DRAINAGE STUDY
FOR
VISTA II BALLFIELDS

(PRELIMINARY ENGINEERING)

Job Number 19253-B

September 30, 2022
Revised: April 7, 2023
Revised: October 16, 2023
Revised: February 27, 2024
Revised: April 17, 2024

RICK ENGINEERING COMPANY

SDC PDS RCVD 09-13-24
TM5647

RICK ENGINEERING COMPANY

DRAINAGE STUDY

FOR

VISTA II BALLFIELDS

(PRELIMINARY ENGINEERING)

Job Number 19253-B

Shavger Rekani
R.C.E #90893, Exp. 3/26

Prepared for:

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September 30, 2022
Revised: April 7, 2023
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Revised: February 27, 2024
Revised: April 17, 2024

DECLARATION OF RESPONSIBLE CHARGE

I hereby declare that I am the engineer of work for this project. That I have exercised responsible charge over the design of the project as defined in Section 6703 of the Business and Professions Code, and that the design is consistent with current standards.

I understand that the check of the project drawings and specifications by the County of San Diego is confined to a review only and does not relieve me, as engineer of work, of my responsibilities for project design.

Shavger Rekani R.C.E #90893, Exp. 3/26

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DRAINAGE STUDY
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Revision Page

April 17, 2024

This Drainage Study presents a revision to the February 27, 2024 report pursuant to a call with the County of San Diego on April 11th, 2024. The following paragraph summarizes the request and the updates to the report.

Based on a call with the County of San Diego on Aprill 11th, 2024, it was requested to show the lot lines for the proposed project on the Hydraulic workmap to show in the limited locations where we see an increase in water surface elevation that it is contained within the proposed project's property limits. Additionally, a note has been added to the exhibit to acknowledge that for one upstream cross-section (1899) that extends beyond the property limits only a 0.01-ft increase is recorded which, considering the limits of the model, effectively rounds to 0.0-ft. The table shows results to two decimal points to be consistent with the HEC-RAS model output.

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Revision Page

February 27, 2024

This Drainage Study presents a revision to the October 16, 2023 report pursuant to the County of San Diego plan check comments received January 24, 2024. The following text identifies the plan check comments along with the responses in bold.

County of San Diego Comments

7-2. The CEQA Drainage Study will be required to be stamped and signed by the Engineer of Record.

Response: Comment noted. The Drainage Study will be stamped and signed for the final submittal.

7-33a. 3rd Review Comment: Revise the n- value used for the channel. Per Section 5.5.2 of the County Hydraulic Design Manual, a mature channel must be assumed and therefore the channel capacity n value must be 0.150.

Response: n- values have been updated for the offsite channel.

7-33b. 3rd Review Comment: The values for the water surface elevation appear to indicate an increase at certain stations. Due to increase in water surface elevation for the existing channel, document if the increase is contained on the project's private property or if the increase will impact existing neighboring property owners.

Response: The water surface elevations and HEC-RAS model has been revised. Any increase to WSEL is contained within the project's private property.

7-35. 3rd Review Comment: Update the Tc, A, and Q100 values for POC 2 in the Pre-Project Drainage Study Map to be consistent with the summary table and analysis in the report.

Response: The pre-project drainage study exhibit has been updated to be consistent with the summary table in the Drainage Study.

7-36. 3rd Review Comment: Both the Pre-Project and Post-Project Drainage exhibits do not indicate the 0.1 subbasin for the right of way public improvements, but the area appears to be included in the analysis.

Revise to show this area in both maps.

Response: The area representing the right of way public improvements has been added to the pre-project and post-project drainage exhibits.

7-37. 3rd Review Comment:
Depending on how the Grading Violation against this property is resolved, the following revisions may be required:

- If the soil from the Vista I project site is allowed to remain on the Vista II site, then the pre-project conditions for this project will need to reflect the soil that was moved onto the site.
- If the soil from the Vista I project site will be removed and the Vista II project site will be restored to its condition prior to the Vista I soil being moved onto the site, then no revisions are needed.

Response: The pre-project exhibit has been updated to show soil from the Vista I project.

DRAINAGE STUDY
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Revision Page

October 16, 2023

This Drainage Study presents a revision to the April 7, 2023 report pursuant to the County of San Diego plan check comments received August 31, 2023 The following text identifies the plan check comments along with the responses in bold.

<u>County of San Diego Comments</u>

7-1. Offsite hydrologic and hydraulic analysis for the existing channel still needed.

Offsite hydraulic and hydraulic analysis has been provided in Appendix I of the report.

7-2. The CEQA Drainage Study will be required to be stamped and signed by the Engineer of Record.

Comment noted. CEQA Drainage Study will be signed and stamped on final submittal.

7-2a. Section 1 Introduction. Revise the narrative to clarify that a network of vaults designed to pond concurrently will drain to a biofiltration BMP. Currently the narrative says one underground vault, which may be how the system was idealized and modeled but is not what will physically be installed.

Section 1 introduction has been revised to clarify the proposed drainage design. The narrative specifies that there are two separate vault systems comprised of individual vaults intended to pond concurrently.

7-3. Specific values are needed in the post-project Q100 mitigated condition. It must be demonstrated through the analysis that the value for the Q100 mitigated condition is less than the pre-project condition, not just stated as such.
V100s needed.

Specific values have been provided for the post-project Q100 and V100 mitigated condition.

7-5. Specific values are needed in the post-project Q100 mitigated condition. It must be demonstrated through the analysis that the value for the Q100 mitigated condition is less than the pre-project condition, not just stated as such.

Specific values have been provided for the post-project Q100 and V100 mitigated condition

7-10. Section 5.0 Conclusion Demonstrate that the existing channel used can safely convey offsite and onsite flow such that the project and neighboring properties will not be flooded.

The conclusion has been updated to demonstrate that the existing channel can safely convey offsite and onsite flow.

7-17. The runoff coefficients used in the AES model.

- Pre-project node 120 to node 120, code 8 - runoff coefficient does not match backup calculations. Still applicable (0.33 in calcs, 0.35 in Appendix C).
- Pre-project node 199 to node 199, code 8 for the 1.3 acre subbasin – runoff coefficient does not match backup calculations. Still applicable (0.33 in calcs, 0.35 in Appendix C).
- Post-project node 199 to node 199, for a 0.1 acre subbasin for the driveway does not appear to be reflected in the AES model. The properties of the self-mitigating 0.1 acre subbasin appear to have been used instead. Still applicable. There are 3 0.1-ac self-mitigating (C=0.35) subbasins added to the mainflow to POC 2, but one of these basins includes added impervious surfaces for the proposed roadway and sidewalk connection to Hanalei Drive. The C value for this subbasin should be higher to reflect the impervious surface added.

The AES Pre- and Post-Project models have been updated to reflect comments.

7-33. Provide offsite hydrologic and hydraulic analysis for the existing channel. Response is only sufficient with regards to connecting to the existing storm drain system.

An offsite hydrologic and hydraulic analysis for the existing report has been included in Section 3 of the Drainage Study.

DRAINAGE STUDY
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Revision Page

April 7, 2023

This Drainage Study presents a revision to the September 30, 2022 report pursuant to the City of San Diego's plan check comments received on March 3, 2023. The following text identifies the plan check comments along with the responses in bold.

<u>County of San Diego Comments</u>

7-1 The comments provided below for the CEQA Drainage Study review are incomplete, as the initial submittal did not include the following:
- Contribution of off-site flows
- Hydraulics for onsite/offsite storm drain and channel
- Inlet sizing
- Energy dissipator design
- discrepancies/missing information in design

Contribution of off-site flows, detailed storm drain hydraulics, inlet sizing, and energy dissipator design will be provided in final engineering. Missing design information has been provided in Drainage Study Appendix H.

7-2 The CEQA Drainage Study will be required to be stamped and signed by the Engineer of Record.

CEQA Drainage Study will be stamped and signed once other comments are addressed.

7-3 In Section 2.3 Results, please provide a summary table of: pre- and post- development C, Tc, I, A, V100, Q100 without mitigation and Q100 with mitigation for each area (or point) where drainage discharges from the project. Peak runoff rates (cfs), velocities (fps) and identification of all erosive velocities (at all points of discharge) calculations for pre-development and post-development must be provided. The comparisons should be made about the same discharge points for each drainage basin affecting the site and adjacent properties.

A summary table of pre- and post- development C, Tc, I, A, and Q100 has been added to section 2.3. V100s will be provided in a future submittal.

7-4 Clarify the discrepancy between the watershed area value in the pre- and post- development (Table 1 and Table 2). It appears that the project proposes to change the drainage pattern. Where is the stormwater from the 0.2 acre difference between the pre and post development project directed?

Offsite area not included in the project boundary had been included in the pre-project model but not the post-project model. This area has been added back into the post-project model to ensure that equivalent areas are compared.

7-5 The Detained 100-year peak flow rate shown in Table 2 in the Results section of the report does not match the value from the HEC-1 model. Revise accordingly to maintain consistency.

Comment noted. Table 2 has been updated for consistency.

7-6 The description of how the stormwater is discharged BMP-A2 to eventually reach the proposed POC on Hannalei Drive is inconsistent through the report. According to the CEQA-Level Drainage Report Section 1, a storm drain is proposed along the existing channel, but Section 3 of the report states it will discharge to the channel. If BMP-A2 will discharge into the existing channel, a second POC is needed.

BMP-A2 will now discharge to the channel due to environmental considerations. The exhibits and report narrative have been updated to reflect these changes, and second POC has been added.

7-7 Section 3 Hydraulics: Correct spelling and/or grammar mistakes.

Comment noted.

7-8 Section 3.2 Pipe Hydraulics: Clarify what pipe size will be used.

Pipe hydraulics will be provided in final engineering. Please refer to the civil plan sheet for specific storm drain pipe sizes.

7-9. Section 5.0 Conclusion: Show the limits of proposed grading within the existing channel in the PGP. Will any portion of the proposed grading fall outside of the property bounds or be within any existing easement?

No grading will be performed within the channel.

7-10. Section 5.0 Conclusion: Demonstrate that the existing channel used can safely convey offsite and onsite flow such that the project and neighboring properties will not be flooded.

Existing drainage patterns will be maintained, and detention will be provided onsite to ensure that post-project Q100s are equal to or less than pre-project flows. Please see detention analysis in Appendix H, the hydraulic section 3.4 of the report, and the hydraulic workmap in Appendix I for more detail.

7-11. Section 5.0 Conclusion: Please discuss explicitly whether or not the proposed project would substantially alter the existing drainage pattern of the site or area, including through alteration of the course of a stream or river, in a manner which would result in substantial erosion or siltation on- or off-site? If so, provide reasons and mitigations proposed.

Conclusion has been updated to discuss project impacts to existing drainage. There are no substantial changes to existing drainage patterns. The channel adjacent to the project will not be modified or impacted. Detention will be provided to ensure that post-project Q100 is equal to or less than pre-project Q100.

7-12. Section 5.0 Conclusion: Discuss whether or not the proposed project would substantially alter the existing drainage pattern of the site or area, including through alteration of the course of a stream or river or substantially increase the rate or amount of surface runoff in a manner which would result in flooding on- or off- site? If so, provide reasons and mitigations proposed.

Conclusion has been updated to discuss project impacts to existing drainage. There are no substantial changes to existing drainage patterns. The channel adjacent to the project will not be modified or impacted. Detention will be provided to ensure that post-project Q100s are equal to or less than pre-project flows.

7-13 Section 5.0 Conclusion: Discuss explicitly whether or not the proposed project would create or contribute runoff water which would exceed the capacity of existing or planned storm water drainage systems? Provide reasons and mitigations proposed.

Report narrative has been updated. The project will include detention vaults to ensure that peak post-project flows are less than peak pre-project flows. Please see detention analysis for more detail.

7-14 Section 5.0 Conclusion: Discuss explicitly whether or not the proposed project would place structures within a 100-year flood hazard area which would impede or redirect flood flows?

A discussion regarding 100-year flood hazard area impediments has been included within section 5.0 of the drainage study.

7-15 Provide Rainfall Isopluvial for 100 Year Rainfall Event - 6 Hours and 24 Hours Maps with project site indicated.

Comment noted. Isopluvial maps have been provided in Appendix C of the drainage study.

7-16 It is not standard practice to use Code 8 subbasins after confluence analysis, as the actual Tc value would be significantly different between the mainline and the subarea being added.

The Code 8 subbasins added after the confluence analysis do not represent a significant portion of the overall basin area. Tc for a confluence is determined primarily based on the longest flow path and ratio between basin areas which would not be heavily impacted due to the addition of small additional areas.

7-17 Verify the runoff coefficients used in the AES model.
- Pre-project node 120 to node 120, code 8 - runoff coefficient does not match backup calculations.
- Pre-project node 199 to node 199, code 8 for the 1.3 acre subbasin - runoff coefficient does not match backup calculations.

- Post-project node 199 to node 199, code 8 for the 0.1 acre subbasin for the driveway does not appear to be reflected in the AES model. The properties of the self-mitigating 0.1 acre subbasin appear to have been used instead.

An AES Rational Method Coefficient chart has been provided in Appendix C of the Drainage Study. NRCS land uses and percent impervious were used to calculate runoff coefficients.

7-18 Include the Weighted Runoff Coefficient Calculations tables in the Drainage Study Maps

An AES Rational Method Coefficient Table (Table 3-1) has been provided in Appendix C of the Drainage Study. NRCS land uses and percent impervious were used to calculate runoff coefficients.

7-19 Post-Project Drainage Study Map: Label which pipes and their sizes are intended for use in the plan.

Please refer to the civil sheets that show the proposed size of the storm drain pipes.

7-20 Post- Project Drainage Study Map: Show the difference between impervious areas from the pervious areas (e.g., using a hatch/shading) on the plan.

Individually hatched pervious/impervious areas will be shown in final engineering. SDCHM Table 3-1 has been used to calculate runoff C values for the Drainage Study.

7-21 Post- Project Drainage Study Map: At the POC, please also indicate the Tc, A, and Q100 for the unmitigated condition.

Tc, A, and Q100 have been provided at POC-1 and POC-2 for the unmitigated condition.

7-22 For the HEC-1, provide all input information, including the input hydrograph and storage volume details.

HEC-1 input hydrograph, storage details, and outlet details have been provided in Appendix H of the drainage study.

7-23 Provide Intensity-Duration Design Chart Figure 3-1 indicating what value(s) were used.

Figure 3-1 has been added to Appendix C of the drainage study.

7-24 Provide Maximum Overland Flow Length and Initial Time of Concentration-Table 3-2.

Table 3-2 has been added to Appendix C of the drainage study.

7-25 Provide Rational Formula for Overland Time of Flow Nomograph Figure 3-3 indicating what value(s) were used.

Figure 3-3 has been added to Appendix C of the drainage study.

7-26 Provide Nomograph for Determination of Tc Figure 3-4 indicating what value(s) were used.

Figure 3-4 has been added to Appendix C of the drainage study.

7-27 Provide Computation of Effective Slope for Natural Watersheds Figure 3-5.

Figure 3-5 has been added to Appendix C of the drainage study.

7-28. Provide Detention storage routing calculations per Section 6.3.2 of the County of San Diego Hydraulic Design Manual (HDM), 2014.

Detention storage calculations have been added to Appendix H of the Drainage Study.

7-29. Provide Tables showing Vault Depth (ft) -Storage (ac-ft) -Outlet (Orifice Size (in)).

Tables showing vault parameters have been added to Appendix H the drainage study.

7-30. Provide Inflow Hydrograph per Section 6.3.1.1 of the HDM, Stage-Storage Curve (Section 6.3.1.2), Stage-Discharge Curve (Section 6.3.1.3).

Detailed inflow hydrograph, Stage-Storage, and Stage-Discharge curves will be provided in final engineering. Preliminary Stage-Storage and Stage-discharge curves representative of the proposed vaults and outlet works have been provided in Appendix H.

7-31. Provide Detention Facility Plans per Section 6.2.5 of the HDM. Show maximum design inflow and velocity, maximum total design outflow and velocity from the outlet, maximum design storage volume and water surface elevation,

Detention facility flow rates have been provided in Appendix H. Velocities out of the storage facilities will be provided in final engineering.

7-32. Provide appropriate details for facility inlet, outlet structures, energy dissipaters, maintenance measures and cross sections.

Storage facility outlet orifice diameters have been provided in Appendix H of the Drainage Study. Inlet structures, energy dissipaters, and maintenance measures will be provided in final design.

7-33. Provide Hydraulic Analysis report. Include a discussion on the existing systems (channel and storm drain) that are proposed to be used in the post-project condition. Discuss the existing connection points to these systems and a comparison of the proposed condition connection points. Discuss how the Hydraulic Grade Line (HGL) in the existing system be affected in the post-project condition. Include a hydraulic model of the existing system from the downstream end to as far upstream as necessary to show any impacts to the existing systems.

Comment noted. A detailed hydraulic analysis will be provided in final engineering. A discussion on pre- and post-project channel hydraulics has been added to Section 3 of the Drainage Study.

1.0 INTRODUCTION

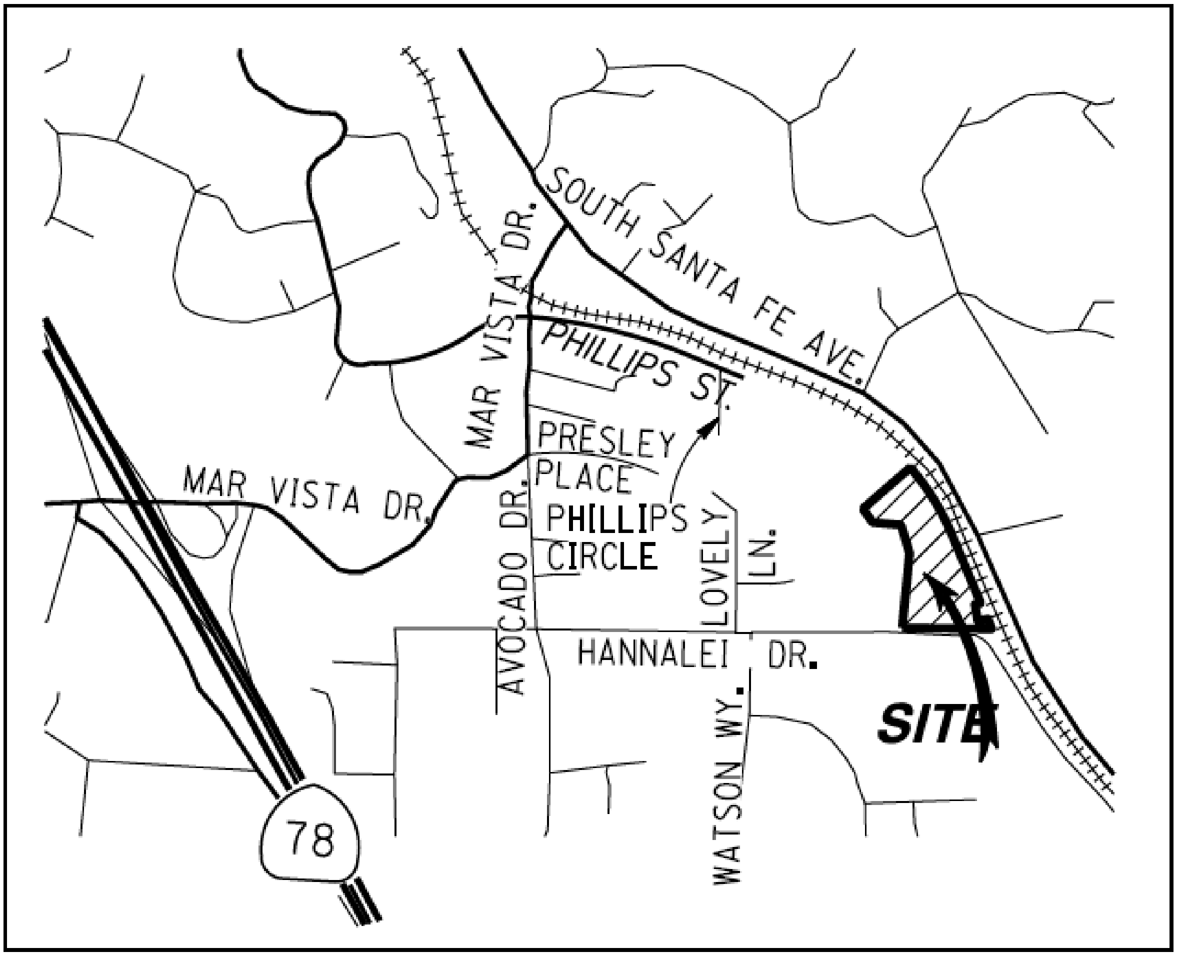
Vista II Ballfields is a residential development to be located in the County of San Diego, northeast of the intersection of Hannalei Drive and South Santa Fe Road. The project proposes 37 dwelling units, a park, roadway features and associated utilities. There are two vault systems intended to provide flood control, hydromodification management control, and detention in the northern and southern portions of the site. The northern vault system (BMP-A1) consists of three vaults that are intended to pond concurrently. The vault floors will be placed at the same elevation and are the same height. The southern vault system (BMP-B1) consists of two vaults that will pond concurrently in a manner similar to BMP-A1. One compact biofiltration BMP will receive low flows from BMP-A1 and BMP-B1 to provide pollutant control. The purpose of this report is to evaluate the impacts to any existing drainage conveyance networks as a result of the proposed project development.

The existing project site is comprised of three (3) baseball fields and associated parking lot. The entire site drains to one (1) point of interest. The site is divided into northern portion and southern portion for the purpose of hydrology. Runoff from the northern portion of the site (3.2 acres) of the project site flows southerly and then easterly to an existing channel via overland flow. The existing channel then flows southerly until it is intercepted by an existing headwall and conveyed into the existing storm drain network. Runoff from the southern portion of the site (1.9 acres) also flows south overland and then east along Hannalei Drive before it is intercepted by the existing headwall. The study of hydrology with regards to the existing channel has not been done as part of this project.

The project is a Tentative Map and Major Use Permit to subdivide an 8.93-acre site into three lots. Lot 1 contains an existing church and an existing driveway that will be improved as a secondary access for Lot 2. Lot 2, which is 5.33 acres, will be improved with 37 multi-family condominium units with associated parking and14,800 square feet of private usable open space. The third lot, Lot A, consists of an existing cellular facility and is not approved for any future development. Access to the site will be from Hannalei Drive with a secondary emergency access in the northwestern area of the site connecting to the adjacent church property to the west (on Lot 1). The project is part of the North County Metro Community Planning Area. Fire service will be provided by the Vista Fire Protection. Sewer will be provided by the Buena Sanitation District and water from the Vista Irrigation District. The site is subject to the General Plan Designation VR-7.3. Zoning for the site is RS. The project includes 111 total parking spaces and 61,462 total square feet of open space. Earthwork will consist of 10,700 cubic yards of cut, 22,500 cubic yards of fill and 11,800 cubic yards of imported material. The site contains a stockpile of approximately 3,500 cubic yards soil spread over a 1-acre area, which is in violation of the County's Grading Ordinance. The stockpile will remain on the site and is considered part of the project. Final Mapping for the project would occur in phases. The first unit would be to create lots 1 and 2 and lot A for finance and conveyance purposes only, not for development. Once the first unit is recorded, Lot 2 will be transferred to the future developer. Lot 2 will then be developed per the conditions of approval for Tentative Tract Map 5647.

The proposed drainage characteristics are similar to the existing drainage characteristics. The northern portion of the site still flows southeasterly via inlets and propose onsite storm drain and is treated by the underground vault and Modular Wetland System (MWS) combination. The mid-flows and high-flows are discharged to the existing channel via proposed onsite storm drain. The outlet to the channel will be protected with a riprap pad. The low flows in the northern portion of the site will be treated by the compact biofiltration system in the southern portion of the site. The southern portion of the site flows in the southerly direction and is treated by the underground vault and compact biofiltration system combination and ultimately ties into the proposed 36-inch RCP. The proposed 36-inch RCP ties into the existing 36-inch RCP across Hannalei Drive. The southern portion contains 0.1 ac. of no impervious area and is self-mitigating per the County of San Diego BMP Design Manual.

Figure 1-1: Vicinity Map



VICINITY MAP

NOT TO SCALE

2.0 ONSITE HYDROLOGY

2.1 CRITERIA

The hydrologic conditions were analyzed in accordance with the County of San Diego's design criteria.

Design Storm:	100-year, 6-hour
100-Year 6-Hour Precip (inches):	P = 3.2 inches

June 2003 San Diego County *Hydrology Manual* Criteria (unit-less) (See Appendix C)

Soil Type:	C and D (See Map Pocket 1)

Intensity-Duration-Frequency (I-D-F) Curves within the June 2003 County of San Diego *Hydrology Manual* (inches per hour)

2.2 MODIFIED RATIONAL METHOD

To calculate the flow rates for Basin 100 pre- and post-project, a Modified Rational Method analysis was performed in accordance with the methodology presented in the June 2003 County of San Diego *Hydrology Manual* to determine pre- and post-project 100-year peak discharge rates for watersheds less than 1 square-mile. The Advanced Engineering Software (AES) Rational Method computer program was used to perform these calculations. The hydrologic model is developed by creating independent node-link models of each interior drainage basin and linking these sub-models together at confluence points. The program has the capability to perform calculations for 15 hydrologic processes. These processes are assigned code numbers that appear in the results. The code numbers and their significance are as follows:

Code 1:	Confluence analysis at a node
Code 2:	Initial subarea analysis
Code 3:	Pipe flow travel time (computer-estimated pipe sizes)
Code 4:	Pipe flow travel time (user-specified pipe size)
Code 5:	Trapezoidal channel travel time
Code 6:	Street flow analysis through a subarea
Code 7:	User-specified information at a node
Code 8:	Addition of the subarea runoff to mainline
Code 9:	V-Gutter flow thru subarea
Code 10:	Copy main-stream data onto a memory bank
Code 11:	Confluence a memory bank with the main-stream memory
Code 12:	Clear a memory bank

Code 13: Clear the main-stream memory
Code 14: Copy a memory bank onto the main-stream memory
Code 15: Hydrologic data bank storage functions

In order for the program to perform the hydrologic analysis; base information for the study area is required. This information includes the land uses, drainage facility locations, flow patterns, drainage basin boundaries, and topographic elevations. The rainfall data, runoff coefficients, and soils information were obtained from the June 2003, County of San Diego *Hydrology Manual*.

2.3 RESULTS

The 100-year Modified Rational Method and Rational Method calculations for pre- and post-project conditions are provided in Appendix A and Appendix B, respectively, and the associated hydrologic drainage exhibits are located in Map Pockets 1 and 2. Preliminary detention analysis is provided in Appendix H. A summary of the results for contributing areas are listed in the following table:

Table 1 - Hydrologic Summary Table (Pre-project)

Basin	Watershed Area (acres)	Runoff "C"	Time of Concentration (min)	Intensity (in/hr)	Q100 (cfs)
100	3.2	0.46	11.1	5.0	7.4
200	5.2	0.5	12.1	4.8	11.2

Table 2 - Hydrologic Summary Table (Post-project)

Basin	Watershed Area (acres)	Runoff "C"	Time of Concentration (min)	Intensity (in/hr)	Q100 – Unmitigated (cfs)	Q100 – Mitigated (cfs)
100	3.2	0.69	6.1	5.1	16.1	5.7
200	5.3	0.71	6.8	6.9	25.9	10.8

Table 3 – Outfall Velocity Table

Basin	V100-Pre (ft/s)	V100-Post (ft/s)	V100-Post (Mitigated, ft/s)
100	n/a - channel	11.2	8.0
200	10.1	12.9	9.0

3.0 OFFSITE HYDROLOGY

3.1 OFFSITE DRAINAGE CHARACTERISTICS

An offsite drainage analysis has been included in this report to provide backup for the flowrates used in the existing earthen channel that boarders the eastern edge of the project site.. This analysis has been conducted to validate the flowrates used in the hydraulic analysis of the channel adjacent to the project.

