5 MINUTES			
B	SHALLOW FLOW	UNPAVED	132	0.010
C	SHALLOW FLOW	PAVED	128	0.011
D	BIORETENTION TIME = 0 MIN (ASSUMED SATURATED)			
E	CHANNEL FLOW	12" RCP SD	28	0.036
F	SHALLOW FLOW	UNPAVED	75	0.077



WATERSHED 1 - Time of Concentration Flow Table

SEGMENT NUMBER	TYPE OF FLOW	SURFACE/PIPE DESCRIPTION	LENGTH (FEET)	AVG SLOPE (FT/FT)
1	SHEET FLOW	SHORT GRASS	300	0.03
2	SHALLOW FLOW	UNPAVED	840	0.005

WATERSHED 2 - Time of Concentration Flow Table

SEGMENT NUMBER	TYPE OF FLOW	SURFACE/PIPE DESCRIPTION	LENGTH (FEET)	AVG SLOPE (FT/FT)
A	SHEET FLOW	SHORT GRASS	300	0.03
B	SHALLOW FLOW	UNPAVED	80	0.0125

Appendix B

Hydrograph Calculation Parameters

RUNOFF CURVE NUMBERS

Table 2-2a Runoff curve numbers for urban areas [1]

Cover description: Cover type and hydrologic condition	Average percent impervious area [2]	Curve numbers for hydrologic soil group: A	B	C	D
Fully developed urban areas (vegetation established)					
Open space (lawns, parks, golf courses, cemeteries, etc.) [3]:					
Poor condition (grass cover < 50%)		68	79	86	89
Fair condition (grass cover 50% to 75%)		49	69	79	84
Good condition (grass cover > 75%)		39	61	74	80
Impervious areas:					
Paved parking lots, roofs, driveways, etc. (excluding right-of-way)		98	98	98	98
Streets and roads:					
Paved; curbs and storm sewers (excluding right-of-way)		98	98	98	98
Paved; open ditches (including right-of-way)		83	89	92	93
Gravel (including right-of-way)		76	85	89	91
Dirt (including right-of-way)		72	82	87	89
Western desert urban areas:					
Natural desert landscaping (pervious areas only) [4]		63	77	85	88
Artificial desert landscaping (impervious weed barrier, desert shrub with 1- to 2-inch sand or gravel mulch and basin borders)		96	96	96	96
Urban districts:					
Commercial and business	85	89	92	94	95
Industrial	72	81	88	91	93
Residential districts by average lot size:					
1/8 acre or less (town houses)	65	77	85	90	92
1/4 acre	38	61	75	83	87
1/3 acre	30	57	72	81	86
1/2 acre	25	54	70	80	85
1 acre	20	51	68	79	84
2 acres	12	46	65	77	82
Developing urban areas					
Newly graded areas (pervious areas only, no vegetation) [5]		77	86	91	94
Idle lands (CN's are determined using cover types similar to those in table 2-2c).					

USED TO CALCULATE EXISTING CN

PROPOSED CN

1 Average runoff condition, and $I_a = 0.2S$.

2 The average percent impervious area shown was used to develop the composite CN's. Other assumptions are as follows: impervious areas are directly connected to the drainage system, impervious areas have a CN of 98, and pervious areas are considered equivalent to open space in good hydrologic condition. CN's for other combinations of conditions may be computed using figure 2-3 or 2-4.

3 CN's shown are equivalent to those of pasture. Composite CN's may be computed for other combinations of open space cover type.

4 Composite CN's for natural desert landscaping should be computed using figures 2-3 or 2-4 based on the impervious area percentage (CN = 98) and the pervious area CN. The pervious area CN's are assumed equivalent to desert shrub in poor hydrologic condition.

5 Composite CN's to use for the design of temporary measures during grading and construction should be computed using figure 2-3 or 2-4 based on the degree of development (impervious area percentage) and the CN's for the newly graded pervious areas.

TABLE 2-2a (TR 55)

CURVE NUMBER CALCULATIONS

Worksheet: Runoff Curve Number (EXISTING)

Project: Zinfandel Subdivision		By: Ray	Date: 8/30/2019
Location: Napa, California		Checked:	Date:
Subshed name: Existing Watershed 1		Check one: ☑ Present ☐ Developed	

RUNOFF CURVE NUMBER

Soil name and hydrologic group (SCS book)	**Cover description** (cover type, treatment and hydrogic condition; percent impervious)	**CN (1)** (Table 2-2)	**Area** ☑ acres ☐ mi2 ☐ %	**Product of CN x Area**
145-D	Open Space (Grass Cover, Fair condition)	84	8.44	708.96
145-D	Impervious Areas (Paved Areas & Roofs)	98	0.70	68.60
		TOTAL:	9.14	777.56

(1) Use only one CN source per line

$$\text{CN (weighted)} = \frac{\text{total product}}{\text{total area}} = \frac{777.56}{9.14} = 85.07$$

; **USE CN** **85**

Worksheet: Runoff Curve Number

Project	Zinfandel Subdivision	By	Ray	Date	8/30/2019
Location	Napa, California	Checked		Date	
Subshed name	Existing Watershed 2	Check one:	☑ Present	☐ Developed	

RUNOFF CURVE NUMBER

Soil name and hydrologic group (SCS book)	**Cover description** (cover type, treatment and hydrogic condition; percent impervious)	**CN** (1) (Table 2-2)	**Area** ☑ acres ☐ mi2 ☐ %	**Product of CN x Area**
116-D	Open Space (Grass Cover, Fair condition)	84	0.95	79.80
116-D	Impervious Areas (Paved Areas & Roofs)	98	0.00	0.00
(1) Use only one CN source per line		**TOTAL:**	0.95	79.80

CN (weighted) = total product / total area = 79.80 / 0.95 = 84.00 ; **USE CN** **84**

2 of 2

Sheet flow

Sheet flow is flow over plane surfaces. It usually occurs in the headwater of streams. With sheet flow, the friction value (Manning's n) is an effective roughness coefficient that includes the effect of raindrop impact; drag over the plane surface; obstacles such as litter, crop ridges, and rocks; and erosion and transportation of sediment. These n values are for very shallow flow depths of about 0.1 foot or so. Table 3-1 gives Manning's n values for sheet flow for various surface conditions.

Table 3-1 Roughness coefficients (Manning's n) for sheet flow

Surface description	n [1]
Smooth surfaces (concrete, asphalt, gravel, or bare soil)	0.011
Fallow (no residue)	0.05
Cultivated soils:	
Residue cover ≤20%	0.06
Residue cover >20%	0.17
Grass:	
Short grass prairie	0.15
Dense grasses [2]	0.24
Bermudagrass	0.41
Range (natural)	0.13
Woods: [3]	
Light underbrush	0.40
Dense underbrush	0.80

1 The n values are a composite of information compiled by Engman (1986).
2 Includes species such as weeping lovegrass, bluegrass, buffalo grass, blue grama grass, and native grass mixtures.
3 When selecting n, consider cover to a height of about 0.1 ft. This is the only part of the plant cover that will obstruct sheet flow.

TABLE 3-1
ROUGHNESS COEFFICIENTS FOR SHEET FLOW

For sheet flow of less than 300 feet, use Manning's kinematic solution (Overtop and Meadows 1976) to compute T_t:

$$T_t = \frac{0.007(nL)^{0.8}}{(P_2)^{0.5} s^{0.4}} \qquad \text{[eq. 3-3]}$$

where:

T_t = travel time (hr),
n = Manning's roughness coefficient (table 3-1)
L = flow length (ft)
P_2 = 2-year, 24-hour rainfall (in)
s = slope of hydraulic grade line (land slope, ft/ft)

This simplified form of the Manning's kinematic solution is based on the following: (1) shallow steady uniform flow, (2) constant intensity of rainfall excess (that part of a rain available for runoff), (3) rainfall duration of 24 hours, and (4) minor effect of infiltration on travel time. Rainfall depth can be obtained from appendix B.

Shallow concentrated flow

After a maximum of 300 feet, sheet flow usually becomes shallow concentrated flow. The average velocity for this flow can be determined from figure 3-1, in which average velocity is a function of watercourse slope and type of channel. For slopes less than 0.005 ft/ft, use equations given in appendix F for figure 3-1. Tillage can affect the direction of shallow concentrated flow. Flow may not always be directly down the watershed slope if tillage runs across the slope.

After determining average velocity in figure 3-1, use equation 3-1 to estimate travel time for the shallow concentrated flow segment.

Open channels

Open channels are assumed to begin where surveyed cross section information has been obtained, where channels are visible on aerial photographs, or where blue lines (indicating streams) appear on United States Geological Survey (USGS) quadrangle sheets. Manning's equation or water surface profile information can be used to estimate average flow velocity. Average flow velocity is usually determined for bank-full elevation.

FREQUENCY	RAINFALL DEPTH/STORM DURATION, INCHES									
	5 M	15 M	1 HR	2 HR	3 HR	6 HR	12 HR	24 HR	2 D	4 D
2-YR	0.15	0.27	0.57	0.82	1.02	1.50	1.98	2.45	3.12	4.03
5-YR	0.20	0.38	0.80	1.16	1.42	2.12	2.79	3.44	4.51	5.77
10-YR	0.25	0.46	0.97	1.39	1.70	2.53	3.33	4.12	5.42	6.94
25-YR	0.30	0.56	1.16	1.66	2.04	3.03	4.00	4.95	6.63	8.38
50-YR	0.32	0.62	1.30	1.87	2.29	3.40	4.48	5.56	7.49	9.44
100-YR	0.36	0.69	1.44	2.07	2.54	3.76	4.96	6.14	8.33	10.45
500-YR	0.45	0.85	1.78	2.55	3.14	4.67	6.15	7.60	10.50	13.01

CHART IS FROM CITY OF NAPA 2006 STORM DRAINAGE MASTER PLAN TABLE 3-2

G:\PubWrks\Standard Details - City of Napa\09-Charts, Figures, and Tables\Table 2.2.dwg

CITY OF NAPA

PUBLIC WORKS DEPARTMENT

RAINFALL DEPTH - DURATION

DRAWN BY:	BRL	CHECKED BY:	TCW
DATE:	05/2018	APPROVED BY:	JRL
SCALE:	NONE	DRAWING NO.	TABLE-2.2
FIELD NOTES:			

122° 19' 2" W
122° 18' 43" W
38° 20' 10" N
38° 20' 10" N

38° 20' 0" N
38° 20' 0" N
122° 19' 2" W
122° 18' 43" W

Map Scale: 1:2,120 if printed on A landscape (11" x 8.5") sheet.

N

Meters: 0, 30, 60, 120, 180

Feet: 0, 100, 200, 400, 600

Map projection: Web Mercator Corner coordinates: WGS84

MAP LEGEND

Area of Interest (AOI)
- Area of Interest (AOI)

Soils

Soil Rating Polygons
- A
- A/D
- B
- B/D
- C
- C/D
- D
- Not rated or not available

Soil Rating Lines
- A
- A/D
- B
- B/D
- C
- C/D
- D
- Not rated or not available

Soil Rating Points
- A
- A/D
- B
- B/D
- C
- C/D
- D
- Not rated or not available

Water Features
- Streams and Canals

Transportation
- Rails
- Interstate Highways
- US Routes
- Major Roads
- Local Roads

Background
- Aerial Photography

MAP INFORMATION

The soil surveys that comprise your AOI were mapped at 1:24,000.

Warning: Soil Map may not be valid at this scale.

Enlargement of maps beyond the scale of mapping can cause misunderstanding of the detail of mapping and accuracy of soil line placement. The maps do not show the small areas of contrasting soils that could have been shown at a more detailed scale.

Please rely on the bar scale on each map sheet for map measurements.

Source of Map: Natural Resources Conservation Service
Web Soil Survey URL:
Coordinate System: Web Mercator (EPSG:3857)

Maps from the Web Soil Survey are based on the Web Mercator projection, which preserves direction and shape but distorts distance and area. A projection that preserves area, such as the Albers equal-area conic projection, should be used if more accurate calculations of distance or area are required.

This product is generated from the USDA-NRCS certified data as of the version date(s) listed below.

Soil Survey Area: Napa County, California
Survey Area Data: Version 10, Sep 25, 2017

Soil map units are labeled (as space allows) for map scales 1:50,000 or larger.

Date(s) aerial images were photographed: Apr 17, 2015—Oct 18, 2016

The orthophoto or other base map on which the soil lines were compiled and digitized probably differs from the background imagery displayed on these maps. As a result, some minor shifting of map unit boundaries may be evident.

Hydrologic Soil Group

Map unit symbol	Map unit name	Rating	Acres in AOI	Percent of AOI
116	Clear Lake clay, drained, 0 to 2 percent slopes, MLRA 14	D	1.2	11.9%
145	Haire loam, 0 to 2 percent slopes	D	9.2	88.1%
Totals for Area of Interest			**10.5**	**100.0%**

Description

Hydrologic soil groups are based on estimates of runoff potential. Soils are assigned to one of four groups according to the rate of water infiltration when the soils are not protected by vegetation, are thoroughly wet, and receive precipitation from long-duration storms.

The soils in the United States are assigned to four groups (A, B, C, and D) and three dual classes (A/D, B/D, and C/D). The groups are defined as follows:

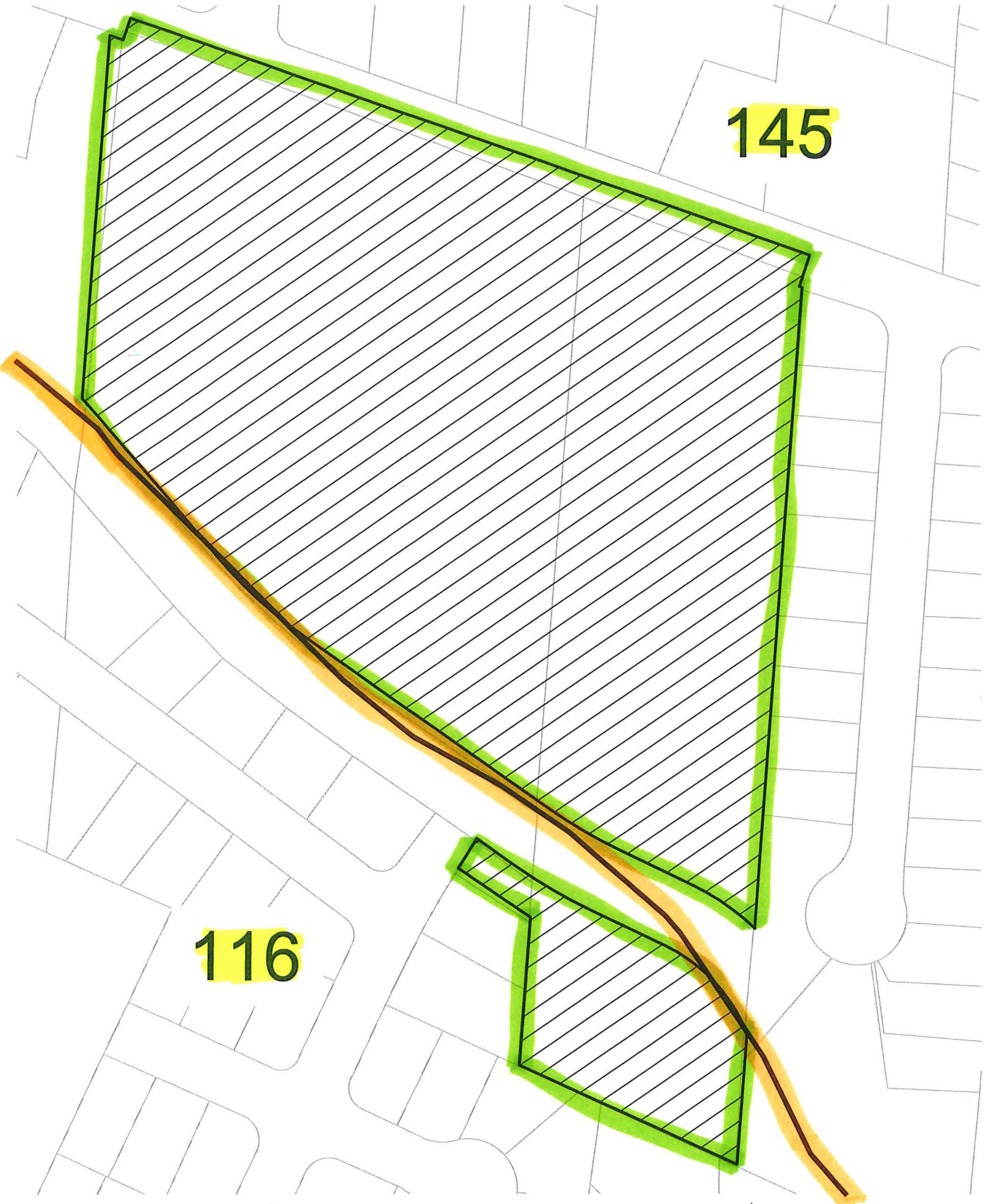
Group A. Soils having a high infiltration rate (low runoff potential) when thoroughly wet. These consist mainly of deep, well drained to excessively drained sands or gravelly sands. These soils have a high rate of water transmission.

Group B. Soils having a moderate infiltration rate when thoroughly wet. These consist chiefly of moderately deep or deep, moderately well drained or well drained soils that have moderately fine texture to moderately coarse texture. These soils have a moderate rate of water transmission.

Group C. Soils having a slow infiltration rate when thoroughly wet. These consist chiefly of soils having a layer that impedes the downward movement of water or soils of moderately fine texture or fine texture. These soils have a slow rate of water transmission.

Group D. Soils having a very slow infiltration rate (high runoff potential) when thoroughly wet. These consist chiefly of clays that have a high shrink-swell potential, soils that have a high water table, soils that have a claypan or clay layer at or near the surface, and soils that are shallow over nearly impervious material. These soils have a very slow rate of water transmission.

If a soil is assigned to a dual hydrologic group (A/D, B/D, or C/D), the first letter is for drained areas and the second is for undrained areas. Only the soils that in their natural condition are in group D are assigned to dual classes.



SCS SOIL CLASSIFICATION EXHIBIT
FOR PRELIMINARY
DETENTION SYSTEM CALCULATION

12. Pipe sizes.

As noted in Table 2.1, several options are available for use in estimating discharge for storm events. Table 2.2 provides the Design Depth Frequency (DDF) for selected storms and Table 2.3 shows Rainfall Intensity Duration.

TABLE 2.2 – RAINFALL DEPTH (DURATION)

	RAINFALL DEPTH/STORM DURATION (INCHES)									
DDF	**5M**	**15M**	**1HR**	**2HR**	**3HR**	**6HR**	**12 HR**	**24 HR**	**2D**	**4D**
2-YR	0.15	0.27	0.57	0.82	1.02	1.50	1.98	2.45	3.12	4.03
5-YR	0.20	0.38	0.80	1.16	1.42	2.12	2.79	3.44	4.51	5.77
10-YR	0.25	0.46	0.97	1.39	1.70	2.53	3.33	4.12	5.42	6.94
25-YR	0.30	0.56	1.16	1.66	2.04	3.03	4.00	4.95	6.63	8.38
50-YR	0.32	0.62	1.30	1.87	2.29	3.40	4.48	5.56	7.49	9.44
100-YR	0.36	0.69	1.44	2.07	2.54	3.76	4.96	6.14	8.33	10.45
500-YR	0.45	0.85	1.78	2.55	3.14	4.67	6.15	7.60	10.50	13.01

Source: City of Napa 2006 Storm Drainage Master Plan Table 3-1

TABLE 2.3 – RAINFALL INTENSITY (DURATION)

	RAINFALL DEPTH/STORM DURATION (INCHES PER HOUR)									
DDF	**5M**	**15M**	**1HR**	**2HR**	**3HR**	**6HR**	**12 HR**	**24 HR**	**2D**	**4D**
2-YR	1.80	1.08	0.80	0.41	0.34	0.25	0.16	0.10	0.06	0.04
5-YR	2.40	1.52	0.08	0.58	0.47	0.35	0.23	0.14	0.09	0.06
10-YR	3.00	1.84	0.97	0.70	0.57	0.42	0.28	0.17	0.11	0.07
25-YR	3.60	2.24	1.16	0.83	0.68	0.50	0.33	0.20	0.14	0.08
50-YR	3.84	2.48	1.30	0.94	0.76	0.57	0.37	0.23	0.16	0.10
100-YR	4.32	2.76	1.44	1.04	0.84	0.63	0.41	0.26	0.17	0.11
500-YR	5.40	3.40	1.78	1.28	1.04	0.78	0.51	0.32	0.22	0.14

Source: City of Napa 2006 Storm Drainage Master Plan Table 3-2

A. Rational Method

The 10-and 100-year peak runoff shall be determined for each analysis point using the Rational Method. The Rational Method provides reasonable estimates of peak runoff for small watersheds. The method relates a peak discharge for the project site, a runoff coefficient (C), and rainfall intensity (i). Runoff coefficients were found to vary between 0.35 and 0.90 for land use and storm frequency.