Existing development in the surrounding area consists of a road, Hannalei Drive, and single-family homes to the south of the project, a graded lot to the north of the project, a church and single family homes to the west of a project, and a railroad, trail, and highway to the east of the project. Soils consist of type "D" and type "C" soil. To the east of the project is a manmade earthen channel that conveys flows from north of the project site to a storm drain on the north side of Hannalei Drive adjacent to the proposed development.

3.2 REFERENCED REPORTS & RESEARCH

Several resources were used to develop an understanding of offsite drainage conditions. The primary report is titled "Inland Rail Trail (IRT) Phase 2B Preliminary Drainage Study", dated December 2014, and prepared by PSOMAS. The purpose of this report is to analyze the hydrology for the IRT bike path project. The project consists of approximately 5,500 feet of new bike path running alongside the North County Transit District railroad tracks. Areas tributary to the offsite channel north of the project site bounded by Phillips Circuit are included in the IRT report. Additionally, the northern half of the project site is included in the report along with the area tributary to the rail trail between the railroad tracks and the project site. The criteria used to develop this project is per the County of San Diego Hydrology Manual, June 2023. The report analyzes the 100-year, 6-hour storm event and includes resources from the CSDHM, hydrology inputs and outputs, and maps of the project watershed. Refer to Appendix J for experts of this report.

Referenced in the IRT report are two additional reports, the first, titled "Drainage Report South Santa Fe Avenue Widening Project", dated September, 2009 (Santa Fe Ave.) and prepared by others, and the second, titled "Sprinter Rail Project (SRP) Drainage Calculations – Oceanside to Escondido", dated August, 2004 and prepared by others. The Santa Fe Ave. report was prepared for the design of the storm drain improvements associated with the Santa Fe Ave. project. The methodology used to develop this report was based on criteria from the 2003 County of San Diego Hydrology manual. The Santa Fe Ave. report includes supporting resources from the CSDHM, hydrology inputs, Advanced Engineering Software (AES) Rational Method outputs, maps denoting drainage delineations, and flow rates for the 100-year, 6-hour storm events. Additionally, the report includes flow rates for the three outfalls that drain to the earthen channel adjacent to the project site. These flow rates provide a basis for the flow rates used in the hydraulic analysis. Appendix J has experts of this report (note: the Sante Fe Ave. Drainage Report is referenced in the IRT report in Appendix D).

The SRP report is intended to summarize the drainage parameters associated with the cross culverts in the railroad right-of-way, the new cross culverts built as part of the SRP, and the side track ditches along the right-of-way. Due to the age of the report and the outdated criteria used for the analysis, this report was not used to develop flow rates for Vista II.

Additionally, Rick Engineering completed a drainage study for a project that boarders the project to the northwest. The report, titled "Drainage Study for Vista Hannalei", revised April, 2023 details hydrologic conditions to the north and west of the project site and provides pre-project, post-project, and mitigated flows an outlet that discharges to the existing channel just upstream from our project site. The 2003 County of San Diego Hydrology Manual was used to develop flows for the 100-year, 6-hour storm. The Advanced Engineering Software (AES) program was used to perform a Modified Rational Method analysis of the drainage area. Please refer to Appendix K for references from this report.

3.3 OFFSITE RESULTS

After review of the reports above and validating the methodology is consistent with the County of San Diego Hydrology manual (2003), the following table summarizes hydrologic flow data used to develop the offsite hydraulic model.

Table 4: Offsite Hydrologic Summary

HEC-RAS Flow Inputs			
Pre-Project		Post-Project	
River Station	Q100	River Station	Q100
1948	64.3	1948	64.3
1923	68.6	1923	68.6
1713	99.7	1713	94.4
-	-	1350	98.7
1271	131.4	1271	130.4
1200	131.4	1200	130.4

4.0 HYDRAULICS

The 100-year post-project peak flow rates determined using the methodology described in Section 2 were used to evaluate the hydraulic capacity of inlets leading into both the BMPs. Inlet sizes will be provided in subsequent submittals. Flows from BMP-A will discharge to the adjacent channel to the east of the project site at Node 125 and BMP-B will tie into the existing Storm Drain within Hannalei Drive. The existing Storm Drain also received offsite flows which were not analyzed as a part of this submittal. It is anticipated that offsite hydrology and hydraulic calculations will be performed as a part of subsequent submittals.

4.1 INLET SIZING

An inlet design calculation will be completed using a computer program based on Equation 1 for inlets on grade and Equation 2 inlets in sump and provided with a subsequent submittal:

Type A Inlets on a Grade

$$Q = 0.7\ L\ (a + y)^{3/2} \qquad \text{(Equation 1)}$$

Where:
y = depth of flow approaching the curb inlet, in feet (ft)
a = depth of depression of curb at inlet, in feet (ft)
L = length of clear opening of inlet for total interception, in feet (ft)
Q = interception capacity of the curb inlet, in cubic feet per second (cfs)

Type B Inlets in a Sump

$$Q/L = 1.5 \text{ cfs/ft} \qquad \text{(Equation 2)}$$

Where:
Q = inlet capacity, in cubic feet per second (cfs)
L = length of clear opening of inlet for total interception, in feet (ft)

4.2 PIPE HYDRAULICS

Storm drain pipe sizes were determined based on a normal depth calculation to verify storm drain capacity based on Manning’s equation.

$$Q = (1.486/n)\ A\ R^{2/3}\ S^{1/2}$$

Where:
Q = Discharge (cfs)
n = Manning’s roughness coefficient
A = Cross-sectional Area of flow (sq. ft.)
R = Hydraulic radius (ft.) (where hydraulic radius is defined as the cross-section area of flow divide by the wetted perimeter, R= A/P)
S = Slope of pipe (ft./ft.)

A Manning's roughness coefficient "n" of 0.013 was used for the hydraulic calculations. This value is typically used for reinforced concrete pipe (RCP), polyvinyl chloride (PVC) and high-density polyethylene pipe (HDPE). The pipe sizes were evaluated based on the Rational Method flow rates with a 30% "bump up" sizing factor to account for hydraulic losses within the system.

Please refer to Appendix E for the storm drain sizes.

4.3 ENERGY DISSIPATOR DESIGN

An energy dissipater (i.e. riprap) at the single storm drain outfall into the existing channel, which lies to the east of the project site at Node 125, will be specified using the 2014 County of San Diego *Hydraulic Design Manual*, which provides rock classifications for design velocities entering riprap outfalls.

The design velocities were determined from both normal depth hydraulic analyses for flow in the final reach of storm drain leading to the outfall and HEC-RAS hydraulic analysis for flow across the riprap pad immediately downstream of the outfall.

HEC-RAS cross sections will be taken at 1-foot intervals across the riprap pad in order to determine the location of the hydraulic jump that is expected to occur on the riprap pad. The flow regime after the hydraulic jump is subcritical flow at normal depth, and the flow velocity after the hydraulic jump is expected to be less than 6 feet per second. The riprap pad length was then specified based on the location of the hydraulic jump, in order to provide 5 feet (or twice the pipe diameter, whichever is greater) of length beyond the hydraulic jump. The riprap pad width is based on the 2012 Edition of the San Diego Regional Standard Drawings Book Riprap Energy Dissipation, drawing number D-40.

The dimensions and size of riprap will be provided in Appendix G as a part of subsequent submittals.

4.4 CHANNEL HYDRAULICS

In the existing condition, the channel adjacent to the eastern boundary of the project is natural earth with a hardened concrete section leading into a storm drain. The project does not propose any grading of the channel. The only change to the stream itself will be a riprap protected outfall discharging from the vault system on the eastern side of the property. A retaining wall is also proposed along the eastern side of the property to allow for pad elevations to be raised above the 100-year water surface elevation (WSEL). A HEC-RAS analysis was performed to ensure that pad elevations would have a minimum freeboard of at least 1-foot above the 100-year water surface elevation, consistent with the County of San Diego Hydraulic Design Manual Section 5.3.7 for open channels conveying more than 10cfs.

HES-RAS cross sections were taken along the length of the channel starting just upstream of the project and ending at the storm drain inlet downstream of the project. Water surface elevations were determined at each cross section, and pads were placed at elevations at least one foot above the closest cross section. Additionally, one cross-section (1105) has a water surface elevation increase, however it is contained within the lot line limits of the proposed project. Also, a note has been added to the exhibit to acknowledge that for one upstream cross-section (1899) that extends beyond the property limits only a 0.01-ft increase is recorded which, considering the limits of the model, effectively rounds to 0.0-ft. The table shows results to two decimal points to be consistent with the HEC-RAS model output. A figure showing HEC-RAS cross sections with their corresponding pad elevations and WSELs can be found in Appendix I.

5.0 DETENTION

For the detention system design, a rational method hydrologic analysis was performed to determine the 100-year peak discharge rates for the post-project condition. Detention will be provided within BMP-A-1 & BMP-B-1 to detain back flows for the 100-year storm event to pre-project conditions.

The sizing of a detention facility requires an inflow hydrograph to obtain the necessary storage volume. The modified rational method only yields a peak discharge and time of concentration and does not yield a hydrograph. In order to convert the peak discharge and time of concentration into a hydrograph, a modified rational method hydrograph synthesizing procedure was used. The modified rational method hydrograph synthesizing procedure methodology and criteria that were used are based on the Rational Method Hydrograph Procedure and Detention Basin Design, of the *San Diego County Hydrology Manual 2003*.

The 100-year hydrograph and elevation-storage-outflow rating curves were used in the HEC-1 hydrologic model to perform routing calculations for two detention systems in the project site to determine the 100-year detention volumes required for the systems to reduce the post-project peak discharge rate back to the pre-project peak discharge rate for the storm event.

The 100-year, 6-hour post-project peak discharge rates were routed using the HEC-1 hydrologic model to determine the detention volume required for the basins to reduce the post-project peak discharge rates back to the pre-project peak discharge rates. Preliminary HEC-1 detention analysis is provided in Appendix H. The detention analysis demonstrates that the post-project Q100 will be detained to pre-project Q100 as shown in Table 2.

6.0 CONCLUSION

Drainage conditions are expected to remain similar between pre- and post-project conditions. No major changes to drainage patterns are proposed, and existing drainage pathways will be preserved. The only proposed change to the existing channel adjacent to the eastern boundary is the addition of a riprap protected outlet for water quality, detention, and HMP flows out of the stormwater vault system. No grading is proposed in the channel. Peak post-project 100-year runoff rates will be detained to be equal to or below pre-project levels which reduces the risk of on- and off-site erosion and siltation. Since the post-project 100-year runoff rates will be equal to or less than existing 100-year runoff rates, the project is not anticipated to impact existing downstream facilities. Pad elevations have been raised to place structures out of the 100-year flood hazard areas in a manner that does not impede or redirect flood flows.

Post-project runoff will be treated for water quality by the underground vault and compact biofiltration BMP combination (i.e. BMP-A1, BMP-B2, BMP-B2). The two underground vault systems will also detain the 100-year 6-hour peak flows back to less than pre-project conditions while also satisfying the hydromodification management requirements per the County of San Diego BMP Design Manual. Grading is not proposed for the existing channel east of the project site. The channel receives flows from offsite areas. Offsite hydrologic and hydraulic analysis for the project has not been performed for this submittal. Flowrates used as part of the analysis have been taken from as-built information. Based on the results of this drainage analysis, it has been determined that the proposed residential development will not adversely impact the existing watershed or drainage patterns. Please refer to the report titled, "Priority Development Project Storm Water Quality Management Plan for Vista II," dated April 7, 2023, and prepared by Rick Engineering Company (Job No. 19253-B), for more information on water quality and hydromodification management.

APPENDIX A

HYDROLOGY ANALYSIS
100-YEAR, MODIFIED RATIONAL METHOD
(PRE-PROJECT)

```
____________________________________________________________________________
****************************************************************************

            RATIONAL METHOD HYDROLOGY COMPUTER PROGRAM PACKAGE
            Reference: SAN DIEGO COUNTY FLOOD CONTROL DISTRICT
                         2003,1985,1981 HYDROLOGY MANUAL
         (c) Copyright 1982-2014 Advanced Engineering Software (aes)
             Ver. 21.0 Release Date: 06/01/2014  License ID 1261

                           Analysis prepared by:

                          RICK ENGINEERING COMPANY
                              5620 Friars Road
                        San Diego, California 92110
                      619-291-0707   Fax 619-291-4165

 ************************** DESCRIPTION OF STUDY **************************
* J-19253B VISTA II BALLFIELDS                                             *
* 100YR, 6-HR PRE-PROJECT CONDITION                                        *
* J:\19253B\WR\HYDROLOGY\RATIONALMETHOD\...                                *
 **************************************************************************

  FILE NAME: V2B1E00.RAT
  TIME/DATE OF STUDY: 13:16 10/03/2023
----------------------------------------------------------------------------
  USER SPECIFIED HYDROLOGY AND HYDRAULIC MODEL INFORMATION:
----------------------------------------------------------------------------
  2003 SAN DIEGO MANUAL CRITERIA

  USER SPECIFIED STORM EVENT(YEAR) = 100.00
  6-HOUR DURATION PRECIPITATION (INCHES) =   3.200
  SPECIFIED MINIMUM PIPE SIZE(INCH) =  18.00
  SPECIFIED PERCENT OF GRADIENTS(DECIMAL) TO USE FOR FRICTION SLOPE = 0.90
  SAN DIEGO HYDROLOGY MANUAL "C"-VALUES USED FOR RATIONAL METHOD
  NOTE: USE MODIFIED RATIONAL METHOD PROCEDURES FOR CONFLUENCE ANALYSIS
  *USER-DEFINED STREET-SECTIONS FOR COUPLED PIPEFLOW AND STREETFLOW MODEL*
     HALF-  CROWN TO   STREET-CROSSFALL:   CURB  GUTTER-GEOMETRIES:  MANNING
     WIDTH  CROSSFALL  IN-  / OUT-/PARK-  HEIGHT  WIDTH  LIP   HIKE  FACTOR
NO.   (FT)     (FT)    SIDE / SIDE/ WAY    (FT)    (FT)  (FT)  (FT)    (n)
===  =====  =========  =================  ======  ===== ====== ===== =======
  1   30.0     20.0    0.018/0.018/0.020   0.67    2.00 0.0313 0.167 0.0150
  2   18.0     13.0    0.020/0.020/0.020   0.50    1.50 0.0100 0.125 0.0180

  GLOBAL STREET FLOW-DEPTH CONSTRAINTS:
    1. Relative Flow-Depth =  0.10 FEET
       as (Maximum Allowable Street Flow Depth) - (Top-of-Curb)
    2. (Depth)*(Velocity) Constraint =  6.0 (FT*FT/S)
  *SIZE PIPE WITH A FLOW CAPACITY GREATER THAN
   OR EQUAL TO THE UPSTREAM TRIBUTARY PIPE.*
```

```
****************************************************************************
  FLOW PROCESS FROM NODE    100.00 TO NODE    102.00 IS CODE =  21
----------------------------------------------------------------------------
  >>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<
============================================================================
  *USER SPECIFIED(SUBAREA):
  USER-SPECIFIED RUNOFF COEFFICIENT = .3500
  S.C.S. CURVE NUMBER (AMC II) =   0
  INITIAL SUBAREA FLOW-LENGTH(FEET) =    66.00
  UPSTREAM ELEVATION(FEET) =    500.00
  DOWNSTREAM ELEVATION(FEET) =    496.00
  ELEVATION DIFFERENCE(FEET) =      4.00
  SUBAREA OVERLAND TIME OF FLOW(MIN.) =    6.016
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  7.483
  SUBAREA RUNOFF(CFS) =      0.26
  TOTAL AREA(ACRES) =      0.10   TOTAL RUNOFF(CFS) =      0.26

****************************************************************************
  FLOW PROCESS FROM NODE    102.00 TO NODE    120.00 IS CODE =  51
----------------------------------------------------------------------------
  >>>>>COMPUTE TRAPEZOIDAL CHANNEL FLOW<<<<<
  >>>>>TRAVELTIME THRU SUBAREA (EXISTING ELEMENT)<<<<<
============================================================================
  ELEVATION DATA: UPSTREAM(FEET) =    496.00  DOWNSTREAM(FEET) =    487.00
  CHANNEL LENGTH THRU SUBAREA(FEET) =   365.00   CHANNEL SLOPE =  0.0247
  CHANNEL BASE(FEET) =   10.00   "Z" FACTOR =  12.000
  MANNING'S FACTOR = 0.030   MAXIMUM DEPTH(FEET) =   2.00
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  5.461
  *USER SPECIFIED(SUBAREA):
  USER-SPECIFIED RUNOFF COEFFICIENT = .3800
  S.C.S. CURVE NUMBER (AMC II) =   0
  TRAVEL TIME COMPUTED USING ESTIMATED FLOW(CFS) =       1.73
  TRAVEL TIME THRU SUBAREA BASED ON VELOCITY(FEET/SEC.) =   1.61
  AVERAGE FLOW DEPTH(FEET) =   0.10   TRAVEL TIME(MIN.) =   3.79
  Tc(MIN.) =    9.80
  SUBAREA AREA(ACRES) =     1.40       SUBAREA RUNOFF(CFS) =    2.91
  AREA-AVERAGE RUNOFF COEFFICIENT =  0.378
  TOTAL AREA(ACRES) =        1.5         PEAK FLOW RATE(CFS) =       3.10

  END OF SUBAREA CHANNEL FLOW HYDRAULICS:
  DEPTH(FEET) =  0.14   FLOW VELOCITY(FEET/SEC.) =   1.89
  LONGEST FLOWPATH FROM NODE    100.00 TO NODE    120.00 =     431.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    120.00 TO NODE    120.00 IS CODE =  81
----------------------------------------------------------------------------
  >>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
============================================================================
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  5.461
  *USER SPECIFIED(SUBAREA):
```

```
  USER-SPECIFIED RUNOFF COEFFICIENT = .3500
  S.C.S. CURVE NUMBER (AMC II) =   0
  AREA-AVERAGE RUNOFF COEFFICIENT = 0.3668
  SUBAREA AREA(ACRES) =    1.00   SUBAREA RUNOFF(CFS) =    1.91
  TOTAL AREA(ACRES) =        2.5   TOTAL RUNOFF(CFS) =       5.01
  TC(MIN.) =    9.80

****************************************************************************
  FLOW PROCESS FROM NODE    120.00 TO NODE    150.00 IS CODE =  51
----------------------------------------------------------------------------
  >>>>>COMPUTE TRAPEZOIDAL CHANNEL FLOW<<<<<
  >>>>>TRAVELTIME THRU SUBAREA (EXISTING ELEMENT)<<<<<
============================================================================
  ELEVATION DATA: UPSTREAM(FEET) =    487.00  DOWNSTREAM(FEET) =    480.00
  CHANNEL LENGTH THRU SUBAREA(FEET) =   298.00   CHANNEL SLOPE =  0.0235
  CHANNEL BASE(FEET) =   10.00   "Z" FACTOR =  12.000
  MANNING'S FACTOR = 0.015   MAXIMUM DEPTH(FEET) =   2.00
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  5.042
  *USER SPECIFIED(SUBAREA):
  USER-SPECIFIED RUNOFF COEFFICIENT = .7900
  S.C.S. CURVE NUMBER (AMC II) =   0
  TRAVEL TIME COMPUTED USING ESTIMATED FLOW(CFS) =       6.40
  TRAVEL TIME THRU SUBAREA BASED ON VELOCITY(FEET/SEC.) =   3.85
  AVERAGE FLOW DEPTH(FEET) =   0.14   TRAVEL TIME(MIN.) =   1.29
  Tc(MIN.) =   11.10
  SUBAREA AREA(ACRES) =     0.70       SUBAREA RUNOFF(CFS) =    2.79
  AREA-AVERAGE RUNOFF COEFFICIENT =  0.459
  TOTAL AREA(ACRES) =        3.2         PEAK FLOW RATE(CFS) =       7.41

  END OF SUBAREA CHANNEL FLOW HYDRAULICS:
  DEPTH(FEET) =  0.16   FLOW VELOCITY(FEET/SEC.) =   4.02
  LONGEST FLOWPATH FROM NODE    100.00 TO NODE    150.00 =     729.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    150.00 TO NODE    199.00 IS CODE =  51
----------------------------------------------------------------------------
  >>>>>COMPUTE TRAPEZOIDAL CHANNEL FLOW<<<<<
  >>>>>TRAVELTIME THRU SUBAREA (EXISTING ELEMENT)<<<<<
============================================================================
  ELEVATION DATA: UPSTREAM(FEET) =    480.00  DOWNSTREAM(FEET) =    472.00
  CHANNEL LENGTH THRU SUBAREA(FEET) =   264.00   CHANNEL SLOPE =  0.0303
  CHANNEL BASE(FEET) =    2.00   "Z" FACTOR =   3.000
  MANNING'S FACTOR = 0.030   MAXIMUM DEPTH(FEET) =   5.00
  CHANNEL FLOW THRU SUBAREA(CFS) =       7.41
  FLOW VELOCITY(FEET/SEC.) =   4.23   FLOW DEPTH(FEET) =   0.50
  TRAVEL TIME(MIN.) =   1.04   Tc(MIN.) =   12.14
  LONGEST FLOWPATH FROM NODE    100.00 TO NODE    199.00 =     993.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    199.00 TO NODE    199.00 IS CODE =  81
```

```
----------------------------------------------------------------------------
  >>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
============================================================================
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  4.759
  *USER SPECIFIED(SUBAREA):
  USER-SPECIFIED RUNOFF COEFFICIENT = .3500
  S.C.S. CURVE NUMBER (AMC II) =   0
  AREA-AVERAGE RUNOFF COEFFICIENT = 0.4278
  SUBAREA AREA(ACRES) =    1.30   SUBAREA RUNOFF(CFS) =    2.17
  TOTAL AREA(ACRES) =        4.5   TOTAL RUNOFF(CFS) =       9.16
  TC(MIN.) =   12.14

****************************************************************************
  FLOW PROCESS FROM NODE    199.00 TO NODE    199.00 IS CODE =  81
----------------------------------------------------------------------------
  >>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
============================================================================
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  4.759
  *USER SPECIFIED(SUBAREA):
  USER-SPECIFIED RUNOFF COEFFICIENT = .6300
  S.C.S. CURVE NUMBER (AMC II) =   0
  AREA-AVERAGE RUNOFF COEFFICIENT = 0.4516
  SUBAREA AREA(ACRES) =    0.60   SUBAREA RUNOFF(CFS) =    1.80
  TOTAL AREA(ACRES) =        5.1   TOTAL RUNOFF(CFS) =      10.96
  TC(MIN.) =   12.14

****************************************************************************
  FLOW PROCESS FROM NODE    199.00 TO NODE    199.00 IS CODE =  81
----------------------------------------------------------------------------
  >>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
============================================================================
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  4.759
  *USER SPECIFIED(SUBAREA):
  USER-SPECIFIED RUNOFF COEFFICIENT = .5700
  S.C.S. CURVE NUMBER (AMC II) =   0
  AREA-AVERAGE RUNOFF COEFFICIENT = 0.4538
  SUBAREA AREA(ACRES) =    0.10   SUBAREA RUNOFF(CFS) =    0.27
  TOTAL AREA(ACRES) =        5.2   TOTAL RUNOFF(CFS) =      11.23
  TC(MIN.) =   12.14
============================================================================
  END OF STUDY SUMMARY:
  TOTAL AREA(ACRES)     =        5.2  TC(MIN.) =     12.14
  PEAK FLOW RATE(CFS)   =      11.23
============================================================================
============================================================================
  END OF RATIONAL METHOD ANALYSIS
```

APPENDIX B

HYDROLOGY ANALYSIS

100 YEAR, RATIONAL METHOD AND MODIFIED RATIONAL METHOD

(POST-PROJECT)

```
****************************************************************************

             RATIONAL METHOD HYDROLOGY COMPUTER PROGRAM PACKAGE
             Reference: SAN DIEGO COUNTY FLOOD CONTROL DISTRICT
                          2003,1985,1981 HYDROLOGY MANUAL
          (c) Copyright 1982-2014 Advanced Engineering Software (aes)
              Ver. 21.0 Release Date: 06/01/2014  License ID 1261

                             Analysis prepared by:

                            RICK ENGINEERING COMPANY
                                5620 Friars Road
                          San Diego, California 92110
                        619-291-0707   Fax 619-291-4165

 ************************** DESCRIPTION OF STUDY **************************
* J-19253B VISTA II BALLFIELDS                                             *
* 100YR, 6-HR POST-PROJECT CONDITION                                       *
* J:\19253B\WR\HYDROLOGY\RATIONALMETHOD\...                                *
 **************************************************************************

  FILE NAME: V2B1P00.RAT
  TIME/DATE OF STUDY: 12:58 10/03/2023
----------------------------------------------------------------------------
  USER SPECIFIED HYDROLOGY AND HYDRAULIC MODEL INFORMATION:
----------------------------------------------------------------------------
  2003 SAN DIEGO MANUAL CRITERIA

  USER SPECIFIED STORM EVENT(YEAR) = 100.00
  6-HOUR DURATION PRECIPITATION (INCHES) =   3.200
  SPECIFIED MINIMUM PIPE SIZE(INCH) =  18.00
  SPECIFIED PERCENT OF GRADIENTS(DECIMAL) TO USE FOR FRICTION SLOPE = 0.90
  SAN DIEGO HYDROLOGY MANUAL "C"-VALUES USED FOR RATIONAL METHOD
  NOTE: USE MODIFIED RATIONAL METHOD PROCEDURES FOR CONFLUENCE ANALYSIS
  *USER-DEFINED STREET-SECTIONS FOR COUPLED PIPEFLOW AND STREETFLOW MODEL*
     HALF-  CROWN TO   STREET-CROSSFALL:   CURB  GUTTER-GEOMETRIES:  MANNING
     WIDTH  CROSSFALL  IN-  / OUT-/PARK-  HEIGHT  WIDTH  LIP   HIKE  FACTOR
NO.   (FT)     (FT)    SIDE / SIDE/ WAY    (FT)    (FT)  (FT)  (FT)    (n)
===  =====  =========  =================  ======  ===== ====== ===== =======
  1   30.0     20.0    0.018/0.018/0.020   0.67    2.00 0.0313 0.167 0.0150
  2   18.0     13.0    0.020/0.020/0.020   0.50    1.50 0.0100 0.125 0.0180

  GLOBAL STREET FLOW-DEPTH CONSTRAINTS:
    1. Relative Flow-Depth =  0.10 FEET
       as (Maximum Allowable Street Flow Depth) - (Top-of-Curb)
    2. (Depth)*(Velocity) Constraint =  6.0 (FT*FT/S)
  *SIZE PIPE WITH A FLOW CAPACITY GREATER THAN
   OR EQUAL TO THE UPSTREAM TRIBUTARY PIPE.*
```

```
****************************************************************************
  FLOW PROCESS FROM NODE    100.00 TO NODE    102.00 IS CODE =  21
----------------------------------------------------------------------------
  >>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<
============================================================================
  *USER SPECIFIED(SUBAREA):
  USER-SPECIFIED RUNOFF COEFFICIENT = .8500
  S.C.S. CURVE NUMBER (AMC II) =   0
  INITIAL SUBAREA FLOW-LENGTH(FEET) =   100.00
  UPSTREAM ELEVATION(FEET) =    504.00
  DOWNSTREAM ELEVATION(FEET) =    503.00
  ELEVATION DIFFERENCE(FEET) =      1.00
  SUBAREA OVERLAND TIME OF FLOW(MIN.) =    3.486
  WARNING: INITIAL SUBAREA FLOW PATH LENGTH IS GREATER THAN
           THE MAXIMUM OVERLAND FLOW LENGTH =    60.00
           (Reference: Table 3-1B of Hydrology Manual)
           THE MAXIMUM OVERLAND FLOW LENGTH IS USED IN Tc CALCULATION!
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  8.431
  NOTE: RAINFALL INTENSITY IS BASED ON Tc = 5-MINUTE.
  SUBAREA RUNOFF(CFS) =      0.72
  TOTAL AREA(ACRES) =      0.10   TOTAL RUNOFF(CFS) =      0.72

****************************************************************************
  FLOW PROCESS FROM NODE    102.00 TO NODE    105.00 IS CODE =  61
----------------------------------------------------------------------------
  >>>>>COMPUTE STREET FLOW TRAVEL TIME THRU SUBAREA<<<<<
  >>>>>(STANDARD CURB SECTION USED)<<<<<
============================================================================
  UPSTREAM ELEVATION(FEET) =  503.00  DOWNSTREAM ELEVATION(FEET) =  498.50
  STREET LENGTH(FEET) =   120.00   CURB HEIGHT(INCHES) =  6.0
  STREET HALFWIDTH(FEET) = 13.00

  DISTANCE FROM CROWN TO CROSSFALL GRADEBREAK(FEET) =   8.00
  INSIDE STREET CROSSFALL(DECIMAL) =  0.020
  OUTSIDE STREET CROSSFALL(DECIMAL)  =  0.020

  SPECIFIED NUMBER OF HALFSTREETS CARRYING RUNOFF =  1
  STREET PARKWAY CROSSFALL(DECIMAL)  =  0.020
  Manning's FRICTION FACTOR for Streetflow Section(curb-to-curb) =   0.0180
  Manning's FRICTION FACTOR for Back-of-Walk Flow Section =   0.0200

    **TRAVEL TIME COMPUTED USING ESTIMATED FLOW(CFS) =       0.90
    STREETFLOW MODEL RESULTS USING ESTIMATED FLOW:
    STREET FLOW DEPTH(FEET) =  0.22
    HALFSTREET FLOOD WIDTH(FEET) =    4.64
    AVERAGE FLOW VELOCITY(FEET/SEC.) =    2.68
    PRODUCT OF DEPTH&VELOCITY(FT*FT/SEC.) =    0.59
  STREET FLOW TRAVEL TIME(MIN.) =   0.75   Tc(MIN.) =    4.23
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  8.431
  NOTE: RAINFALL INTENSITY IS BASED ON Tc = 5-MINUTE.
```

```
 *USER SPECIFIED(SUBAREA):
 USER-SPECIFIED RUNOFF COEFFICIENT = .8500
 S.C.S. CURVE NUMBER (AMC II) =   0
 AREA-AVERAGE RUNOFF COEFFICIENT =  0.850
 SUBAREA AREA(ACRES) =    0.05      SUBAREA RUNOFF(CFS) =    0.36
 TOTAL AREA(ACRES) =        0.2        PEAK FLOW RATE(CFS) =       1.07

 END OF SUBAREA STREET FLOW HYDRAULICS:
 DEPTH(FEET) = 0.23   HALFSTREET FLOOD WIDTH(FEET) =   5.22
 FLOW VELOCITY(FEET/SEC.) =  2.75   DEPTH*VELOCITY(FT*FT/SEC.) =   0.63
 LONGEST FLOWPATH FROM NODE    100.00 TO NODE    105.00 =     220.00 FEET.

****************************************************************************
 FLOW PROCESS FROM NODE    105.00 TO NODE    115.00 IS CODE =  61
----------------------------------------------------------------------------
 >>>>>COMPUTE STREET FLOW TRAVEL TIME THRU SUBAREA<<<<<
 >>>>>(STANDARD CURB SECTION USED)<<<<<
============================================================================
 UPSTREAM ELEVATION(FEET) =  498.50  DOWNSTREAM ELEVATION(FEET) =  488.50
 STREET LENGTH(FEET) =   411.00   CURB HEIGHT(INCHES) =  6.0
 STREET HALFWIDTH(FEET) = 13.00

 DISTANCE FROM CROWN TO CROSSFALL GRADEBREAK(FEET) =   8.00
 INSIDE STREET CROSSFALL(DECIMAL) =  0.020
 OUTSIDE STREET CROSSFALL(DECIMAL)  =  0.020

 SPECIFIED NUMBER OF HALFSTREETS CARRYING RUNOFF =  1
 STREET PARKWAY CROSSFALL(DECIMAL)  =  0.020
 Manning's FRICTION FACTOR for Streetflow Section(curb-to-curb) =   0.0180
 Manning's FRICTION FACTOR for Back-of-Walk Flow Section =   0.0200

   **TRAVEL TIME COMPUTED USING ESTIMATED FLOW(CFS) =       1.89
   STREETFLOW MODEL RESULTS USING ESTIMATED FLOW:
   STREET FLOW DEPTH(FEET) =  0.28
   HALFSTREET FLOOD WIDTH(FEET) =    7.84
   AVERAGE FLOW VELOCITY(FEET/SEC.) =    2.58
   PRODUCT OF DEPTH&VELOCITY(FT*FT/SEC.) =    0.73
 STREET FLOW TRAVEL TIME(MIN.) =   2.65   Tc(MIN.) =    6.89
  100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  6.859
 *USER SPECIFIED(SUBAREA):
 USER-SPECIFIED RUNOFF COEFFICIENT = .7900
 S.C.S. CURVE NUMBER (AMC II) =   0
 AREA-AVERAGE RUNOFF COEFFICIENT =  0.810
 SUBAREA AREA(ACRES) =    0.30      SUBAREA RUNOFF(CFS) =    1.63
 TOTAL AREA(ACRES) =        0.5        PEAK FLOW RATE(CFS) =       2.50

 END OF SUBAREA STREET FLOW HYDRAULICS:
 DEPTH(FEET) = 0.30   HALFSTREET FLOOD WIDTH(FEET) =   8.91
 FLOW VELOCITY(FEET/SEC.) =  2.74   DEPTH*VELOCITY(FT*FT/SEC.) =   0.83
 LONGEST FLOWPATH FROM NODE    100.00 TO NODE    115.00 =     631.00 FEET.
```

```
****************************************************************************
  FLOW PROCESS FROM NODE    115.00 TO NODE    120.00 IS CODE =  31
----------------------------------------------------------------------------
  >>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
  >>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<
============================================================================
  ELEVATION DATA: UPSTREAM(FEET) =   489.00  DOWNSTREAM(FEET) =   487.00
  FLOW LENGTH(FEET) =   100.00   MANNING'S N =  0.013
  ESTIMATED PIPE DIAMETER(INCH) INCREASED TO 18.000
  DEPTH OF FLOW IN  18.0 INCH PIPE IS   5.1 INCHES
  PIPE-FLOW VELOCITY(FEET/SEC.) =   6.03
  ESTIMATED PIPE DIAMETER(INCH) =  18.00    NUMBER OF PIPES =   1
  PIPE-FLOW(CFS) =       2.50
  PIPE TRAVEL TIME(MIN.) =   0.28    Tc(MIN.) =    7.16
  LONGEST FLOWPATH FROM NODE    100.00 TO NODE    120.00 =     731.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    120.00 TO NODE    120.00 IS CODE =   1
----------------------------------------------------------------------------
  >>>>>DESIGNATE INDEPENDENT STREAM FOR CONFLUENCE<<<<<
============================================================================
  TOTAL NUMBER OF STREAMS =  2
  CONFLUENCE VALUES USED FOR INDEPENDENT STREAM  1 ARE:
  TIME OF CONCENTRATION(MIN.) =    7.16
  RAINFALL INTENSITY(INCH/HR) =   6.69
  TOTAL STREAM AREA(ACRES) =     0.45
  PEAK FLOW RATE(CFS) AT CONFLUENCE =      2.50

****************************************************************************
  FLOW PROCESS FROM NODE    110.00 TO NODE    112.00 IS CODE =  21
----------------------------------------------------------------------------
  >>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<
============================================================================
  *USER SPECIFIED(SUBAREA):
  USER-SPECIFIED RUNOFF COEFFICIENT = .8500
  S.C.S. CURVE NUMBER (AMC II) =   0
  INITIAL SUBAREA FLOW-LENGTH(FEET) =   100.00
  UPSTREAM ELEVATION(FEET) =    504.00
  DOWNSTREAM ELEVATION(FEET) =    503.00
  ELEVATION DIFFERENCE(FEET) =      1.00
  SUBAREA OVERLAND TIME OF FLOW(MIN.) =    3.486
  WARNING: INITIAL SUBAREA FLOW PATH LENGTH IS GREATER THAN
           THE MAXIMUM OVERLAND FLOW LENGTH =    60.00
           (Reference: Table 3-1B of Hydrology Manual)
           THE MAXIMUM OVERLAND FLOW LENGTH IS USED IN Tc CALCULATION!
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  8.431
  NOTE: RAINFALL INTENSITY IS BASED ON Tc = 5-MINUTE.
  SUBAREA RUNOFF(CFS) =      0.72
  TOTAL AREA(ACRES) =      0.10   TOTAL RUNOFF(CFS) =      0.72
```

```
****************************************************************************
  FLOW PROCESS FROM NODE    112.00 TO NODE    115.00 IS CODE =  61
----------------------------------------------------------------------------
  >>>>>COMPUTE STREET FLOW TRAVEL TIME THRU SUBAREA<<<<<
  >>>>>(STANDARD CURB SECTION USED)<<<<<
============================================================================
  UPSTREAM ELEVATION(FEET) =  503.00  DOWNSTREAM ELEVATION(FEET) =  490.50
  STREET LENGTH(FEET) =   411.00   CURB HEIGHT(INCHES) =  6.0
  STREET HALFWIDTH(FEET) = 13.00

  DISTANCE FROM CROWN TO CROSSFALL GRADEBREAK(FEET) =   8.00
  INSIDE STREET CROSSFALL(DECIMAL) =  0.020
  OUTSIDE STREET CROSSFALL(DECIMAL)  =  0.020

  SPECIFIED NUMBER OF HALFSTREETS CARRYING RUNOFF =  1
  STREET PARKWAY CROSSFALL(DECIMAL)  =  0.020
  Manning's FRICTION FACTOR for Streetflow Section(curb-to-curb) =   0.0180
  Manning's FRICTION FACTOR for Back-of-Walk Flow Section =   0.0200

    **TRAVEL TIME COMPUTED USING ESTIMATED FLOW(CFS) =       2.87
    STREETFLOW MODEL RESULTS USING ESTIMATED FLOW:
    STREET FLOW DEPTH(FEET) =  0.31
    HALFSTREET FLOOD WIDTH(FEET) =    9.03
    AVERAGE FLOW VELOCITY(FEET/SEC.) =    3.08
    PRODUCT OF DEPTH&VELOCITY(FT*FT/SEC.) =    0.94
  STREET FLOW TRAVEL TIME(MIN.) =   2.23   Tc(MIN.) =    5.71
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  7.738
  *USER SPECIFIED(SUBAREA):
  USER-SPECIFIED RUNOFF COEFFICIENT = .7900
  S.C.S. CURVE NUMBER (AMC II) =   0
  AREA-AVERAGE RUNOFF COEFFICIENT =  0.797
  SUBAREA AREA(ACRES) =    0.70      SUBAREA RUNOFF(CFS) =    4.28
  TOTAL AREA(ACRES) =        0.8        PEAK FLOW RATE(CFS) =       4.94

  END OF SUBAREA STREET FLOW HYDRAULICS:
  DEPTH(FEET) = 0.35   HALFSTREET FLOOD WIDTH(FEET) =  11.34
  FLOW VELOCITY(FEET/SEC.) =  3.51   DEPTH*VELOCITY(FT*FT/SEC.) =   1.24
  LONGEST FLOWPATH FROM NODE    110.00 TO NODE    115.00 =     511.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    115.00 TO NODE    117.00 IS CODE =  41
----------------------------------------------------------------------------
  >>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
  >>>>>USING USER-SPECIFIED PIPESIZE (EXISTING ELEMENT)<<<<<
============================================================================
  ELEVATION DATA: UPSTREAM(FEET) =   487.00  DOWNSTREAM(FEET) =   486.00
  FLOW LENGTH(FEET) =   105.00   MANNING'S N =  0.013
  DEPTH OF FLOW IN  18.0 INCH PIPE IS   9.1 INCHES
  PIPE-FLOW VELOCITY(FEET/SEC.) =   5.52
```

```
  GIVEN PIPE DIAMETER(INCH) =  18.00    NUMBER OF PIPES =   1
  PIPE-FLOW(CFS) =       4.94
  PIPE TRAVEL TIME(MIN.) =   0.32    Tc(MIN.) =    6.03
  LONGEST FLOWPATH FROM NODE    110.00 TO NODE    117.00 =     616.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    117.00 TO NODE    117.00 IS CODE =  81
----------------------------------------------------------------------------
  >>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
============================================================================
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  7.473
  *USER SPECIFIED(SUBAREA):
  USER-SPECIFIED RUNOFF COEFFICIENT = .7900
  S.C.S. CURVE NUMBER (AMC II) =   0
  AREA-AVERAGE RUNOFF COEFFICIENT = 0.7943
  SUBAREA AREA(ACRES) =    0.60   SUBAREA RUNOFF(CFS) =    3.54
  TOTAL AREA(ACRES) =        1.4   TOTAL RUNOFF(CFS) =       8.31
  TC(MIN.) =    6.03

****************************************************************************
  FLOW PROCESS FROM NODE    117.00 TO NODE    120.00 IS CODE =  41
----------------------------------------------------------------------------
  >>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
  >>>>>USING USER-SPECIFIED PIPESIZE (EXISTING ELEMENT)<<<<<
============================================================================
  ELEVATION DATA: UPSTREAM(FEET) =   486.00  DOWNSTREAM(FEET) =   485.50
  FLOW LENGTH(FEET) =    26.00   MANNING'S N =  0.013
  DEPTH OF FLOW IN  18.0 INCH PIPE IS  10.0 INCHES
  PIPE-FLOW VELOCITY(FEET/SEC.) =   8.20
  GIVEN PIPE DIAMETER(INCH) =  18.00    NUMBER OF PIPES =   1
  PIPE-FLOW(CFS) =       8.31
  PIPE TRAVEL TIME(MIN.) =   0.05    Tc(MIN.) =    6.08
  LONGEST FLOWPATH FROM NODE    110.00 TO NODE    120.00 =     642.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    120.00 TO NODE    120.00 IS CODE =   1
----------------------------------------------------------------------------
  >>>>>DESIGNATE INDEPENDENT STREAM FOR CONFLUENCE<<<<<
  >>>>>AND COMPUTE VARIOUS CONFLUENCED STREAM VALUES<<<<<
============================================================================
  TOTAL NUMBER OF STREAMS =  2
  CONFLUENCE VALUES USED FOR INDEPENDENT STREAM  2 ARE:
  TIME OF CONCENTRATION(MIN.) =    6.08
  RAINFALL INTENSITY(INCH/HR) =   7.43
  TOTAL STREAM AREA(ACRES) =     1.40
  PEAK FLOW RATE(CFS) AT CONFLUENCE =      8.31

  ** CONFLUENCE DATA **
  STREAM     RUNOFF       Tc      INTENSITY      AREA
  NUMBER      (CFS)     (MIN.)   (INCH/HOUR)    (ACRE)
```

```
     1        2.50     7.16        6.687          0.45
     2        8.31     6.08        7.431          1.40

 RAINFALL INTENSITY AND TIME OF CONCENTRATION RATIO
 CONFLUENCE FORMULA USED FOR  2 STREAMS.

 ** PEAK FLOW RATE TABLE **
 STREAM     RUNOFF      Tc      INTENSITY
 NUMBER      (CFS)    (MIN.)   (INCH/HOUR)
     1       10.43     6.08       7.431
     2        9.98     7.16       6.687

 COMPUTED CONFLUENCE ESTIMATES ARE AS FOLLOWS:
 PEAK FLOW RATE(CFS) =      10.43   Tc(MIN.) =    6.08
 TOTAL AREA(ACRES) =        1.9
 LONGEST FLOWPATH FROM NODE    100.00 TO NODE    120.00 =     731.00 FEET.

****************************************************************************
 FLOW PROCESS FROM NODE    120.00 TO NODE    120.00 IS CODE =  81
----------------------------------------------------------------------------
 >>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
============================================================================
  100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  7.431
 *USER SPECIFIED(SUBAREA):
 USER-SPECIFIED RUNOFF COEFFICIENT = .6800
 S.C.S. CURVE NUMBER (AMC II) =   0
 AREA-AVERAGE RUNOFF COEFFICIENT = 0.7771
 SUBAREA AREA(ACRES) =    0.40   SUBAREA RUNOFF(CFS) =    2.02
 TOTAL AREA(ACRES) =        2.3   TOTAL RUNOFF(CFS) =      12.99
 TC(MIN.) =    6.08

****************************************************************************
 FLOW PROCESS FROM NODE    120.00 TO NODE    120.00 IS CODE =  81
----------------------------------------------------------------------------
 >>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
============================================================================
  100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  7.431
 *USER SPECIFIED(SUBAREA):
 USER-SPECIFIED RUNOFF COEFFICIENT = .7900
 S.C.S. CURVE NUMBER (AMC II) =   0
 AREA-AVERAGE RUNOFF COEFFICIENT = 0.7782
 SUBAREA AREA(ACRES) =    0.20   SUBAREA RUNOFF(CFS) =    1.17
 TOTAL AREA(ACRES) =        2.5   TOTAL RUNOFF(CFS) =      14.17
 TC(MIN.) =    6.08

****************************************************************************
 FLOW PROCESS FROM NODE    120.00 TO NODE    120.00 IS CODE =  81
----------------------------------------------------------------------------
 >>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
============================================================================
```

```
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  7.431
  *USER SPECIFIED(SUBAREA):
  USER-SPECIFIED RUNOFF COEFFICIENT = .3800
  S.C.S. CURVE NUMBER (AMC II) =   0
  AREA-AVERAGE RUNOFF COEFFICIENT = 0.7481
  SUBAREA AREA(ACRES) =    0.20   SUBAREA RUNOFF(CFS) =    0.56
  TOTAL AREA(ACRES) =        2.7   TOTAL RUNOFF(CFS) =      14.73
  TC(MIN.) =    6.08

****************************************************************************
  FLOW PROCESS FROM NODE    120.00 TO NODE    120.00 IS CODE =  81
----------------------------------------------------------------------------
  >>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
============================================================================
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  7.431
  *USER SPECIFIED(SUBAREA):
  USER-SPECIFIED RUNOFF COEFFICIENT = .3800
  S.C.S. CURVE NUMBER (AMC II) =   0
  AREA-AVERAGE RUNOFF COEFFICIENT = 0.6897
  SUBAREA AREA(ACRES) =    0.50   SUBAREA RUNOFF(CFS) =    1.41
  TOTAL AREA(ACRES) =        3.2   TOTAL RUNOFF(CFS) =      16.14
  TC(MIN.) =    6.08

****************************************************************************
  FLOW PROCESS FROM NODE    120.00 TO NODE    199.00 IS CODE =  41
----------------------------------------------------------------------------
  >>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
  >>>>>USING USER-SPECIFIED PIPESIZE (EXISTING ELEMENT)<<<<<
============================================================================
  ELEVATION DATA: UPSTREAM(FEET) =   474.00  DOWNSTREAM(FEET) =   468.80
  FLOW LENGTH(FEET) =   380.00   MANNING'S N =  0.013
  DEPTH OF FLOW IN  24.0 INCH PIPE IS  14.0 INCHES
  PIPE-FLOW VELOCITY(FEET/SEC.) =   8.50
  GIVEN PIPE DIAMETER(INCH) =  24.00    NUMBER OF PIPES =   1
  PIPE-FLOW(CFS) =      16.14
  PIPE TRAVEL TIME(MIN.) =   0.75    Tc(MIN.) =    6.83
  LONGEST FLOWPATH FROM NODE    100.00 TO NODE    199.00 =    1111.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    199.00 TO NODE    199.00 IS CODE =   1
----------------------------------------------------------------------------
  >>>>>DESIGNATE INDEPENDENT STREAM FOR CONFLUENCE<<<<<
============================================================================
  TOTAL NUMBER OF STREAMS =  2
  CONFLUENCE VALUES USED FOR INDEPENDENT STREAM  1 ARE:
  TIME OF CONCENTRATION(MIN.) =    6.83
  RAINFALL INTENSITY(INCH/HR) =   6.90
  TOTAL STREAM AREA(ACRES) =     3.15
  PEAK FLOW RATE(CFS) AT CONFLUENCE =     16.14
```

```
****************************************************************************
  FLOW PROCESS FROM NODE    130.00 TO NODE    132.00 IS CODE =  21
----------------------------------------------------------------------------
  >>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<
============================================================================
  *USER SPECIFIED(SUBAREA):
  USER-SPECIFIED RUNOFF COEFFICIENT = .7900
  S.C.S. CURVE NUMBER (AMC II) =   0
  INITIAL SUBAREA FLOW-LENGTH(FEET) =   100.00
  UPSTREAM ELEVATION(FEET) =    484.00
  DOWNSTREAM ELEVATION(FEET) =    483.00
  ELEVATION DIFFERENCE(FEET) =      1.00
  SUBAREA OVERLAND TIME OF FLOW(MIN.) =    4.499
  WARNING: INITIAL SUBAREA FLOW PATH LENGTH IS GREATER THAN
           THE MAXIMUM OVERLAND FLOW LENGTH =    65.00
           (Reference: Table 3-1B of Hydrology Manual)
           THE MAXIMUM OVERLAND FLOW LENGTH IS USED IN Tc CALCULATION!
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  8.431
  NOTE: RAINFALL INTENSITY IS BASED ON Tc = 5-MINUTE.
  SUBAREA RUNOFF(CFS) =      0.67
  TOTAL AREA(ACRES) =      0.10   TOTAL RUNOFF(CFS) =      0.67

****************************************************************************
  FLOW PROCESS FROM NODE    132.00 TO NODE    135.00 IS CODE =  61
----------------------------------------------------------------------------
  >>>>>COMPUTE STREET FLOW TRAVEL TIME THRU SUBAREA<<<<<
  >>>>>(STANDARD CURB SECTION USED)<<<<<
============================================================================
  UPSTREAM ELEVATION(FEET) =  483.00  DOWNSTREAM ELEVATION(FEET) =  480.00
  STREET LENGTH(FEET) =   210.00   CURB HEIGHT(INCHES) =  6.0
  STREET HALFWIDTH(FEET) = 13.00

  DISTANCE FROM CROWN TO CROSSFALL GRADEBREAK(FEET) =   8.00
  INSIDE STREET CROSSFALL(DECIMAL) =  0.020
  OUTSIDE STREET CROSSFALL(DECIMAL)  =  0.020

  SPECIFIED NUMBER OF HALFSTREETS CARRYING RUNOFF =  1
  STREET PARKWAY CROSSFALL(DECIMAL)  =  0.020
  Manning's FRICTION FACTOR for Streetflow Section(curb-to-curb) =   0.0180
  Manning's FRICTION FACTOR for Back-of-Walk Flow Section =   0.0200

    **TRAVEL TIME COMPUTED USING ESTIMATED FLOW(CFS) =       1.83
    STREETFLOW MODEL RESULTS USING ESTIMATED FLOW:
    STREET FLOW DEPTH(FEET) =  0.30
    HALFSTREET FLOOD WIDTH(FEET) =    8.72
    AVERAGE FLOW VELOCITY(FEET/SEC.) =    2.08
    PRODUCT OF DEPTH&VELOCITY(FT*FT/SEC.) =    0.63
  STREET FLOW TRAVEL TIME(MIN.) =   1.68   Tc(MIN.) =    6.18
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  7.356
  *USER SPECIFIED(SUBAREA):
```

```
  USER-SPECIFIED RUNOFF COEFFICIENT = .7900
  S.C.S. CURVE NUMBER (AMC II) =   0
  AREA-AVERAGE RUNOFF COEFFICIENT =  0.790
  SUBAREA AREA(ACRES) =    0.40      SUBAREA RUNOFF(CFS) =    2.32
  TOTAL AREA(ACRES) =        0.5        PEAK FLOW RATE(CFS) =       2.91

  END OF SUBAREA STREET FLOW HYDRAULICS:
  DEPTH(FEET) = 0.34   HALFSTREET FLOOD WIDTH(FEET) =  10.66
  FLOW VELOCITY(FEET/SEC.) =  2.32   DEPTH*VELOCITY(FT*FT/SEC.) =   0.79
  LONGEST FLOWPATH FROM NODE    130.00 TO NODE    135.00 =     310.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    135.00 TO NODE    140.00 IS CODE =  41
----------------------------------------------------------------------------
  >>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
  >>>>>USING USER-SPECIFIED PIPESIZE (EXISTING ELEMENT)<<<<<
============================================================================
  ELEVATION DATA: UPSTREAM(FEET) =   477.00  DOWNSTREAM(FEET) =   476.50
  FLOW LENGTH(FEET) =    26.00   MANNING'S N =  0.013
  DEPTH OF FLOW IN  18.0 INCH PIPE IS   5.6 INCHES
  PIPE-FLOW VELOCITY(FEET/SEC.) =   6.20
  GIVEN PIPE DIAMETER(INCH) =  18.00    NUMBER OF PIPES =   1
  PIPE-FLOW(CFS) =       2.91
  PIPE TRAVEL TIME(MIN.) =   0.07    Tc(MIN.) =    6.25
  LONGEST FLOWPATH FROM NODE    130.00 TO NODE    140.00 =     336.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    140.00 TO NODE    140.00 IS CODE =  81
----------------------------------------------------------------------------
  >>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
============================================================================
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  7.303
  *USER SPECIFIED(SUBAREA):
  USER-SPECIFIED RUNOFF COEFFICIENT = .7900
  S.C.S. CURVE NUMBER (AMC II) =   0
  AREA-AVERAGE RUNOFF COEFFICIENT = 0.7900
  SUBAREA AREA(ACRES) =    0.10   SUBAREA RUNOFF(CFS) =    0.58
  TOTAL AREA(ACRES) =        0.6   TOTAL RUNOFF(CFS) =       3.46
  TC(MIN.) =    6.25

****************************************************************************
  FLOW PROCESS FROM NODE    140.00 TO NODE    140.00 IS CODE =  81
----------------------------------------------------------------------------
  >>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
============================================================================
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  7.303
  *USER SPECIFIED(SUBAREA):
  USER-SPECIFIED RUNOFF COEFFICIENT = .7900
  S.C.S. CURVE NUMBER (AMC II) =   0
  AREA-AVERAGE RUNOFF COEFFICIENT = 0.7900
```

```
  SUBAREA AREA(ACRES) =    0.50   SUBAREA RUNOFF(CFS) =    2.88
  TOTAL AREA(ACRES) =        1.1   TOTAL RUNOFF(CFS) =      6.35
  TC(MIN.) =    6.25

****************************************************************************
  FLOW PROCESS FROM NODE    140.00 TO NODE    140.00 IS CODE =  81
----------------------------------------------------------------------------
  >>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
============================================================================
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  7.303
  *USER SPECIFIED(SUBAREA):
  USER-SPECIFIED RUNOFF COEFFICIENT = .7900
  S.C.S. CURVE NUMBER (AMC II) =   0
  AREA-AVERAGE RUNOFF COEFFICIENT = 0.7900
  SUBAREA AREA(ACRES) =    0.60   SUBAREA RUNOFF(CFS) =    3.46
  TOTAL AREA(ACRES) =        1.7   TOTAL RUNOFF(CFS) =      9.81
  TC(MIN.) =    6.25

****************************************************************************
  FLOW PROCESS FROM NODE    140.00 TO NODE    145.00 IS CODE =  41
----------------------------------------------------------------------------
  >>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
  >>>>>USING USER-SPECIFIED PIPESIZE (EXISTING ELEMENT)<<<<<
============================================================================
  ELEVATION DATA: UPSTREAM(FEET) =   471.50  DOWNSTREAM(FEET) =   471.00
  FLOW LENGTH(FEET) =   125.00   MANNING'S N =  0.013
  DEPTH OF FLOW IN  24.0 INCH PIPE IS  15.1 INCHES
  PIPE-FLOW VELOCITY(FEET/SEC.) =   4.72
  GIVEN PIPE DIAMETER(INCH) =  24.00    NUMBER OF PIPES =   1
  PIPE-FLOW(CFS) =       9.81
  PIPE TRAVEL TIME(MIN.) =   0.44    Tc(MIN.) =    6.69
  LONGEST FLOWPATH FROM NODE    130.00 TO NODE    145.00 =    461.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    145.00 TO NODE    150.00 IS CODE =  51
----------------------------------------------------------------------------
  >>>>>COMPUTE TRAPEZOIDAL CHANNEL FLOW<<<<<
  >>>>>TRAVELTIME THRU SUBAREA (EXISTING ELEMENT)<<<<<
============================================================================
  ELEVATION DATA: UPSTREAM(FEET) =    471.00  DOWNSTREAM(FEET) =    470.90
  CHANNEL LENGTH THRU SUBAREA(FEET) =   100.00   CHANNEL SLOPE =  0.0010
  CHANNEL BASE(FEET) =   10.00   "Z" FACTOR =  12.000
  MANNING'S FACTOR = 0.060   MAXIMUM DEPTH(FEET) =   3.00
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  5.480
  *USER SPECIFIED(SUBAREA):
  USER-SPECIFIED RUNOFF COEFFICIENT = .3500
  S.C.S. CURVE NUMBER (AMC II) =   0
  TRAVEL TIME COMPUTED USING ESTIMATED FLOW(CFS) =       9.86
  TRAVEL TIME THRU SUBAREA BASED ON VELOCITY(FEET/SEC.) =   0.54
  AVERAGE FLOW DEPTH(FEET) =   0.88   TRAVEL TIME(MIN.) =   3.06
```

```
  Tc(MIN.) =    9.75
  SUBAREA AREA(ACRES) =     0.05       SUBAREA RUNOFF(CFS) =    0.10
  AREA-AVERAGE RUNOFF COEFFICIENT =  0.777
  TOTAL AREA(ACRES) =        1.8         PEAK FLOW RATE(CFS) =       9.81

  END OF SUBAREA CHANNEL FLOW HYDRAULICS:
  DEPTH(FEET) =  0.88   FLOW VELOCITY(FEET/SEC.) =   0.54
  LONGEST FLOWPATH FROM NODE    130.00 TO NODE    150.00 =     561.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    199.00 TO NODE    199.00 IS CODE =   1
----------------------------------------------------------------------------
  >>>>>DESIGNATE INDEPENDENT STREAM FOR CONFLUENCE<<<<<
  >>>>>AND COMPUTE VARIOUS CONFLUENCED STREAM VALUES<<<<<
============================================================================
  TOTAL NUMBER OF STREAMS =  2
  CONFLUENCE VALUES USED FOR INDEPENDENT STREAM  2 ARE:
  TIME OF CONCENTRATION(MIN.) =    9.75
  RAINFALL INTENSITY(INCH/HR) =   5.48
  TOTAL STREAM AREA(ACRES) =     1.75
  PEAK FLOW RATE(CFS) AT CONFLUENCE =      9.81

  ** CONFLUENCE DATA **
  STREAM     RUNOFF       Tc      INTENSITY      AREA
  NUMBER      (CFS)     (MIN.)   (INCH/HOUR)    (ACRE)
      1       16.14     6.83        6.897          3.15
      2        9.81     9.75        5.480          1.75

  RAINFALL INTENSITY AND TIME OF CONCENTRATION RATIO
  CONFLUENCE FORMULA USED FOR  2 STREAMS.

  ** PEAK FLOW RATE TABLE **
  STREAM     RUNOFF      Tc      INTENSITY
  NUMBER      (CFS)    (MIN.)   (INCH/HOUR)
      1       23.01     6.83       6.897
      2       22.64     9.75       5.480

  COMPUTED CONFLUENCE ESTIMATES ARE AS FOLLOWS:
  PEAK FLOW RATE(CFS) =      23.01   Tc(MIN.) =    6.83
  TOTAL AREA(ACRES) =        4.9
  LONGEST FLOWPATH FROM NODE    100.00 TO NODE    199.00 =    1111.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    199.00 TO NODE    199.00 IS CODE =  81
----------------------------------------------------------------------------
  >>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
============================================================================
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  6.897
  *USER SPECIFIED(SUBAREA):
  USER-SPECIFIED RUNOFF COEFFICIENT = .3500
```

```
  S.C.S. CURVE NUMBER (AMC II) =   0
  AREA-AVERAGE RUNOFF COEFFICIENT = 0.7136
  SUBAREA AREA(ACRES) =    0.10   SUBAREA RUNOFF(CFS) =    0.24
  TOTAL AREA(ACRES) =        5.0   TOTAL RUNOFF(CFS) =      24.61
  TC(MIN.) =    6.83

****************************************************************************
  FLOW PROCESS FROM NODE    199.00 TO NODE    199.00 IS CODE =  81
----------------------------------------------------------------------------
  >>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
============================================================================
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  6.897
  *USER SPECIFIED(SUBAREA):
  USER-SPECIFIED RUNOFF COEFFICIENT = .3500
  S.C.S. CURVE NUMBER (AMC II) =   0
  AREA-AVERAGE RUNOFF COEFFICIENT = 0.7065
  SUBAREA AREA(ACRES) =    0.10   SUBAREA RUNOFF(CFS) =    0.24
  TOTAL AREA(ACRES) =        5.1   TOTAL RUNOFF(CFS) =      24.85
  TC(MIN.) =    6.83

****************************************************************************
  FLOW PROCESS FROM NODE    199.00 TO NODE    199.00 IS CODE =  81
----------------------------------------------------------------------------
  >>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
============================================================================
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  6.897
  *USER SPECIFIED(SUBAREA):
  USER-SPECIFIED RUNOFF COEFFICIENT = .6800
  S.C.S. CURVE NUMBER (AMC II) =   0
  AREA-AVERAGE RUNOFF COEFFICIENT = 0.7060
  SUBAREA AREA(ACRES) =    0.10   SUBAREA RUNOFF(CFS) =    0.47
  TOTAL AREA(ACRES) =        5.2   TOTAL RUNOFF(CFS) =      25.32
  TC(MIN.) =    6.83

****************************************************************************
  FLOW PROCESS FROM NODE    199.00 TO NODE    199.00 IS CODE =  81
----------------------------------------------------------------------------
  >>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
============================================================================
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  6.897
  *USER SPECIFIED(SUBAREA):
  USER-SPECIFIED RUNOFF COEFFICIENT = .7900
  S.C.S. CURVE NUMBER (AMC II) =   0
  AREA-AVERAGE RUNOFF COEFFICIENT = 0.7075
  SUBAREA AREA(ACRES) =    0.10   SUBAREA RUNOFF(CFS) =    0.54
  TOTAL AREA(ACRES) =        5.3   TOTAL RUNOFF(CFS) =      25.86
  TC(MIN.) =    6.83
============================================================================
  END OF STUDY SUMMARY:
  TOTAL AREA(ACRES)     =        5.3  TC(MIN.) =      6.83
```