The Rational Method equation has the form: $Q = CiA$

Where:

Q = rate of runoff, acre-inches per hour or cubic feet per second

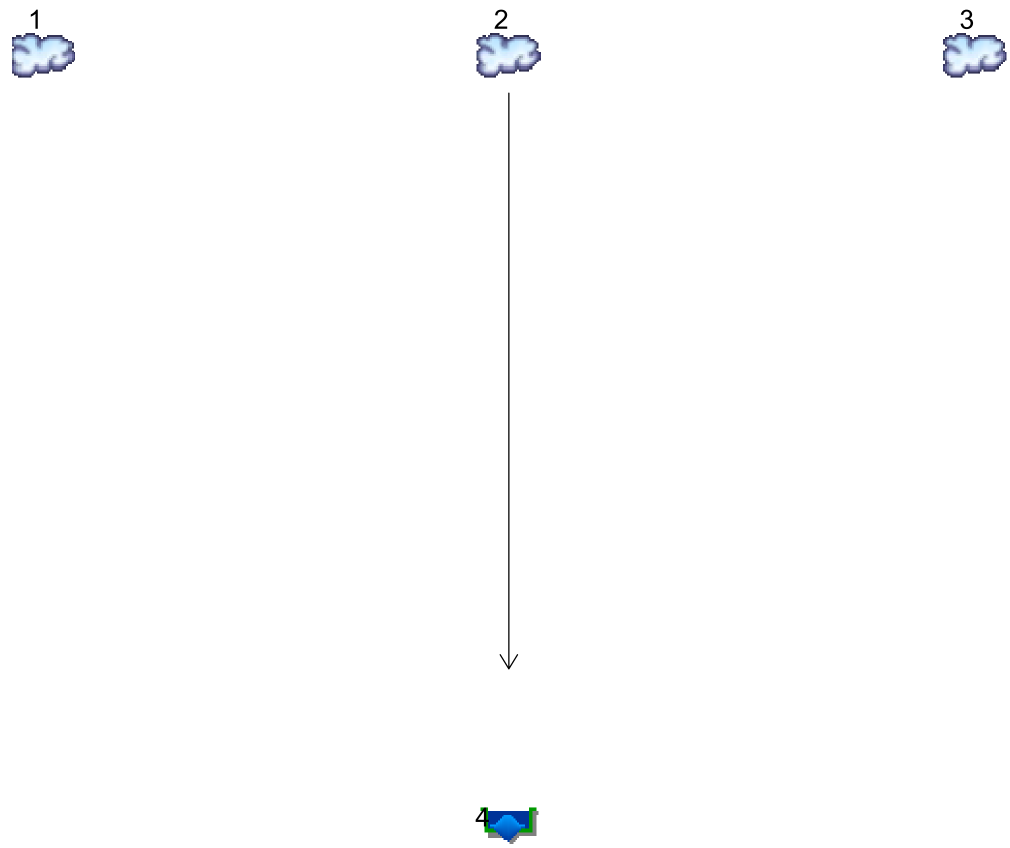
C = runoff coefficient, which is the ratio of peak runoff to average rainfall intensity

Appendix C

Detention Calculations using Hydraflow

Watershed Model Schematic

Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2020



Legend

Hyd.	Origin	Description
1	SCS Runoff	Existing 100-yr Watershed 1
2	SCS Runoff	Proposed Pre-Routed 100-yr Watershed 1
3	SCS Runoff	Proposed Undetained 100-yr Watershed 1
4	Reservoir	DETAINED ROUTED (100)

Project: DB1-100yrDetention Minus BRF Tc.gpw

Thursday, 07 / 6 / 2023

Hydrograph Return Period Recap

Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2020

Hyd. No.	Hydrograph type (origin)	Inflow hyd(s)	Peak Outflow (cfs)								Hydrograph Description
			1-yr	2-yr	3-yr	5-yr	10-yr	25-yr	50-yr	100-yr	
1	SCS Runoff	------	-------	-------	-------	-------	-------	-------	-------	8.663	Existing 100-yr Watershed 1
2	SCS Runoff	------	-------	-------	-------	-------	-------	-------	-------	9.121	Proposed Pre-Routed 100-yr Watersh
3	SCS Runoff	------	-------	-------	-------	-------	-------	-------	-------	2.698	Proposed Undetained 100-yr Watersh
4	Reservoir	2	-------	-------	-------	-------	-------	-------	-------	5.912	DETAINED ROUTED (100)

Proj. file: DB1-100yrDetention Minus BRF Tc.gpw

Thursday, 07 / 6 / 2023

Hydrograph Summary Report

Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2020

Hyd. No.	Hydrograph type (origin)	Peak flow (cfs)	Time interval (min)	Time to Peak (min)	Hyd. volume (cuft)	Inflow hyd(s)	Maximum elevation (ft)	Total strge used (cuft)	Hydrograph Description
1	SCS Runoff	8.663	1	496	147,137	------	------	------	Existing 100-yr Watershed 1
2	SCS Runoff	9.121	1	477	127,455	------	------	------	Proposed Pre-Routed 100-yr Watersh
3	SCS Runoff	2.698	1	492	43,474	------	------	------	Proposed Undetained 100-yr Watersh
4	Reservoir	5.912	1	493	119,651	2	68.94	16,710	DETAINED ROUTED (100)

DB1-100yrDetention Minus BRF Tc.gpw	Return Period: 100 Year	Thursday, 07 / 6 / 2023

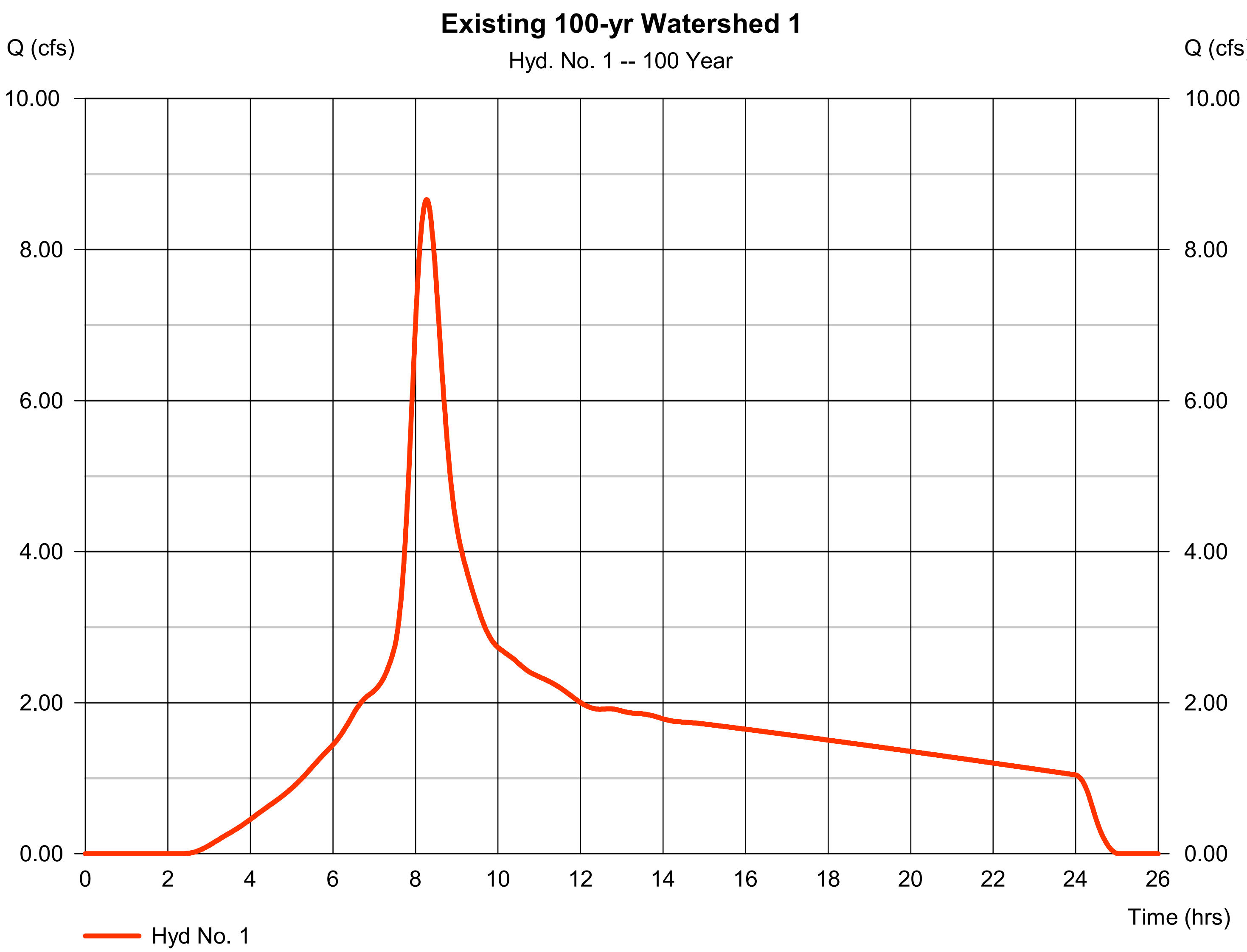
Hydrograph Report

Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2020 Thursday, 07 / 6 / 2023

Hyd. No. 1

Existing 100-yr Watershed 1

Hydrograph type	= SCS Runoff	Peak discharge	= 8.663 cfs
Storm frequency	= 100 yrs	Time to peak	= 8.27 hrs
Time interval	= 1 min	Hyd. volume	= 147,137 cuft
Drainage area	= 9.140 ac	Curve number	= 85
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 40.50 min
Total precip.	= 6.14 in	Distribution	= Type IA
Storm duration	= 24 hrs	Shape factor	= 484



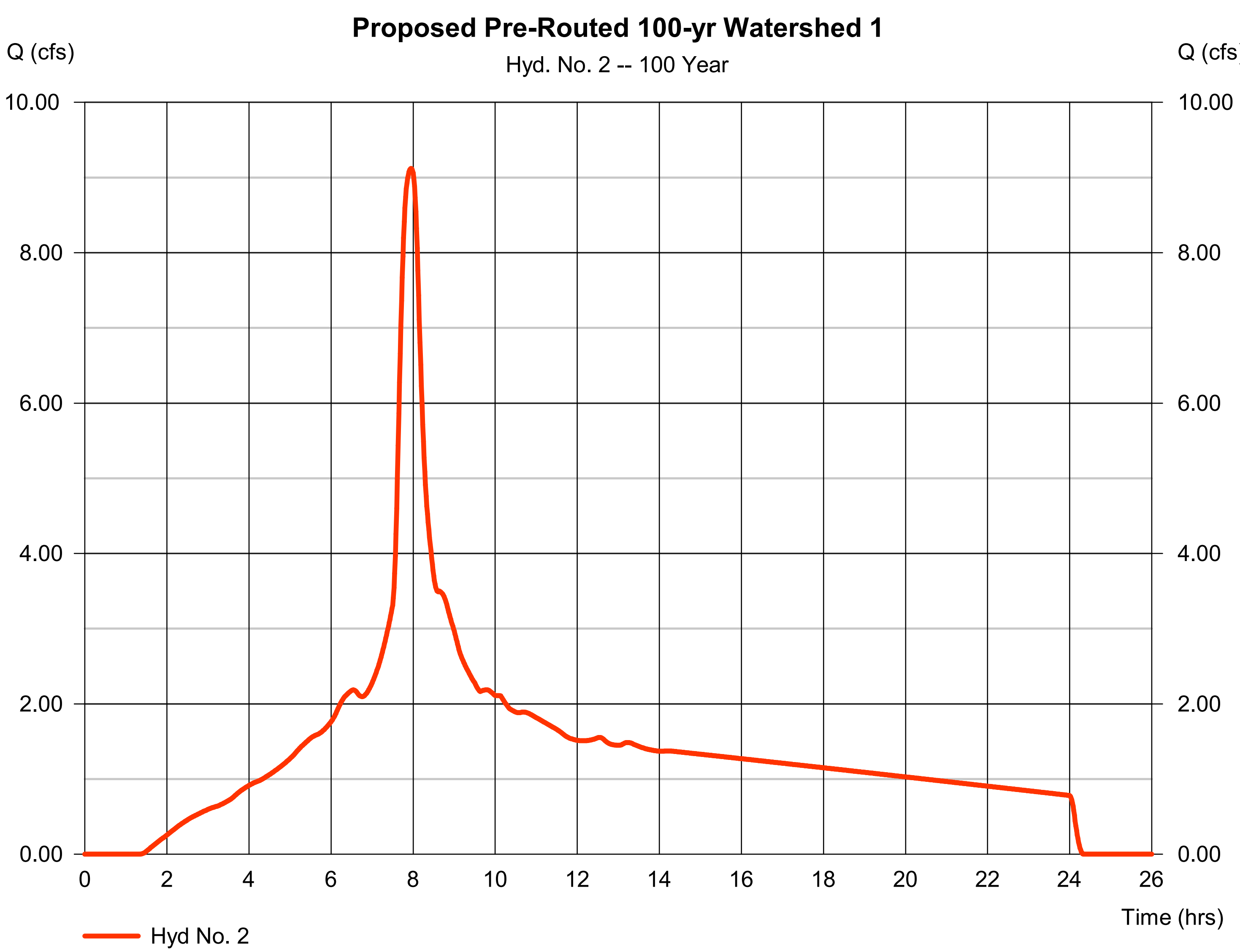
Hydrograph Report

Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2020 — Thursday, 07 / 6 / 2023

Hyd. No. 2

Proposed Pre-Routed 100-yr Watershed 1

Hydrograph type	= SCS Runoff	Peak discharge	= 9.121 cfs
Storm frequency	= 100 yrs	Time to peak	= 7.95 hrs
Time interval	= 1 min	Hyd. volume	= 127,455 cuft
Drainage area	= 6.850 ac	Curve number	= 92
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 12.30 min
Total precip.	= 6.14 in	Distribution	= Type IA
Storm duration	= 24 hrs	Shape factor	= 484



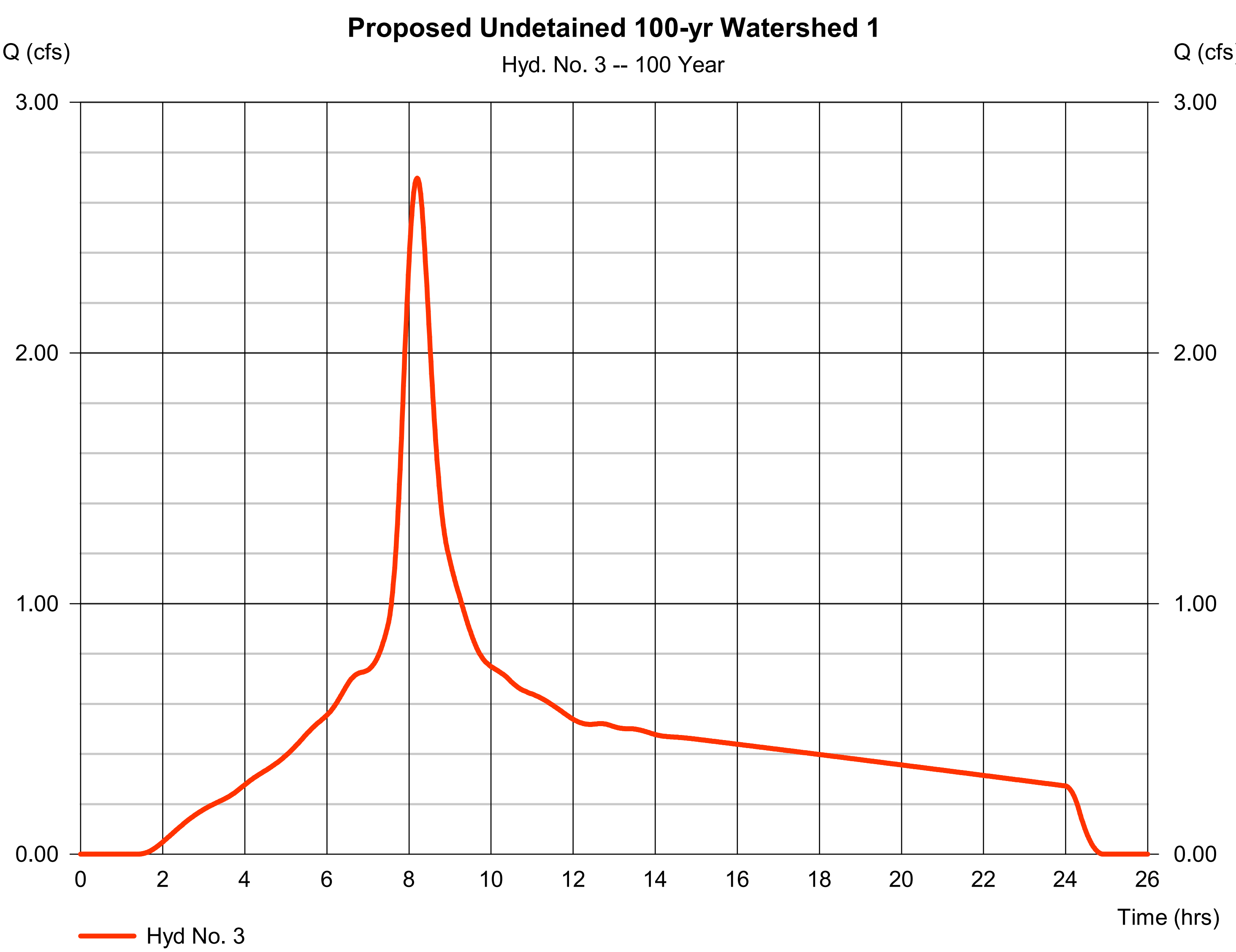
Hydrograph Report

Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2020 Thursday, 07 / 6 / 2023

Hyd. No. 3

Proposed Undetained 100-yr Watershed 1

Hydrograph type	= SCS Runoff	Peak discharge	= 2.698 cfs
Storm frequency	= 100 yrs	Time to peak	= 8.20 hrs
Time interval	= 1 min	Hyd. volume	= 43,474 cuft
Drainage area	= 2.300 ac	Curve number	= 92
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 35.20 min
Total precip.	= 6.14 in	Distribution	= Type IA
Storm duration	= 24 hrs	Shape factor	= 484



Hydrograph Report

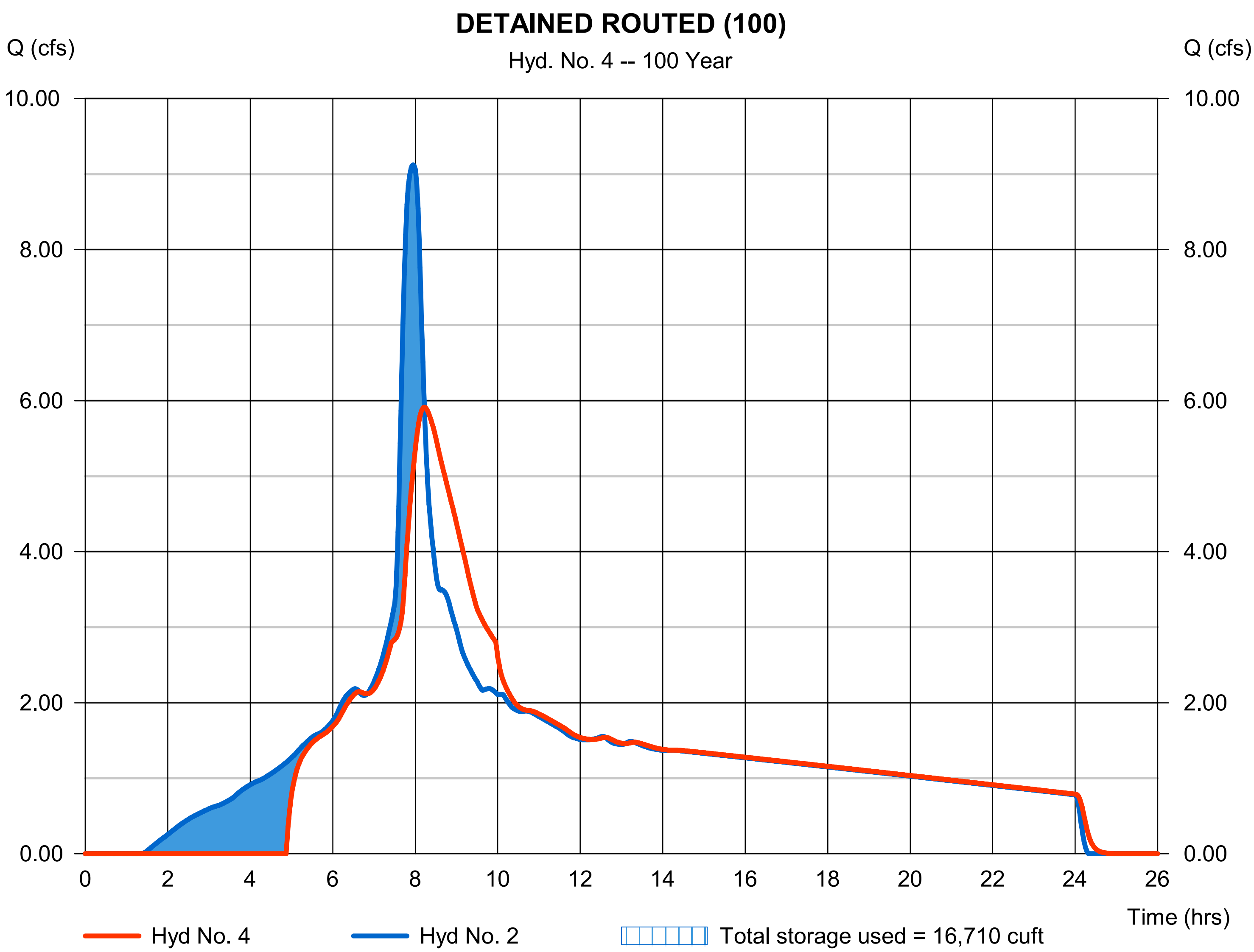
Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2020 — Thursday, 07 / 6 / 2023

Hyd. No. 4

DETAINED ROUTED (100)

Hydrograph type	= Reservoir	Peak discharge	= 5.912 cfs
Storm frequency	= 100 yrs	Time to peak	= 8.22 hrs
Time interval	= 1 min	Hyd. volume	= 119,651 cuft
Inflow hyd. No.	= 2 - Proposed Pre-Routed 100-yr Watershed 1	Max. Elevation	= 68.94 ft
Reservoir name	= DETENTION BASIN 1	Max. Storage	= 16,710 cuft

Storage Indication method used.



Pond Report

Pond No. 1 - DETENTION BASIN 1

Pond Data

Contours -User-defined contour areas. Conic method used for volume calculation. Begining Elevation = 66.80 ft

Stage / Storage Table

Stage (ft)	Elevation (ft)	Contour area (sqft)	Incr. Storage (cuft)	Total storage (cuft)
0.00	66.80	7,804	0	0
1.00	67.80	7,804	7,803	7,803
2.70	69.60	7,804	13,265	21,069

Culvert / Orifice Structures

	[A]	[B]	[C]	[PrfRsr]
Rise (in)	= 9.20	0.00	0.00	0.00
Span (in)	= 18.00	0.00	0.00	0.00
No. Barrels	= 1	1	1	0
Invert El. (ft)	= 67.30	0.00	0.00	0.00
Length (ft)	= 0.00	0.00	0.00	0.00
Slope (%)	= 0.00	0.00	0.00	n/a
N-Value	= .013	.013	.013	n/a
Orifice Coeff.	= 0.60	0.60	0.60	0.60
Multi-Stage	= n/a	No	No	No

Weir Structures

	[A]	[B]	[C]	[D]
Crest Len (ft)	= 0.00	0.00	0.00	0.00
Crest El. (ft)	= 0.00	0.00	0.00	0.00
Weir Coeff.	= 3.33	3.33	3.33	3.33
Weir Type	= ---	---	---	---
Multi-Stage	= No	No	No	No

Exfil.(in/hr) = 0.000 (by Contour)
TW Elev. (ft) = 67.80

Note: Culvert/Orifice outflows are analyzed under inlet (ic) and outlet (oc) control. Weir risers checked for orifice conditions (ic) and submergence (s).

Stage / Storage / Discharge Table

Stage ft	Storage cuft	Elevation ft	Clv A cfs	Clv B cfs	Clv C cfs	PrfRsr cfs	Wr A cfs	Wr B cfs	Wr C cfs	Wr D cfs	Exfil cfs	User cfs	Total cfs
0.00	0	66.80	0.00	---	---	---	---	---	---	---	---	---	0.000
1.00	7,803	67.80	0.00	---	---	---	---	---	---	---	---	---	0.000
2.70	21,069	69.60	7.43 ic	---	---	---	---	---	---	---	---	---	7.429

Hydraflow Rainfall Report

Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2020 Thursday, 07 / 6 / 2023

Return Period (Yrs)	Intensity-Duration-Frequency Equation Coefficients (FHA)			
	B	D	E	(N/A)
1	0.0000	0.0000	0.0000	--------
2	5.1978	2.3000	0.5349	--------
3	0.0000	0.0000	0.0000	--------
5	0.0000	0.0000	0.0000	--------
10	12.1604	5.1000	0.6055	--------
25	16.1806	5.9000	0.6293	--------
50	0.0000	0.0000	0.0000	--------
100	22.1077	7.4000	0.6487	--------

File name: NAPA.IDF

Intensity = B / (Tc + D)^E

Return Period (Yrs)	Intensity Values (in/hr)											
	5 min	10	15	20	25	30	35	40	45	50	55	60
1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2	1.79	1.36	1.13	0.99	0.89	0.81	0.75	0.70	0.66	0.63	0.60	0.57
3	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
5	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
10	3.00	2.35	1.98	1.73	1.55	1.41	1.30	1.21	1.14	1.07	1.02	0.97
25	3.60	2.84	2.39	2.09	1.87	1.70	1.57	1.46	1.36	1.29	1.22	1.16
50	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
100	4.32	3.47	2.94	2.58	2.32	2.11	1.94	1.81	1.70	1.60	1.51	1.44

Tc = time in minutes. Values may exceed 60.

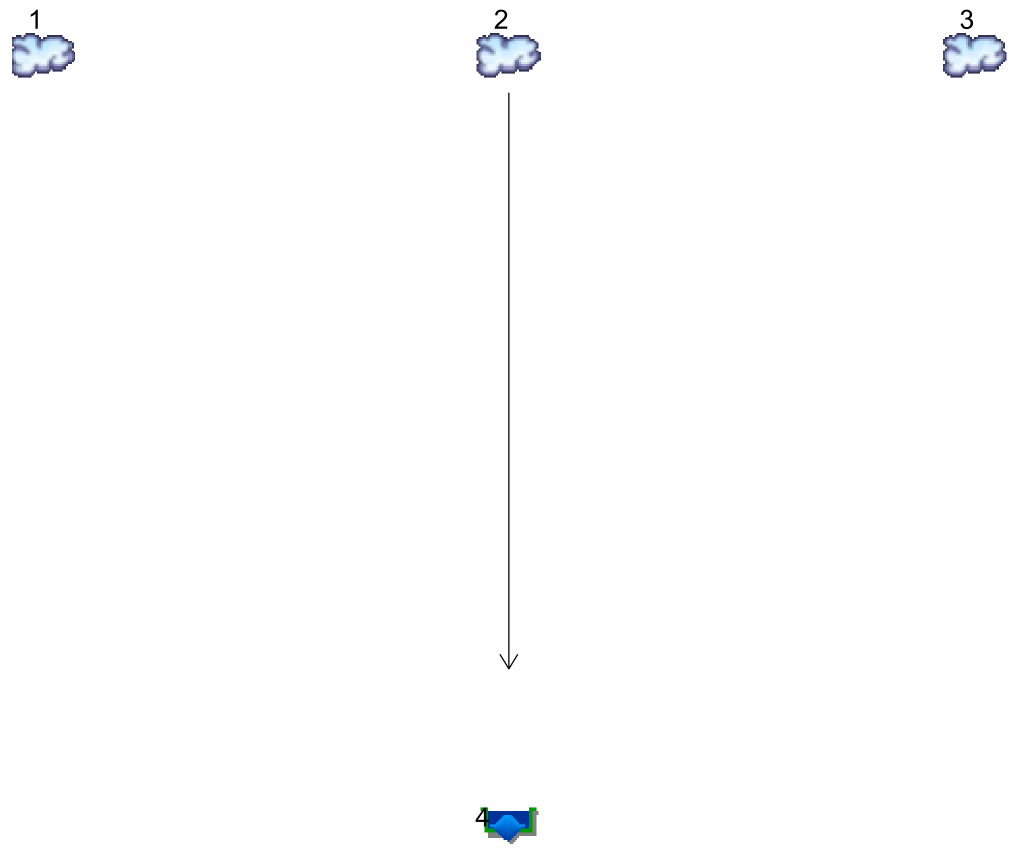
Precip. file name: NAPA.pcp

Storm Distribution	Rainfall Precipitation Table (in)							
	1-yr	2-yr	3-yr	5-yr	10-yr	25-yr	50-yr	100-yr
SCS 24-hour	0.00	2.45	0.00	3.44	4.12	4.95	5.56	6.14
SCS 6-Hr	0.00	1.50	0.00	2.12	2.53	3.03	3.40	3.76
Huff-1st	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Huff-2nd	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Huff-3rd	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Huff-4th	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Huff-Indy	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Custom	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

Hydraflow Table of Contents

Watershed Model Schematic

Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2020



Legend

Hyd.	Origin	Description
1	SCS Runoff	Existing 100-yr Watershed 2
2	SCS Runoff	Proposed Pre-Routed 100-yr Watershed 2
3	SCS Runoff	Proposed Undetained 100-yr Watershed 2
4	Reservoir	DETAINED ROUTED (100)

Project: DB2-100yrDetention Minus BRF Tc.gpw

Thursday, 07 / 6 / 2023

Hydrograph Return Period Recap

Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2020

Hyd. No.	Hydrograph type (origin)	Inflow hyd(s)	Peak Outflow (cfs)								Hydrograph Description
			1-yr	2-yr	3-yr	5-yr	10-yr	25-yr	50-yr	100-yr	
1	SCS Runoff	------	-------	-------	-------	-------	-------	-------	-------	1.035	Existing 100-yr Watershed 2
2	SCS Runoff	------	-------	-------	-------	-------	-------	-------	-------	0.837	Proposed Pre-Routed 100-yr Watersh
3	SCS Runoff	------	-------	-------	-------	-------	-------	-------	-------	0.411	Proposed Undetained 100-yr Watersh
4	Reservoir	2	-------	-------	-------	-------	-------	-------	-------	0.563	DETAINED ROUTED (100)

Hydrograph Summary Report

Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2020

Hyd. No.	Hydrograph type (origin)	Peak flow (cfs)	Time interval (min)	Time to Peak (min)	Hyd. volume (cuft)	Inflow hyd(s)	Maximum elevation (ft)	Total strge used (cuft)	Hydrograph Description
1	SCS Runoff	1.035	1	482	14,924	------	------	------	Existing 100-yr Watershed 2
2	SCS Runoff	0.837	1	473	11,611	------	------	------	Proposed Pre-Routed 100-yr Watersh
3	SCS Runoff	0.411	1	485	6,049	------	------	------	Proposed Undetained 100-yr Watersh
4	Reservoir	0.563	1	488	10,670	2	68.91	1,740	DETAINED ROUTED (100)

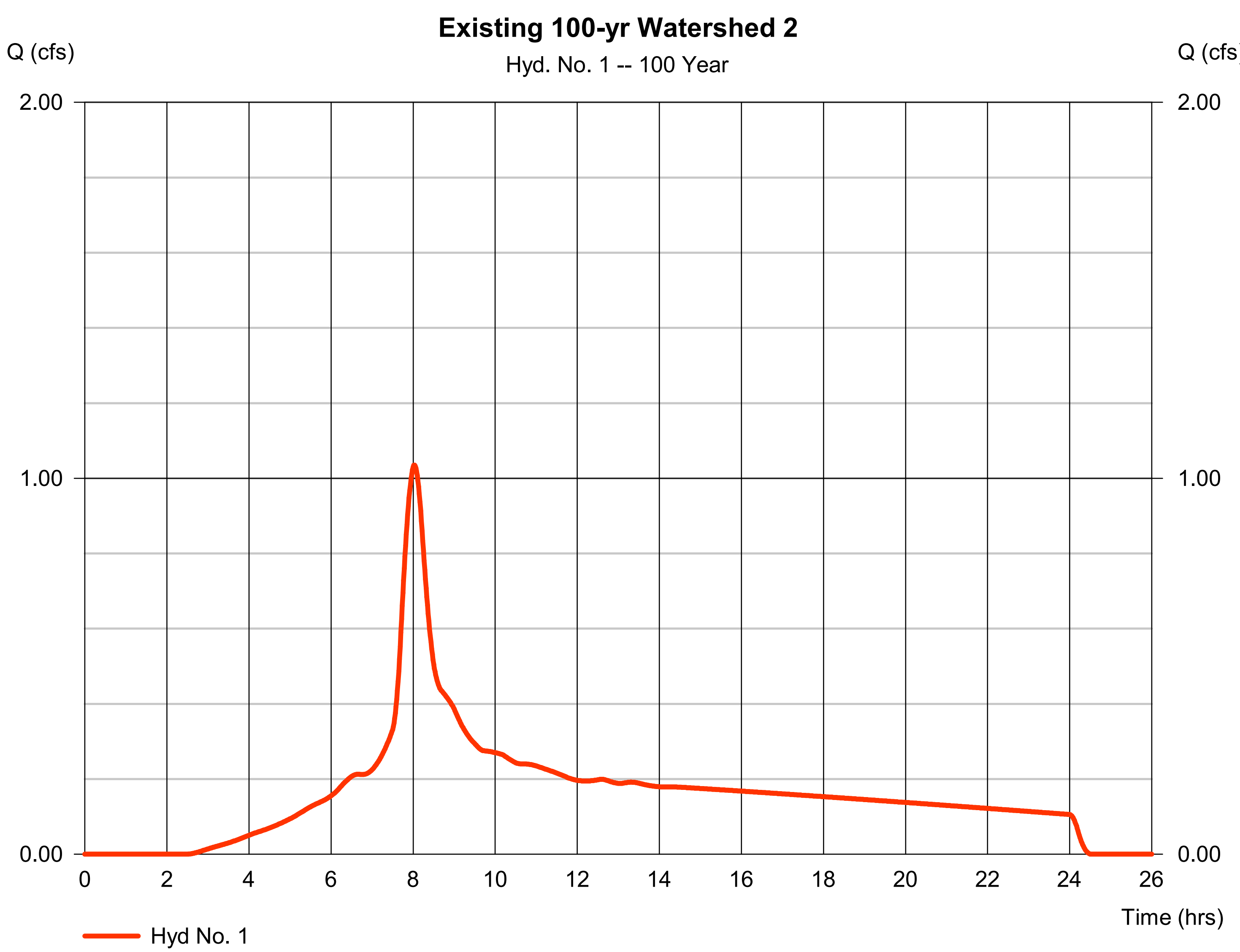
Hydrograph Report

Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2020 Thursday, 07 / 6 / 2023

Hyd. No. 1

Existing 100-yr Watershed 2

Hydrograph type	= SCS Runoff	Peak discharge	= 1.035 cfs
Storm frequency	= 100 yrs	Time to peak	= 8.03 hrs
Time interval	= 1 min	Hyd. volume	= 14,924 cuft
Drainage area	= 0.950 ac	Curve number	= 84
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 19.10 min
Total precip.	= 6.14 in	Distribution	= Type IA
Storm duration	= 24 hrs	Shape factor	= 484



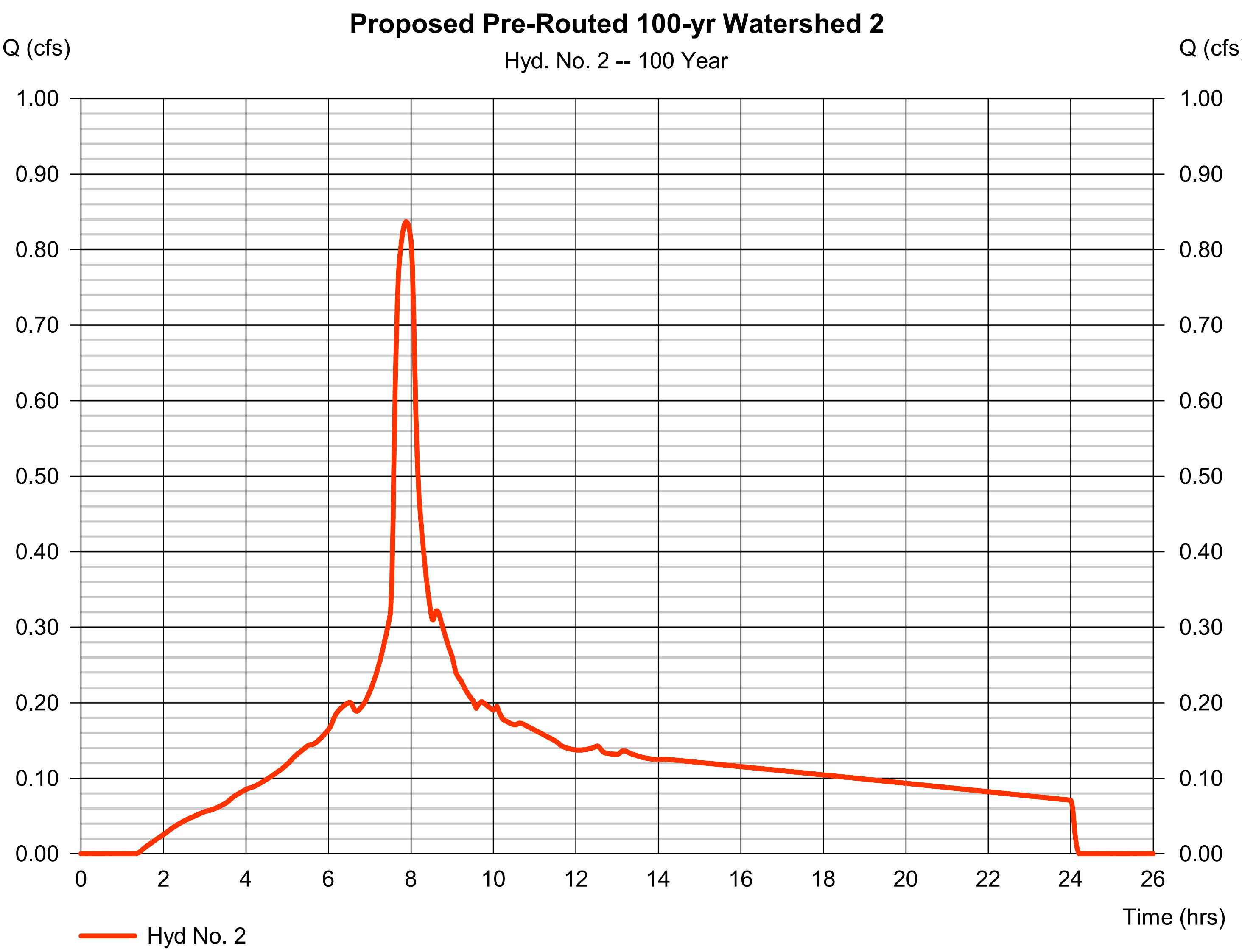
Hydrograph Report

Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2020 Thursday, 07 / 6 / 2023

Hyd. No. 2

Proposed Pre-Routed 100-yr Watershed 2

Hydrograph type	= SCS Runoff	Peak discharge	= 0.837 cfs
Storm frequency	= 100 yrs	Time to peak	= 7.88 hrs
Time interval	= 1 min	Hyd. volume	= 11,611 cuft
Drainage area	= 0.630 ac	Curve number	= 92
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 7.30 min
Total precip.	= 6.14 in	Distribution	= Type IA
Storm duration	= 24 hrs	Shape factor	= 484

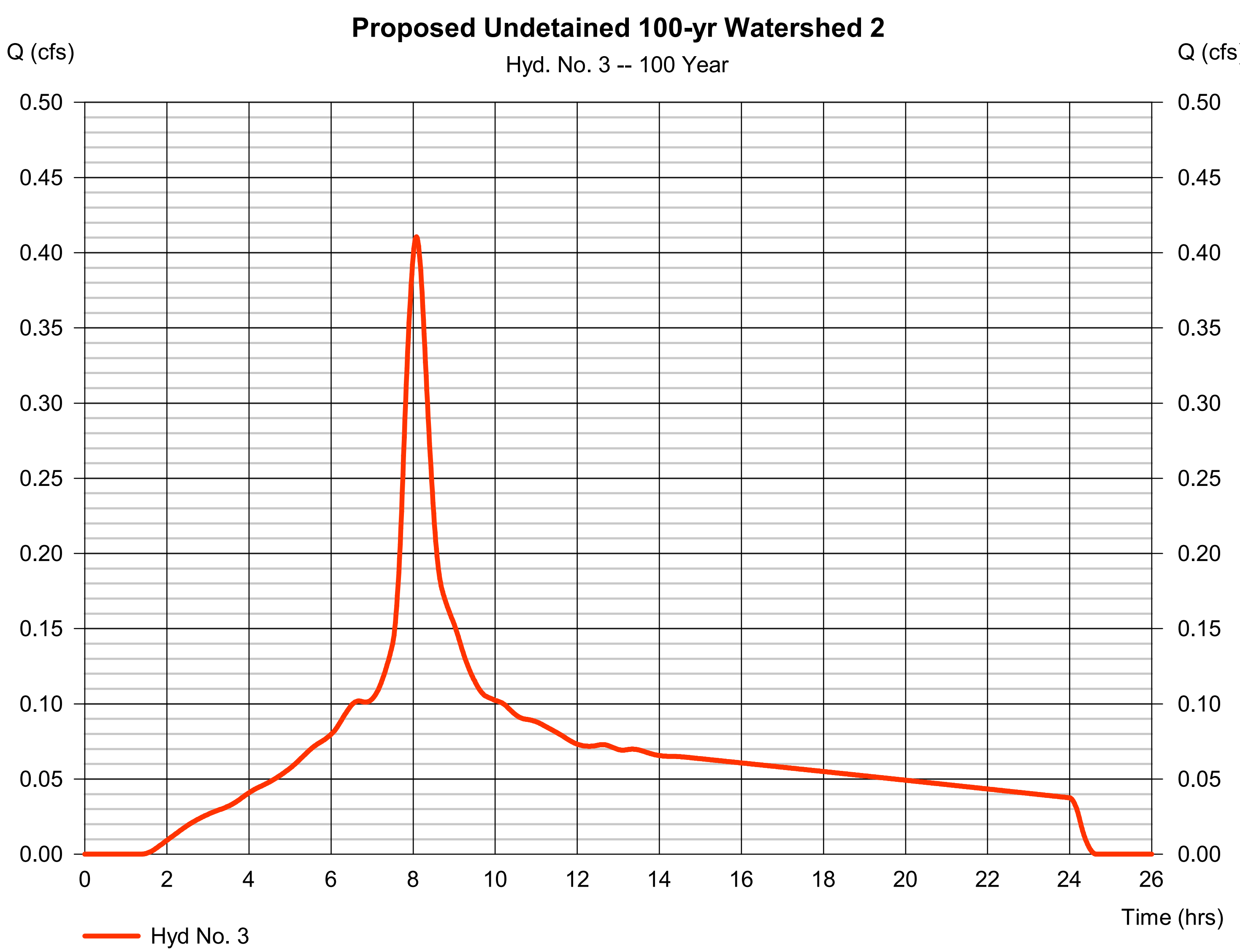


Hydrograph Report

Hyd. No. 3

Proposed Undetained 100-yr Watershed 2

Hydrograph type	= SCS Runoff	Peak discharge	= 0.411 cfs
Storm frequency	= 100 yrs	Time to peak	= 8.08 hrs
Time interval	= 1 min	Hyd. volume	= 6,049 cuft
Drainage area	= 0.320 ac	Curve number	= 92
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 23.70 min
Total precip.	= 6.14 in	Distribution	= Type IA
Storm duration	= 24 hrs	Shape factor	= 484



Hydrograph Report

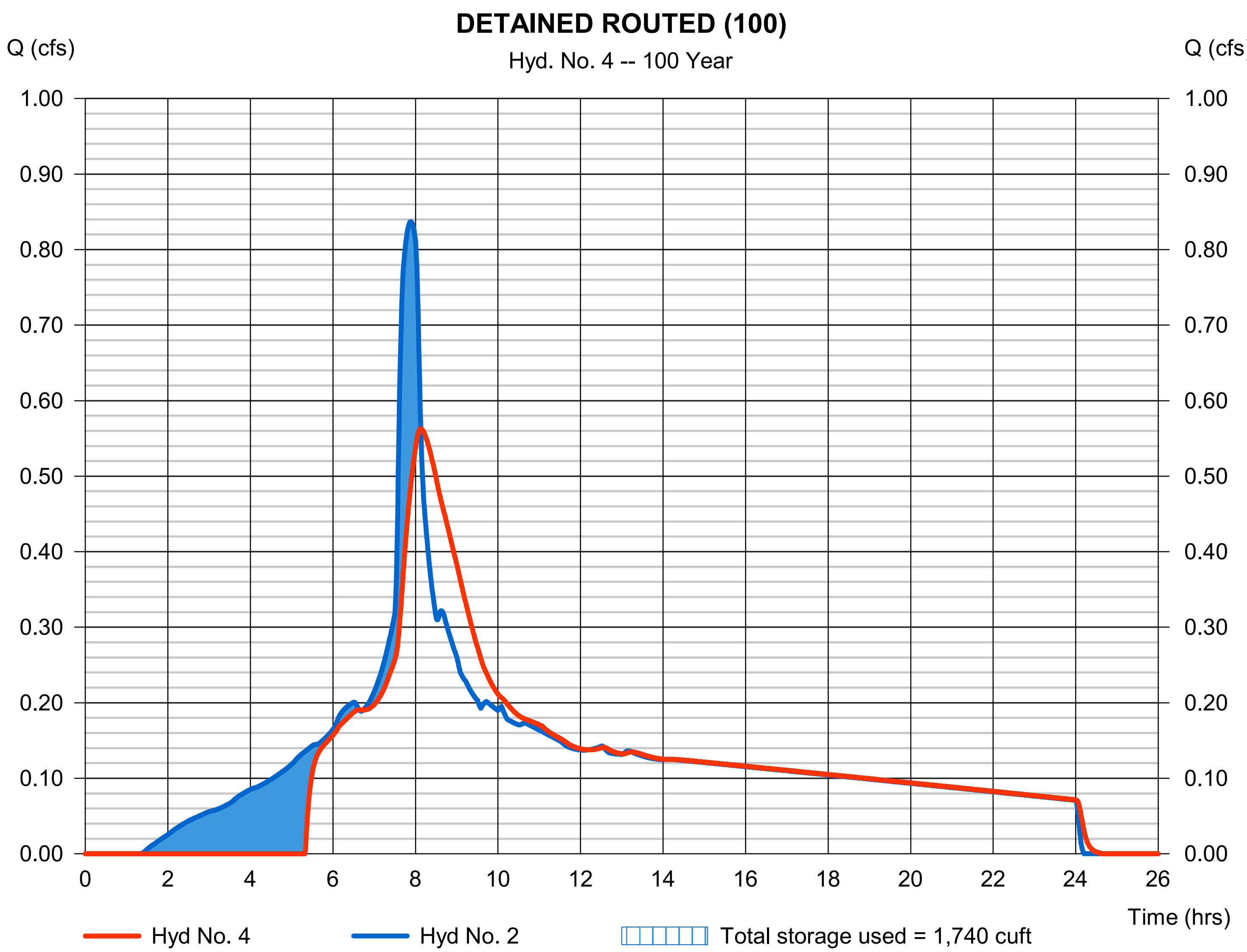
Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2020 Thursday, 07 / 6 / 2023

Hyd. No. 4

DETAINED ROUTED (100)

Hydrograph type	= Reservoir	Peak discharge	= 0.563 cfs
Storm frequency	= 100 yrs	Time to peak	= 8.13 hrs
Time interval	= 1 min	Hyd. volume	= 10,670 cuft
Inflow hyd. No.	= 2 - Proposed Pre-Routed 100-yr W atershed 2	Max. Elevation	= 68.91 ft
Reservoir name	= DETENTION BASIN 2	Max. Storage	= 1,740 cuft

Storage Indication method used.