PEAK FLOW RATE(CFS) = $\boxed{25.86}$

==

==

END OF RATIONAL METHOD ANALYSIS

APPENDIX C

RATIONAL METHOD BACK-UP

Weighted Runoff Coefficient Calculations

Pre-Project Condition

Basin 100

U/S Node #	D/S Node #	Sub Basin Area, A (sf)	Sub Basin Area, A (acres)	% Impervious (%)	Hydrologic Soil Group	Weighted Runoff Coefficient[1], C
100	102	3622	0.1	0%	D	0.35
102	120	60407	1.4	5%	C/D	0.38
120	120	45970	1	0%	C/D	0.35
120	150	29306	0.7	80%	C/D	0.79
199	199	55401	1.3	0%	C/D	0.35
199	199	27249	0.6	50%	C/D	0.63
199	199	4165	0.1	0.4	C/D	0.57
Total Area (acres) =			5.2			

Notes:

1. Based on County of San Diego Hydrology Manual (June 2003) Table 3-1 (Runoff Coefficients for Urban Areas), measured % impervious areas are matched up with the % impervious and soil type on the table to determine the appropriate runoff coefficient. If the measured % impervious is not provided in the table, runoff coefficients were interpolated accordingly.

2. The 100-year, 6-hour precipitation of 3.2 inches is referenced from the County of San Diego Hydrology Manual (June 2003) isopluvial map.

Weighted Runoff Coefficient Calculations

Post-Project Condition

Basin 100

U/S Node #	D/S Node #	Sub Basin Area, A (sf)	Sub Basin Area, A (acres)	% Impervious (%)	Hydrologic Soil Group	Weighted Runoff Coefficient[1], C
100	102	4550	0.1	90%	D	0.85
102	105	2047	0.05	90%	D	0.85
105	115	12104	0.3	80%	D	0.79
110	112	4081	0.1	90%	D	0.85
112	115	30017	0.7	80%	D	0.79
117	117	26417	0.6	80%	D	0.79
120	120	19184	0.4	60%	D	0.68
120	120	8425	0.2	80%	D	0.79
120	120	8712	0.2	5%	D	0.38
120	120	20905	0.5	5%	D	0.38
130	132	4460	0.1	80%	D	0.79
132	135	18821	0.4	80%	D	0.79
140	140	4095	0.1	80%	D	0.79
140	140	21260	0.5	80%	D	0.79
140	140	27432	0.6	80%	D	0.79
145	150	1961	0.05	0%	D	0.35
199	199	4048	0.1	0%	D	0.35
199	199	3779	0.1	0%	D	0.35
199	199	4356	0.1	60%	D	0.68
199	199	4165	0.1	80%	D	0.79
Total Area (acres) =		230819	5.3			

Notes:
1. Based on County of San Diego Hydrology Manual (June 2003) Table 3-1 (Runoff Coefficients for Urban Areas), measured % impervious areas are matched up with the % impervious and soil type on the table to determine the appropriate runoff coefficient. If the measured % impervious is not provided in the table, runoff coefficients were interpolated accordingly.

2. The 100-year, 6-hour precipitation of 3.2 inches is referenced from the County of San Diego Hydrology Manual (June 2003) isopluvial map.

=Used for Pre-Project AES Hydrology

=Used for Post-Project AES Hydrology

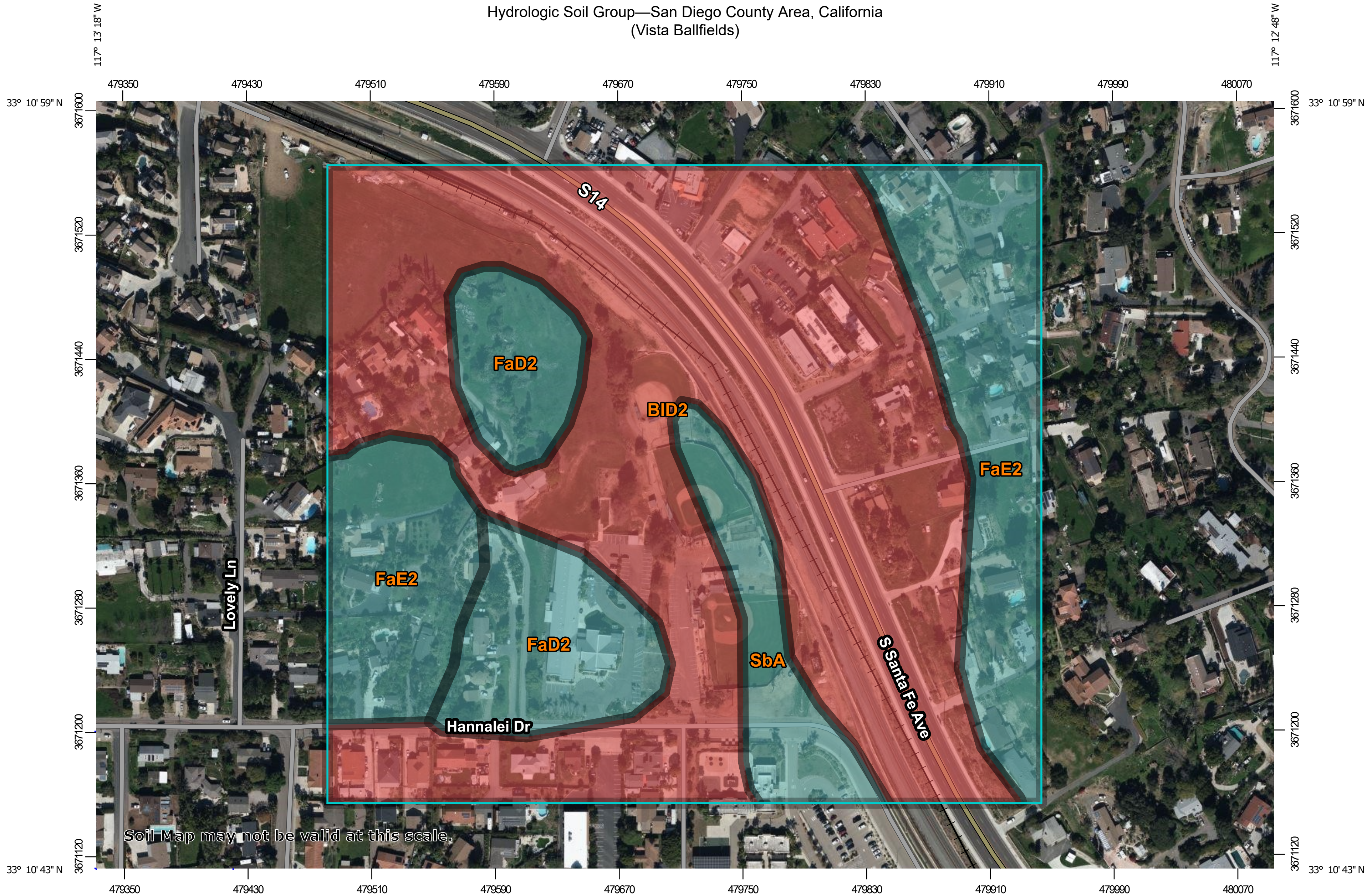
Table 3-1
RUNOFF COEFFICIENTS FOR URBAN AREAS

Land Use		Runoff Coefficient "C"				
			Soil Type			
NRCS Elements	County Elements	% IMPER.	A	B	C	D
Undisturbed Natural Terrain (Natural)	Permanent Open Space	0*	0.20	0.25	0.30	0.35
Low Density Residential (LDR)	Residential, 1.0 DU/A or less	10	0.27	0.32	0.36	0.41
Low Density Residential (LDR)	Residential, 2.0 DU/A or less	20	0.34	0.38	0.42	0.46
Low Density Residential (LDR)	Residential, 2.9 DU/A or less	25	0.38	0.41	0.45	0.49
Medium Density Residential (MDR)	Residential, 4.3 DU/A or less	30	0.41	0.45	0.48	0.52
Medium Density Residential (MDR)	Residential, 7.3 DU/A or less	40	0.48	0.51	0.54	0.57
Medium Density Residential (MDR)	Residential, 10.9 DU/A or less	45	0.52	0.54	0.57	0.60
Medium Density Residential (MDR)	Residential, 14.5 DU/A or less	50	0.55	0.58	0.60	0.63
High Density Residential (HDR)	Residential, 24.0 DU/A or less	65	0.66	0.67	0.69	0.71
High Density Residential (HDR)	Residential, 43.0 DU/A or less	80	0.76	0.77	0.78	0.79
Commercial/Industrial (N. Com)	Neighborhood Commercial	80	0.76	0.77	0.78	0.79
Commercial/Industrial (G. Com)	General Commercial	85	0.80	0.80	0.81	0.82
Commercial/Industrial (O.P. Com)	Office Professional/Commercial	90	0.83	0.84	0.84	0.85
Commercial/Industrial (Limited I.)	Limited Industrial	90	0.83	0.84	0.84	0.85
Commercial/Industrial (General I.)	General Industrial	95	0.87	0.87	0.87	0.87

*The values associated with 0% impervious may be used for direct calculation of the runoff coefficient as described in Section 3.1.2 (representing the pervious runoff coefficient, Cp, for the soil type), or for areas that will remain undisturbed in perpetuity. Justification must be given that the area will remain natural forever (e.g., the area is located in Cleveland National Forest).

DU/A = dwelling units per acre

NRCS = National Resources Conservation Service



Map Scale: 1:3,480 if printed on A landscape (11" x 8.5") sheet.

N

Meters: 0, 50, 100, 200, 300

Feet: 0, 150, 300, 600, 900

Map projection: Web Mercator Corner coordinates: WGS84 Edge tics: UTM Zone 11N 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: San Diego County Area, California
Survey Area Data: Version 15, May 27, 2020

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

Date(s) aerial images were photographed: Jan 24, 2020—Feb 12, 2020

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
BlD2	Bonsall sandy loam, 9 to 15 percent slopes, eroded	D	28.3	60.1%
FaD2	Fallbrook sandy loam, 9 to 15 percent slopes, eroded	C	5.7	12.2%
FaE2	Fallbrook sandy loam, 15 to 30 percent slopes, eroded	C	10.2	21.7%
SbA	Salinas clay loam, 0 to 2 percent slopes, warm MAAT, MLRA 19	C	2.8	6.0%
Totals for Area of Interest			**47.0**	**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.

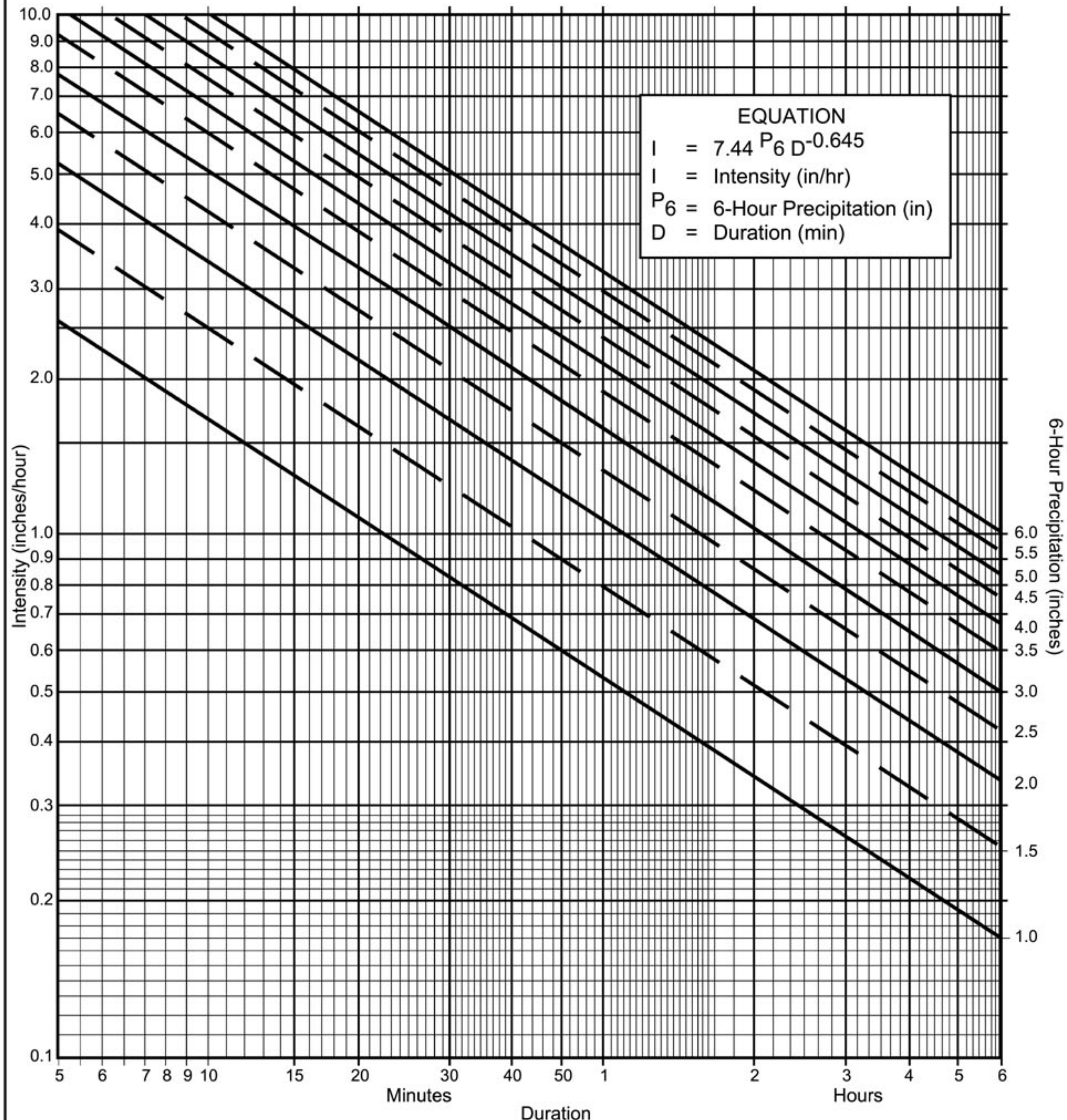
If a soil is assigned to a dual hydrologic group (A/D, B/D, or C/D), the first letter is for drained areas and the second is for undrained areas. Only the soils that in their natural condition are in group D are assigned to dual classes.

Rating Options

Aggregation Method: Dominant Condition

Component Percent Cutoff: None Specified

Tie-break Rule: Higher



Directions for Application:

(1) From precipitation maps determine 6 hr and 24 hr amounts for the selected frequency. These maps are included in the County Hydrology Manual (10, 50, and 100 yr maps included in the Design and Procedure Manual).

(2) Adjust 6 hr precipitation (if necessary) so that it is within the range of 45% to 65% of the 24 hr precipitation (not applicable to Desert).

(3) Plot 6 hr precipitation on the right side of the chart.

(4) Draw a line through the point parallel to the plotted lines.

(5) This line is the intensity-duration curve for the location being analyzed.

Application Form:

(a) Selected frequency ______ year

(b) P_6 = ______ in., P_{24} = ______, $\frac{P_6}{P_{24}}$ = ______ %(2)

(c) Adjusted P_6(2) = ______ in.

(d) t_x = ______ min.

(e) I = ______ in./hr.

Note: This chart replaces the Intensity-Duration-Frequency curves used since 1965.

P6	1	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6
Duration	I	I	I	I	I	I	I	I	I	I	I
5	2.63	3.95	5.27	6.59	7.90	9.22	10.54	11.86	13.17	14.49	15.81
7	2.12	3.18	4.24	5.30	6.36	7.42	8.48	9.54	10.60	11.66	12.72
10	1.68	2.53	3.37	4.21	5.05	5.90	6.74	7.58	8.42	9.27	10.11
15	1.30	1.95	2.59	3.24	3.89	4.54	5.19	5.84	6.49	7.13	7.78
20	1.08	1.62	2.15	2.69	3.23	3.77	4.31	4.85	5.39	5.93	6.46
25	0.93	1.40	1.87	2.33	2.80	3.27	3.73	4.20	4.67	5.13	5.60
30	0.83	1.24	1.66	2.07	2.49	2.90	3.32	3.73	4.15	4.56	4.98
40	0.69	1.03	1.38	1.72	2.07	2.41	2.76	3.10	3.45	3.79	4.13
50	0.60	0.90	1.19	1.49	1.79	2.09	2.39	2.69	2.98	3.28	3.58
60	0.53	0.80	1.06	1.33	1.59	1.86	2.12	2.39	2.65	2.92	3.18
90	0.41	0.61	0.82	1.02	1.23	1.43	1.63	1.84	2.04	2.25	2.45
120	0.34	0.51	0.68	0.85	1.02	1.19	1.36	1.53	1.70	1.87	2.04
150	0.29	0.44	0.59	0.73	0.88	1.03	1.18	1.32	1.47	1.62	1.76
180	0.26	0.39	0.52	0.65	0.78	0.91	1.04	1.18	1.31	1.44	1.57
240	0.22	0.33	0.43	0.54	0.65	0.76	0.87	0.98	1.08	1.19	1.30
300	0.19	0.28	0.38	0.47	0.56	0.66	0.75	0.85	0.94	1.03	1.13
360	0.17	0.25	0.33	0.42	0.50	0.58	0.67	0.75	0.84	0.92	1.00

Intensity-Duration Design Chart - Template

FIGURE

3-1

Note that the Initial Time of Concentration should be reflective of the general land-use at the upstream end of a drainage basin. A single lot with an area of two or less acres does not have a significant effect where the drainage basin area is 20 to 600 acres.

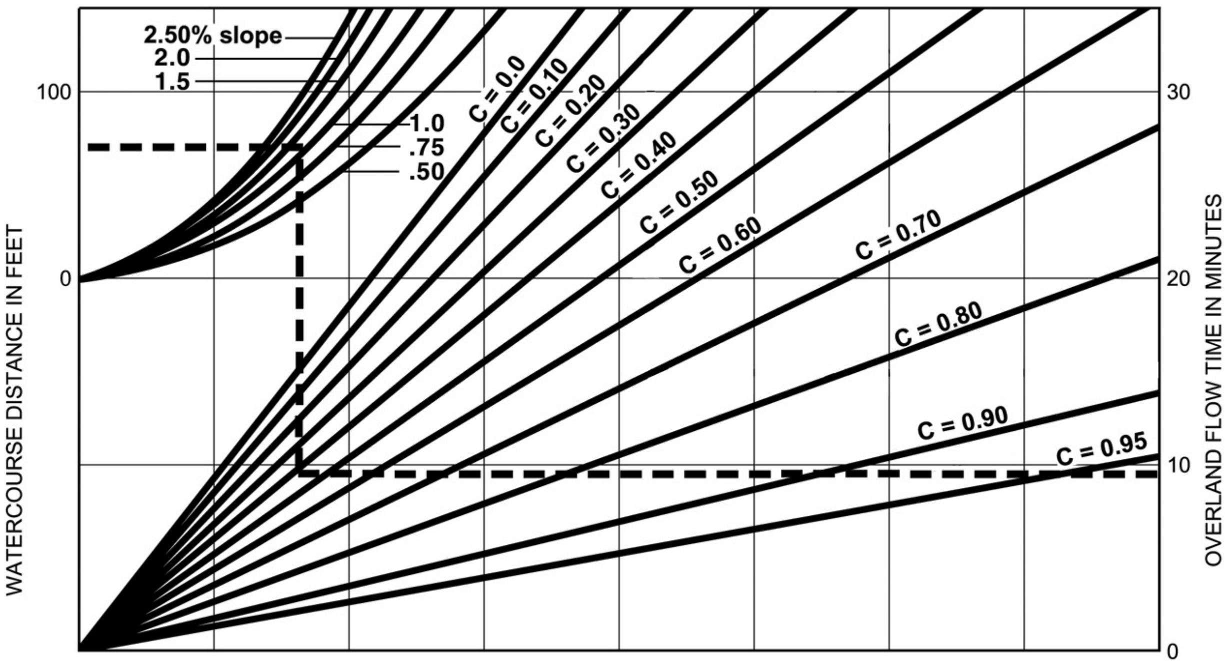
Table 3-2 provides limits of the length (Maximum Length (L_M)) of sheet flow to be used in hydrology studies. Initial T_i values based on average C values for the Land Use Element are also included. These values can be used in planning and design applications as described below. Exceptions may be approved by the "Regulating Agency" when submitted with a detailed study.