Pond Report

Pond No. 1 - DETENTION BASIN 2

Pond Data

Contours -User-defined contour areas. Conic method used for volume calculation. Begining Elevation = 66.50 ft

Stage / Storage Table

Stage (ft)	Elevation (ft)	Contour area (sqft)	Incr. Storage (cuft)	Total storage (cuft)
0.00	66.50	723	0	0
0.50	67.00	723	361	361
1.50	68.00	723	723	1,084
2.00	68.50	723	361	1,446
2.50	69.00	723	361	1,807
2.80	69.30	723	217	2,024

Culvert / Orifice Structures

	[A]	[B]	[C]	[PrfRsr]
Rise (in)	= 2.00	0.00	0.00	0.00
Span (in)	= 8.00	0.00	0.00	0.00
No. Barrels	= 1	1	1	0
Invert El. (ft)	= 67.00	0.00	0.00	0.00
Length (ft)	= 0.00	0.00	0.00	0.00
Slope (%)	= 0.00	0.00	0.00	n/a
N-Value	= .013	.013	.013	n/a
Orifice Coeff.	= 0.60	0.60	0.60	0.60
Multi-Stage	= n/a	No	No	No

Weir Structures

	[A]	[B]	[C]	[D]
Crest Len (ft)	= 0.00	0.00	0.00	0.00
Crest El. (ft)	= 0.00	0.00	0.00	0.00
Weir Coeff.	= 3.33	3.33	3.33	3.33
Weir Type	= ---	---	---	---
Multi-Stage	= No	No	No	No

Exfil.(in/hr) = 0.000 (by Contour)
TW Elev. (ft) = 67.80

Note: Culvert/Orifice outflows are analyzed under inlet (ic) and outlet (oc) control. Weir risers checked for orifice conditions (ic) and submergence (s).

Stage / Storage / Discharge Table

Stage ft	Storage cuft	Elevation ft	Clv A cfs	Clv B cfs	Clv C cfs	PrfRsr cfs	Wr A cfs	Wr B cfs	Wr C cfs	Wr D cfs	Exfil cfs	User cfs	Total cfs
0.00	0	66.50	0.00	---	---	---	---	---	---	---	---	---	0.000
0.50	361	67.00	0.00	---	---	---	---	---	---	---	---	---	0.000
1.50	1,084	68.00	0.24 ic	---	---	---	---	---	---	---	---	---	0.239
2.00	1,446	68.50	0.45 ic	---	---	---	---	---	---	---	---	---	0.448
2.50	1,807	69.00	0.59 ic	---	---	---	---	---	---	---	---	---	0.586
2.80	2,024	69.30	0.66 ic	---	---	---	---	---	---	---	---	---	0.655

Hydraflow Rainfall Report

Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2020 Thursday, 07 / 6 / 2023

Return Period (Yrs)	Intensity-Duration-Frequency Equation Coefficients (FHA)			
	B	D	E	(N/A)
1	0.0000	0.0000	0.0000	--------
2	5.1978	2.3000	0.5349	--------
3	0.0000	0.0000	0.0000	--------
5	0.0000	0.0000	0.0000	--------
10	12.1604	5.1000	0.6055	--------
25	16.1806	5.9000	0.6293	--------
50	0.0000	0.0000	0.0000	--------
100	22.1077	7.4000	0.6487	--------

File name: NAPA.IDF

Intensity = B / (Tc + D)^E

Return Period (Yrs)	Intensity Values (in/hr)											
	5 min	10	15	20	25	30	35	40	45	50	55	60
1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2	1.79	1.36	1.13	0.99	0.89	0.81	0.75	0.70	0.66	0.63	0.60	0.57
3	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
5	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
10	3.00	2.35	1.98	1.73	1.55	1.41	1.30	1.21	1.14	1.07	1.02	0.97
25	3.60	2.84	2.39	2.09	1.87	1.70	1.57	1.46	1.36	1.29	1.22	1.16
50	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
100	4.32	3.47	2.94	2.58	2.32	2.11	1.94	1.81	1.70	1.60	1.51	1.44

Tc = time in minutes. Values may exceed 60.

Precip. file name: NAPA.pcp

Storm Distribution	Rainfall Precipitation Table (in)							
	1-yr	2-yr	3-yr	5-yr	10-yr	25-yr	50-yr	100-yr
SCS 24-hour	0.00	2.45	0.00	3.44	4.12	4.95	5.56	6.14
SCS 6-Hr	0.00	1.50	0.00	2.12	2.53	3.03	3.40	3.76
Huff-1st	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Huff-2nd	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Huff-3rd	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Huff-4th	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Huff-Indy	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Custom	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

Hydraflow Table of Contents

DB2-100yrDetention Minus BRF Tc.gpw

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An ADEC Innovation

F.3 - Hydraulic Analysis

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October 26, 2023

TO: Derek Dittman, P.E.
RSA+ Civil Engineers

PROFESSIONAL GEOLOGIST • JEREMY KOBOR • No. 9501 • STATE OF CALIFORNIA

FROM: ______________________________
Jeremy Kobor, Senior Hydrologist, PG #9501

Matt O'Connor, President, CEG #2449
O'Connor Environmental, Inc.

SUBJECT: Hydraulic Analysis of the Proposed Zinfandel Estate Subdivision

Introduction

This document supersedes the previous hydraulic analysis submitted for the project dated May 4, 2021. Detailed in-channel water surface elevation results are presented in Appendix A and responses to review comments by River Focus Water Resources Consultants received June 9, 2021, are presented in Appendix B.

RSA+ has developed several alternative design concepts for the proposed Zinfandel Estate Subdivision adjacent to Salvador Creek on APNs 038-361-009 & 038-361-010. The designs involve elevating portions of the subject properties above 100-yr flood elevations along with measures to mitigate against potential increases in flooding associated with the loss of floodplain. A MIKE FLOOD hydraulic model of the creek and floodplains was developed by DHI, Inc. and the Napa County Resource Conservation District in 2008. This model served as the basis for developing 100-yr Base Flood Elevations (BFEs) and FEMA Flood Insurance Rate Maps (FIRMs) in 2010. The 2010 BFEs and FIRMs were later revised with an effective date of February 20th, 2012 and they provide the basis for defining pre-project hydraulic conditions. The existing model was used to evaluate two alternative design concepts leading to selection of a preferred design alternative. A geomorphic assessment was also performed to evaluate the stability and likely maintenance requirements associated with the preferred design; the geomorphic assessment is presented in a separate technical memorandum. Given that the proposed preferred alternative presented here is very similar to the original preferred alternative evaluated in the May 4, 2021 geomorphic assessment, this document was not revised.

Limitations

The modeling analysis is based on an existing hydraulic model originally developed in 2008. This model uses channel cross sections that were surveyed in 2002 and 2005 and LiDAR data for the floodplain that was collected in 2002. The existing and proposed topography in the model outside of the proposed work area was retained since it was used to define the existing FEMA regulatory floodplain and thus serves as the baseline for evaluating proposed project effects. It is noted that these topographic data sources are relatively old and that a higher-resolution 2018 LiDAR dataset is now available which could be used for a revised study of the system in the future (a task beyond the scope of this parcel-specific project). In the absence of a revised study, it is important to acknowledge the uncertainty associated with the analysis due to the age of the off-site topographic information and potential changes in channel morphology or floodplain development that may have occurred since 2002. For example, field observations in early 2021 revealed the presence of a beaver dam at Lassen Street and evidence of beaver activity downstream of Lassen Street.

Design Alternatives

Both designs include removal of the Biale footbridge and retention/upgrades to the south bank pathway/access road in addition to the features discussed below.

<u>Alternative 1</u>: This design elevates most of the floodplain above the 100-yr BFE and allows for shallow street flooding on both sides of the creek (Figure 1).

<u>Alternative 2</u>: This design reduces the area of elevated floodplain and creates ~830 lineal feet of terracing on the north bank and ~450 lineal feet on the south bank. Street flooding is excluded on the north bank but retained on the south bank (Figure 2).

Figure 1: Design plans provided by RSA+ for Alternative 1.

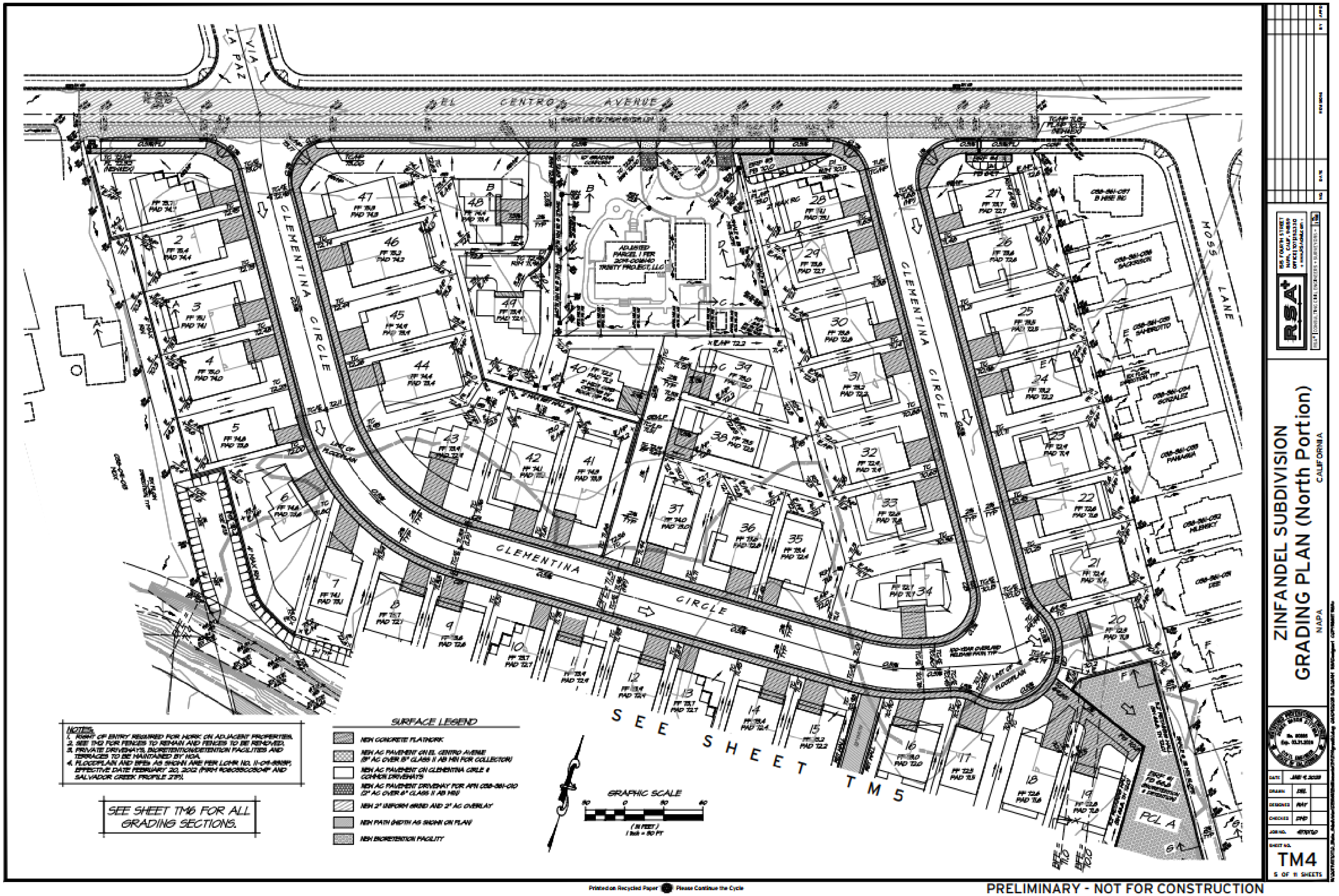


Figure 2a: Design plans provided by RSA+ for Alternative 2 (north section).

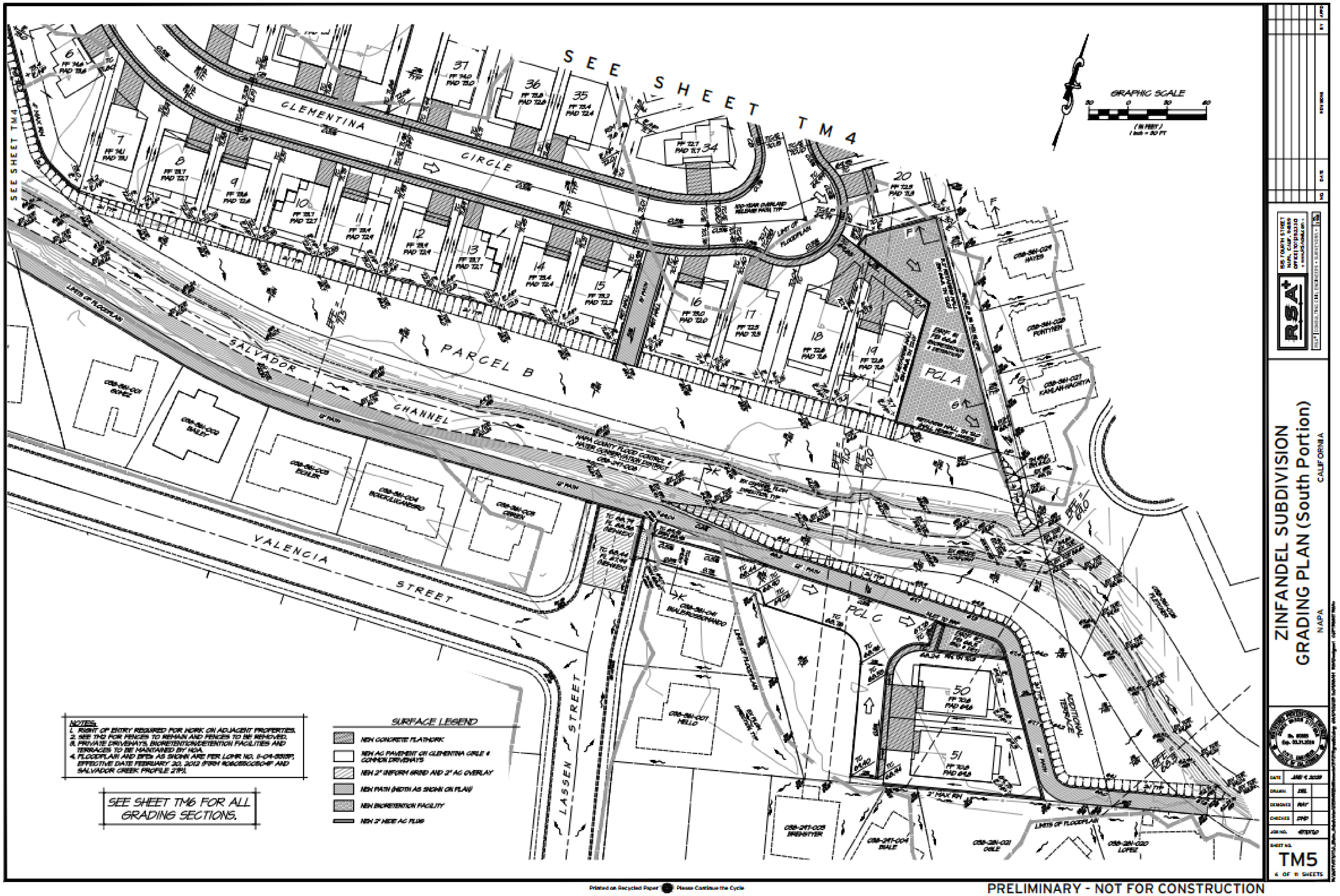


Figure 2b: Design plans provided by RSA+ for Alternative 2 (south section).

Hydrologic Model Development

Hydrologic models for the December 2005 flood and the 100-yr flood were developed previously as part of the 2008 flood study. To enable evaluation of the proposed design alternatives over a wider range of flow conditions, new hydrologic models were developed for the 2- and 10-yr floods using the hydrologic model parameters developed previously for the other flood events in combination with NOAA ATLAS 14 24-hr duration rainfall depths and an SCS Type 1A distribution (same procedure used previously to simulate the 100-yr flood). Resulting peak flows contributing to the project reach range from 1,277 cfs during the 2-yr event to 3,934 cfs for the 100-yr flood (Figure 3).

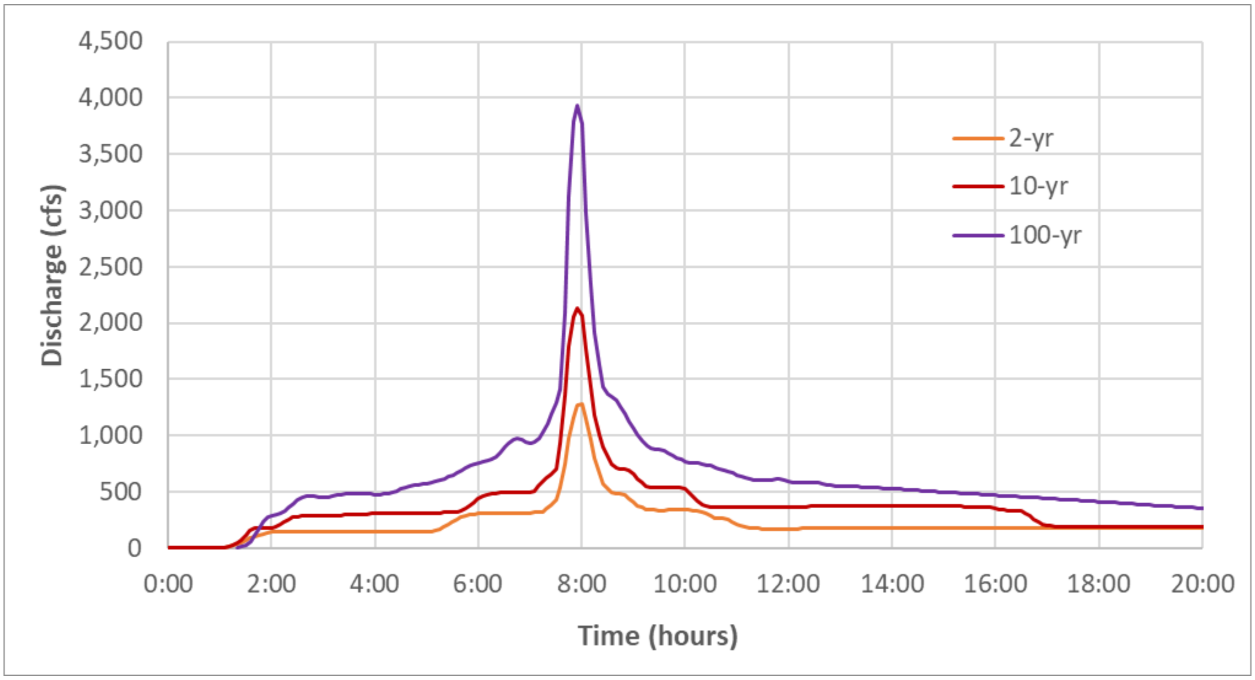


Figure 3: Simulated runoff hydrographs contributing to the project reach of Salvador Creek for the 2-, 10- and 100-yr flood events.

Hydraulic Model Development

Topographic surfaces representing the finished grade of the proposed design alternatives (Figures 4 & 5) were interpolated from points representing proposed elevations at the project site provided by RSA+. Each alternative includes placement of fill to elevate building sites above the 100-yr floodplain. Excluding the building footprints and proposed roads, finished grades in the development footprint are generally between 1- and 2-ft above existing grade on the north side of the creek and between 2- and 3-ft above existing grade on the south side of the creek. In Alternative 1, the proposed roads on both sides of the creek are designed to be below 100-yr floodplain elevations to allow for some shallow street flooding to help to mitigate offsite impacts. This feature was retained on the south side of the creek in Alternative 2, however roads were not lowered on the north side of the creek with the entirety of the building envelope on the north side of the creek excluded from the 100-yr floodplain.

In addition to elevating the site, Alternative 2 includes a 50- to 70-ft wide 830-ft long terrace along the north bank of the creek and a 15- to 75-ft wide 450-ft long terrace along the south bank of the creek. The terracing along the south bank extends downstream of the primary project parcels by ~110-ft to include portions of APNs 038-361-026 and -027. Terrace elevations were set based on a field determination of Ordinary High Water performed by RSA+. In most locations, the terrace elevations range from 2 to 4-ft below existing grade. The design also includes the removal of the bridge near the downstream edge of the project area which was referred to as the Biale Bridge in the existing 2008 flood study. Note that the proposed topography on the north side of the creek for Alternative 2 does not include the details of proposed elevated building pads.

To implement the proposed project in the model, the floodplain elevations in the 2-dimensional component of the model were modified by replacing the existing elevations with proposed elevations within the project site. The terraces were also included in the 2-dimensional component of the model. Cross section bank elevations within the 1-dimensional component of the model (which represents the channels) were lowered to correspond with the proposed terrace grades and several new cross sections were interpolated from adjacent cross sections to more accurately capture the transitions between terraced and unterraced reaches. To accommodate the new positions of the top of banks associated with the terrace design, the locations of the transition from 1- to 2-dimensional flow (known as lateral links) were adjusted accordingly. The Biale bridge was also removed from the model.

The existing condition model was re-run with the inclusion of the new interpolated cross sections to ensure that the comparisons between existing and proposed conditions reflect only the proposed changes to the creek and floodplain and not the changes resulting from the change in cross section density in the model. The changes in water surface elevations and inundation extents resulting from the cross section additions were very minor. The original existing conditions model was calibrated using a uniform floodplain roughness of 0.033 and channel roughness that varied in the project reach from 0.06 to 0.10. The terraces will be re-vegetated with native grasses, shrubs, and trees. We selected a roughness value of 0.08 for the terraces to reflect conditions following the establishment and maturation of the new vegetation. No changes are proposed within the active channel below Ordinary High Water therefore the existing in-channel roughness values were retained.

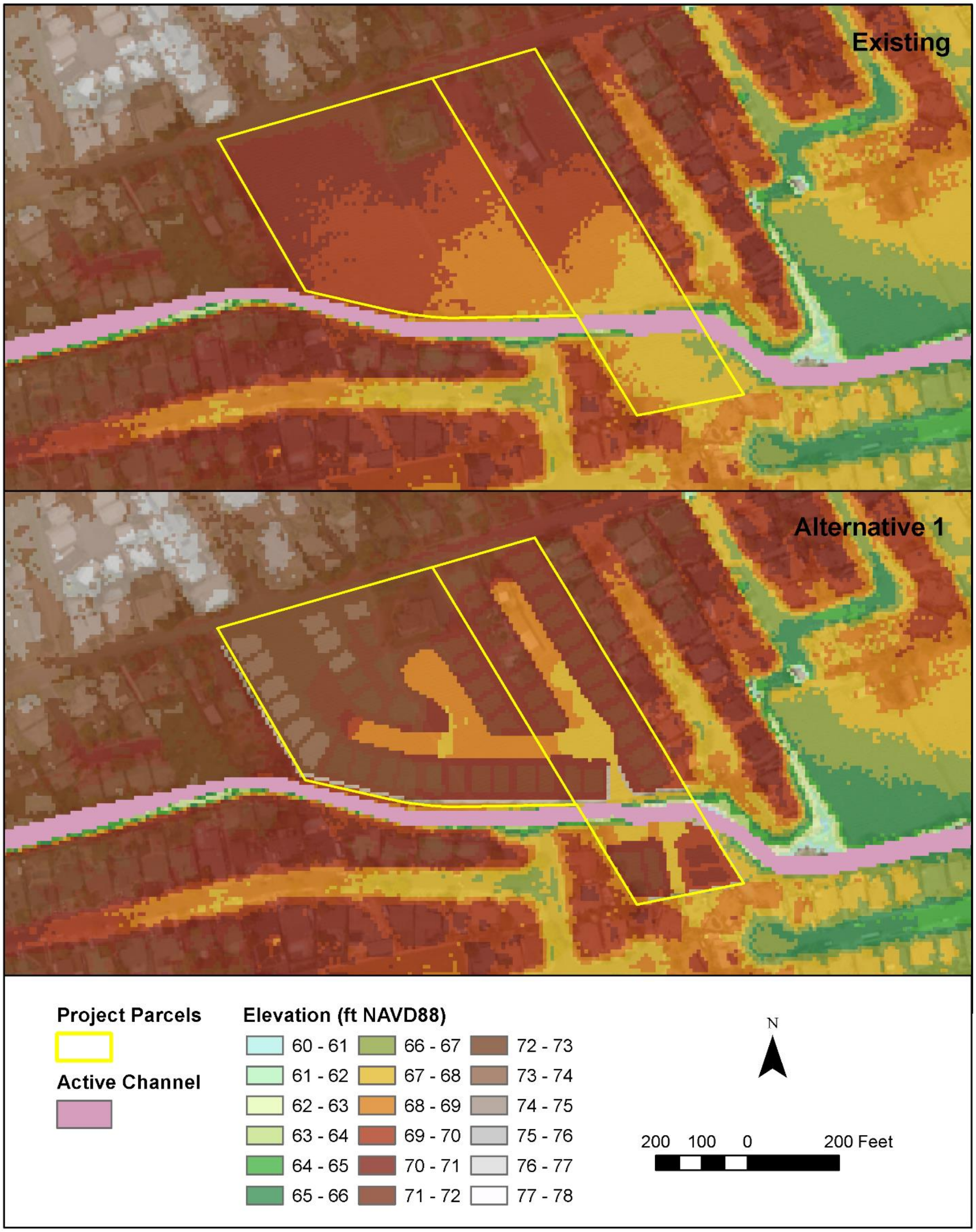


Figure 4: Comparison of existing and proposed topography used in the hydraulic model for Alternative 1.

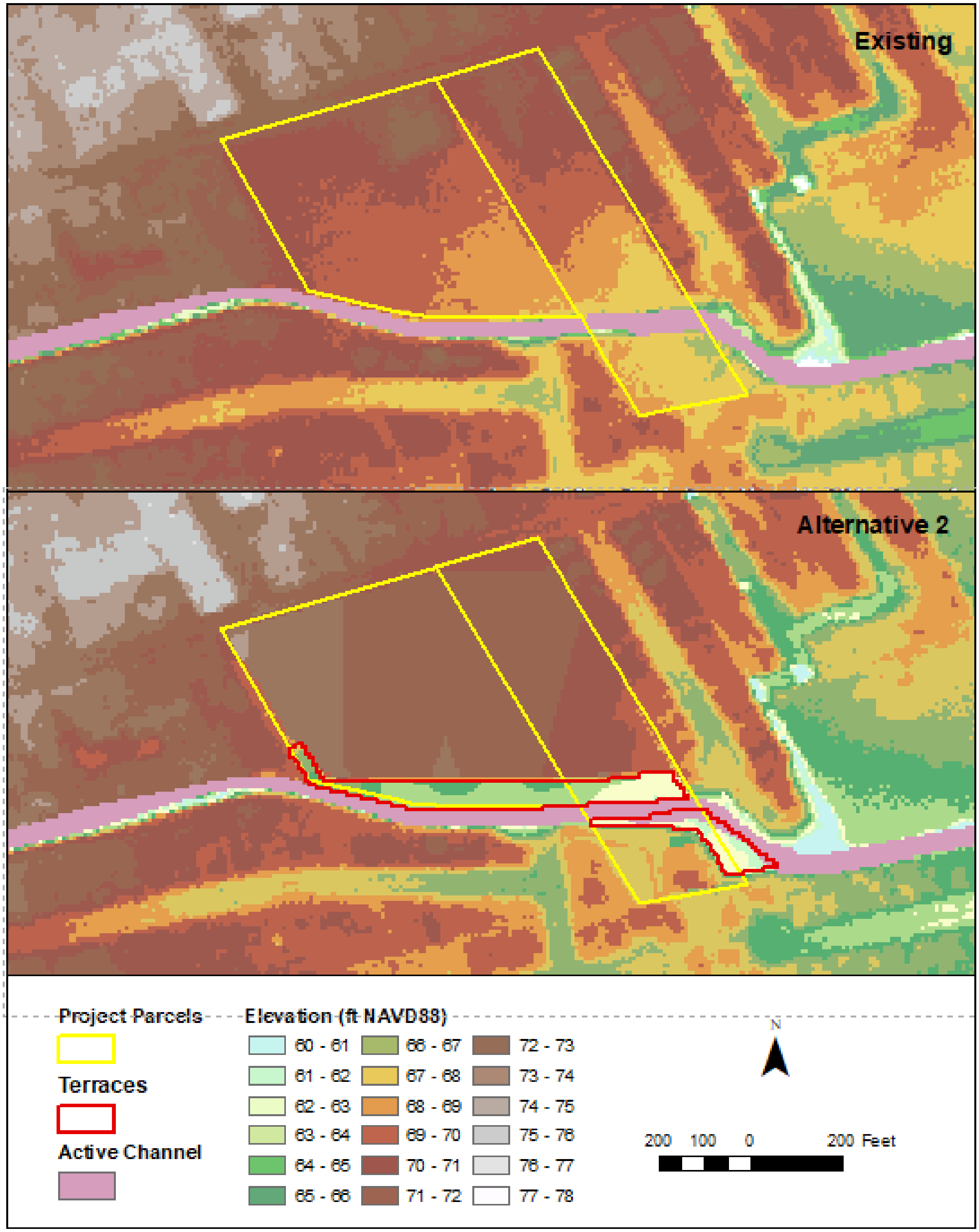


Figure 5: Comparison of existing and proposed topography used in the hydraulic model for Alternative 2. Note proposed grades do not include street and building pad details on the north side of the creek.

Results

Preface

The combination of multiple alternatives, flood events, and various ways to examine the model outputs results in the generation of a very large number of datasets. Results have been summarized for all alternative/event combinations and figures have been included for select combinations that serve to best illustrate the findings of the modeling analysis without overwhelming the reader with information.

Alternative 1

Comparison of maximum water surface elevation (WSE) profiles between the updated existing and proposed Alternative 1 conditions reveals that Alternative 1 produces increases in WSEs upstream of Lassen Street during each of the flow events (Figure 6). The maximum increases range from 0.45 to 0.64-ft (Table 1). This can be attributed to the decrease in floodplain area on the north bank which results in more water remaining in-channel through this reach. Comparison of maximum floodplain inundation extents and depths for the 2- and 100-yr floods shows how the design substantially increases flood extents on the south side of the creek throughout the range of evaluated flows (Figures 7 & 8).

In contrast, the design results in decreases in WSEs downstream of Lassen Street due to the increased overbank flows at Lassen Street which result in less water remaining in-channel to be routed to the downstream reaches (Figure 6). This effect manifests with modest reductions in flood extent downstream of the project parcels on the north side the creek (Figures 7 & 8). Overall, the mean change in WSE over the project reach plus 1,000-ft upstream and downstream is a small decrease of between 0.04 and 0.11-ft (Table 1).

Table 1: Summary of changes in channel water surface elevations for Alternative 1.

		2-yr	10-yr	100-yr
Change in WSE (ft)	Min	-0.33	-0.54	-0.37
	Max	0.45	0.52	0.64
	Mean	-0.05	-0.11	-0.04

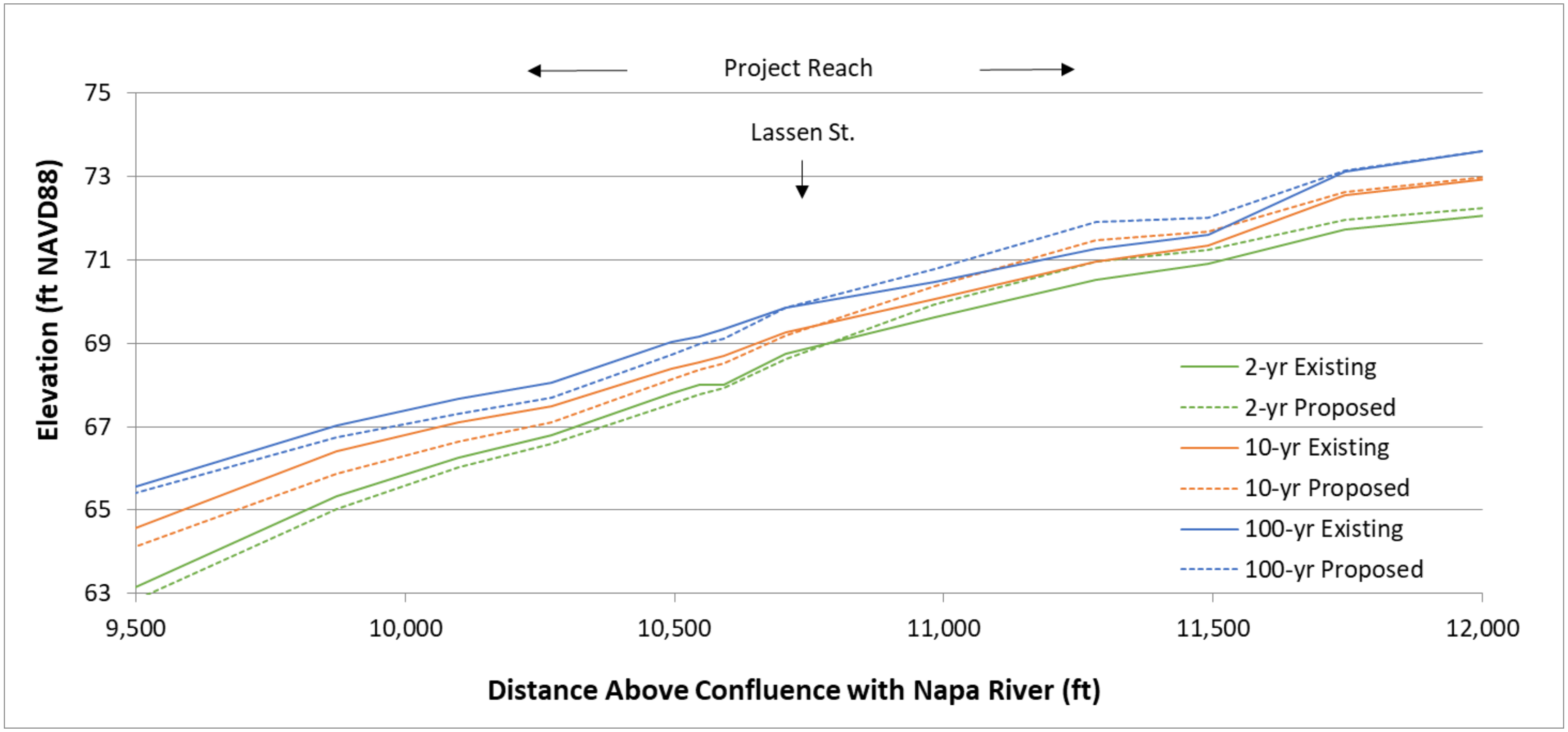


Figure 6: Comparison of existing and proposed water surface profiles for Alternative 1.

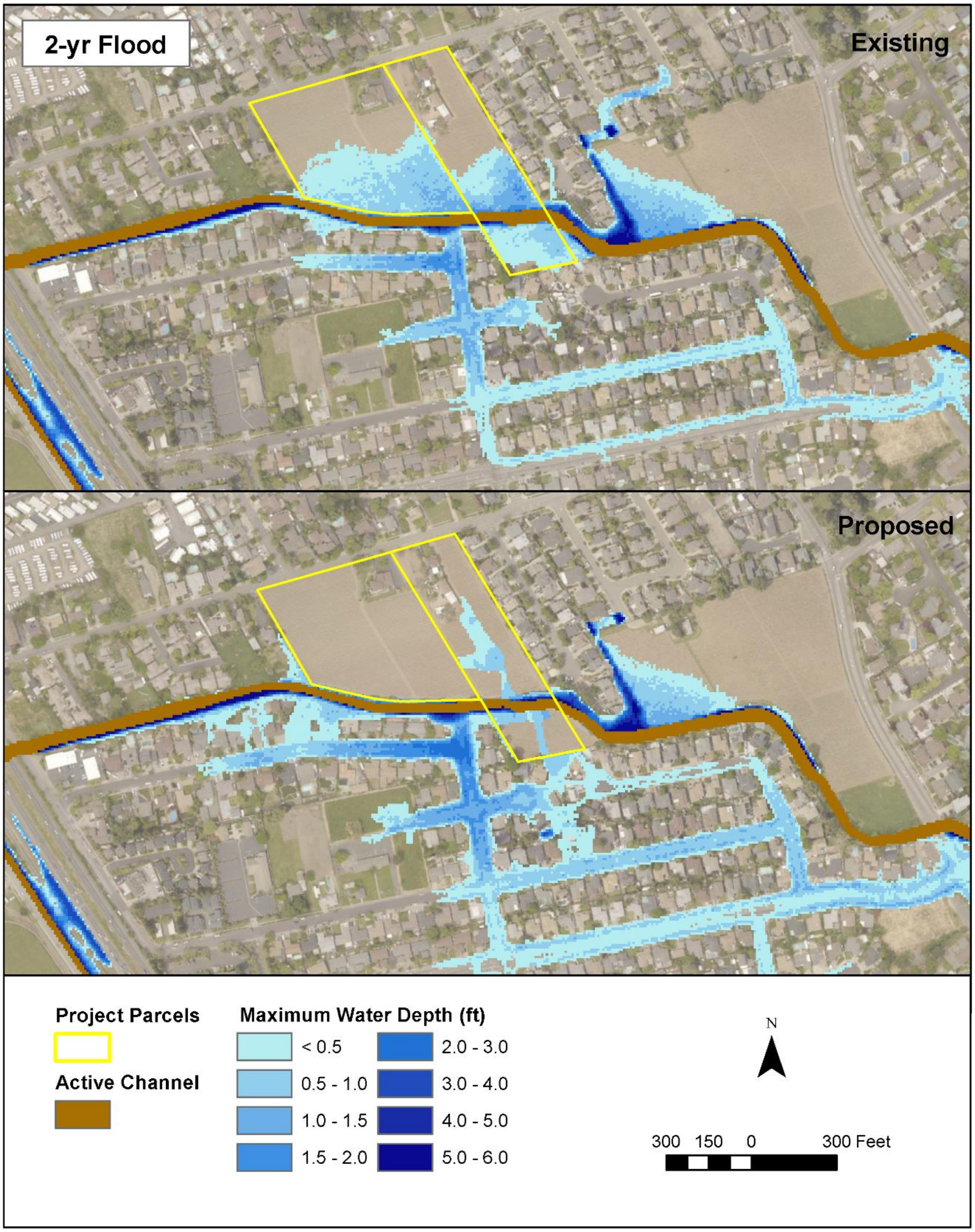


Figure 7: Comparison of existing and proposed 2-yr water depths and inundation extents for Alternative 1.

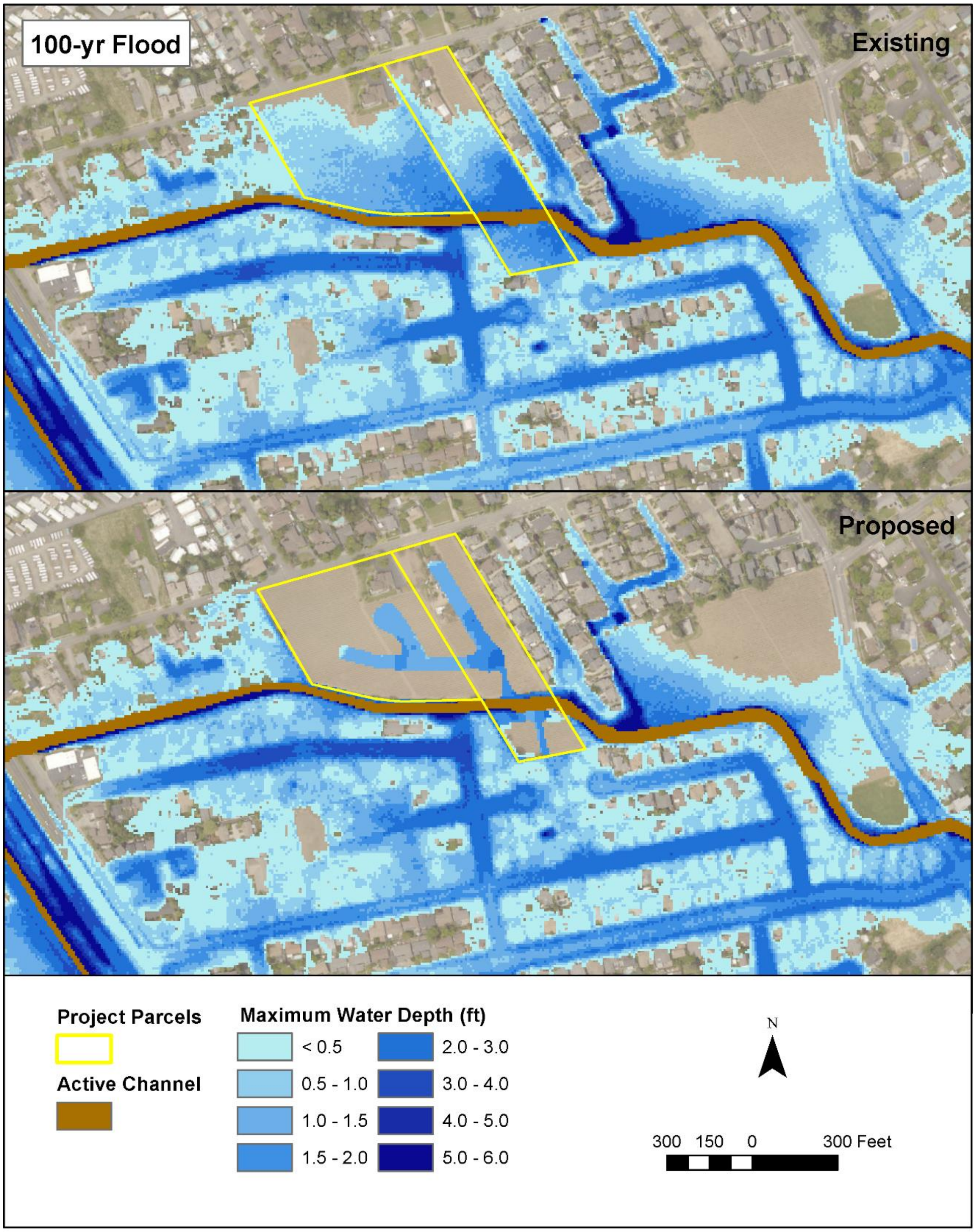


Figure 8: Comparison of existing and proposed 100-yr water depths and inundation extents for Alternative 1.

<u>Alternative 2</u>

Comparison of maximum WSE profiles between the updated existing and Alternative 2 conditions reveals that the design results in decreases in WSEs upstream of Lassen Street during the 2-yr and to a lesser extent during the 10-yr event, with maximum decreases of 0.57-ft during the 2-yr flood (Figure 9; Table 2). This can be attributed to the increase in channel capacity due to the addition of the terraces which, at smaller flood flows, is more than enough to compensate for the loss of floodplain associated with the proposed fill in the development footprint. The reduced WSEs result in substantially reduced overbank flows at Lassen Street and associated reductions in flood extents and depths south of the creek during the 2-yr flood and to a lesser extent during the 10- and 100-yr floods (Figures 10-12). This effect diminishes with increasing flow as changes on the floodplain become more important relative to changes in channel capacity. Except for the area along the edge of the fill prism immediately upstream of the project reach where local increases in depth of up to 0.8-ft occur during the 10- and 100-yr floods, increases in inundation extent and flood depth upstream of Lassen Street are minor (Figures 11-13). As discussed in greater detail in Appendix B, these increases do not affect structure flooding.

Downstream of Lassen Street, Alternative 2 results in increases in WSEs due to reduced overbank flows and inundation at Lassen Street (Figure 9). These overbank flow decreases result in more water remaining in-channel to be routed to downstream reaches resulting in a maximum increase in WSE of 0.28-ft during the 2-yr flood (Table 2). This effect diminishes with increasing flow, and the maximum increase in WSE during the 100-yr flood is 0.10-ft (Table 2). Increased flow in the downstream reaches results in small increases in inundation extents and depths in the vicinity of Bryce Court, along the high flow path on the north bank through the downstream Biale vineyard and crossing Jefferson Street north of Trower, and farther downstream at Vintage High School. These changes are relatively minor and primarily represent less than a 0.1-ft increase in inundation depth, except in low-lying portions of the street networks (Figures 11-13). Overall, the mean change in WSE over the project reach plus 1,000-ft upstream and downstream is near zero during all flood events (Table 2).