Table 3-2

**MAXIMUM OVERLAND FLOW LENGTH (L_M)
& INITIAL TIME OF CONCENTRATION (T_i)**

Element*	DU/ Acre	.5%		1%		2%		3%		5%		10%	
		L_M	T_i	L_M	T_i	L_M	T_i	L_M	T_i	L_M	T_i	L_M	T_i
Natural		50	13.2	70	12.5	85	10.9	100	10.3	100	8.7	100	6.9
LDR	1	50	12.2	70	11.5	85	10.0	100	9.5	100	8.0	100	6.4
LDR	2	50	11.3	70	10.5	85	9.2	100	8.8	100	7.4	100	5.8
LDR	2.9	50	10.7	70	10.0	85	8.8	95	8.1	100	7.0	100	5.6
MDR	4.3	50	10.2	70	9.6	80	8.1	95	7.8	100	6.7	100	5.3
MDR	7.3	50	9.2	65	8.4	80	7.4	95	7.0	100	6.0	100	4.8
MDR	10.9	50	8.7	65	7.9	80	6.9	90	6.4	100	5.7	100	4.5
MDR	14.5	50	8.2	65	7.4	80	6.5	90	6.0	100	5.4	100	4.3
HDR	24	50	6.7	65	6.1	75	5.1	90	4.9	95	4.3	100	3.5
HDR	43	50	5.3	65	4.7	75	4.0	85	3.8	95	3.4	100	2.7
N. Com		50	5.3	60	4.5	75	4.0	85	3.8	95	3.4	100	2.7
G. Com		50	4.7	60	4.1	75	3.6	85	3.4	90	2.9	100	2.4
O.P./Com		50	4.2	60	3.7	70	3.1	80	2.9	90	2.6	100	2.2
Limited I.		50	4.2	60	3.7	70	3.1	80	2.9	90	2.6	100	2.2
General I.		50	3.7	60	3.2	70	2.7	80	2.6	90	2.3	100	1.9

*See Table 3-1 for more detailed description



EXAMPLE:

Given: Watercourse Distance (D) = 70 Feet

Slope (s) = 1.3%

Runoff Coefficient (C) = 0.41

Overland Flow Time (T) = 9.5 Minutes

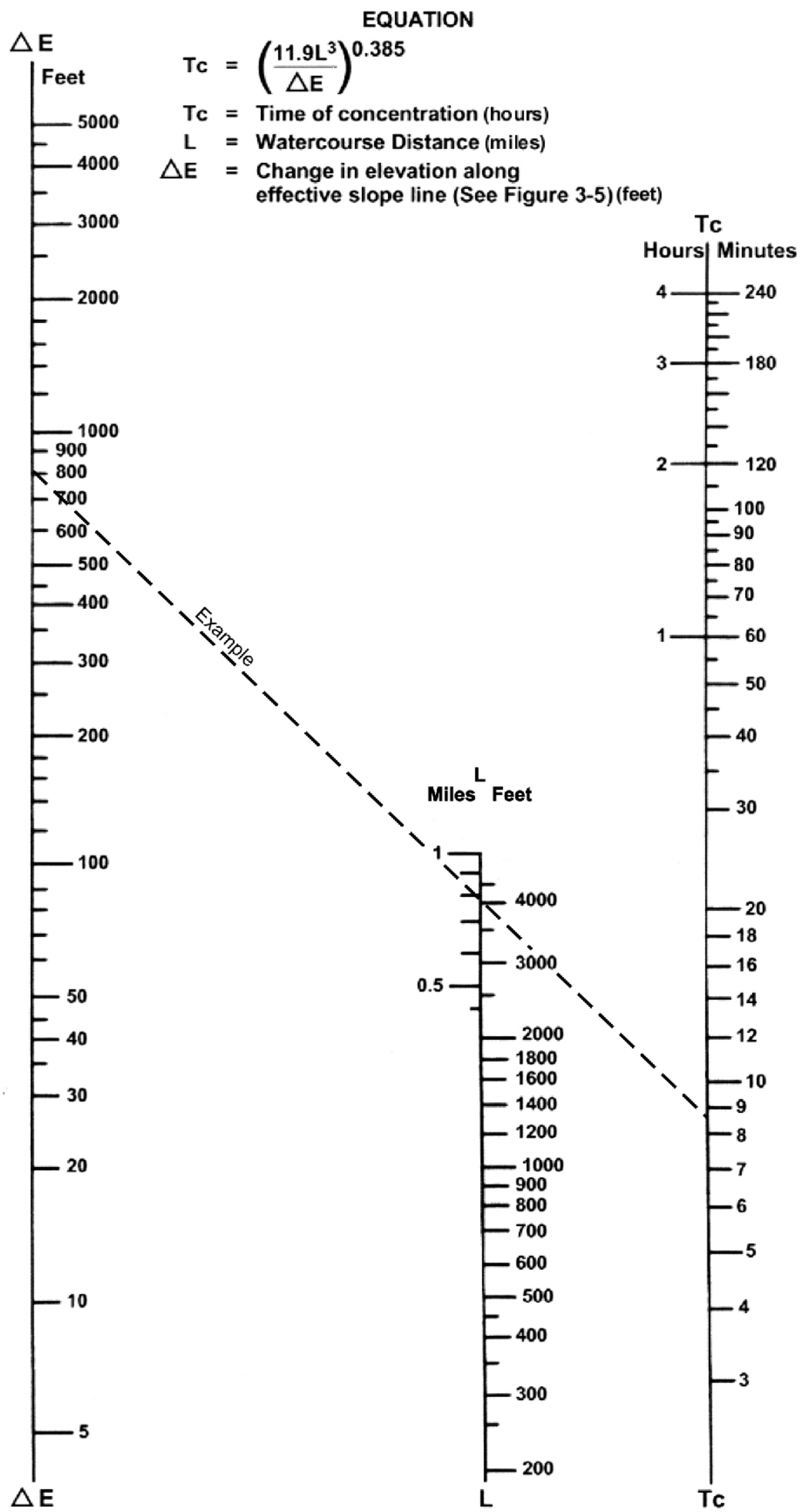
$$T = \frac{1.8\,(1.1-C)\sqrt{D}}{\sqrt[3]{s}}$$

SOURCE: Airport Drainage, Federal Aviation Administration, 1965

FIGURE

3-3

Rational Formula - Overland Time of Flow Nomograph

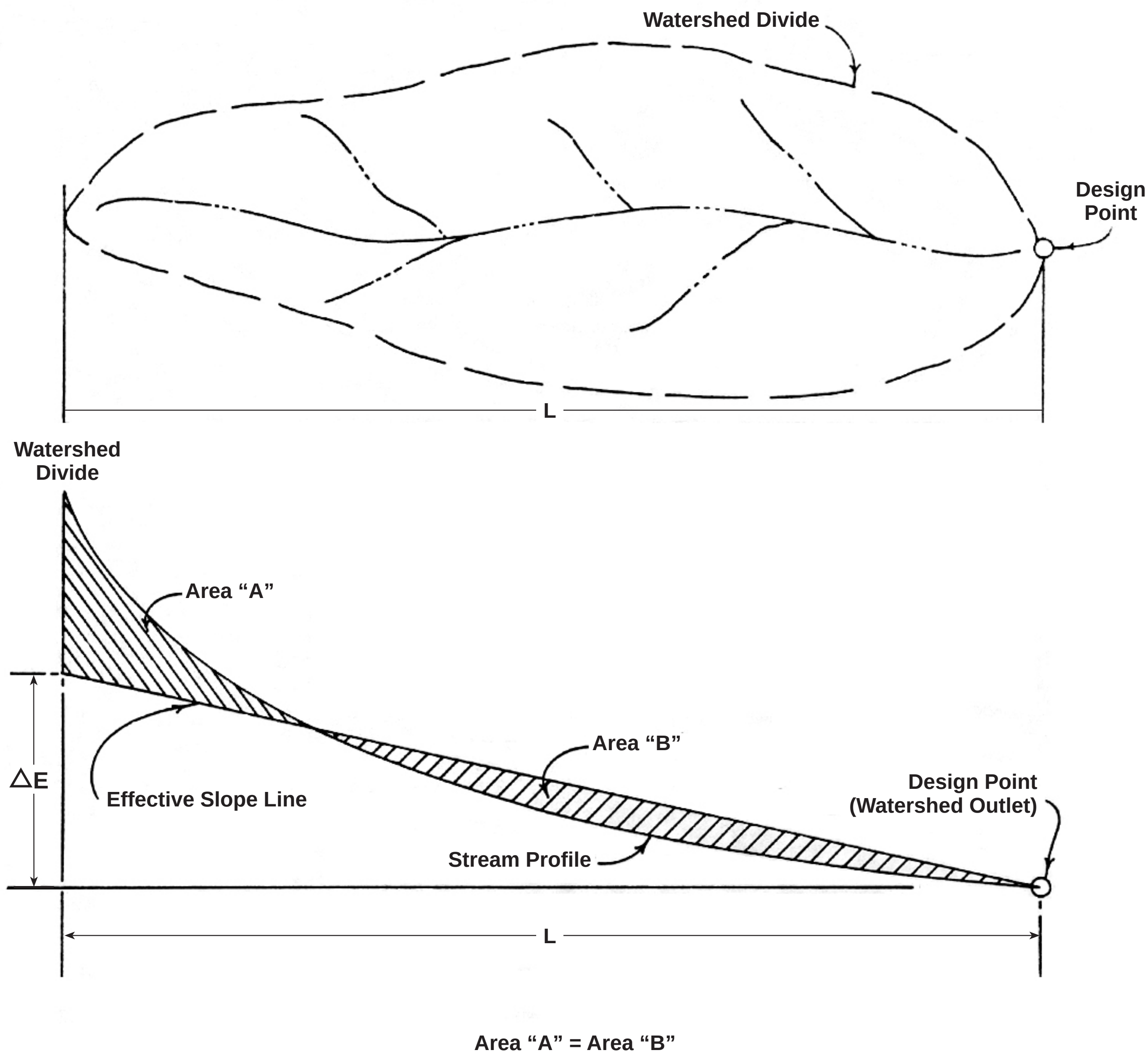


SOURCE: California Division of Highways (1941) and Kirpich (1940)

FIGURE

3-4

Nomograph for Determination of
Time of Concentration (Tc) or Travel Time (Tt) for Natural Watersheds



SOURCE: California Division of Highways (1941) and Kirpich (1940)

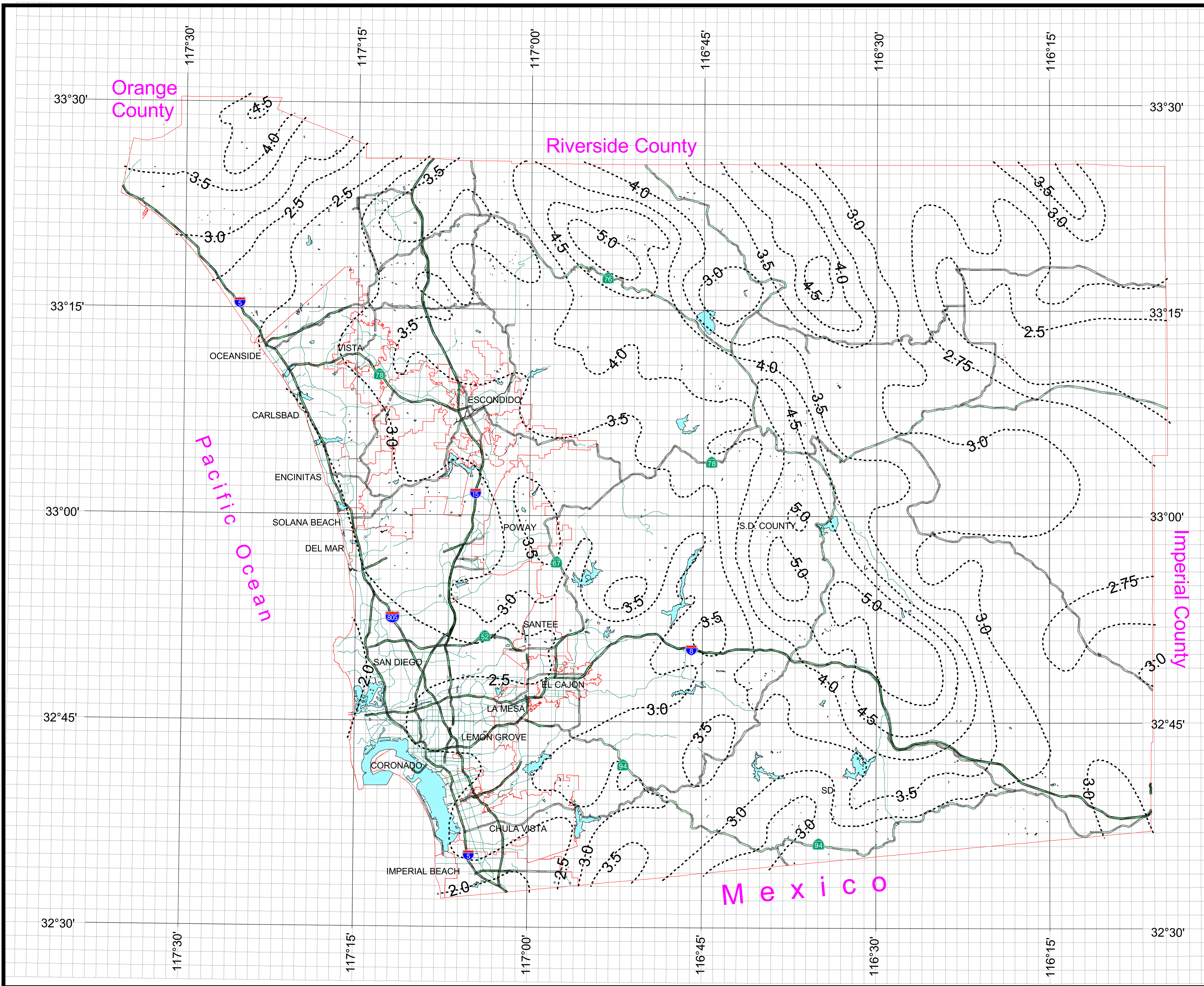
Computation of Effective Slope for Natural Watersheds

FIGURE 3-5

County of San Diego Hydrology Manual

Rainfall Isopluvials

100 Year Rainfall Event - 6 Hours



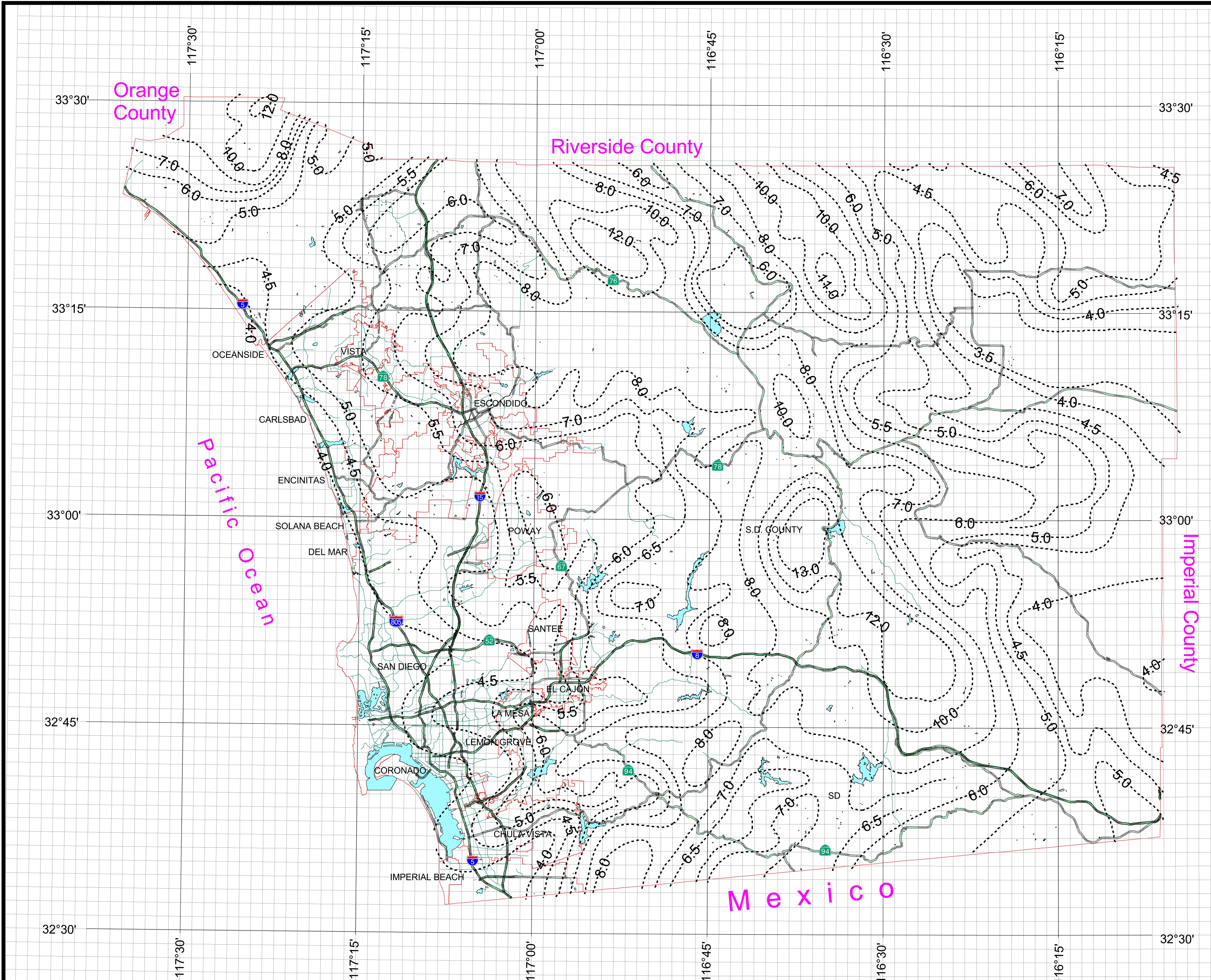
Isopluvial (inches)

N
W
E
S

3 0 3 Miles

Department of Public Works
Geographic Information Services

We Have San Diego Covered!



County of San Diego Hydrology Manual

Rainfall Isopluvials

100 Year Rainfall Event - 24 Hours

Isopluvial (inches)

DPW GIS
Department of Public Works
Geographic Information Services

SanGIS
We Have San Diego Covered!

N
W E
S

3 0 3 Miles

APPENDIX D

INLET SIZING CALCULATIONS

This will be included during final engineering.

APPENDIX E

PRELIMINARY STORM DRAIN SIZING

Normal Depth Storm Drain Sizing Table

The purpose of this table is to provide an estimated pipe size to convey the 100-year flow rates with a sizing factor.

Manning's n: 0.013

Sizing Factor (%): 30

	Slope at:	0.5%		1.0%		2.0%		3.0%	
Q_{100} (cfs[1])	Q_{100} with Sizing Factor (cfs[1])	Minimum Pipe Size[2] (feet)	Recommended Pipe Size (inches)	Minimum Pipe Size[2] (feet)	Recommended Pipe Size (inches)	Minimum Pipe Size [2] (feet)	Recommended Pipe Size (inches)	Minimum Pipe Size[2] (feet)	Recommended Pipe Size (inches)
0.8	1.0	0.70	10"	0.61	8"	0.54	8"	0.50	6"
1.3	1.7	0.86	12"	0.76	10"	0.66	8"	0.61	8"
3.0	3.9	1.18	18"	1.03	18"	0.91	12"	0.84	10"
6.0	7.8	1.53	24"	1.34	18"	1.18	18"	1.09	18"
10.5	13.7	1.88	24"	1.65	24"	1.45	18"	1.35	18"
16.1	20.9	2.21	30"	1.94	24"	1.70	24"	1.58	24"
25.0	32.5	2.61	36"	2.29	30"	2.01	24"	1.86	24"
30.0	39.0	2.79	36"	2.45	30"	2.15	30"	1.99	24"
35.0	45.5	2.96	36"	2.60	36"	2.28	30"	2.11	30"
40.0	52.0	3.11	42"	2.73	36"	2.40	30"	2.22	30"
50.0	65.0	3.38	42"	2.97	36"	2.61	36"	2.42	30"
60.0	78.0	3.62	48"	3.18	42"	2.79	36"	2.59	36"
70.0	91.0	3.83	48"	3.37	42"	2.96	36"	2.74	36"
80.0	104.0	4.03	54"	3.54	48"	3.11	42"	2.88	36"
90.0	117.0	4.21	54"	3.70	48"	3.25	42"	3.01	42"
100.0	130.0	4.38	54"	3.85	48"	3.38	42"	3.13	42"
150.0	195.0	5.10	72"	4.48	54"	3.94	48"	3.65	48"
200.0	260.0	5.68	72"	4.99	60"	4.38	54"	4.06	54"
250.0	325.0	6.18	84"	5.43	72"	4.77	60"	4.42	54"
300.0	390.0	6.62	84"	5.81	72"	5.10	72"	4.73	60"

Note:

1. "cfs" = cubic feet per second.

2. Minimum pipe sizes are calculated using the Manning's equation and are based on the flow rates with 30% factor.

APPENDIX F

EMERGENCY OVERFLOW CALCULATIONS

RICK™ ENGINEERING COMPANY
5620 Friars Road
San Diego, CA 92110-2596
Tel: (619) 291-0707
Fax: (619) 291-4165

Date 7/19/2021
Job No. 19253-B
Page
Done By BV
Checked By

Vista-II Emergency Overflow Calcs

BMP-A-1

Post-project undetained Q_{100} = 15.6 cfs

$H = 0.5\ ft$

Solving for length ⇒ $Q = CLH^{3/2}$

$15.6\ cfs = 3 \times L \times 0.5^{3/2}$

$L = 15\text{-feet}$

BMP-B-1

Post-project undetained Q_{100} = 9.8 cfs

$H = 0.5\ ft$

Solving for length ⇒ $Q = CLH^{3/2}$

$9.8\ cfs = 3 \times L \times 0.5^{3/2}$

$L = 10\text{-feet}$

APPENDIX G

ENERGY DISSIPATER DESIGN

This will be included during final engineering.