Table 2: Summary of changes in channel water surface elevations for Alternative 2.

		2-yr	**10-yr**	**100-yr**
Change in WSE (ft)	Min	-0.57	-0.27	-0.13
	Max	0.28	0.14	0.10
	Mean	-0.03	-0.02	-0.01

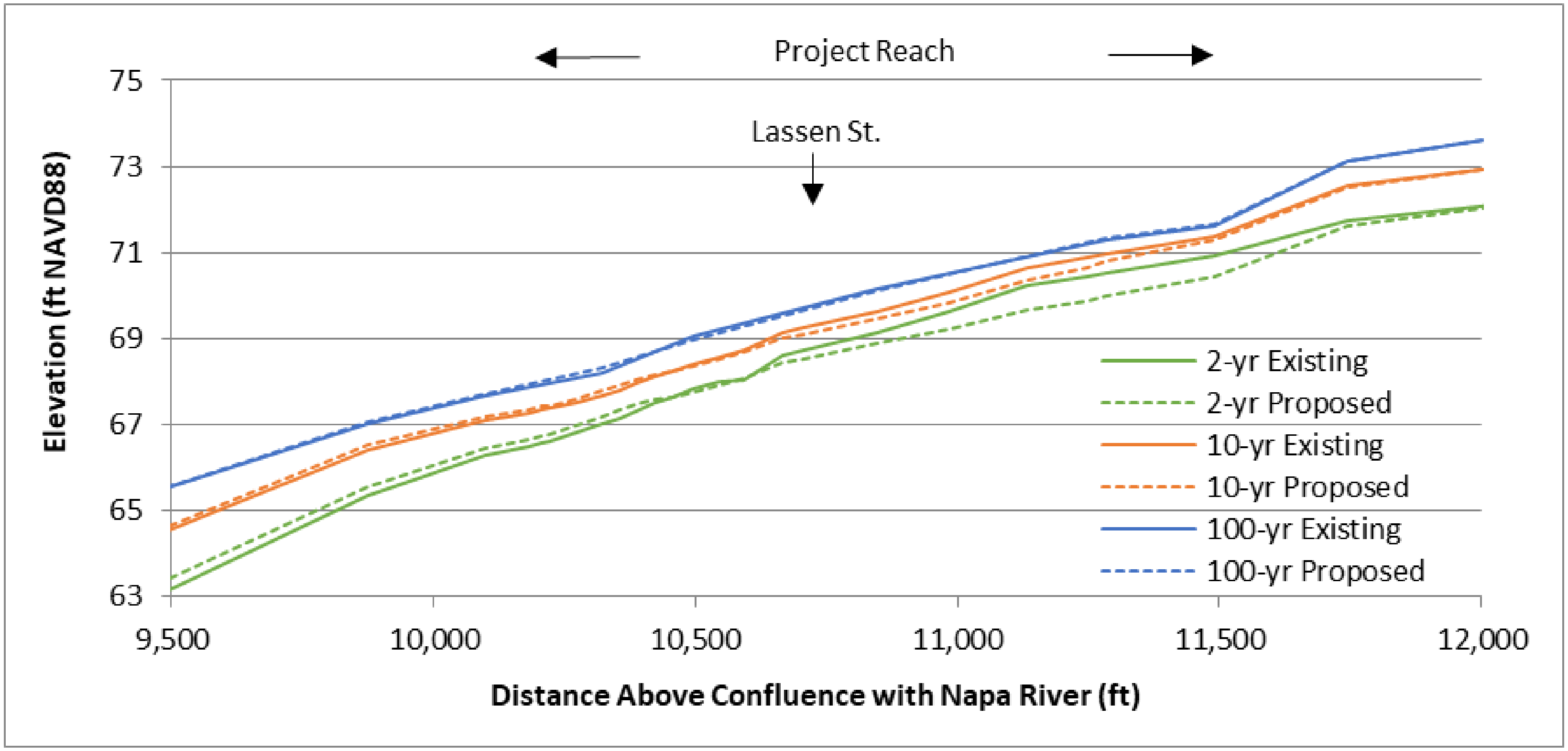


Figure 9: Comparison of existing and proposed water surface profiles for Alternative 2.

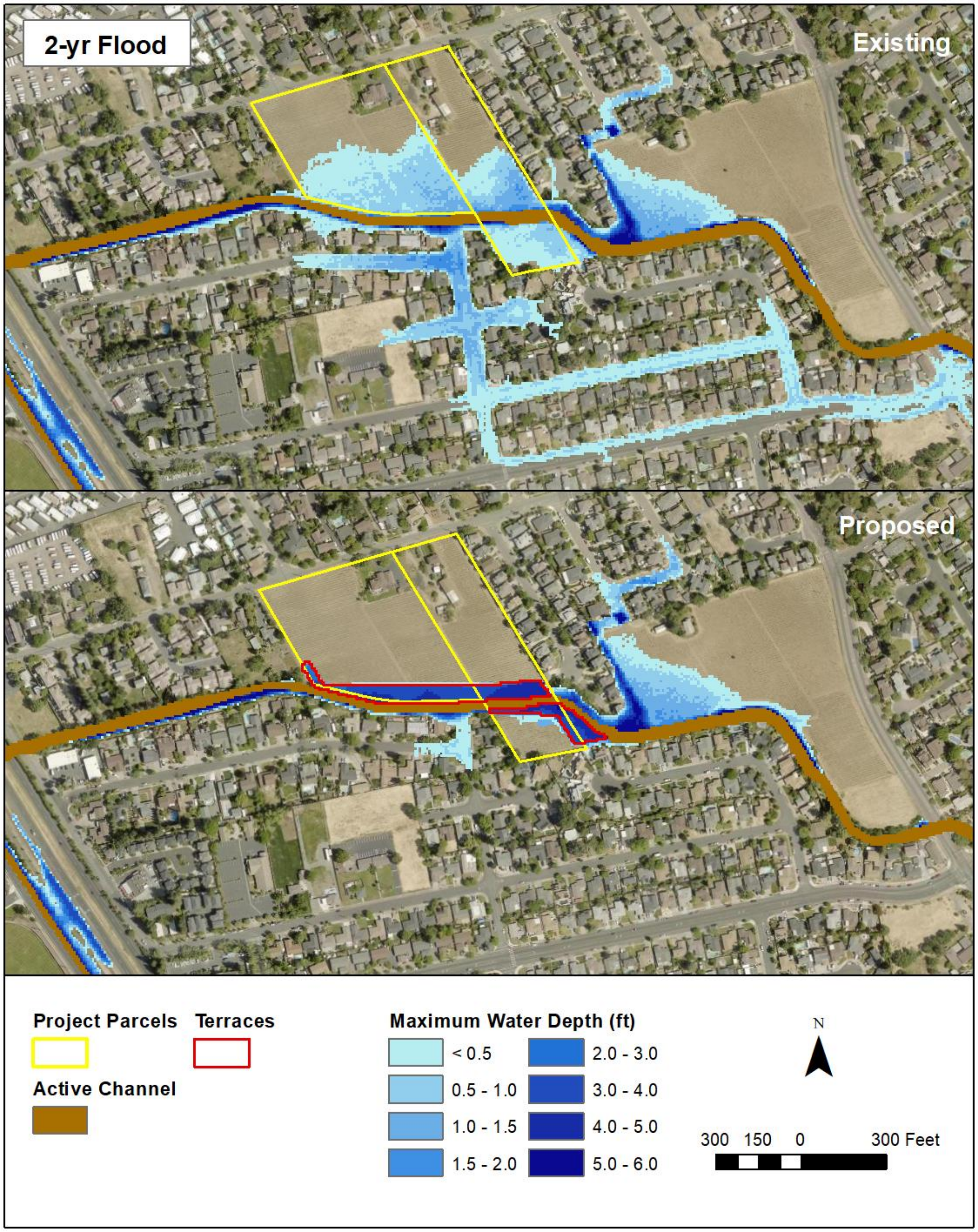


Figure 10: Comparison of existing and proposed 2-yr water depths and inundation extents for Alternative 2.

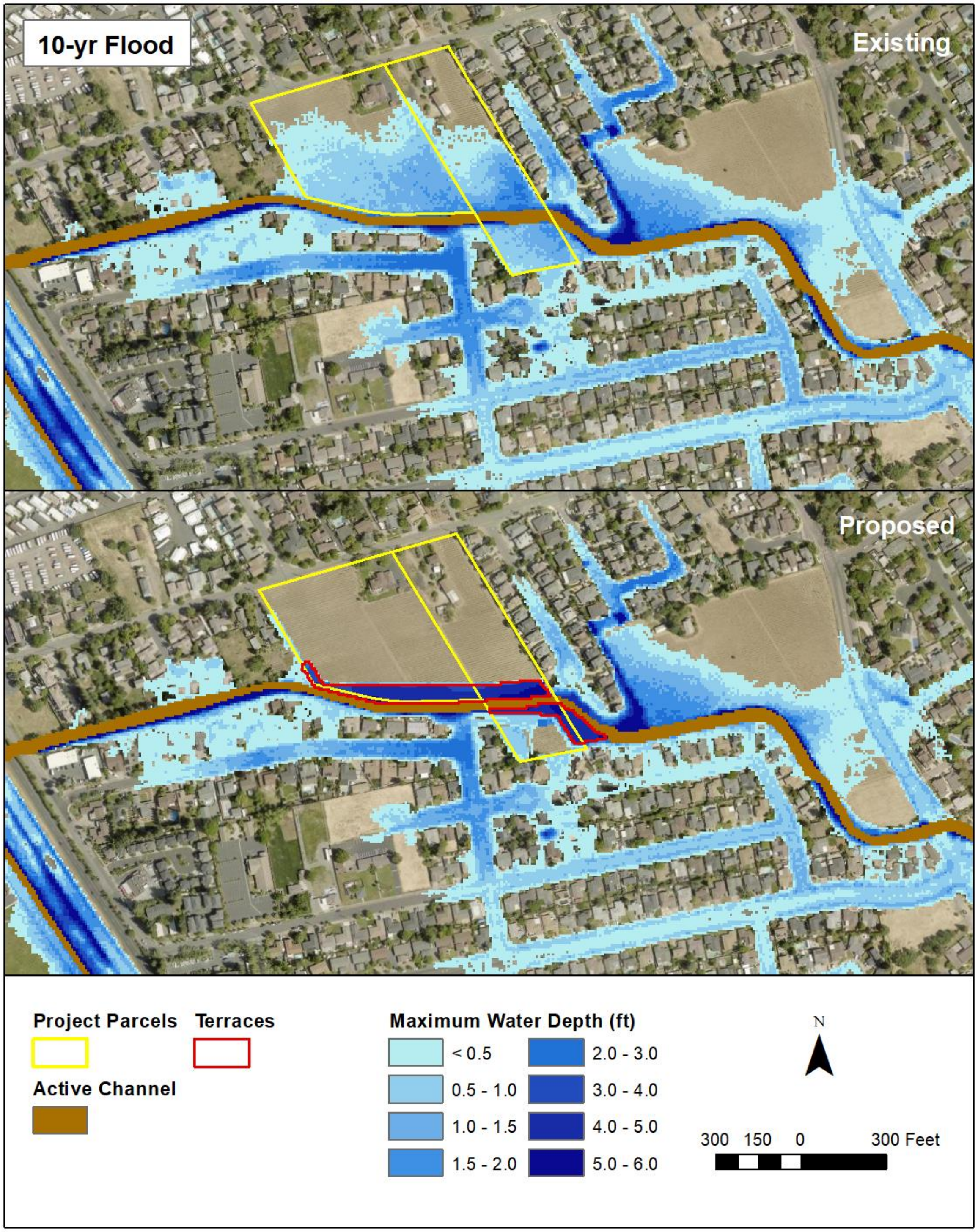


Figure 11: Comparison of existing and proposed 10-yr water depths and inundation extents for Alternative 2.

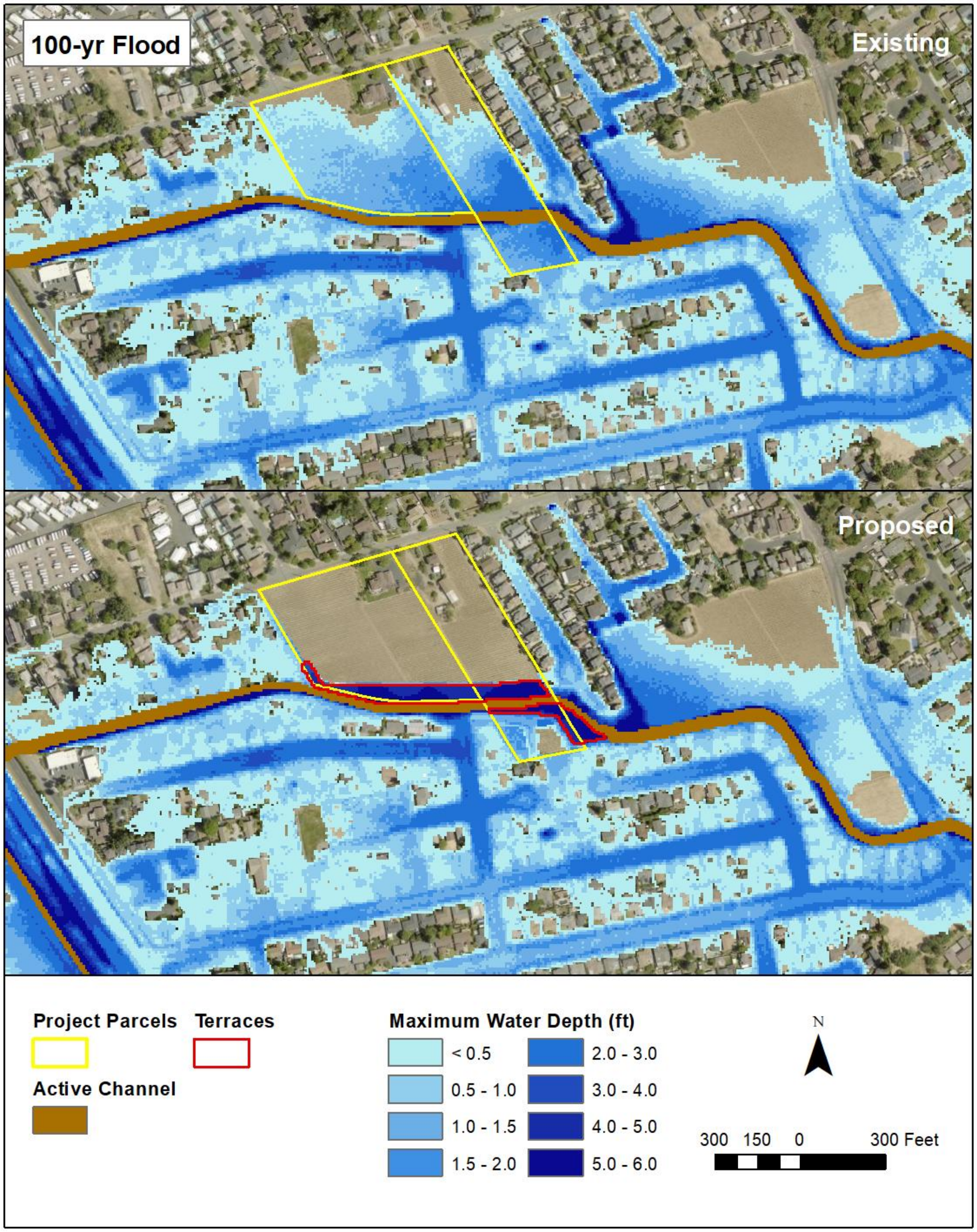


Figure 12: Comparison of existing and proposed 100-yr water depths and inundation extents for Alternative 2.

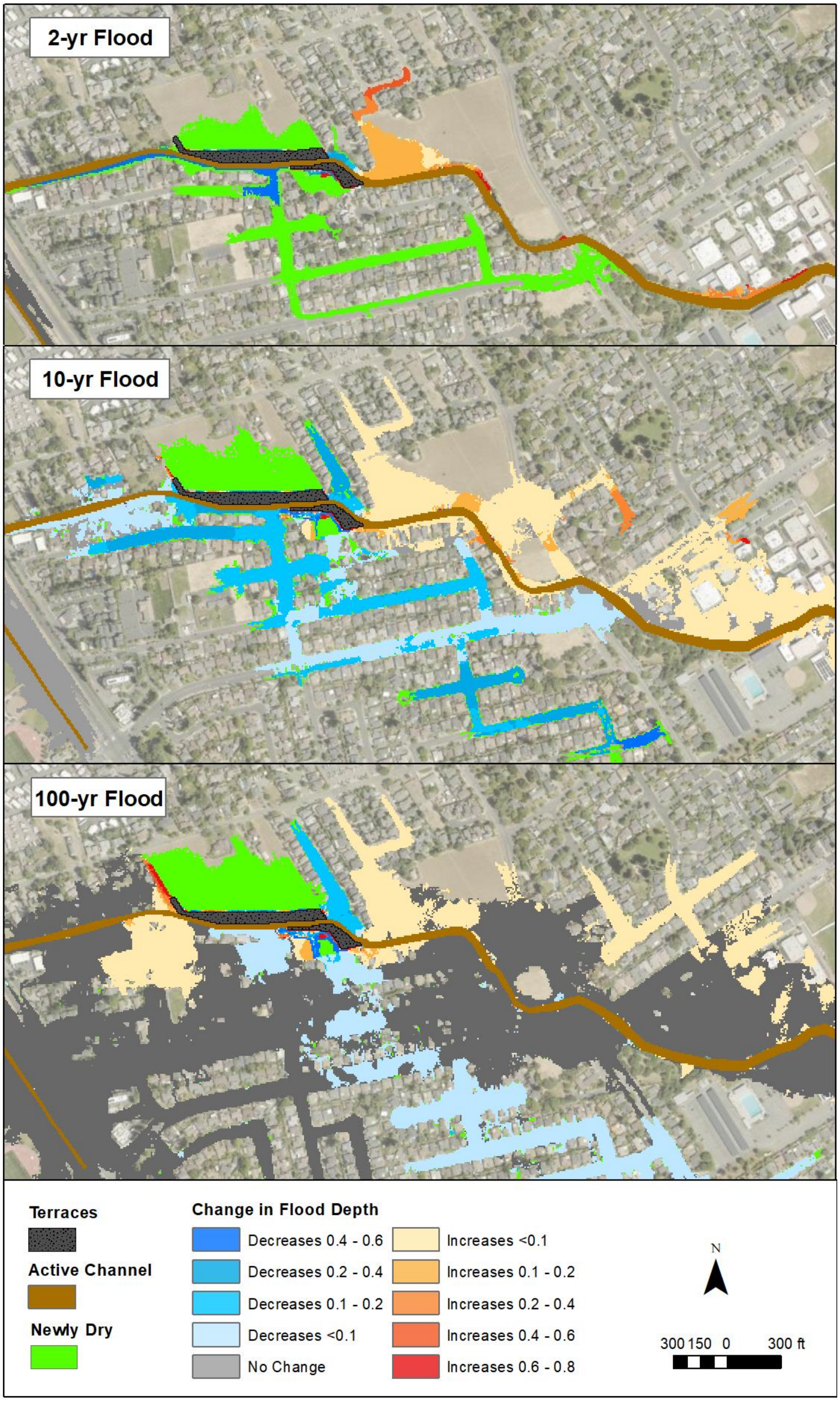


Figure 13: Change in 2-, 10- and 100-yr water depths and inundation extents for Alternative 2 relative to existing conditions.

Discussion & Selection of Preferred Alterative

The modeling analysis reveals several important aspects to the flooding situation along Salvador Creek. Overall, the analysis shows that flooding patterns are highly sensitive to changes in grade or WSEs that affect overbank flows at Lassen Street. The results indicate that these overbank flows are providing some important flood attenuation benefits to the reaches farther downstream, and that any measures that reduce overbank flows and inundation at Lassen Street will be accompanied by increases in WSEs and inundation extents downstream (or vice versa). This is because the channel downstream does not have sufficient capacity to contain increases in flow generated from reduced overbank flow at Lassen. This suggests that in the absence of a comprehensive flood mitigation strategy for the creek that addresses capacity limitations both upstream and downstream of the project reach, this site-specific project should seek to avoid significant reductions in overbank flows at Lassen Street to prevent downstream impacts.

Results for Alternative 1 indicate that filling the development footprint as a stand-alone measure results in significant increases in WSEs and inundation upstream of the project and in the Lassen Street neighborhood. Results for Alternative 2 indicate that pairing the fill with terracing can successfully mitigate these increases. As discussed above, flooding patterns are very sensitive to changes in overbank flows at Lassen Street, and the design does result in some reduction in overbank flows and inundation in the Lassen neighborhood which in turn results in minor increases in flows and inundation in the downstream reaches.

Comparison of the maximum increases in WSEs between the two alternatives reveals that Alternative 2 results in the smallest increases for each of the four simulated events (Tables 1 & 2). Comparison of the mean changes in WSEs reveals that Alternative 1 results in reductions in WSEs for all four events. This result indicates the potential pitfalls of basing decisions on WSE changes alone since the overall reductions are the result of significant increases in flooding in the Lassen neighborhood and the associated reductions in flows downstream. Alternative 2 results in near zero change to the mean WSE for all four events (Table 2).

Alternative 2 has been selected as the preferred alternative because it results in the smallest increases in WSEs for all four events as well as the smallest increases in inundation extents and depths. The degree of terracing represented by the design appears to be near optimal since less terracing would be expected to result in significant increases in WSEs and flooding upstream of the project reach and more terracing would be expected to result in less overbank flow at Lassen Street accompanied by significant increases in flooding downstream.

Though not likely to significantly affect peak riverine flooding or this analysis, it is worth noting that the City of Napa has developed a concept study for stormwater drainage system improvement for the Trower & Lassen area which it is planning to implement. The proposed improvements consist of abandoning five outfalls including one at Lassen Street and re-directing stormwater flows to an existing concrete culvert at Trower and Jefferson. The runoff from the relatively small (compared to total upstream drainage area) drainage areas served by these outfalls is not expected to significantly mitigate peak riverine flooding, however it should help alleviate the frequency of nuisance street flooding during small events and the duration of inundation associated with overtopping at Lassen during larger events.

Erosion and sedimentation considerations associated with Alternative 2, particularly potential sediment deposition on proposed terrace surfaces and potential change of in-channel sediment transport capacity are the subject of a companion geomorphic assessment.

Appendix A - Comparison of water surface elevations in Salvador Creek between existing and proposed preferred alternative (Alternative 2) conditions.

<u>2-yr Flood</u>

		Water Surface Elevation (ft)		
	Station (ft)	Existing	Proposed	Change (ft)
	12503.7	73.91	73.91	0.00
	12480.6	73.89	73.89	0.00
	12328.0	72.33	72.31	-0.02
	12303.7	72.28	72.26	-0.02
	12216.2	72.19	72.17	-0.02
	12025.2	72.09	72.06	-0.03
	11744.5	71.73	71.60	-0.13
Project Reach	11491.6	70.92	70.42	-0.50
Project Reach	11282.1	70.53	69.99	-0.54
Project Reach	11246.0	70.45	69.88	-0.57
Project Reach	11131.4	70.22	69.66	-0.56
Project Reach	10982.9	69.63	69.22	-0.41
Project Reach	10844.7	69.13	68.88	-0.25
Project Reach	10706.5	68.75	68.51	-0.24
Project Reach	10665.0	68.59	68.44	-0.15
Project Reach	10591.9	68.01	68.07	0.06
Project Reach	10545.5	68.01	67.91	-0.10
Project Reach	10494.5	67.81	67.77	-0.05
Project Reach	10456.1	67.64	67.65	0.00
Project Reach	10423.3	67.49	67.58	0.09
Project Reach	10384.9	67.30	67.45	0.15
Project Reach	10355.4	67.15	67.34	0.19
Project Reach	10322.6	67.00	67.18	0.18
Project Reach	10272.1	66.80	66.96	0.15
Project Reach	10220.1	66.63	66.76	0.13
Project Reach	10203.7	66.57	66.72	0.15
	10181.7	66.50	66.66	0.16
	10098.2	66.27	66.44	0.17
	9872.4	65.33	65.55	0.22
	9360.9	62.36	62.64	0.28
	9303.4	62.34	62.62	0.28
	9249.2	61.47	61.74	0.28
	9071.1	61.29	61.57	0.28
	9010.0	61.10	61.38	0.27
	8869.8	60.82	61.10	0.28
	8650.1	60.51	60.78	0.28
	8480.9	58.67	58.89	0.23
	8254.6	58.13	58.35	0.22
	7921.5	57.73	57.94	0.21

10-yr Flood

		Water Surface Elevation (ft)		
	Station (ft)	Existing	Proposed	Change (ft)
	12503.7	75.92	75.92	0.00
	12480.6	75.90	75.90	0.00
	12328.0	73.29	73.27	-0.01
	12303.7	73.22	73.21	-0.01
	12216.2	73.12	73.10	-0.02
	12025.2	72.97	72.96	-0.02
	11744.5	72.55	72.52	-0.03
Project Reach	11491.6	71.35	71.28	-0.07
	11282.1	70.96	70.81	-0.16
	11246.0	70.87	70.64	-0.23
	11131.4	70.63	70.36	-0.27
	10982.9	70.05	69.83	-0.23
	10844.7	69.60	69.46	-0.14
	10706.5	69.27	69.08	-0.18
	10665.0	69.14	69.01	-0.12
	10591.9	68.71	68.67	-0.04
	10545.5	68.55	68.53	-0.02
	10494.5	68.39	68.36	-0.03
	10456.1	68.25	68.23	-0.02
	10423.3	68.13	68.17	0.04
	10384.9	67.97	68.03	0.06
	10355.4	67.78	67.93	0.14
	10322.6	67.66	67.77	0.11
	10272.1	67.49	67.57	0.08
	10220.1	67.37	67.43	0.05
	10203.7	67.33	67.41	0.08
	10181.7	67.28	67.36	0.08
	10098.2	67.10	67.18	0.09
	9872.4	66.41	66.51	0.10
	9360.9	63.89	63.95	0.06
	9303.4	63.88	63.96	0.08
	9249.2	62.85	62.90	0.04
	9071.1	62.71	62.75	0.04
	9010.0	62.52	62.56	0.04
	8869.8	62.27	62.29	0.02
	8650.1	61.85	61.87	0.02
	8480.9	59.66	59.67	0.01
	8254.6	59.11	59.13	0.01
	7921.5	58.69	58.71	0.02

<u>100-yr Flood</u>

		Water Surface Elevation (ft)		
	Station (ft)	Existing	Proposed	Change (ft)
	12503.7	77.57	77.57	0.00
	12480.6	77.56	77.55	0.00
	12328.0	74.24	74.24	0.00
	12303.7	74.12	74.11	-0.01
	12216.2	73.93	73.92	-0.01
	12025.2	73.65	73.66	0.01
	11744.5	73.12	73.12	-0.01
Project Reach	11491.6	71.61	71.66	0.06
Project Reach	11282.1	71.28	71.33	0.06
Project Reach	11246.0	71.17	71.14	-0.03
Project Reach	11131.4	70.95	70.89	-0.06
Project Reach	10982.9	70.48	70.42	-0.06
Project Reach	10844.7	70.14	70.10	-0.04
Project Reach	10706.5	69.86	69.73	-0.13
Project Reach	10665.0	69.73	69.65	-0.08
Project Reach	10591.9	69.34	69.27	-0.07
Project Reach	10545.5	69.17	69.16	0.00
Project Reach	10494.5	69.03	68.98	-0.06
Project Reach	10456.1	68.92	68.84	-0.08
Project Reach	10423.3	68.80	68.78	-0.01
Project Reach	10384.9	68.60	68.62	0.02
Project Reach	10355.4	68.41	68.48	0.07
Project Reach	10322.6	68.21	68.31	0.10
Project Reach	10272.1	68.05	68.08	0.03
Project Reach	10220.1	67.96	67.94	-0.01
Project Reach	10203.7	67.92	67.93	0.01
	10181.7	67.86	67.88	0.02
	10098.2	67.69	67.70	0.01
	9872.4	67.03	67.05	0.02
	9360.9	65.02	65.03	0.01
	9303.4	65.02	65.03	0.01
	9249.2	64.20	64.21	0.01
	9071.1	64.11	64.09	-0.01
	9010.0	63.91	63.91	0.00
	8869.8	63.63	63.63	0.00
	8650.1	62.96	62.97	0.00
	8480.9	60.82	60.82	0.00
	8254.6	60.09	60.09	0.00
	7921.5	59.80	59.80	0.00

Appendix B – Response to June 9, 2021 comments received from River Focus Water Resources Consultants.

1. The preferred Alternative 2 causes a 0.2 to 0.4 ft increase in computed 100-year flood elevations on the north (left overbank) floodplain, immediately upstream of the project, as well as on the south (right overbank) floodplain (see Figure 1). These increases appear to be the result of floodplain flows being blocked by the proposed fill (Figure 2). On the north side, the proposed floodplain terracing tapers off at the upstream end of the project to the point where there is no connection between the upstream floodplain flow and the proposed terracing during the 100-year event.

   Note that the terracing was likely tapered back to the channel because this matches the effective FEMA Zone AE (100-year) floodplain in this area. However, on the upstream side of the development, the existing conditions 100-year floodplain computed by the current study is larger than the effective FEMA floodplain.

   To reduce the increase in flood elevations, some of the floodplain conveyance may have to be preserved (for example, see Figure 3). Alternatively, floodplain relief culverts through the fill, and discharging to the terracing, might be an option. Note: All figures in this review document were taken from the *Hydraulic Analysis* tech memo with annotated comments, labels, and arrows added by River Focus.

Several changes to the Alternative 2 design were made to mitigate the effects of floodplain blockage at the upstream extents of the project area on the north and south banks. On the north bank, the terrace was widened at its upstream end as suggested by River Focus. Additionally, the fill prism was set back 15-ft from the property line such that increases in inundation associated with floodplain flows interacting with the fill will largely be contained on the project parcel. We also investigated using relief culverts to mitigate the north bank increases, however this strategy was not as effective as the adopted terrace changes. On the south bank, the extent of the fill prism was reduced by eliminating the western-most lot and instead lowering the grades in this area. The proposed driveway parallel to the channel was also lowered. The changes on the north side resulted in a decreased area where inundation depth increases extend onto the neighboring parcel to the west. Additional topographic survey was collected surrounding the neighbor's house and the existing and proposed 100-yr WSEs were mapped at a finer scale than is possible using the hydraulic model alone. This exercise reveals that although Alternative 2 does result in the WSE in this area increasing from 70.9-71.9 to 71.3-71.9, neither the exiting nor proposed WSEs are high enough to result in inundation that extends to the edges of the house (Figure A1). The changes on the south side resulted in all increases in inundation depth in excess of 0.1 ft being contained on the project parcels thus fully mitigating against significant off-site impacts.

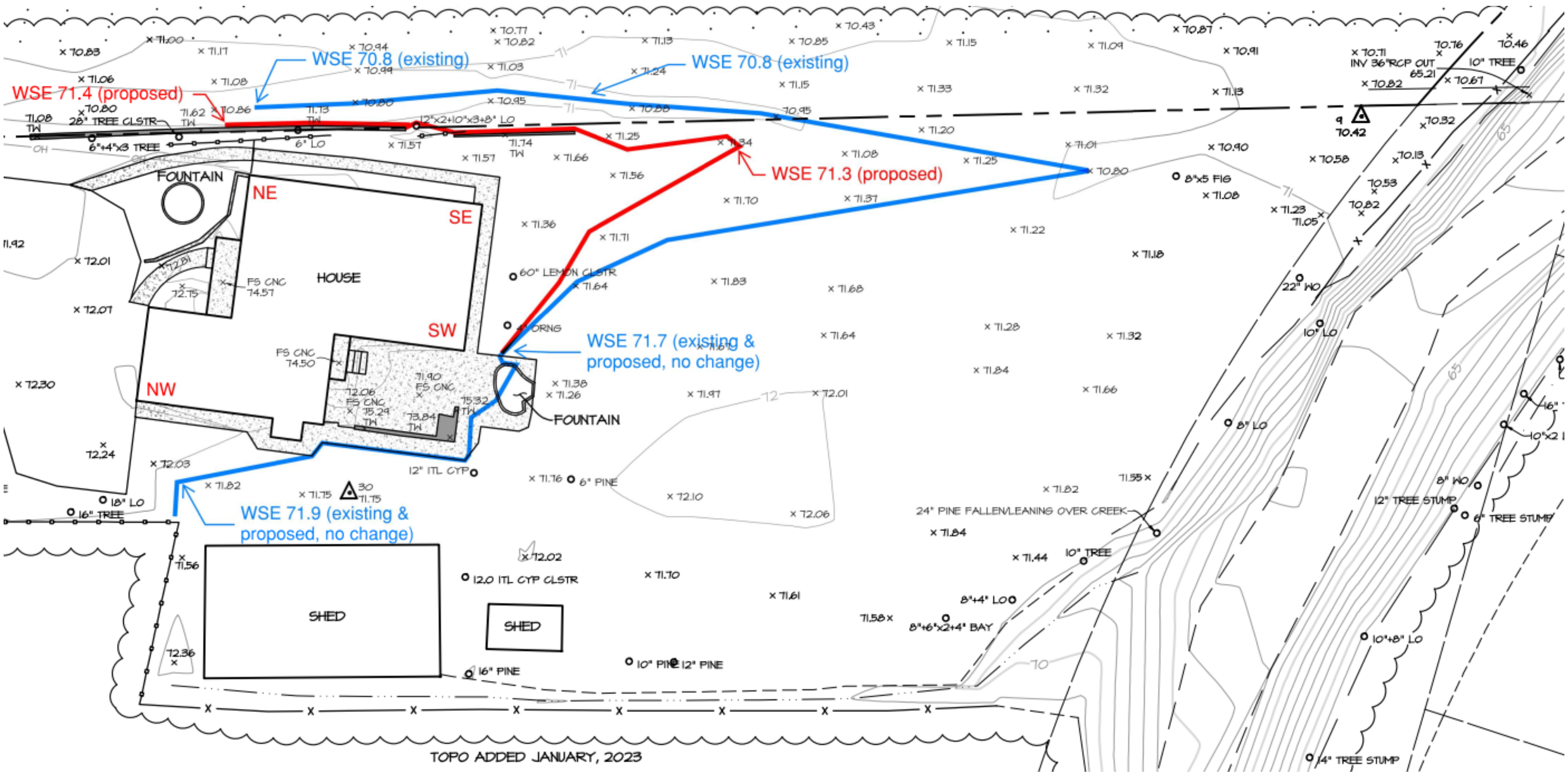


Figure A1: Detailed mapping of existing and proposed WSEs in the vicinity of the house located on the neighboring parcel west of the proposed project on the north side of Salvador Creek.

2. Alternatives 3a and 3b include an elevated pathway (0.5 ft for 3a or 0.9 ft for 3b) to block flow from leaving the channel at Lassen Street. The proposed berm keeps significantly more flow in the channel and causes increased flooding throughout the study reach. Based on a close review of the model results, we believe that Alternative 2 may be a better option for proposed conditions in terms of minimizing offsite flood impacts until a more comprehensive, regional flood reduction solution can be implemented in the future. However, in consultation with the NCFC&WCD, more information is needed to make a definitive conclusion (see comment #3).

We completely agree that Alternative 2 is the preferred option and have eliminated discussion of Alternatives 3a and 3b from the revised hydraulic report for simplicity and because those alternatives resulted in significant increases in flooding.

3. For Alternatives 2 and 3, it is difficult to determine whether the floodplain benefits outweigh the areas where flood depths are increasing. This is because not all increases (and decreases) are alike. For example, flood depth increases on undeveloped land may be more acceptable than increases affecting existing structures. For each recurrence interval event, please provide a comparison of:
    a) The number of insurable structures and the total floodplain area with WSE increases greater than 0.1 ft; and
    b) The number of insurable structures and the total floodplain area with WSE decreases less than 0.1 ft.

The table below provides the requested information for Alternative 2. As discussed above in the response to item #2, Alternative 3 has been removed from consideration due to unacceptable impacts. Changes in floodplain inundation on the proposed project parcels were excluded from the tabulation of floodplain area change to avoid skewing the results. The proposed condition results in reductions in offsite floodplain inundation area for all three flood events. The number of structures with decreases in WSE is greater than the number of structures with increases for all three flood events. The raw model outputs do indicate increases in 100-yr WSEs greater than 0.1 ft in the vicinity of two houses, however additional surveying and more detailed floodplain mapping was performed in the vicinity of the houses which revealed that the finished floor elevations are above both the existing and proposed 100-yr WSEs, therefore these structures were excluded from the table. The detailed mapping for the house upstream of the project on the north bank is discussed above under Item #1 and presented in Figure A1, and the detailed mapping for the other house downstream of the project on the south bank is presented in Figure A2. These statistics clearly show that the project results in a net reduction in flood risk to the surrounding neighborhoods.

	floodplain area (acres)			# of structures		
	increase	decrease	net change	increase	decrease	net change
2-yr	6.9	7.3	-0.4	7	15	-8
10-yr	4.4	13.6	-9.3	11	57	-46
100-yr	0.6	7.4	-6.8	0	2	-2

Figure A2: Surveyed elevations in the vicinity of the houses located downstream of the proposed project on the south side of Salvador Creek (the existing and proposed 100-yr water surface elevations in this reach are 68.1 and 68.2 ft respectively).

4. A Manning's *n* value of 0.08 was selected for the terracing. The report mentions that this value is based on mature vegetation conditions. Please provide additional information on what type of vegetation is planned or expected for the terraces and how that corresponds to the selected *n* value.

The terraces will be planted with native grasses, shrubs, and trees. A detailed riparian restoration plan will be prepared prior to construction. A Manning's *n* value of 0.08 was selected as a 'conservative' value representing the high-end of plausible roughness associated with the proposed planting strategy.

5. Figure 23 of the Hydraulic Analysis tech memo shows the maximum change in WSE for each of the alternatives for each modeled flood event. Please confirm the maximum change values. For example, for preferred Alternative 2 the 100-year max change is shown as 0.1 ft; however, the WSE difference plot (Figure 1 above) shows a max change of 0.2 to 0.4 ft.

The maximum WSE change values reported in Figure 23 of the prior report represent changes within the channel of Salvador Creek as simulated at cross sections within the 1-dimensional component of the model, whereas the changes shown in the WSE difference plots represent changes in inundation on the floodplains as simulated with the 2-dimensional component of the model. In the current report, the maximum changes in the channel and on the floodplain for Alternative 2 are shown in Table 2 and Figure 13 respectively.

6. Overall: The geomorphic analysis provides a well-reasoned analysis of potential stream sedimentation—based on the computed Rouse Number—through the project reach. We do not have any specific comments or suggested revisions on the geomorphic analysis.

No response necessary.

7. The preferred Alternative 2 has additional flow in the main channel downstream of the project, which could have an impact on stream stability. Please provide a discussion—either in the hydraulic analysis or with the geomorphic analysis—of whether any increased velocity or shear stress will adversely impact stream stability given the existing bank vegetation and/or protection. Please include figures showing computed channel velocities and shear stresses for existing and proposed conditions, as well as difference plots. Note: For the velocity and shear stress figures, please do not cover the main/active channel with a brown polygon (as in the flood elevation plots).

As requested, simulated channel velocities and shear stresses for existing and proposed conditions are shown below in Figures A3 and A4. Note, that the channel is simulated using a 1-dimensional formulation which is why the flood elevation plots show the channel as a brown polygon. The polygon is not covering up information but rather is intended to delineate where detailed 2-dimensional information is not available and where results are instead presented as longitudinal profiles.

The velocity and shear stress comparisons reveal that the increased flow in the channel downstream of the project associated with Alternative 2 results in only very minor increases in these parameters. Matt O'Connor performed an additional reconnaissance survey of the creek between Lassen Avenue and Jefferson Avenue on June 1, 2023 to assess existing channel stability and potential vulnerability to erosion. This reconnaissance revealed that much of the right lower bank is armored by a vertical rock revetment about 3 ft or less in height that appears to have been placed in a stable stacked arrangement, probably at the time of construction of the channel. Portions of the left bank are armored by plates of concrete slab that appear to be waste material. These revetments appeared stable; no areas of undermining or failure of revetment were observed.

There was little evidence of significant sediment deposition in the channel of Salvador Creek. There is one substantial gravel bar that was observed in both 2020 and 2023 formed about 200 ft downstream of the private bridge, and it was not particularly large. The sediment deposited on the bar had a median diameter estimated to be 30 mm and the largest clasts were about 100 mm in diameter. A sand deposit along the bank on the bar top had a median diameter estimated to be about 0.5 mm. The general absence of mobile gravel bars in this area reduces the potential bank erosion. The only other gravel bar observed in the survey is at the Jefferson Avenue bridge at the downstream end of the surveyed reach.

The downstream reach has relatively significant woody vegetation canopy in the narrow riparian zone between the adjacent vineyards and subdivisions. The riparian canopy is dominated by young willow and more mature oak trees; the young willow comprises a substantial understory component. The abundance of relatively mature trees is greater on the right bank. The density of shrubs, grasses, and herbaceous plants on the banks is variable. The width of the riparian zone is limited, and the overstory canopy generally extends not more than one crown diameter from either bank.

The density of riparian vegetation is sufficient to provide ground cover and some root reinforcement of soils and generally appears to prevent surface erosion. The density of woody stems and branches in the channel is substantial but not extreme. This density appears to provide some balance between excessive density that could significantly increase flow resistance and sparse or absent woody vegetation that could leave inadequate ground cover and low flow resistance along the banks that could increase the likelihood of erosion.

This reach of Salvador Creek is inhabited by beaver. Four active dams about 3 ft in height were observed during the survey. The upstream most of these is at Lassen Avenue with another active dam about 200 ft downstream near the existing private bridge; these two locations were also occupied by beaver dams in 2020. The next dam downstream is about 850 ft downstream of the private bridge, with another dam about 300 ft further downstream. The presence of beaver is indicative of a reliable food source and perennial flow. It is generally understood that impounded ponds formed upstream of dams promote the growth of willows and other woody plants utilized by beaver. The presence of this beaver population suggests that this reach of Salvador Creek is relatively stable.

Given hydraulic simulations that indicate very little change in velocity and bed shear stress in Salvador Creek with the proposed project and the observed channel conditions, the proposed project does not pose a substantial risk of destabilizing this reach of Salvador Creek.