APPENDIX H

PRELIMINARY HEC-1 DETENTION ANALYSIS

```
*FREE
*DIAGRAM
ID VISTA BALLFIELDS, J-19253-B PROJECT SITE
ID 100-YEAR DETENTION ANALYSIS
ID SEPTEMBER 19, 2023 - FILE NAME: VB100.HC1
IT 1 01JAN90 1200 1000
IO 5 0
KK VB100.hc1
KM RUN DATE   9/19/2023
KM RATIONAL METHOD HYDROGRAPH PROGRAM
KM COPYRIGHT 1992, 2014, RICK ENGINEERING COMPANY
KM 6HR RAINFALL IS 3.2 INCHES
KM RATIONAL METHOD RUNOFF COEFFICIENT IS 0.78
KM RATIONAL METHOD TIME OF CONCENTRATION IS 6 MIN.
KM FOR THIS DATA TO RUN PROPERLY THIS IT CARD MUST BE ADDED TO YOUR HEC-1
KM IT 2 01JAN90 1200 200
BA 0.0039
IN 6 01JAN90 1157
QI 0  0.4  0.4  0.4  0.4  0.4  0.4  0.4  0.4  0.4
QI 0.4  0.4  0.5  0.5  0.5  0.5  0.5  0.5  0.5  0.5
QI 0.6  0.6  0.6  0.6  0.6  0.7  0.7  0.7  0.8  0.8
QI 0.9  0.9  1  1.1  1.1  1.3  1.4  1.7  2  2.9
QI 4.5  14.2  2.3  1.6  1.2  1  0.9  0.8  0.7  0.7
QI 0.6  0.6  0.5  0.5  0.5  0.5  0.4  0.4  0.4  0.4
QI 0.4  0  0  0  0  0  0  0  0  0
QI 0  0
*
KK DET-A
KO 0 0 0 0 21
RS 1 ELEV 0
SV 0 0.185 0.21 0.229 0.249 0.322 0.351
SQ 0.04 0.083 0.674 1.025 1.278 1.945 5.373
SE 0 3.17 3.58 3.92 4.25 5.5 6
*
ZZ
```

```
1****************************************                                                   ***************************************
 *                                       *                                                  *                                     *
 *   FLOOD HYDROGRAPH PACKAGE  (HEC-1)   *                                                  *   U.S. ARMY CORPS OF ENGINEERS      *
 *             JUN   1998                *                                                  *   HYDROLOGIC ENGINEERING CENTER     *
 *           VERSION 4.1                 *                                                  *          609 SECOND STREET          *
 *                                       *                                                  *      DAVIS, CALIFORNIA 95616        *
 *  RUN DATE   02OCT23  TIME  14:51:28   *                                                  *          (916) 756-1104             *
 *                                       *                                                  *                                     *
 ****************************************                                                   ***************************************


                                        X     X  XXXXXXX   XXXXX          X
                                        X     X  X        X     X        XX
                                        X     X  X        X               X
                                        XXXXXXX  XXXX     X       XXXXX   X
                                        X     X  X        X               X
                                        X     X  X        X     X         X
                                        X     X  XXXXXXX   XXXXX         XXX


           THIS PROGRAM REPLACES ALL PREVIOUS VERSIONS OF HEC-1 KNOWN AS HEC1 (JAN 73), HEC1GS, HEC1DB, AND HEC1KW.

           THE DEFINITIONS OF VARIABLES -RTIMP- AND -RTIOR- HAVE CHANGED FROM THOSE USED WITH THE 1973-STYLE INPUT STRUCTURE.
           THE DEFINITION OF -AMSKK- ON RM-CARD WAS CHANGED WITH REVISIONS DATED 28 SEP 81. THIS IS THE FORTRAN77 VERSION
           NEW OPTIONS: DAMBREAK OUTFLOW SUBMERGENCE , SINGLE EVENT DAMAGE CALCULATION, DSS:WRITE STAGE FREQUENCY,
           DSS:READ TIME SERIES AT DESIRED CALCULATION INTERVAL   LOSS RATE:GREEN AND AMPT INFILTRATION
           KINEMATIC WAVE: NEW FINITE DIFFERENCE ALGORITHM
```

```
1                                        HEC-1 INPUT                                              PAGE  1

          LINE          ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10

 *** FREE ***
                        *DIAGRAM
            1           ID   VISTA BALLFIELDS, J-19253-B PROJECT SITE
            2           ID   100-YEAR DETENTION ANALYSIS
            3           ID   SEPTEMBER 19, 2023 - FILE NAME: VB100.HC1
            4           IT       1 01JAN90    1200    1000
            5           IO       5       0

            6           KKVB100.hc1
            7           KM   RUN DATE   9/19/2023
            8           KM   RATIONAL METHOD HYDROGRAPH PROGRAM
            9           KM   COPYRIGHT 1992, 2014, RICK ENGINEERING COMPANY
           10           KM   6HR RAINFALL IS 3.2 INCHES
           11           KM   RATIONAL METHOD RUNOFF COEFFICIENT IS 0.78
           12           KM   RATIONAL METHOD TIME OF CONCENTRATION IS 6 MIN.
           13           KM   FOR THIS DATA TO RUN PROPERLY THIS IT CARD MUST BE ADDED TO YOUR HEC-1
           14           KM   IT 2 01JAN90 1200 200
           15           BA  0.0039
           16           IN       6 01JAN90    1157
           17           QI       0     0.4     0.4     0.4     0.4     0.4     0.4     0.4     0.4     0.4
           18           QI     0.4     0.4     0.5     0.5     0.5     0.5     0.5     0.5     0.5     0.5
           19           QI     0.6     0.6     0.6     0.6     0.6     0.7     0.7     0.7     0.8     0.8
           20           QI     0.9     0.9       1     1.1     1.1     1.3     1.4     1.7       2     2.9
           21           QI     4.5    14.2     2.3     1.6     1.2       1     0.9     0.8     0.7     0.7
           22           QI     0.6     0.6     0.5     0.5     0.5     0.5     0.4     0.4     0.4     0.4
           23           QI     0.4       0       0       0       0       0       0       0       0       0
           24           QI       0       0
                        *

           25           KK   DET-A
           26           KO       0       0       0       0      21
           27           RS       1    ELEV       0
           28           SV       0   0.185    0.21   0.229   0.249   0.322   0.351
           29           SQ    0.04   0.083   0.674   1.025   1.278   1.945   5.373
           30           SE       0    3.17    3.58    3.92    4.25     5.5       6
                        *
           31           ZZ
```

```
1
              SCHEMATIC DIAGRAM OF STREAM NETWORK
 INPUT
  LINE      (V) ROUTING         (--->) DIVERSION OR PUMP FLOW

   NO.      (.) CONNECTOR       (<---) RETURN OF DIVERTED OR PUMPED FLOW

     6    VB100.hc
              V
              V
    25     DET-A

 (***) RUNOFF ALSO COMPUTED AT THIS LOCATION
```


```
1****************************************                                                   ***************************************
 *                                       *                                                  *                                     *
 *   FLOOD HYDROGRAPH PACKAGE  (HEC-1)   *                                                  *   U.S. ARMY CORPS OF ENGINEERS      *
 *             JUN   1998                *                                                  *   HYDROLOGIC ENGINEERING CENTER     *
 *           VERSION 4.1                 *                                                  *          609 SECOND STREET          *
 *                                       *                                                  *      DAVIS, CALIFORNIA 95616        *
 *  RUN DATE   02OCT23  TIME  14:51:28   *                                                  *          (916) 756-1104             *
 *                                       *                                                  *                                     *
 ****************************************                                                   ***************************************
```


```
                              VISTA BALLFIELDS, J-19253-B PROJECT SITE
```

```
                         100-YEAR DETENTION ANALYSIS
                         SEPTEMBER 19, 2023 - FILE NAME: VB100.HC1

   5 IO         OUTPUT CONTROL VARIABLES
                      IPRNT           5  PRINT CONTROL
                      IPLOT           0  PLOT CONTROL
                      QSCAL          0.  HYDROGRAPH PLOT SCALE

     IT         HYDROGRAPH TIME DATA
                       NMIN           1  MINUTES IN COMPUTATION INTERVAL
                      IDATE      1JAN90  STARTING DATE
                      ITIME        1200  STARTING TIME
                         NQ        1000  NUMBER OF HYDROGRAPH ORDINATES
                     NDDATE      2JAN90  ENDING DATE
                     NDTIME        0439  ENDING TIME
                     ICENT           19  CENTURY MARK

                  COMPUTATION INTERVAL     .02 HOURS
                       TOTAL TIME BASE   16.65 HOURS

         ENGLISH UNITS
              DRAINAGE AREA          SQUARE MILES
              PRECIPITATION DEPTH    INCHES
              LENGTH, ELEVATION      FEET
              FLOW                   CUBIC FEET PER SECOND
              STORAGE VOLUME         ACRE-FEET
              SURFACE AREA           ACRES
              TEMPERATURE            DEGREES FAHRENHEIT


 *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** ***

           **************
           *            *
  25 KK    *    DET-A   *
           *            *
           **************

  26 KO         OUTPUT CONTROL VARIABLES
                      IPRNT           5  PRINT CONTROL
                      IPLOT           0  PLOT CONTROL
                      QSCAL          0.  HYDROGRAPH PLOT SCALE
                      IPNCH           0  PUNCH COMPUTED HYDROGRAPH
                       IOUT          21  SAVE HYDROGRAPH ON THIS UNIT
                      ISAV1           1  FIRST ORDINATE PUNCHED OR SAVED
                      ISAV2        1000  LAST ORDINATE PUNCHED OR SAVED
                     TIMINT        .017  TIME INTERVAL IN HOURS

1
                                                   RUNOFF SUMMARY
                                           FLOW IN CUBIC FEET PER SECOND
                                        TIME IN HOURS,  AREA IN SQUARE MILES

                                         PEAK   TIME OF     AVERAGE FLOW FOR MAXIMUM PERIOD      BASIN     MAXIMUM     TIME OF
          OPERATION       STATION        FLOW     PEAK                                            AREA       STAGE    MAX STAGE
+                                                              6-HOUR     24-HOUR     72-HOUR

          HYDROGRAPH AT
+                        VB100.hc         14.     4.05           1.          0.          0.        .00

          ROUTED TO
+                           DET-A          4.     4.13           1.          0.          0.        .00
+                                                                                                             5.84        4.13


 *** NORMAL END OF HEC-1 ***
```

LAG TIME FOR AES DETENTION RUN

POC-1

PEAK FLOW FOR AES DETENTION RUN

DET-A 11200 1JAN90 0 1 11000 .004

.040	.040	.040	.040	.040	.040	.041	.041	.041	.041
.041	.041	.041	.041	.042	.042	.042	.042	.042	.042
.042	.042	.042	.043	.043	.043	.043	.043	.043	.043
.043	.043	.044	.044	.044	.044	.044	.044	.044	.044
.044	.045	.045	.045	.045	.045	.045	.045	.045	.046
.046	.046	.046	.046	.046	.046	.046	.046	.047	.047
.047	.047	.047	.047	.047	.047	.047	.048	.048	.048
.048	.048	.048	.048	.049	.049	.049	.049	.049	.049
.049	.050	.050	.050	.050	.050	.050	.050	.051	.051
.051	.051	.051	.051	.051	.052	.052	.052	.052	.052
.052	.052	.053	.053	.053	.053	.053	.053	.053	.054
.054	.054	.054	.054	.054	.055	.055	.055	.055	.055
.055	.056	.056	.056	.056	.056	.056	.057	.057	.057
.057	.057	.057	.058	.058	.058	.058	.058	.059	.059
.059	.059	.059	.059	.060	.060	.060	.060	.060	.061
.061	.061	.061	.061	.062	.062	.062	.062	.062	.063
.063	.063	.063	.063	.064	.064	.064	.064	.065	.065
.065	.065	.066	.066	.066	.066	.067	.067	.067	.067
.068	.068	.068	.068	.069	.069	.069	.070	.070	.070
.070	.071	.071	.071	.072	.072	.072	.073	.073	.073
.074	.074	.074	.075	.075	.075	.076	.076	.077	.077
.077	.078	.078	.079	.079	.079	.080	.080	.081	.081
.082	.082	.089	.146	.203	.260	.318	.379	.443	.510
.579	.651	.716	.781	.851	.926	1.005	1.070	1.143	1.243
1.344	1.454	1.583	1.731	1.874	2.501	3.516	4.080	4.260	4.114
3.832	3.575	3.339	3.121	2.918	2.729	2.554	2.395	2.250	2.117
1.994	1.940	1.930	1.920	1.910	1.900	1.889	1.878	1.867	1.856
1.845	1.834	1.822	1.811	1.799	1.788	1.776	1.764	1.753	1.741
1.729	1.717	1.705	1.693	1.681	1.669	1.657	1.645	1.633	1.621
1.610	1.598	1.587	1.576	1.564	1.553	1.541	1.529	1.518	1.506
1.495	1.484	1.473	1.462	1.451	1.440	1.429	1.418	1.407	1.395
1.384	1.373	1.362	1.351	1.341	1.330	1.320	1.310	1.299	1.289
1.280	1.267	1.254	1.240	1.228	1.215	1.203	1.191	1.179	1.166
1.154	1.142	1.129	1.117	1.105	1.092	1.081	1.069	1.057	1.046
1.035	1.023	1.008	.992	.977	.963	.949	.935	.921	.908
.896	.883	.871	.859	.848	.836	.825	.815	.803	.791
.777	.761	.745	.727	.709	.691	.673	.652	.631	.611
.591	.572	.554	.536	.519	.502	.486	.471	.456	.441
.427	.413	.400	.387	.375	.363	.351	.340	.329	.318
.308	.298	.289	.280	.271	.262	.254	.245	.238	.230
.223	.215	.209	.202	.195	.189	.183	.177	.172	.166
.161	.156	.151	.146	.141	.137	.132	.128	.124	.120
.116	.112	.109	.105	.102	.099	.095	.092	.089	.087
.084	.083	.083	.083	.083	.083	.083	.083	.083	.083
.083	.083	.083	.083	.083	.083	.083	.083	.083	.083
.082	.082	.082	.082	.082	.082	.082	.082	.082	.082
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.076	.076	.076	.076	.076	.076	.076	.076	.076	.076
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.076	.076	.076	.076	.076	.076	.076	.076	.076	.076
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.075	.075	.075	.075	.075	.075	.075	.075	.075	.075
.075	.075	.075	.075	.075	.075	.075	.075	.075	.075
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.074	.074	.074	.074	.074	.074	.074	.074	.074	.074
.073	.073	.073	.073	.073	.073	.073	.073	.073	.073

.073	.073	.073	.073	.073	.073	.073	.073	.073	.073
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.071	.070	.070	.070	.070	.070	.070	.070	.070	.070
.070	.070	.070	.070	.070	.070	.070	.070	.070	.070
.070	.070	.070	.070	.070	.070	.070	.070	.070	.070
.070	.070	.070	.070	.070	.070	.070	.070	.070	.070
.070	.070	.070	.070	.070	.069	.069	.069	.069	.069
.069	.069	.069	.069	.069	.069	.069	.069	.069	.069

```
*FREE
*DIAGRAM
ID VISTA BALLFIELDS, J-19253-B PROJECT SITE
ID 100-YEAR DETENTION ANALYSIS
ID SEPTEMBER 19, 2023 - FILE NAME: VB200.HC1
IT 1 01JAN90 1200 1000
IO 5 0
KK VB200.hc1
KM RUN DATE   9/19/2023
KM RATIONAL METHOD HYDROGRAPH PROGRAM
KM COPYRIGHT 1992, 2014, RICK ENGINEERING COMPANY
KM 6HR RAINFALL IS 3.2 INCHES
KM RATIONAL METHOD RUNOFF COEFFICIENT IS 0.78
KM RATIONAL METHOD TIME OF CONCENTRATION IS 10 MIN.
KM FOR THIS DATA TO RUN PROPERLY THIS IT CARD MUST BE ADDED TO YOUR HEC-1
KM IT 2 01JAN90 1200 200
BA 0.0028
IN 10 01JAN90 1155
QI 0  0.3  0.3  0.3  0.3  0.3  0.3  0.3  0.3  0.4
QI 0.4  0.4  0.4  0.4  0.4  0.5  0.5  0.6  0.6  0.7
QI 0.7  0.9  1  1.5  3.1  9.8  1.2  0.8  0.6  0.5
QI 0.5  0.4  0.4  0.3  0.3  0.3  0.3  0  0  0
QI 0  0  0  0  0  0  0  0
*
KK DET-B1/B3
KO 0 0 0 0 21
RS 1 ELEV 0
SV 0 0.111 0.123 0.144 0.191 0.267 0.277 0.286
SQ 0.033 0.092 0.44 0.831 1.306 1.836 6.389 14.669
SE 0 3 3.33 3.92 5.17 7.25 7.5 7.75
*
ZZ
```

```
1****************************************                                             **************************************
 *                                       *                                            *                                    *
 *   FLOOD HYDROGRAPH PACKAGE  (HEC-1)   *                                            *    U.S. ARMY CORPS OF ENGINEERS    *
 *             JUN  1998                 *                                            *    HYDROLOGIC ENGINEERING CENTER   *
 *          VERSION 4.1                  *                                            *         609 SECOND STREET          *
 *                                       *                                            *      DAVIS, CALIFORNIA 95616       *
 *  RUN DATE   02OCT23  TIME  14:24:26   *                                            *          (916) 756-1104            *
 *                                       *                                            *                                    *
 ****************************************                                             **************************************



                                                   X     X  XXXXXXX   XXXXX          X
                                                   X     X  X        X     X        XX
                                                   X     X  X        X               X
                                                   XXXXXXX  XXXX     X       XXXXX   X
                                                   X     X  X        X               X
                                                   X     X  X        X     X         X
                                                   X     X  XXXXXXX   XXXXX         XXX



           THIS PROGRAM REPLACES ALL PREVIOUS VERSIONS OF HEC-1 KNOWN AS HEC1 (JAN 73), HEC1GS, HEC1DB, AND HEC1KW.

           THE DEFINITIONS OF VARIABLES -RTIMP- AND -RTIOR- HAVE CHANGED FROM THOSE USED WITH THE 1973-STYLE INPUT STRUCTURE.
           THE DEFINITION OF -AMSKK- ON RM-CARD WAS CHANGED WITH REVISIONS DATED 28 SEP 81. THIS IS THE FORTRAN77 VERSION
           NEW OPTIONS: DAMBREAK OUTFLOW SUBMERGENCE , SINGLE EVENT DAMAGE CALCULATION, DSS:WRITE STAGE FREQUENCY,
           DSS:READ TIME SERIES AT DESIRED CALCULATION INTERVAL   LOSS RATE:GREEN AND AMPT INFILTRATION
           KINEMATIC WAVE: NEW FINITE DIFFERENCE ALGORITHM


1                                            HEC-1 INPUT                                                 PAGE  1

          LINE          ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10

 *** FREE ***
                        *DIAGRAM
             1          ID   VISTA BALLFIELDS, J-19253-B PROJECT SITE
             2          ID   100-YEAR DETENTION ANALYSIS
             3          ID   SEPTEMBER 19, 2023 - FILE NAME: VB200.HC1
             4          IT       1 01JAN90    1200    1000
             5          IO       5       0

             6          KKVB200.hc1
             7          KM   RUN DATE   9/19/2023
             8          KM   RATIONAL METHOD HYDROGRAPH PROGRAM
             9          KM   COPYRIGHT 1992, 2014, RICK ENGINEERING COMPANY
            10          KM   6HR RAINFALL IS 3.2 INCHES
            11          KM   RATIONAL METHOD RUNOFF COEFFICIENT IS 0.78
            12          KM   RATIONAL METHOD TIME OF CONCENTRATION IS 10 MIN.
            13          KM   FOR THIS DATA TO RUN PROPERLY THIS IT CARD MUST BE ADDED TO YOUR HEC-1
            14          KM   IT 2 01JAN90 1200 200
            15          BA  0.0028
            16          IN      10 01JAN90     1155
            17          QI       0     0.3     0.3     0.3     0.3     0.3     0.3     0.3     0.3     0.4
            18          QI     0.4     0.4     0.4     0.4     0.4     0.5     0.5     0.6     0.6     0.7
            19          QI     0.7     0.9       1     1.5     3.1     9.8     1.2     0.8     0.6     0.5
            20          QI     0.5     0.4     0.4     0.3     0.3     0.3     0.3       0       0       0
            21          QI       0       0       0       0       0       0       0       0
                        *

            22          KKDET-B1/B3
            23          KO       0       0       0       0      21
            24          RS       1    ELEV       0
            25          SV       0   0.111   0.123   0.144   0.191   0.267   0.277   0.286
            26          SQ   0.033   0.092    0.44   0.831   1.306   1.836   6.389  14.669
            27          SE       0       3    3.33    3.92    5.17    7.25     7.5    7.75
                        *
            28          ZZ
1
             SCHEMATIC DIAGRAM OF STREAM NETWORK
 INPUT
  LINE      (V) ROUTING         (--->) DIVERSION OR PUMP FLOW

   NO.      (.) CONNECTOR       (<---) RETURN OF DIVERTED OR PUMPED FLOW

     6    VB200.hc
                 V
                 V
    22    DET-B1/B

 (***) RUNOFF ALSO COMPUTED AT THIS LOCATION
```


```
1****************************************                                             **************************************
 *                                       *                                            *                                    *
 *   FLOOD HYDROGRAPH PACKAGE  (HEC-1)   *                                            *    U.S. ARMY CORPS OF ENGINEERS    *
 *             JUN  1998                 *                                            *    HYDROLOGIC ENGINEERING CENTER   *
 *          VERSION 4.1                  *                                            *         609 SECOND STREET          *
 *                                       *                                            *      DAVIS, CALIFORNIA 95616       *
 *  RUN DATE   02OCT23  TIME  14:24:26   *                                            *          (916) 756-1104            *
 *                                       *                                            *                                    *
 ****************************************                                             **************************************
```


```
                         VISTA BALLFIELDS, J-19253-B PROJECT SITE
                         100-YEAR DETENTION ANALYSIS
                         SEPTEMBER 19, 2023 - FILE NAME: VB200.HC1
```

```
   5 IO         OUTPUT CONTROL VARIABLES
                      IPRNT          5  PRINT CONTROL
                      IPLOT          0  PLOT CONTROL
                      QSCAL         0.  HYDROGRAPH PLOT SCALE

     IT         HYDROGRAPH TIME DATA
                       NMIN          1  MINUTES IN COMPUTATION INTERVAL
                      IDATE      1JAN90  STARTING DATE
                      ITIME       1200  STARTING TIME
                         NQ       1000  NUMBER OF HYDROGRAPH ORDINATES
                     NDDATE      2JAN90  ENDING DATE
                     NDTIME       0439  ENDING TIME
                     ICENT          19  CENTURY MARK

                COMPUTATION INTERVAL     .02 HOURS
                     TOTAL TIME BASE   16.65 HOURS

         ENGLISH UNITS
              DRAINAGE AREA         SQUARE MILES
              PRECIPITATION DEPTH   INCHES
              LENGTH, ELEVATION     FEET
              FLOW                  CUBIC FEET PER SECOND
              STORAGE VOLUME        ACRE-FEET
              SURFACE AREA          ACRES
              TEMPERATURE           DEGREES FAHRENHEIT


*** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** ***

            **************
            *            *
  22 KK     *  DET-B1/B  *     3
            *            *
            **************

  23 KO         OUTPUT CONTROL VARIABLES
                      IPRNT          5  PRINT CONTROL
                      IPLOT          0  PLOT CONTROL
                      QSCAL         0.  HYDROGRAPH PLOT SCALE
                      IPNCH          0  PUNCH COMPUTED HYDROGRAPH
                       IOUT         21  SAVE HYDROGRAPH ON THIS UNIT
                      ISAV1          1  FIRST ORDINATE PUNCHED OR SAVED
                      ISAV2       1000  LAST ORDINATE PUNCHED OR SAVED
                     TIMINT       .017  TIME INTERVAL IN HOURS

1
                                                             RUNOFF SUMMARY
                                                     FLOW IN CUBIC FEET PER SECOND
                                                  TIME IN HOURS,  AREA IN SQUARE MILES

                                        PEAK   TIME OF     AVERAGE FLOW FOR MAXIMUM PERIOD      BASIN     MAXIMUM     TIME OF
          OPERATION       STATION       FLOW    PEAK                                             AREA      STAGE     MAX STAGE
+                                                           6-HOUR     24-HOUR      72-HOUR

          HYDROGRAPH AT
+                        VB200.hc         10.   4.08           1.          0.          0.        .00

          ROUTED TO
+                        DET-B1/B          3.   4.22           1.          0.          0.        .00
+                                                                                                            7.33        4.22


 *** NORMAL END OF HEC-1 ***
```

LAG TIME FOR AES
DETENTION RUN

POC-2

PEAK FLOW FOR AES DETENTION RUN

DET-B1/B   11200 1JAN90 0 1 11000   .003

.033	.033	.033	.033	.034	.034	.034	.034	.034	.034
.035	.035	.035	.035	.035	.036	.036	.036	.036	.036
.037	.037	.037	.037	.037	.038	.038	.038	.038	.038
.039	.039	.039	.039	.039	.039	.040	.040	.040	.040
.040	.041	.041	.041	.041	.041	.042	.042	.042	.042
.042	.043	.043	.043	.043	.043	.043	.044	.044	.044
.044	.044	.045	.045	.045	.045	.045	.046	.046	.046
.046	.046	.046	.047	.047	.047	.047	.047	.048	.048
.048	.048	.048	.049	.049	.049	.049	.050	.050	.050
.050	.051	.051	.051	.052	.052	.052	.052	.053	.053
.053	.053	.054	.054	.054	.054	.055	.055	.055	.055
.056	.056	.056	.056	.057	.057	.057	.057	.058	.058
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.061	.061	.061	.061	.062	.062	.062	.062	.063	.063
.063	.063	.064	.064	.064	.065	.065	.065	.066	.066
.066	.067	.067	.067	.068	.068	.068	.068	.069	.069
.070	.070	.070	.071	.071	.071	.072	.072	.073	.073
.073	.074	.074	.074	.075	.075	.076	.076	.076	.077
.077	.078	.078	.079	.079	.079	.080	.080	.081	.081
.082	.082	.083	.083	.083	.084	.084	.085	.085	.086
.086	.087	.087	.088	.089	.089	.090	.090	.091	.092
.103	.136	.169	.200	.231	.261	.291	.322	.353	.385
.418	.448	.470	.493	.516	.541	.567	.597	.630	.666
.705	.748	.793	.837	.865	.894	.930	.973	1.026	1.087
1.157	1.234	1.316	1.381	1.452	1.529	1.603	1.669	1.727	1.775
1.815	2.331	3.228	3.286	2.906	2.297	1.835	1.828	1.821	1.814
1.806	1.798	1.790	1.781	1.773	1.764	1.754	1.745	1.735	1.726
1.716	1.706	1.696	1.686	1.676	1.666	1.656	1.646	1.635	1.625
1.615	1.605	1.594	1.584	1.574	1.564	1.554	1.543	1.534	1.524
1.514	1.504	1.495	1.485	1.476	1.466	1.457	1.448	1.438	1.429
1.420	1.410	1.401	1.392	1.382	1.373	1.364	1.355	1.346	1.336
1.328	1.319	1.310	1.299	1.287	1.274	1.262	1.250	1.238	1.226
1.214	1.202	1.190	1.178	1.166	1.154	1.142	1.131	1.119	1.108
1.097	1.086	1.075	1.064	1.053	1.043	1.033	1.023	1.013	1.003
.993	.984	.974	.965	.956	.947	.938	.929	.920	.911
.903	.895	.886	.878	.870	.862	.855	.846	.838	.827
.810	.793	.776	.758	.740	.721	.703	.685	.668	.651
.635	.619	.603	.588	.573	.558	.544	.530	.517	.504
.491	.479	.467	.455	.443	.428	.411	.395	.379	.364
.350	.336	.323	.311	.298	.287	.276	.265	.254	.244
.235	.226	.217	.208	.200	.192	.185	.178	.171	.164
.157	.151	.145	.140	.134	.129	.124	.119	.114	.110
.106	.101	.098	.094	.092	.092	.092	.092	.092	.092
.092	.091	.091	.091	.091	.091	.091	.091	.091	.091
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.061	.061	.061	.061	.061	.061	.061	.061	.061	.061
.061	.061	.061	.061	.061	.061	.060	.060	.060	.060

VISTA II AES,
DETAINED
100-YR, 6-HR STORM

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****************************************************************************

              RATIONAL METHOD HYDROLOGY COMPUTER PROGRAM PACKAGE
              Reference: SAN DIEGO COUNTY FLOOD CONTROL DISTRICT
                           2003,1985,1981 HYDROLOGY MANUAL
           (c) Copyright 1982-2014 Advanced Engineering Software (aes)
               Ver. 21.0 Release Date: 06/01/2014  License ID 1261

                             Analysis prepared by:

                            RICK ENGINEERING COMPANY
                                5620 Friars Road
                          San Diego, California 92110
                        619-291-0707   Fax 619-291-4165

 ************************** DESCRIPTION OF STUDY **************************
* J-19253B VISTA II BALLFIELDS                                             *
* 100YR, 6-HR POST-PROJECT - DETAINED                                      *
* J:\19253B\WR\HYDROLOGY\RATIONALMETHOD\...                                *
 **************************************************************************

  FILE NAME: V2B1D00.RAT
  TIME/DATE OF STUDY: 14:02 10/03/2023
----------------------------------------------------------------------------
  USER SPECIFIED HYDROLOGY AND HYDRAULIC MODEL INFORMATION:
----------------------------------------------------------------------------
  2003 SAN DIEGO MANUAL CRITERIA

  USER SPECIFIED STORM EVENT(YEAR) = 100.00
  6-HOUR DURATION PRECIPITATION (INCHES) =   3.200
  SPECIFIED MINIMUM PIPE SIZE(INCH) =  18.00
  SPECIFIED PERCENT OF GRADIENTS(DECIMAL) TO USE FOR FRICTION SLOPE = 0.90
  SAN DIEGO HYDROLOGY MANUAL "C"-VALUES USED FOR RATIONAL METHOD
  NOTE: USE MODIFIED RATIONAL METHOD PROCEDURES FOR CONFLUENCE ANALYSIS
  *USER-DEFINED STREET-SECTIONS FOR COUPLED PIPEFLOW AND STREETFLOW MODEL*
     HALF-  CROWN TO   STREET-CROSSFALL:   CURB  GUTTER-GEOMETRIES:  MANNING
     WIDTH  CROSSFALL  IN-  / OUT-/PARK-  HEIGHT  WIDTH  LIP   HIKE  FACTOR
NO.   (FT)     (FT)    SIDE / SIDE/ WAY    (FT)    (FT)  (FT)  (FT)    (n)
===  =====  =========  =================  ======  ===== ====== ===== =======
  1   30.0     20.0    0.018/0.018/0.020   0.67    2.00 0.0313 0.167 0.0150
  2   18.0     13.0    0.020/0.020/0.020   0.50    1.50 0.0100 0.125 0.0180

  GLOBAL STREET FLOW-DEPTH CONSTRAINTS:
    1. Relative Flow-Depth =  0.10 FEET
       as (Maximum Allowable Street Flow Depth) - (Top-of-Curb)
    2. (Depth)*(Velocity) Constraint =  6.0 (FT*FT/S)
  *SIZE PIPE WITH A FLOW CAPACITY GREATER THAN
   OR EQUAL TO THE UPSTREAM TRIBUTARY PIPE.*
```

```
****************************************************************************
  FLOW PROCESS FROM NODE    100.00 TO NODE    102.00 IS CODE =  21
----------------------------------------------------------------------------
  >>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<
============================================================================
  *USER SPECIFIED(SUBAREA):
  USER-SPECIFIED RUNOFF COEFFICIENT = .8500
  S.C.S. CURVE NUMBER (AMC II) =   0
  INITIAL SUBAREA FLOW-LENGTH(FEET) =   100.00
  UPSTREAM ELEVATION(FEET) =    504.00
  DOWNSTREAM ELEVATION(FEET) =    503.00
  ELEVATION DIFFERENCE(FEET) =      1.00
  SUBAREA OVERLAND TIME OF FLOW(MIN.) =    3.486
  WARNING: INITIAL SUBAREA FLOW PATH LENGTH IS GREATER THAN
           THE MAXIMUM OVERLAND FLOW LENGTH =    60.00
           (Reference: Table 3-1B of Hydrology Manual)
           THE MAXIMUM OVERLAND FLOW LENGTH IS USED IN Tc CALCULATION!
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  8.431
  NOTE: RAINFALL INTENSITY IS BASED ON Tc = 5-MINUTE.
  SUBAREA RUNOFF(CFS) =      0.72
  TOTAL AREA(ACRES) =      0.10   TOTAL RUNOFF(CFS) =      0.72

****************************************************************************
  FLOW PROCESS FROM NODE    102.00 TO NODE    105.00 IS CODE =  61
----------------------------------------------------------------------------
  >>>>>COMPUTE STREET FLOW TRAVEL TIME THRU SUBAREA<<<<<
  >>>>>(STANDARD CURB SECTION USED)<<<<<
============================================================================
  UPSTREAM ELEVATION(FEET) =  503.00  DOWNSTREAM ELEVATION(FEET) =  498.50
  STREET LENGTH(FEET) =   120.00   CURB HEIGHT(INCHES) =  6.0
  STREET HALFWIDTH(FEET) = 13.00

  DISTANCE FROM CROWN TO CROSSFALL GRADEBREAK(FEET) =   8.00
  INSIDE STREET CROSSFALL(DECIMAL) =  0.020
  OUTSIDE STREET CROSSFALL(DECIMAL)  =  0.020

  SPECIFIED NUMBER OF HALFSTREETS CARRYING RUNOFF =  1
  STREET PARKWAY CROSSFALL(DECIMAL)  =  0.020
  Manning's FRICTION FACTOR for Streetflow Section(curb-to-curb) =   0.0180
  Manning's FRICTION FACTOR for Back-of-Walk Flow Section =   0.0200

    **TRAVEL TIME COMPUTED USING ESTIMATED FLOW(CFS) =       0.90
    STREETFLOW MODEL RESULTS USING ESTIMATED FLOW:
    STREET FLOW DEPTH(FEET) =  0.22
    HALFSTREET FLOOD WIDTH(FEET) =    4.64
    AVERAGE FLOW VELOCITY(FEET/SEC.) =    2.68
    PRODUCT OF DEPTH&VELOCITY(FT*FT/SEC.) =    0.59
  STREET FLOW TRAVEL TIME(MIN.) =   0.75   Tc(MIN.) =    4.23
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  8.431
  NOTE: RAINFALL INTENSITY IS BASED ON Tc = 5-MINUTE.
```

```
*USER SPECIFIED(SUBAREA):
USER-SPECIFIED RUNOFF COEFFICIENT = .8500
S.C.S. CURVE NUMBER (AMC II) =   0
AREA-AVERAGE RUNOFF COEFFICIENT =  0.850
SUBAREA AREA(ACRES) =    0.05      SUBAREA RUNOFF(CFS) =    0.36
TOTAL AREA(ACRES) =        0.2        PEAK FLOW RATE(CFS) =       1.07

END OF SUBAREA STREET FLOW HYDRAULICS:
DEPTH(FEET) = 0.23   HALFSTREET FLOOD WIDTH(FEET) =   5.22
FLOW VELOCITY(FEET/SEC.) =  2.75   DEPTH*VELOCITY(FT*FT/SEC.) =   0.63
LONGEST FLOWPATH FROM NODE    100.00 TO NODE    105.00 =     220.00 FEET.