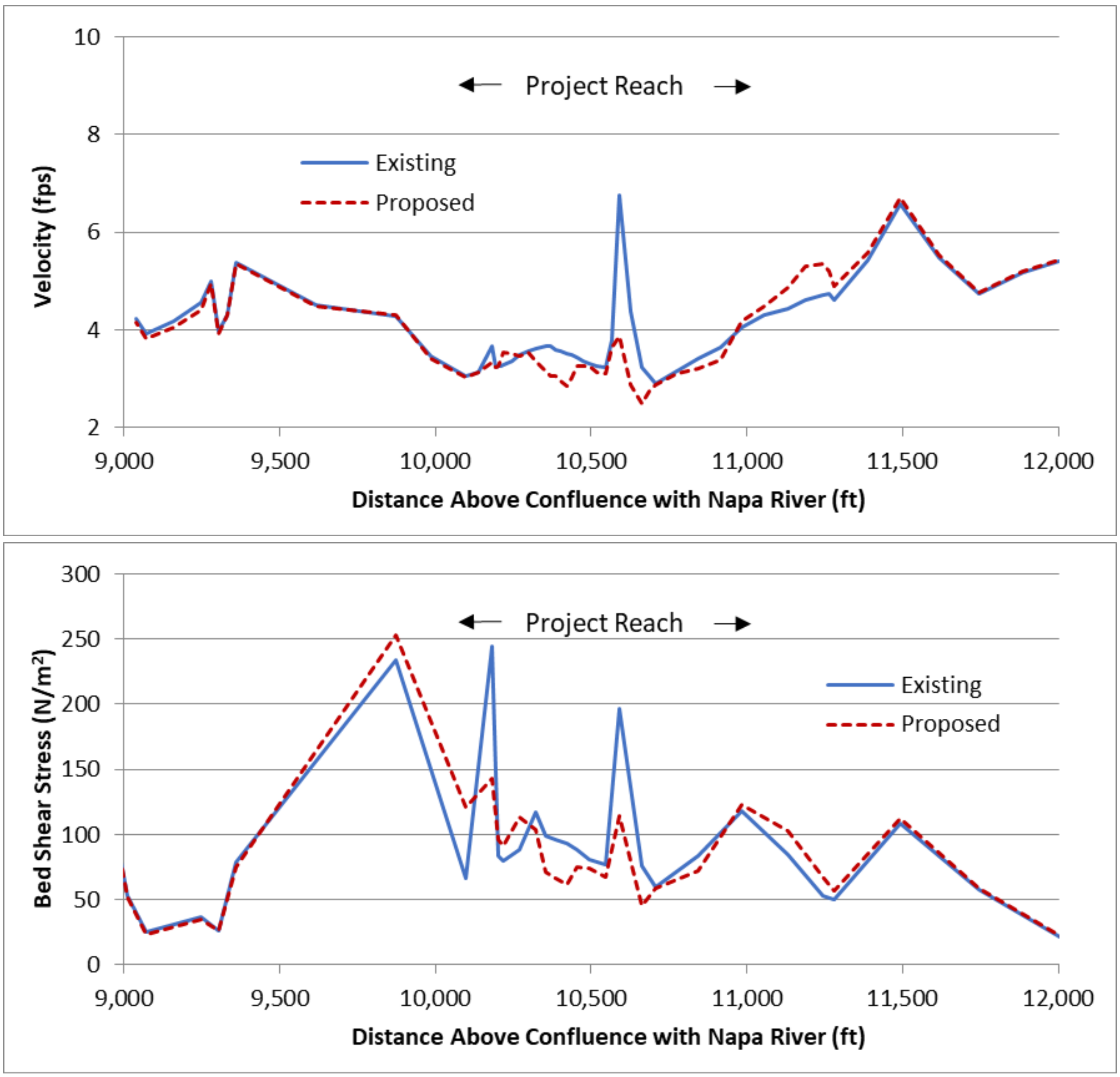
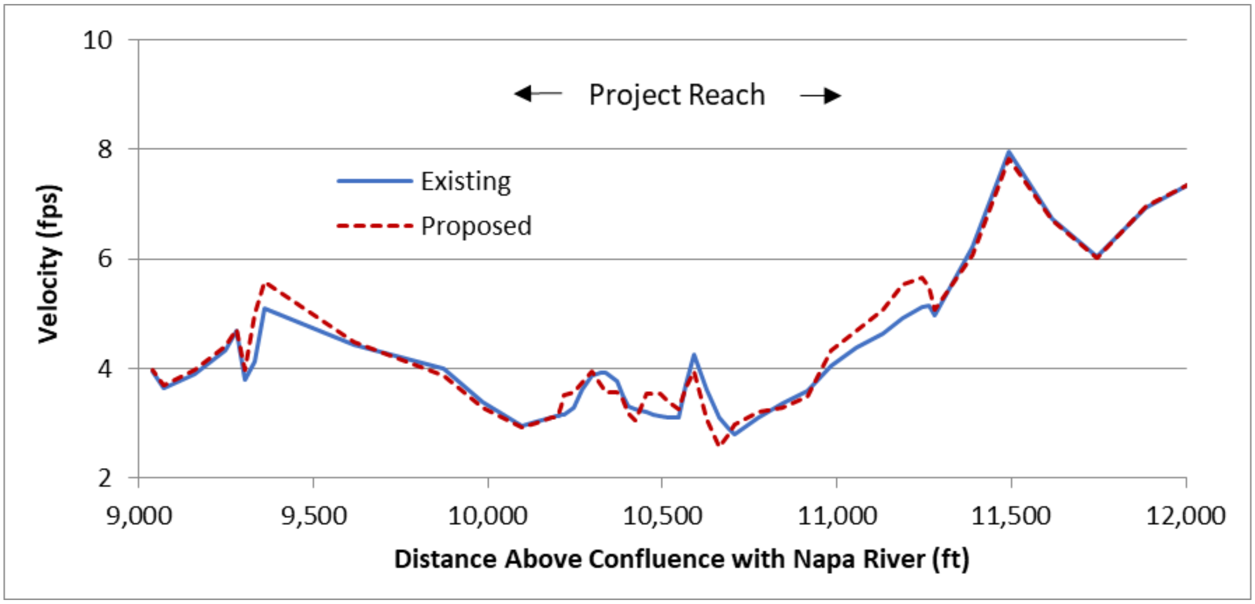


Figure A3: Comparison of existing and proposed velocities (top) and shear stresses (bottom) for the 10-yr flood.



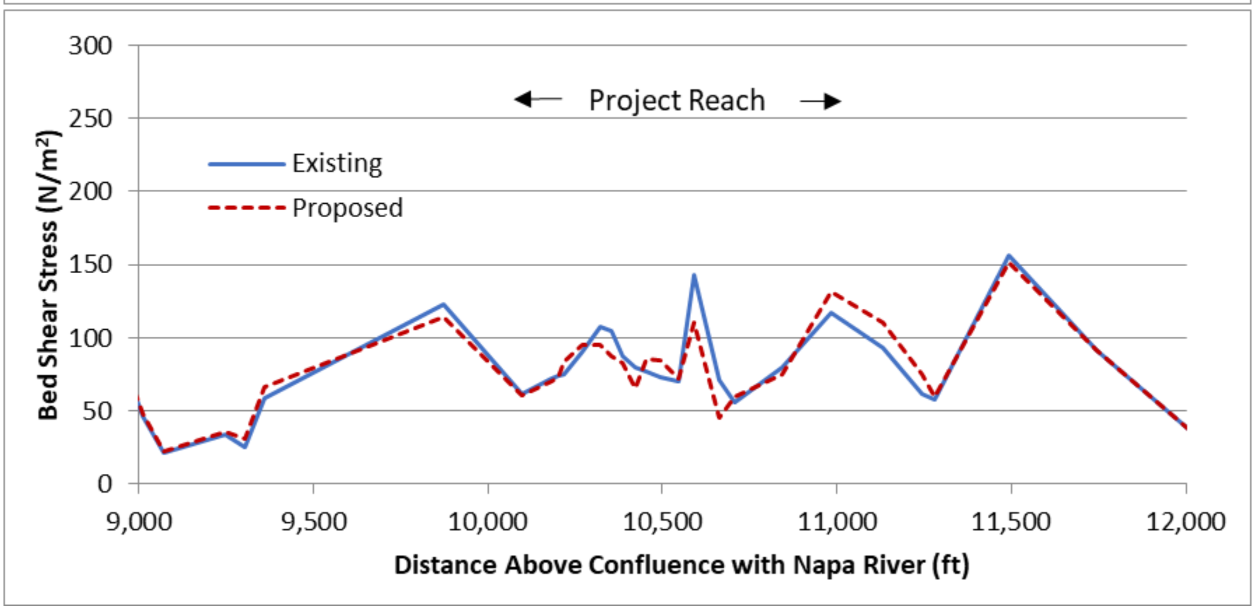


Figure A4: Comparison of existing and proposed velocities (top) and shear stresses (bottom) for the 100-yr flood.

8. Please provide velocity and shear stress results for the terraces and fill slopes along the terraces and verify that no additional protection is required.

The requested results are provided below in Figures A5 & A6. Maximum velocities and shear stresses on the terraces are ~2.3 fps and 35 N/m^2 respectively. These values indicate that additional protection beyond the planned planting of native grasses, shrubs, and trees is not necessary. Maximum velocities and shear stresses on the side slopes leading down to the terraces are ~7.2 fps and 100 N/m^2 respectively. These values indicate that these fill slopes do require additional protection in the form of 9-in d_{50} or larger rock rip-rap which will be incorporated in the final design plans.

9. Where floodplain flows are returning to the channel at the upstream end of the project (see Figure 4), the likelihood of bank erosion may increase. Please examine proposed vs. existing velocity and shear stresses specifically in this area.

Floodplain velocities and shear stresses are both quite low in the area where floodplain flows return to the channel at the upstream end of the project on the north side of the creek (Figures A5 & A6) and there aren't significant increases in channel velocities associated with the project in this area (Figures A3 & A4), therefore increases in bank erosion are not likely to occur in this area and additional bank protection is unnecessary.

10. We reviewed selected MIKE 11 and MIKE 21 models for the project and the model parameters look reasonable. The only model revisions needed will be based on design changes required by any of the previous comments.

No response necessary.

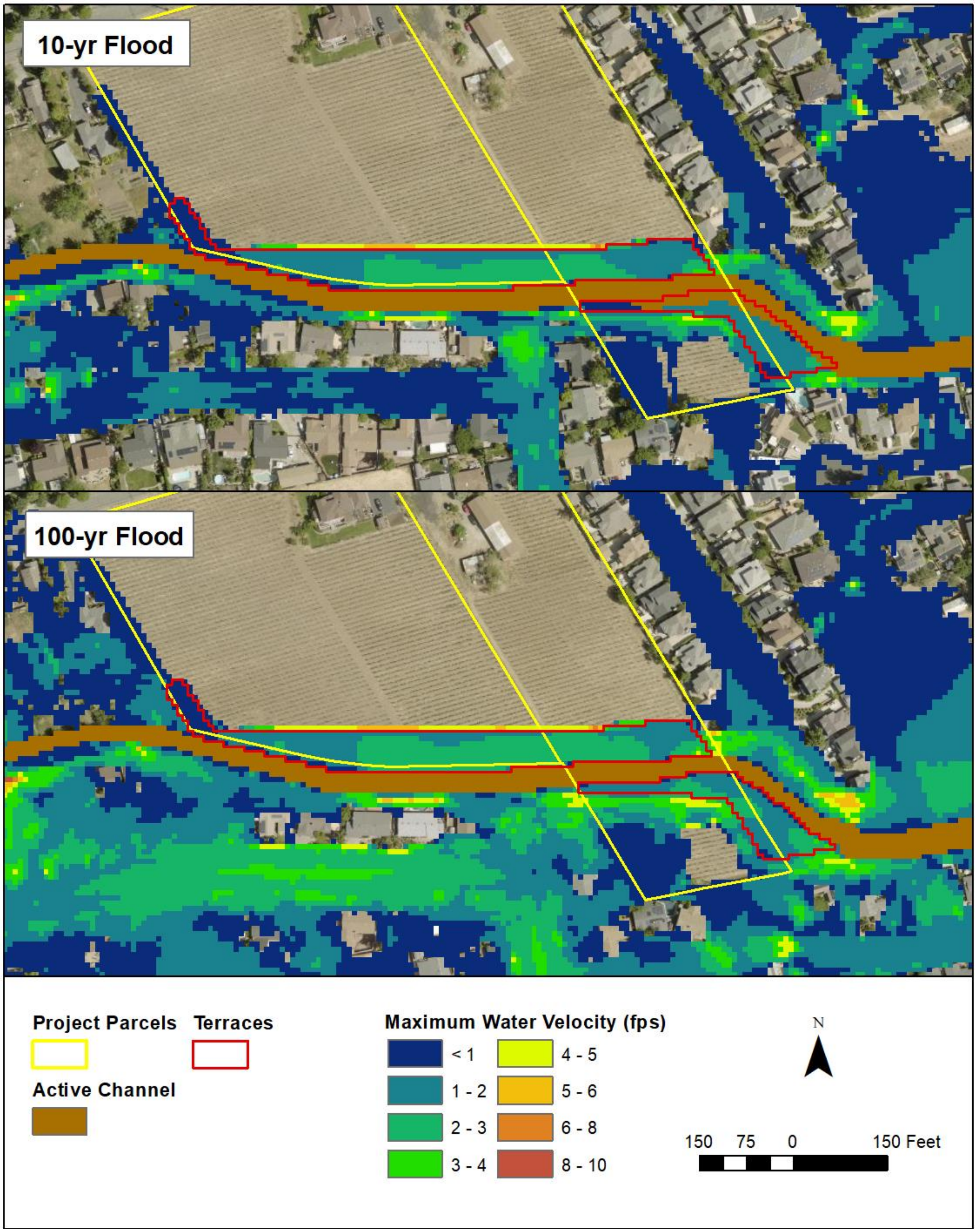


Figure A5: Proposed velocities for the 10-yr (top) and 100-yr (bottom) floods.

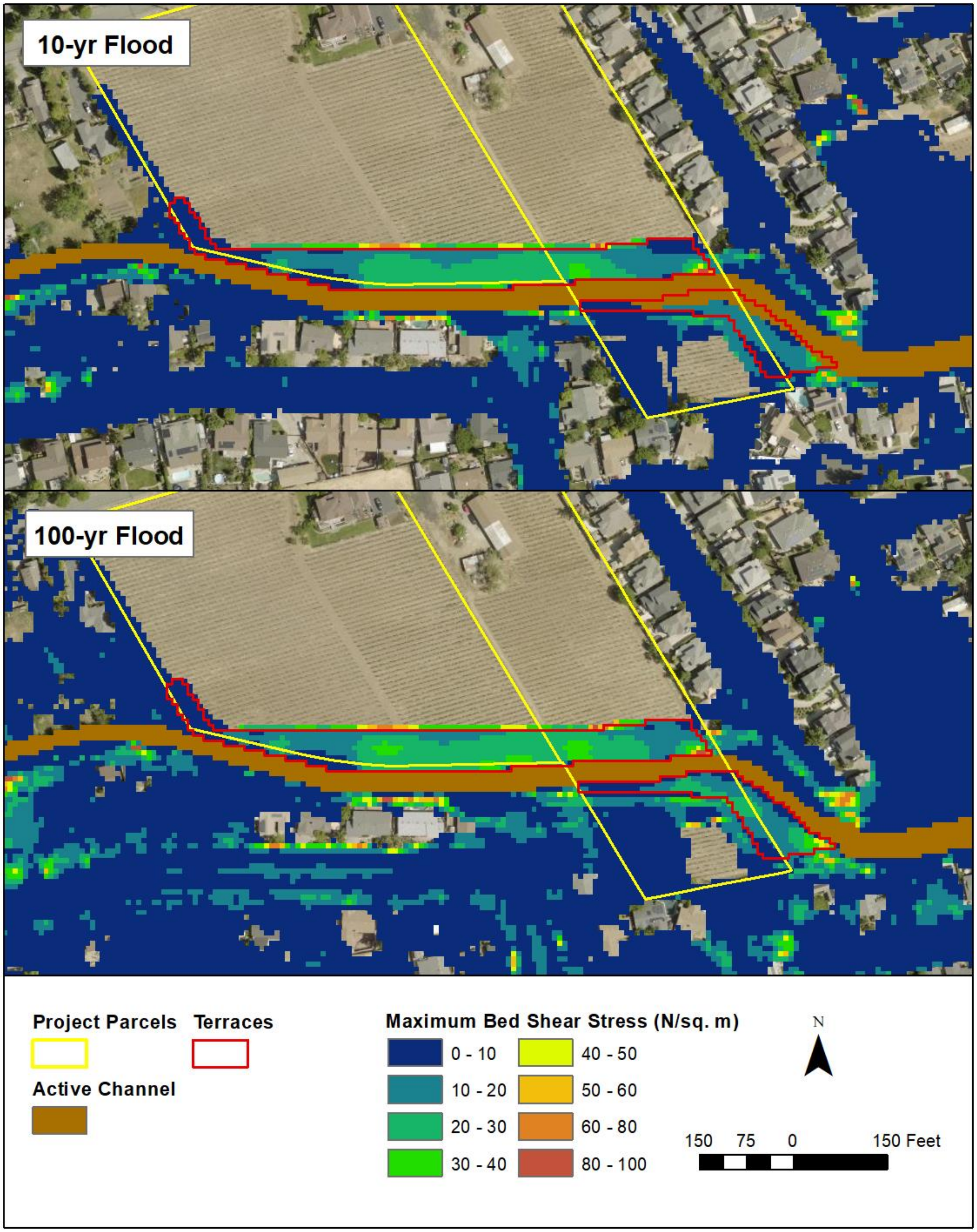


Figure A6: Proposed shear stresses for the 10-yr (top) and 100-yr (bottom) floods.

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F.4 - Review of Stormwater Management Documentation

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June 7, 2024

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via email to: rkrusenoski@fcs-intl.com

RE: Review of Stormwater Management Documentation for the Zinfandel Subdivision, City of Napa, Napa County, California

Dear Rachel Krusenoski,

Thank you again for offering Balance Hydrologics the opportunity to review documentation related to stormwater management pertinent to the proposed Zinfandel Subdivision in the City of Napa. Balance staff has reviewed the materials that were provided to understand the proposed stormwater management approach and evaluate whether the technical analyses provide reasonable assurance that potential impacts related to runoff water quality and flow rates can be mitigated with implementation of the infrastructure measures proposed. This letter summarizes our review of that information and associated conclusions.

Documents Reviewed

The following documents have been reviewed:

- Preliminary Detention Calculations, dated September 15, 2023, prepared by RSA+
- Stormwater Control Plan, dated September 15, 2023, prepared by RSA+
- Hydraulic Analysis, dated October 25, 2023, prepared by O'Connor Environmental, Inc.
- Peer Review of Development Project's Hydraulic Analysis, dated February 12, 2024, prepared by RSA+

Summary of Review and Findings

The review was carried out in light of the requirements of the City of Napa and the Phase II Municipal Stormwater NPDES permit issued by the California State Water Resources Control Board. City of Napa design standards require projects to mitigate potential increases in peak stormwater flow rates, while the State Phase II permit is the primary regulatory framework for compliance with federal Clean Water Act and state Porter-Cologne Act requirements. Specific City requirements and design standards are found in the City of Napa Standard Specifications (January 2022). Requirements associated with meeting the pertinent regulations regarding water quality and hydromodification management are found in the Design Guidance for Stormwater Treatment and Control for Projects in Marin, Sonoma, Napa, and Solano Counties published by the Bay Area Stormwater Management Agencies Association (BASMAA, January 2019).

The following discussion summarizes the findings of our review, and, where appropriate, identifies additional information that will likely be needed before final project approvals.

Water Quality. The Stormwater Control Plan provided is carefully prepared, and specifically addresses post-construction water-quality management approaches and facilities in a consistent and clear manner. Characterization of the site conditions, including underlying soils and pre-project drainage patterns and land cover, are appropriate for sites of this scale. Self-treating areas are appropriately delineated and characterized. The sizing of the four proposed stormwater control measures was reviewed and is consistent with the guidance set forth in the BASMAA manual, and the Plan conclusions that the bioretention facilities can meet runoff treatment and trash management requirements is supported.

> *Recommended Additional Information*. None.

Hydromodification Management. The proposed impervious cover associated with the project is well above the threshold of one acre at which the provisions of Section E.12.f of the Phase II permit apply. This provision requires regulated projects to mitigate increases in peak flow rates associated with the 2-year storm event. The Stormwater Control Plan as reviewed does not appear to acknowledge this requirement and/or the manner in which mitigation is provided. This omission is not design related as the requirements of Section E.12.f are actually met through the proposed stormwater treatment measures.

> *Recommended Additional Information*. The final version of the Stormwater Control Plan should include a concise discussion of the concept of potential impacts due to changes in runoff flow-duration relationships (hydromodification) and reiterate that sizing of the bioretention facilities per the BASMAA Manual meets the E.12.f criteria.

Peak Flow Control. Our understanding is that compliance with City Standard Specifications regarding peak flow control are summarized in the Preliminary Detention Calculations document. That document presents a discussion of the modeling completed using the TR-55 methodology. Review of the information presented, and the selection of hydrologic parameters show that the modeling was done in a manner consistent with standard practice and generally conforming to the requirements embodied in the City standards. The model output supports the conclusion that the bioretention facilities can be configured in a manner that mitigates potential increases in peak flow during a 100-year event. The document correctly notes that there are special provisions that apply to projects in the Salvador Creek watershed (see Section 2.10.01 of the City Standard Specifications) but does not discuss the rationale for only including modeling of the 100-year event, whereas the standards call for modeling of the 10- and 25-year events as well.

Recommended Additional Information. Future updates of the detention calculations should include a discussion and/or hydrologic modeling per the following considerations:

- Pre-project curve number. The composite pre-project curve number cites a pervious area value of 84 per guidance for open space in fair condition on hydrologic soil group D soils. The characterization of the soil type is appropriate; however, the majority of the site has apparently been dedicated to vineyard uses for many years. A reference should be included supporting the selection of open space as the pervious area curve number.

- Design storms. As noted above, our understanding is that the full range of requirements in the City standards apply. Therefore, the modeling should be expanded to include the 10- and 25-year design events.

- Basin routing. The discussion of the hydrograph routing through the two bioretention facilities should be revised to clarify whether the underdrain outlet is included in the routing calculations and/or why it is not. The text should also clarify the flowline elevations of the proposed outlet orifices, since a minimum of 6 inches of ponding depth is typically necessary to achieve the biofiltration benefits of each facility and the orifices should not reduce that storage volume. Additional information should also be provided regarding any emergency spillways and how the facilities will provide the one foot of freeboard called for in the standards.

Summary

The documents reviewed provide information that shows the proposed project can mitigate potential impacts related to water quality, hydromodification, and peak flow released to Salvador Creek.
Our understanding is that the information provided is consistent with an early stage of the project design and that future design revisions and associated documentation will address the additional information identified in our review. It is reasonable to assume that inclusion of the appropriate additional modeling and discussion can show that the project can fully mitigate the pertinent potential impacts associated with stormwater management at the site.

Closing

Thank you again for the opportunity to assist with this peer review process. Do not hesitate to contact us if you have questions related to observations presented in this letter.

Sincerely,

BALANCE HYDROLOGICS, Inc.

Edward D. Ballman, P.E.
Principal Engineer

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F.5 - Review of the Hydraulic Analysis

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August 23, 2024

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RE: Review of the Hydraulic Analysis for the Zinfandel Subdivision, City of Napa, Napa County, California

Dear Rachel Krusenoski,

Thank you again for offering Balance Hydrologics the opportunity to review documentation related to stormwater management pertinent to the proposed Zinfandel Subdivision in the City of Napa. Balance staff has re-reviewed the Hydraulic Analysis documents that were provided to understand the effects of the proposed developments on Salvador Creek and the surrounding floodplain. This letter summarizes our review of that information and associated conclusions. It is intended to supplement the prior letter that we submitted on June 7, 2024, which covered our review of the proposed stormwater management approach for the project.

Documents Reviewed

The following documents have been reviewed:

- Hydraulic Analysis, dated October 25, 2023, prepared by O'Connor Environmental, Inc.
- Peer Review of Development Project's Hydraulic Analysis, dated February 12, 2024, prepared by RSA+

Summary of Review and Findings

The review was carried out to assess for reasonable model input and evaluation of model results consistent with the standard of care for such analyses. It is our understanding that no specific requirements exist for hydraulic modeling, other than the demonstration that the proposed project will not worsen flood conditions within the project reach and the surrounding area. The following discussion summarizes the findings of our review.

Mitigation of Flood Events. The hydraulic modeling approach is carefully prepared and addresses the impacts of two proposed alternatives with the existing conditions. Both alternatives are clearly described, and the discussed parameter assumptions are logical and appear well-justified. The results of the study clearly describe Alternative 2 as the superior option, as it results in the smallest increases in water surface elevation and inundation extents and depths for the simulated 2-, 10-, and 100-year floods.

Recommended Additional Information. None.

Response to Original Peer Review Comments. The Hydraulic Analysis 2023 document is updated to address comments received from River Focus Water Resources Consultants. The responses demonstrate that the updated proposed alternatives have improved in their ability to lessen floodplain impacts. Each comment has been carefully and clearly addressed.

Recommended Additional Information. None.

Summary

The documents reviewed provide information that shows the proposed project, specifically Alternative 2, can mitigate potential impacts related to flooding during 2-, 10, and 100-year flood events, both within the project reach and upstream and downstream along Salvador Creek.

Closing

Thank you again for the opportunity to assist with this peer review process. Do not hesitate to contact us if you have questions related to observations presented in this letter.

Sincerely,

BALANCE HYDROLOGICS, Inc.

Edward D. Ballman, P.E.
Principal Engineer