****************************************************************************
FLOW PROCESS FROM NODE    105.00 TO NODE    115.00 IS CODE =  61
----------------------------------------------------------------------------
>>>>>COMPUTE STREET FLOW TRAVEL TIME THRU SUBAREA<<<<<
>>>>>(STANDARD CURB SECTION USED)<<<<<
============================================================================
UPSTREAM ELEVATION(FEET) =  498.50  DOWNSTREAM ELEVATION(FEET) =  488.50
STREET LENGTH(FEET) =   411.00   CURB HEIGHT(INCHES) =  6.0
STREET HALFWIDTH(FEET) = 13.00

DISTANCE FROM CROWN TO CROSSFALL GRADEBREAK(FEET) =   8.00
INSIDE STREET CROSSFALL(DECIMAL) =  0.020
OUTSIDE STREET CROSSFALL(DECIMAL)  =  0.020

SPECIFIED NUMBER OF HALFSTREETS CARRYING RUNOFF =  1
STREET PARKWAY CROSSFALL(DECIMAL)  =  0.020
Manning's FRICTION FACTOR for Streetflow Section(curb-to-curb) =   0.0180
Manning's FRICTION FACTOR for Back-of-Walk Flow Section =   0.0200

  **TRAVEL TIME COMPUTED USING ESTIMATED FLOW(CFS) =       1.89
  STREETFLOW MODEL RESULTS USING ESTIMATED FLOW:
  STREET FLOW DEPTH(FEET) =  0.28
  HALFSTREET FLOOD WIDTH(FEET) =    7.84
  AVERAGE FLOW VELOCITY(FEET/SEC.) =    2.58
  PRODUCT OF DEPTH&VELOCITY(FT*FT/SEC.) =    0.73
STREET FLOW TRAVEL TIME(MIN.) =   2.65   Tc(MIN.) =    6.89
 100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  6.859
*USER SPECIFIED(SUBAREA):
USER-SPECIFIED RUNOFF COEFFICIENT = .7900
S.C.S. CURVE NUMBER (AMC II) =   0
AREA-AVERAGE RUNOFF COEFFICIENT =  0.810
SUBAREA AREA(ACRES) =    0.30      SUBAREA RUNOFF(CFS) =    1.63
TOTAL AREA(ACRES) =        0.5        PEAK FLOW RATE(CFS) =       2.50

END OF SUBAREA STREET FLOW HYDRAULICS:
DEPTH(FEET) = 0.30   HALFSTREET FLOOD WIDTH(FEET) =   8.91
FLOW VELOCITY(FEET/SEC.) =  2.74   DEPTH*VELOCITY(FT*FT/SEC.) =   0.83
LONGEST FLOWPATH FROM NODE    100.00 TO NODE    115.00 =     631.00 FEET.
```

```
****************************************************************************
  FLOW PROCESS FROM NODE    115.00 TO NODE    120.00 IS CODE =  31
----------------------------------------------------------------------------
  >>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
  >>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<
============================================================================
  ELEVATION DATA: UPSTREAM(FEET) =   489.00  DOWNSTREAM(FEET) =   487.00
  FLOW LENGTH(FEET) =   100.00   MANNING'S N =  0.013
  ESTIMATED PIPE DIAMETER(INCH) INCREASED TO 18.000
  DEPTH OF FLOW IN  18.0 INCH PIPE IS   5.1 INCHES
  PIPE-FLOW VELOCITY(FEET/SEC.) =   6.03
  ESTIMATED PIPE DIAMETER(INCH) =  18.00    NUMBER OF PIPES =   1
  PIPE-FLOW(CFS) =       2.50
  PIPE TRAVEL TIME(MIN.) =   0.28    Tc(MIN.) =    7.16
  LONGEST FLOWPATH FROM NODE    100.00 TO NODE    120.00 =     731.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    120.00 TO NODE    120.00 IS CODE =   1
----------------------------------------------------------------------------
  >>>>>DESIGNATE INDEPENDENT STREAM FOR CONFLUENCE<<<<<
============================================================================
  TOTAL NUMBER OF STREAMS =  2
  CONFLUENCE VALUES USED FOR INDEPENDENT STREAM  1 ARE:
  TIME OF CONCENTRATION(MIN.) =    7.16
  RAINFALL INTENSITY(INCH/HR) =   6.69
  TOTAL STREAM AREA(ACRES) =     0.45
  PEAK FLOW RATE(CFS) AT CONFLUENCE =      2.50

****************************************************************************
  FLOW PROCESS FROM NODE    110.00 TO NODE    112.00 IS CODE =  21
----------------------------------------------------------------------------
  >>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<
============================================================================
  *USER SPECIFIED(SUBAREA):
  USER-SPECIFIED RUNOFF COEFFICIENT = .8500
  S.C.S. CURVE NUMBER (AMC II) =   0
  INITIAL SUBAREA FLOW-LENGTH(FEET) =   100.00
  UPSTREAM ELEVATION(FEET) =    504.00
  DOWNSTREAM ELEVATION(FEET) =    503.00
  ELEVATION DIFFERENCE(FEET) =      1.00
  SUBAREA OVERLAND TIME OF FLOW(MIN.) =    3.486
  WARNING: INITIAL SUBAREA FLOW PATH LENGTH IS GREATER THAN
           THE MAXIMUM OVERLAND FLOW LENGTH =    60.00
           (Reference: Table 3-1B of Hydrology Manual)
           THE MAXIMUM OVERLAND FLOW LENGTH IS USED IN Tc CALCULATION!
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  8.431
  NOTE: RAINFALL INTENSITY IS BASED ON Tc = 5-MINUTE.
  SUBAREA RUNOFF(CFS) =      0.72
  TOTAL AREA(ACRES) =      0.10   TOTAL RUNOFF(CFS) =      0.72
```

```
****************************************************************************
  FLOW PROCESS FROM NODE    112.00 TO NODE    115.00 IS CODE =  61
----------------------------------------------------------------------------
  >>>>>COMPUTE STREET FLOW TRAVEL TIME THRU SUBAREA<<<<<
  >>>>>(STANDARD CURB SECTION USED)<<<<<
============================================================================
  UPSTREAM ELEVATION(FEET) =  503.00  DOWNSTREAM ELEVATION(FEET) =  490.50
  STREET LENGTH(FEET) =   411.00   CURB HEIGHT(INCHES) =  6.0
  STREET HALFWIDTH(FEET) = 13.00

  DISTANCE FROM CROWN TO CROSSFALL GRADEBREAK(FEET) =   8.00
  INSIDE STREET CROSSFALL(DECIMAL) =  0.020
  OUTSIDE STREET CROSSFALL(DECIMAL)  =  0.020

  SPECIFIED NUMBER OF HALFSTREETS CARRYING RUNOFF =  1
  STREET PARKWAY CROSSFALL(DECIMAL)  =  0.020
  Manning's FRICTION FACTOR for Streetflow Section(curb-to-curb) =   0.0180
  Manning's FRICTION FACTOR for Back-of-Walk Flow Section =   0.0200

    **TRAVEL TIME COMPUTED USING ESTIMATED FLOW(CFS) =       2.87
    STREETFLOW MODEL RESULTS USING ESTIMATED FLOW:
    STREET FLOW DEPTH(FEET) =  0.31
    HALFSTREET FLOOD WIDTH(FEET) =    9.03
    AVERAGE FLOW VELOCITY(FEET/SEC.) =    3.08
    PRODUCT OF DEPTH&VELOCITY(FT*FT/SEC.) =    0.94
  STREET FLOW TRAVEL TIME(MIN.) =   2.23   Tc(MIN.) =    5.71
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  7.738
  *USER SPECIFIED(SUBAREA):
  USER-SPECIFIED RUNOFF COEFFICIENT = .7900
  S.C.S. CURVE NUMBER (AMC II) =   0
  AREA-AVERAGE RUNOFF COEFFICIENT =  0.797
  SUBAREA AREA(ACRES) =    0.70      SUBAREA RUNOFF(CFS) =    4.28
  TOTAL AREA(ACRES) =        0.8        PEAK FLOW RATE(CFS) =       4.94

  END OF SUBAREA STREET FLOW HYDRAULICS:
  DEPTH(FEET) = 0.35   HALFSTREET FLOOD WIDTH(FEET) =  11.34
  FLOW VELOCITY(FEET/SEC.) =  3.51   DEPTH*VELOCITY(FT*FT/SEC.) =   1.24
  LONGEST FLOWPATH FROM NODE    110.00 TO NODE    115.00 =     511.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    115.00 TO NODE    117.00 IS CODE =  41
----------------------------------------------------------------------------
  >>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
  >>>>>USING USER-SPECIFIED PIPESIZE (EXISTING ELEMENT)<<<<<
============================================================================
  ELEVATION DATA: UPSTREAM(FEET) =   487.00  DOWNSTREAM(FEET) =   486.00
  FLOW LENGTH(FEET) =   105.00   MANNING'S N =  0.013
  DEPTH OF FLOW IN  18.0 INCH PIPE IS   9.1 INCHES
  PIPE-FLOW VELOCITY(FEET/SEC.) =   5.52
```

```
  GIVEN PIPE DIAMETER(INCH) =  18.00    NUMBER OF PIPES =   1
  PIPE-FLOW(CFS) =       4.94
  PIPE TRAVEL TIME(MIN.) =   0.32    Tc(MIN.) =    6.03
  LONGEST FLOWPATH FROM NODE    110.00 TO NODE    117.00 =     616.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    117.00 TO NODE    117.00 IS CODE =  81
----------------------------------------------------------------------------
  >>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
============================================================================
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  7.473
  *USER SPECIFIED(SUBAREA):
  USER-SPECIFIED RUNOFF COEFFICIENT = .7900
  S.C.S. CURVE NUMBER (AMC II) =   0
  AREA-AVERAGE RUNOFF COEFFICIENT = 0.7943
  SUBAREA AREA(ACRES) =    0.60   SUBAREA RUNOFF(CFS) =    3.54
  TOTAL AREA(ACRES) =        1.4   TOTAL RUNOFF(CFS) =       8.31
  TC(MIN.) =    6.03

****************************************************************************
  FLOW PROCESS FROM NODE    117.00 TO NODE    120.00 IS CODE =  41
----------------------------------------------------------------------------
  >>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
  >>>>>USING USER-SPECIFIED PIPESIZE (EXISTING ELEMENT)<<<<<
============================================================================
  ELEVATION DATA: UPSTREAM(FEET) =   486.00  DOWNSTREAM(FEET) =   485.50
  FLOW LENGTH(FEET) =    26.00   MANNING'S N =  0.013
  DEPTH OF FLOW IN  18.0 INCH PIPE IS  10.0 INCHES
  PIPE-FLOW VELOCITY(FEET/SEC.) =   8.20
  GIVEN PIPE DIAMETER(INCH) =  18.00    NUMBER OF PIPES =   1
  PIPE-FLOW(CFS) =       8.31
  PIPE TRAVEL TIME(MIN.) =   0.05    Tc(MIN.) =    6.08
  LONGEST FLOWPATH FROM NODE    110.00 TO NODE    120.00 =     642.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    120.00 TO NODE    120.00 IS CODE =   1
----------------------------------------------------------------------------
  >>>>>DESIGNATE INDEPENDENT STREAM FOR CONFLUENCE<<<<<
  >>>>>AND COMPUTE VARIOUS CONFLUENCED STREAM VALUES<<<<<
============================================================================
  TOTAL NUMBER OF STREAMS =  2
  CONFLUENCE VALUES USED FOR INDEPENDENT STREAM  2 ARE:
  TIME OF CONCENTRATION(MIN.) =    6.08
  RAINFALL INTENSITY(INCH/HR) =   7.43
  TOTAL STREAM AREA(ACRES) =     1.40
  PEAK FLOW RATE(CFS) AT CONFLUENCE =      8.31

  ** CONFLUENCE DATA **
  STREAM     RUNOFF       Tc      INTENSITY      AREA
  NUMBER      (CFS)     (MIN.)   (INCH/HOUR)    (ACRE)
```

```
     1        2.50     7.16        6.687          0.45
     2        8.31     6.08        7.431          1.40

  RAINFALL INTENSITY AND TIME OF CONCENTRATION RATIO
  CONFLUENCE FORMULA USED FOR  2 STREAMS.

  ** PEAK FLOW RATE TABLE **
  STREAM     RUNOFF      Tc      INTENSITY
  NUMBER      (CFS)    (MIN.)   (INCH/HOUR)
     1       10.43     6.08       7.431
     2        9.98     7.16       6.687

  COMPUTED CONFLUENCE ESTIMATES ARE AS FOLLOWS:
  PEAK FLOW RATE(CFS) =      10.43   Tc(MIN.) =    6.08
  TOTAL AREA(ACRES) =        1.9
  LONGEST FLOWPATH FROM NODE    100.00 TO NODE    120.00 =     731.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    120.00 TO NODE    120.00 IS CODE =  81
----------------------------------------------------------------------------
  >>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
============================================================================
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  7.431
  *USER SPECIFIED(SUBAREA):
  USER-SPECIFIED RUNOFF COEFFICIENT = .6800
  S.C.S. CURVE NUMBER (AMC II) =   0
  AREA-AVERAGE RUNOFF COEFFICIENT = 0.7771
  SUBAREA AREA(ACRES) =    0.40   SUBAREA RUNOFF(CFS) =    2.02
  TOTAL AREA(ACRES) =        2.3   TOTAL RUNOFF(CFS) =      12.99
  TC(MIN.) =    6.08

****************************************************************************
  FLOW PROCESS FROM NODE    120.00 TO NODE    120.00 IS CODE =  81
----------------------------------------------------------------------------
  >>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
============================================================================
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  7.431
  *USER SPECIFIED(SUBAREA):
  USER-SPECIFIED RUNOFF COEFFICIENT = .7900
  S.C.S. CURVE NUMBER (AMC II) =   0
  AREA-AVERAGE RUNOFF COEFFICIENT = 0.7782
  SUBAREA AREA(ACRES) =    0.20   SUBAREA RUNOFF(CFS) =    1.17
  TOTAL AREA(ACRES) =        2.5   TOTAL RUNOFF(CFS) =      14.17
  TC(MIN.) =    6.08

****************************************************************************
  FLOW PROCESS FROM NODE    120.00 TO NODE    120.00 IS CODE =   7
----------------------------------------------------------------------------
  >>>>>USER SPECIFIED HYDROLOGY INFORMATION AT NODE<<<<<
============================================================================
```

AES ROUNDS TO 2.5 AC - ACTUAL AREA 2.45 ACRES AS INPUT IN CODE 7. SEE RUNOFF COEFFICIENT/AREA TABLE FOR BACKUP

```
 USER-SPECIFIED VALUES ARE AS FOLLOWS:
 TC(MIN) =  10.88   RAIN INTENSITY(INCH/HOUR) =  5.11
 TOTAL AREA(ACRES) =     2.45   TOTAL RUNOFF(CFS) =     4.30

****************************************************************************
  FLOW PROCESS FROM NODE    120.00 TO NODE    120.00 IS CODE =  81
----------------------------------------------------------------------------
  >>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
============================================================================
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  5.106
  *USER SPECIFIED(SUBAREA):
  USER-SPECIFIED RUNOFF COEFFICIENT = .3800
  S.C.S. CURVE NUMBER (AMC II) =   0
  AREA-AVERAGE RUNOFF COEFFICIENT = 0.3465
  SUBAREA AREA(ACRES) =    0.20   SUBAREA RUNOFF(CFS) =    0.39
  TOTAL AREA(ACRES) =        2.7   TOTAL RUNOFF(CFS) =       4.69
  TC(MIN.) =   10.88

****************************************************************************
  FLOW PROCESS FROM NODE    120.00 TO NODE    120.00 IS CODE =  81
----------------------------------------------------------------------------
  >>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
============================================================================
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  5.106
  *USER SPECIFIED(SUBAREA):
  USER-SPECIFIED RUNOFF COEFFICIENT = .3800
  S.C.S. CURVE NUMBER (AMC II) =   0
  AREA-AVERAGE RUNOFF COEFFICIENT = 0.3518
  SUBAREA AREA(ACRES) =    0.50   SUBAREA RUNOFF(CFS) =    0.97
  TOTAL AREA(ACRES) =        3.2   TOTAL RUNOFF(CFS) =       5.66
  TC(MIN.) =   10.88

****************************************************************************
  FLOW PROCESS FROM NODE    120.00 TO NODE    199.00 IS CODE =  41
----------------------------------------------------------------------------
  >>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
  >>>>>USING USER-SPECIFIED PIPESIZE (EXISTING ELEMENT)<<<<<
============================================================================
  ELEVATION DATA: UPSTREAM(FEET) =   474.00  DOWNSTREAM(FEET) =   468.80
  FLOW LENGTH(FEET) =   380.00   MANNING'S N =  0.013
  DEPTH OF FLOW IN  24.0 INCH PIPE IS   7.7 INCHES
  PIPE-FLOW VELOCITY(FEET/SEC.) =   6.47
  GIVEN PIPE DIAMETER(INCH) =  24.00    NUMBER OF PIPES =   1
  PIPE-FLOW(CFS) =       5.66
  PIPE TRAVEL TIME(MIN.) =   0.98    Tc(MIN.) =   11.86
  LONGEST FLOWPATH FROM NODE    100.00 TO NODE    199.00 =    1111.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    199.00 TO NODE    199.00 IS CODE =   1
----------------------------------------------------------------------------
```

```
  >>>>>DESIGNATE INDEPENDENT STREAM FOR CONFLUENCE<<<<<
============================================================================
  TOTAL NUMBER OF STREAMS =  2
  CONFLUENCE VALUES USED FOR INDEPENDENT STREAM  1 ARE:
  TIME OF CONCENTRATION(MIN.) =   11.86
  RAINFALL INTENSITY(INCH/HR) =   4.83
  TOTAL STREAM AREA(ACRES) =     3.15
  PEAK FLOW RATE(CFS) AT CONFLUENCE =      5.66

****************************************************************************
  FLOW PROCESS FROM NODE    130.00 TO NODE    132.00 IS CODE =  21
----------------------------------------------------------------------------
  >>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<
============================================================================
  *USER SPECIFIED(SUBAREA):
  USER-SPECIFIED RUNOFF COEFFICIENT = .7900
  S.C.S. CURVE NUMBER (AMC II) =   0
  INITIAL SUBAREA FLOW-LENGTH(FEET) =   100.00
  UPSTREAM ELEVATION(FEET) =    484.00
  DOWNSTREAM ELEVATION(FEET) =    483.00
  ELEVATION DIFFERENCE(FEET) =      1.00
  SUBAREA OVERLAND TIME OF FLOW(MIN.) =    4.499
  WARNING: INITIAL SUBAREA FLOW PATH LENGTH IS GREATER THAN
           THE MAXIMUM OVERLAND FLOW LENGTH =    65.00
           (Reference: Table 3-1B of Hydrology Manual)
           THE MAXIMUM OVERLAND FLOW LENGTH IS USED IN Tc CALCULATION!
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  8.431
  NOTE: RAINFALL INTENSITY IS BASED ON Tc = 5-MINUTE.
  SUBAREA RUNOFF(CFS) =      0.67
  TOTAL AREA(ACRES) =      0.10   TOTAL RUNOFF(CFS) =      0.67

****************************************************************************
  FLOW PROCESS FROM NODE    132.00 TO NODE    135.00 IS CODE =  61
----------------------------------------------------------------------------
  >>>>>COMPUTE STREET FLOW TRAVEL TIME THRU SUBAREA<<<<<
  >>>>>(STANDARD CURB SECTION USED)<<<<<
============================================================================
  UPSTREAM ELEVATION(FEET) =  483.00  DOWNSTREAM ELEVATION(FEET) =  480.00
  STREET LENGTH(FEET) =   210.00   CURB HEIGHT(INCHES) =  6.0
  STREET HALFWIDTH(FEET) = 13.00

  DISTANCE FROM CROWN TO CROSSFALL GRADEBREAK(FEET) =   8.00
  INSIDE STREET CROSSFALL(DECIMAL) =  0.020
  OUTSIDE STREET CROSSFALL(DECIMAL)  =  0.020

  SPECIFIED NUMBER OF HALFSTREETS CARRYING RUNOFF =  1
  STREET PARKWAY CROSSFALL(DECIMAL)  =  0.020
  Manning's FRICTION FACTOR for Streetflow Section(curb-to-curb) =   0.0180
  Manning's FRICTION FACTOR for Back-of-Walk Flow Section =   0.0200
```

```
    **TRAVEL TIME COMPUTED USING ESTIMATED FLOW(CFS) =       1.83
    STREETFLOW MODEL RESULTS USING ESTIMATED FLOW:
    STREET FLOW DEPTH(FEET) =  0.30
    HALFSTREET FLOOD WIDTH(FEET) =    8.72
    AVERAGE FLOW VELOCITY(FEET/SEC.) =    2.08
    PRODUCT OF DEPTH&VELOCITY(FT*FT/SEC.) =    0.63
  STREET FLOW TRAVEL TIME(MIN.) =   1.68   Tc(MIN.) =    6.18
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  7.356
  *USER SPECIFIED(SUBAREA):
  USER-SPECIFIED RUNOFF COEFFICIENT = .7900
  S.C.S. CURVE NUMBER (AMC II) =   0
  AREA-AVERAGE RUNOFF COEFFICIENT =  0.790
  SUBAREA AREA(ACRES) =    0.40      SUBAREA RUNOFF(CFS) =    2.32
  TOTAL AREA(ACRES) =        0.5        PEAK FLOW RATE(CFS) =       2.91

  END OF SUBAREA STREET FLOW HYDRAULICS:
  DEPTH(FEET) = 0.34   HALFSTREET FLOOD WIDTH(FEET) =  10.66
  FLOW VELOCITY(FEET/SEC.) =  2.32   DEPTH*VELOCITY(FT*FT/SEC.) =   0.79
  LONGEST FLOWPATH FROM NODE    130.00 TO NODE    135.00 =     310.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    135.00 TO NODE    140.00 IS CODE =  41
----------------------------------------------------------------------------
  >>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
  >>>>>USING USER-SPECIFIED PIPESIZE (EXISTING ELEMENT)<<<<<
============================================================================
  ELEVATION DATA: UPSTREAM(FEET) =   477.00  DOWNSTREAM(FEET) =   476.50
  FLOW LENGTH(FEET) =    26.00   MANNING'S N =  0.013
  DEPTH OF FLOW IN  18.0 INCH PIPE IS   5.6 INCHES
  PIPE-FLOW VELOCITY(FEET/SEC.) =   6.20
  GIVEN PIPE DIAMETER(INCH) =  18.00    NUMBER OF PIPES =   1
  PIPE-FLOW(CFS) =       2.91
  PIPE TRAVEL TIME(MIN.) =   0.07    Tc(MIN.) =    6.25
  LONGEST FLOWPATH FROM NODE    130.00 TO NODE    140.00 =     336.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    140.00 TO NODE    140.00 IS CODE =  81
----------------------------------------------------------------------------
  >>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
============================================================================
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  7.303
  *USER SPECIFIED(SUBAREA):
  USER-SPECIFIED RUNOFF COEFFICIENT = .7900
  S.C.S. CURVE NUMBER (AMC II) =   0
  AREA-AVERAGE RUNOFF COEFFICIENT = 0.7900
  SUBAREA AREA(ACRES) =    0.10   SUBAREA RUNOFF(CFS) =    0.58
  TOTAL AREA(ACRES) =        0.6   TOTAL RUNOFF(CFS) =       3.46
  TC(MIN.) =    6.25

****************************************************************************
```

```
  FLOW PROCESS FROM NODE    140.00 TO NODE    140.00 IS CODE =  81
----------------------------------------------------------------------------
  >>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
============================================================================
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  7.303
  *USER SPECIFIED(SUBAREA):
  USER-SPECIFIED RUNOFF COEFFICIENT = .7900
  S.C.S. CURVE NUMBER (AMC II) =   0
  AREA-AVERAGE RUNOFF COEFFICIENT = 0.7900
  SUBAREA AREA(ACRES) =    0.50   SUBAREA RUNOFF(CFS) =    2.88
  TOTAL AREA(ACRES) =        1.1   TOTAL RUNOFF(CFS) =       6.35
  TC(MIN.) =    6.25

****************************************************************************
  FLOW PROCESS FROM NODE    140.00 TO NODE    140.00 IS CODE =  81
----------------------------------------------------------------------------
  >>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
============================================================================
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  7.303
  *USER SPECIFIED(SUBAREA):
  USER-SPECIFIED RUNOFF COEFFICIENT = .7900
  S.C.S. CURVE NUMBER (AMC II) =   0
  AREA-AVERAGE RUNOFF COEFFICIENT = 0.7900
  SUBAREA AREA(ACRES) =    0.60   SUBAREA RUNOFF(CFS) =    3.46
  TOTAL AREA(ACRES) =        1.7   TOTAL RUNOFF(CFS) =       9.81
  TC(MIN.) =    6.25

****************************************************************************
  FLOW PROCESS FROM NODE    140.00 TO NODE    145.00 IS CODE =  41
----------------------------------------------------------------------------
  >>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
  >>>>>USING USER-SPECIFIED PIPESIZE (EXISTING ELEMENT)<<<<<
============================================================================
  ELEVATION DATA: UPSTREAM(FEET) =   471.50  DOWNSTREAM(FEET) =   471.00
  FLOW LENGTH(FEET) =   125.00   MANNING'S N =  0.013
  DEPTH OF FLOW IN  24.0 INCH PIPE IS  15.1 INCHES
  PIPE-FLOW VELOCITY(FEET/SEC.) =   4.72
  GIVEN PIPE DIAMETER(INCH) =  24.00    NUMBER OF PIPES =   1
  PIPE-FLOW(CFS) =       9.81
  PIPE TRAVEL TIME(MIN.) =   0.44    Tc(MIN.) =    6.69
  LONGEST FLOWPATH FROM NODE    130.00 TO NODE    145.00 =     461.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    145.00 TO NODE    150.00 IS CODE =  51
----------------------------------------------------------------------------
  >>>>>COMPUTE TRAPEZOIDAL CHANNEL FLOW<<<<<
  >>>>>TRAVELTIME THRU SUBAREA (EXISTING ELEMENT)<<<<<
============================================================================
  ELEVATION DATA: UPSTREAM(FEET) =    471.00  DOWNSTREAM(FEET) =    470.90
  CHANNEL LENGTH THRU SUBAREA(FEET) =   100.00   CHANNEL SLOPE =  0.0010
```

AES ROUNDS TO 1.8 AC - ACTUAL AREA 1.75 ACRES AS INPUT IN CODE 7. SEE RUNOFF COEFFICIENT/AREA TABLE FOR BACKUP

```
  CHANNEL BASE(FEET) =   10.00   "Z" FACTOR =  12.000
  MANNING'S FACTOR = 0.060   MAXIMUM DEPTH(FEET) =   3.00
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  5.480
  *USER SPECIFIED(SUBAREA):
  USER-SPECIFIED RUNOFF COEFFICIENT = .3500
  S.C.S. CURVE NUMBER (AMC II) =   0
  TRAVEL TIME COMPUTED USING ESTIMATED FLOW(CFS) =       9.86
  TRAVEL TIME THRU SUBAREA BASED ON VELOCITY(FEET/SEC.) =   0.54
  AVERAGE FLOW DEPTH(FEET) =   0.88   TRAVEL TIME(MIN.) =   3.06
  Tc(MIN.) =    9.75
  SUBAREA AREA(ACRES) =     0.05       SUBAREA RUNOFF(CFS) =    0.10
  AREA-AVERAGE RUNOFF COEFFICIENT =  0.777
  TOTAL AREA(ACRES) =         1.8          PEAK FLOW RATE(CFS) =       9.81

  END OF SUBAREA CHANNEL FLOW HYDRAULICS:
  DEPTH(FEET) =  0.88   FLOW VELOCITY(FEET/SEC.) =   0.54
  LONGEST FLOWPATH FROM NODE    130.00 TO NODE    150.00 =     561.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    150.00 TO NODE    150.00 IS CODE =   7
----------------------------------------------------------------------------
  >>>>>USER SPECIFIED HYDROLOGY INFORMATION AT NODE<<<<<
============================================================================
  USER-SPECIFIED VALUES ARE AS FOLLOWS:
  TC(MIN) =  18.20   RAIN INTENSITY(INCH/HOUR) =  3.66
  TOTAL AREA(ACRES) =     1.75   TOTAL RUNOFF(CFS) =      3.30

****************************************************************************
  FLOW PROCESS FROM NODE    199.00 TO NODE    199.00 IS CODE =   1
----------------------------------------------------------------------------
  >>>>>DESIGNATE INDEPENDENT STREAM FOR CONFLUENCE<<<<<
  >>>>>AND COMPUTE VARIOUS CONFLUENCED STREAM VALUES<<<<<
============================================================================
  TOTAL NUMBER OF STREAMS =  2
  CONFLUENCE VALUES USED FOR INDEPENDENT STREAM  2 ARE:
  TIME OF CONCENTRATION(MIN.) =   18.20
  RAINFALL INTENSITY(INCH/HR) =   3.66
  TOTAL STREAM AREA(ACRES) =     1.75
  PEAK FLOW RATE(CFS) AT CONFLUENCE =      3.30

  ** CONFLUENCE DATA **
  STREAM     RUNOFF       Tc      INTENSITY      AREA
  NUMBER      (CFS)     (MIN.)   (INCH/HOUR)    (ACRE)
     1         5.66    11.86        4.830          3.15
     2         3.30    18.20        3.664          1.75

  RAINFALL INTENSITY AND TIME OF CONCENTRATION RATIO
  CONFLUENCE FORMULA USED FOR  2 STREAMS.

  ** PEAK FLOW RATE TABLE **
```

```
  STREAM     RUNOFF      Tc      INTENSITY
  NUMBER      (CFS)    (MIN.)   (INCH/HOUR)
      1        7.81    11.86       4.830
      2        7.59    18.20       3.664

  COMPUTED CONFLUENCE ESTIMATES ARE AS FOLLOWS:
  PEAK FLOW RATE(CFS) =       7.81   Tc(MIN.) =   11.86
  TOTAL AREA(ACRES) =        4.9
  LONGEST FLOWPATH FROM NODE    100.00 TO NODE    199.00 =    1111.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    199.00 TO NODE    199.00 IS CODE =  81
----------------------------------------------------------------------------
  >>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
============================================================================
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  4.830
  *USER SPECIFIED(SUBAREA):
  USER-SPECIFIED RUNOFF COEFFICIENT = .3500
  S.C.S. CURVE NUMBER (AMC II) =   0
  AREA-AVERAGE RUNOFF COEFFICIENT = 0.4087
  SUBAREA AREA(ACRES) =    0.10   SUBAREA RUNOFF(CFS) =    0.17
  TOTAL AREA(ACRES) =        5.0   TOTAL RUNOFF(CFS) =       9.87
  TC(MIN.) =   11.86

****************************************************************************
  FLOW PROCESS FROM NODE    199.00 TO NODE    199.00 IS CODE =  81
----------------------------------------------------------------------------
  >>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
============================================================================
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  4.830
  *USER SPECIFIED(SUBAREA):
  USER-SPECIFIED RUNOFF COEFFICIENT = .3500
  S.C.S. CURVE NUMBER (AMC II) =   0
  AREA-AVERAGE RUNOFF COEFFICIENT = 0.4076
  SUBAREA AREA(ACRES) =    0.10   SUBAREA RUNOFF(CFS) =    0.17
  TOTAL AREA(ACRES) =        5.1   TOTAL RUNOFF(CFS) =      10.04
  TC(MIN.) =   11.86

****************************************************************************
  FLOW PROCESS FROM NODE    199.00 TO NODE    199.00 IS CODE =  81
----------------------------------------------------------------------------
  >>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
============================================================================
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  4.830
  *USER SPECIFIED(SUBAREA):
  USER-SPECIFIED RUNOFF COEFFICIENT = .6800
  S.C.S. CURVE NUMBER (AMC II) =   0
  AREA-AVERAGE RUNOFF COEFFICIENT = 0.4128
  SUBAREA AREA(ACRES) =    0.10   SUBAREA RUNOFF(CFS) =    0.33
  TOTAL AREA(ACRES) =        5.2   TOTAL RUNOFF(CFS) =      10.37
```

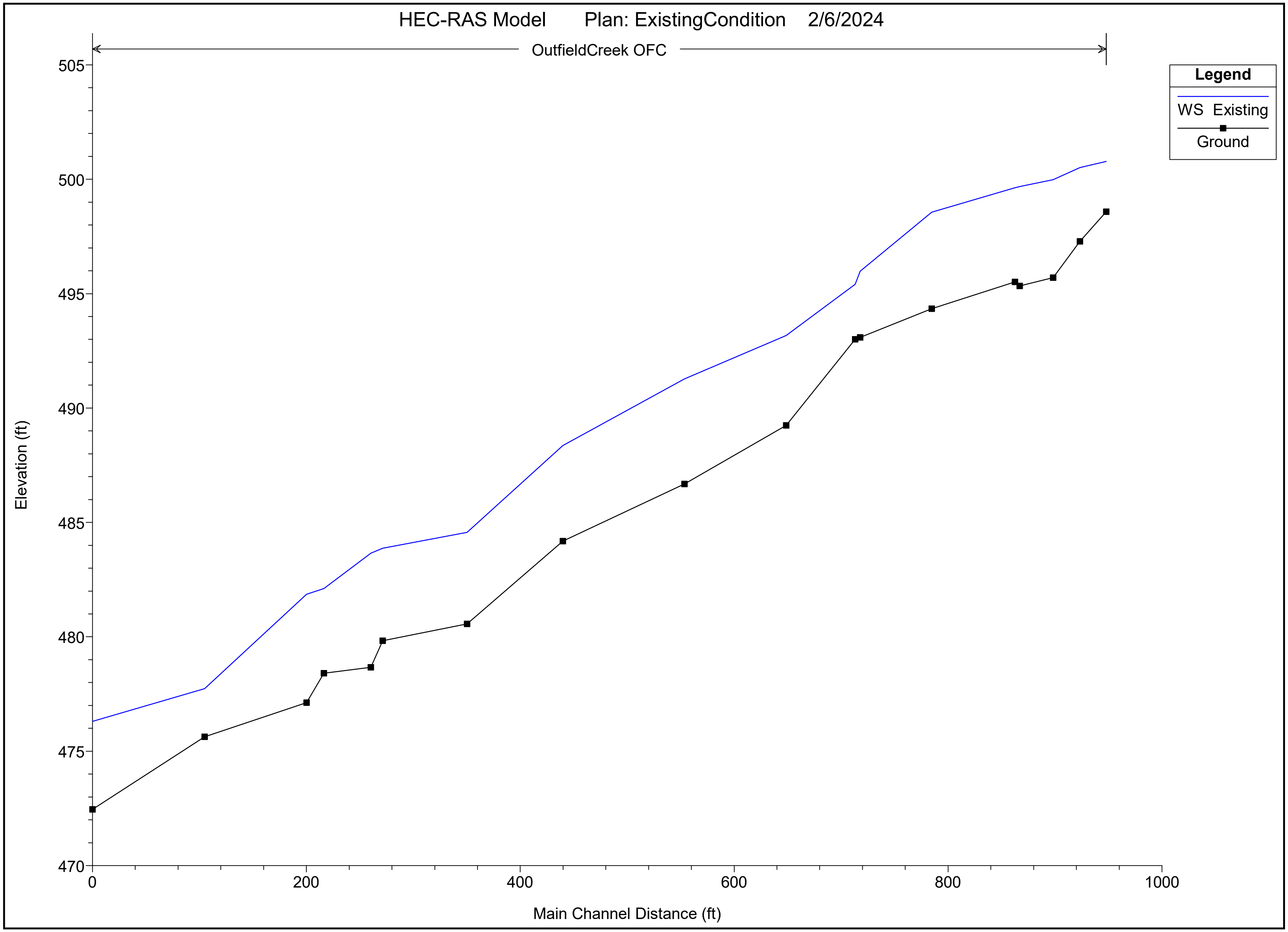
```
  TC(MIN.) =   11.86

****************************************************************************
  FLOW PROCESS FROM NODE     199.00 TO NODE     199.00 IS CODE =  81
----------------------------------------------------------------------------
  >>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
============================================================================
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  4.830
  *USER SPECIFIED(SUBAREA):
  USER-SPECIFIED RUNOFF COEFFICIENT = .7900
  S.C.S. CURVE NUMBER (AMC II) =   0
  AREA-AVERAGE RUNOFF COEFFICIENT = 0.4199
  SUBAREA AREA(ACRES) =    0.10   SUBAREA RUNOFF(CFS) =    0.38
  TOTAL AREA(ACRES) =        5.3   TOTAL RUNOFF(CFS) =      10.75
  TC(MIN.) =   11.86
============================================================================
  END OF STUDY SUMMARY:
  TOTAL AREA(ACRES)     =        5.3  TC(MIN.) =     11.86
  PEAK FLOW RATE(CFS)   =      10.75
============================================================================
============================================================================
  END OF RATIONAL METHOD ANALYSIS
```

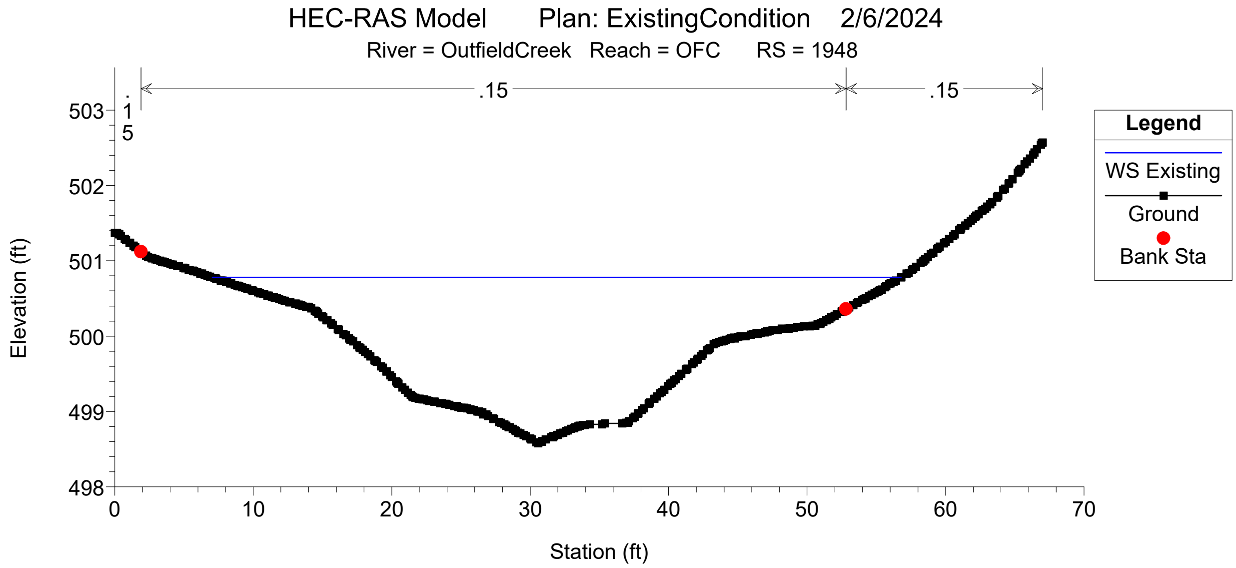
Q100-DETAINED
(Q100-PRE = 11.2 CFS)

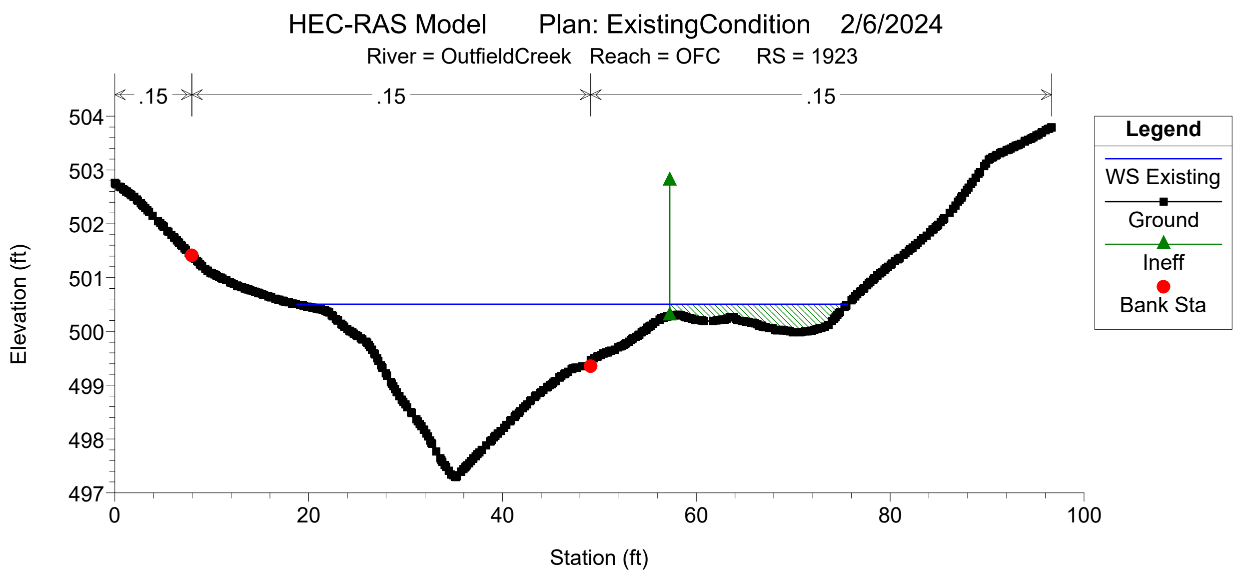
APPENDIX I

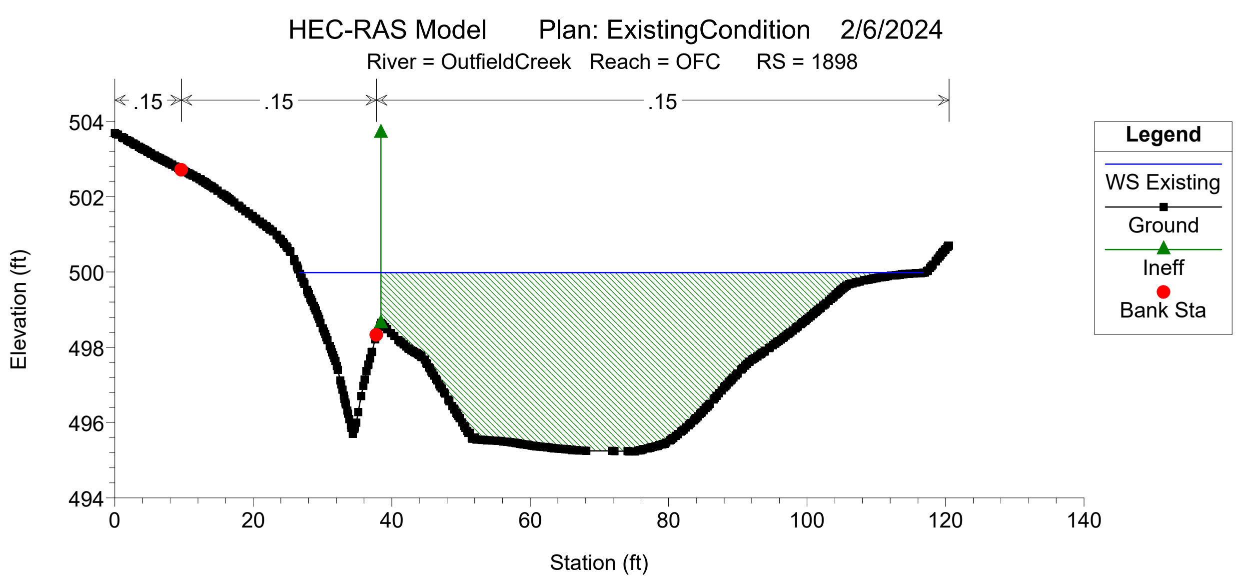
HEC-RAS CROSS SECTIONS AND WATER SURFACE ELEVATIONS

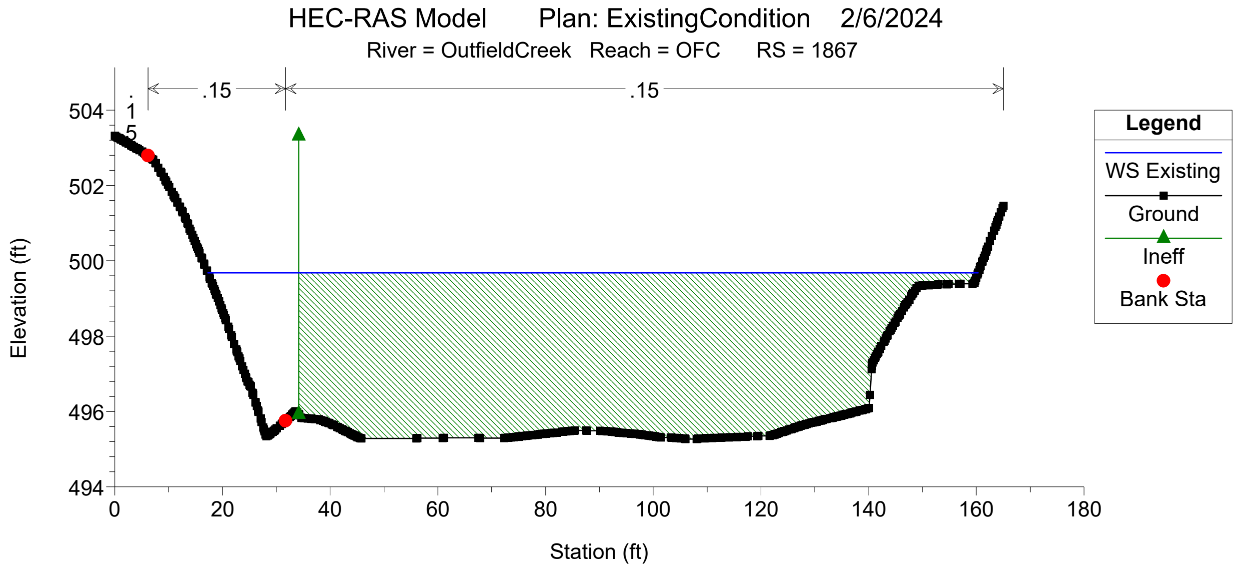


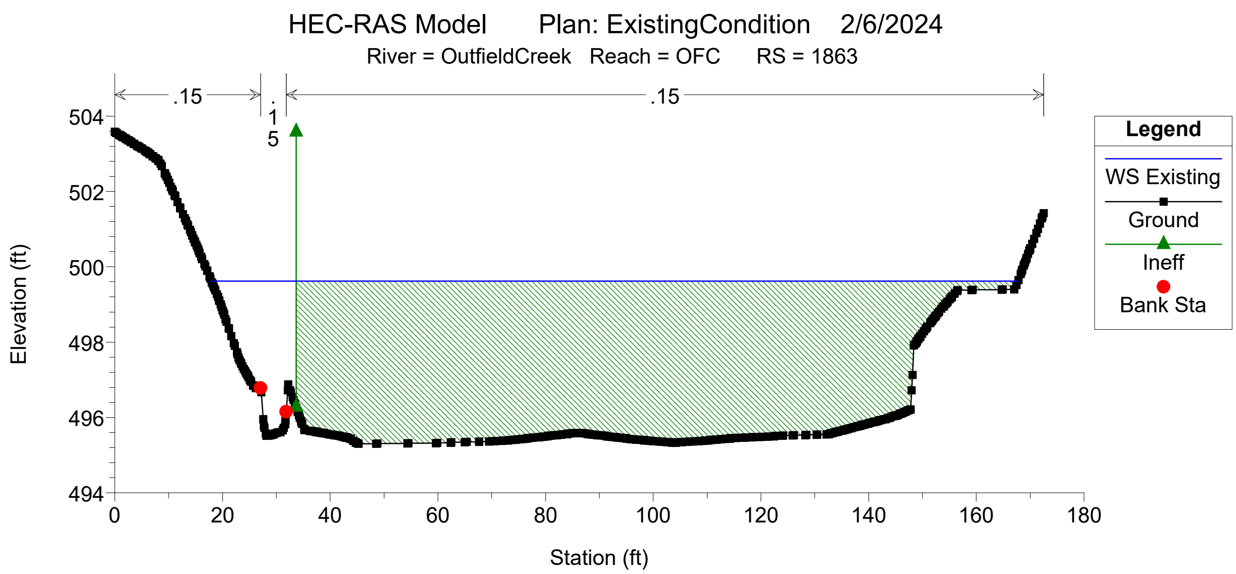
HEC-RAS Model Plan: ExistingCondition 2/6/2024
OutfieldCreek OFC
Legend
WS Existing
Ground
Elevation (ft)
505
500
495
490
485
480
475
470
0
200
400
600
800
1000
Main Channel Distance (ft)

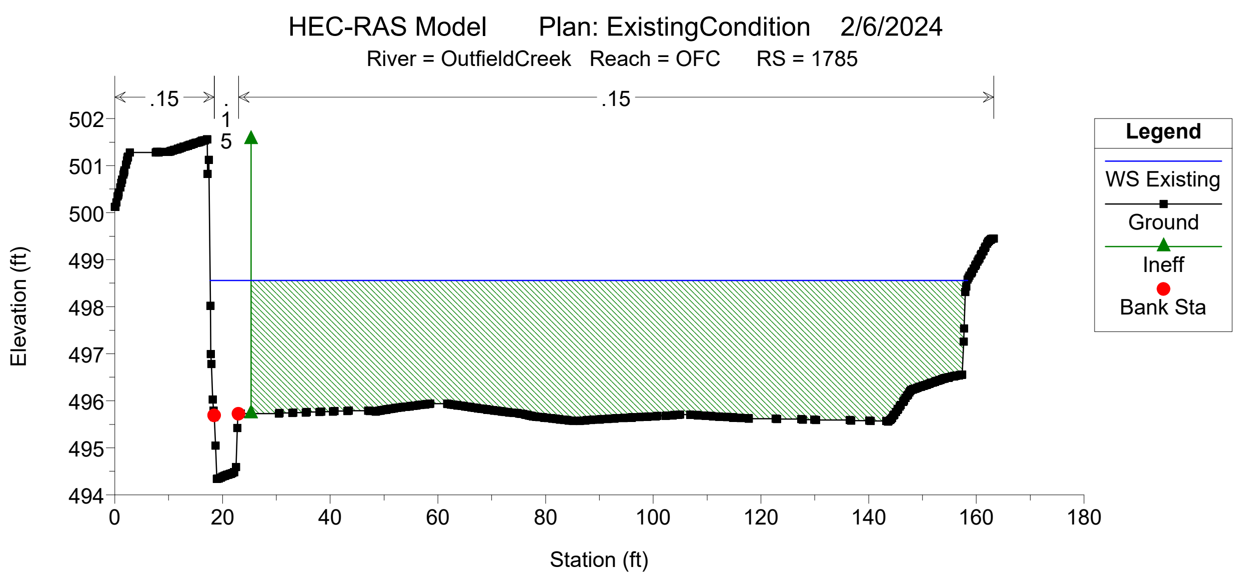

HEC-RAS Model Plan: ExistingCondition 2/6/2024
River = OutfieldCreek Reach = OFC RS = 1948
.15
.15
.15
Elevation (ft)
Station (ft)
Legend
WS Existing
Ground
Bank Sta

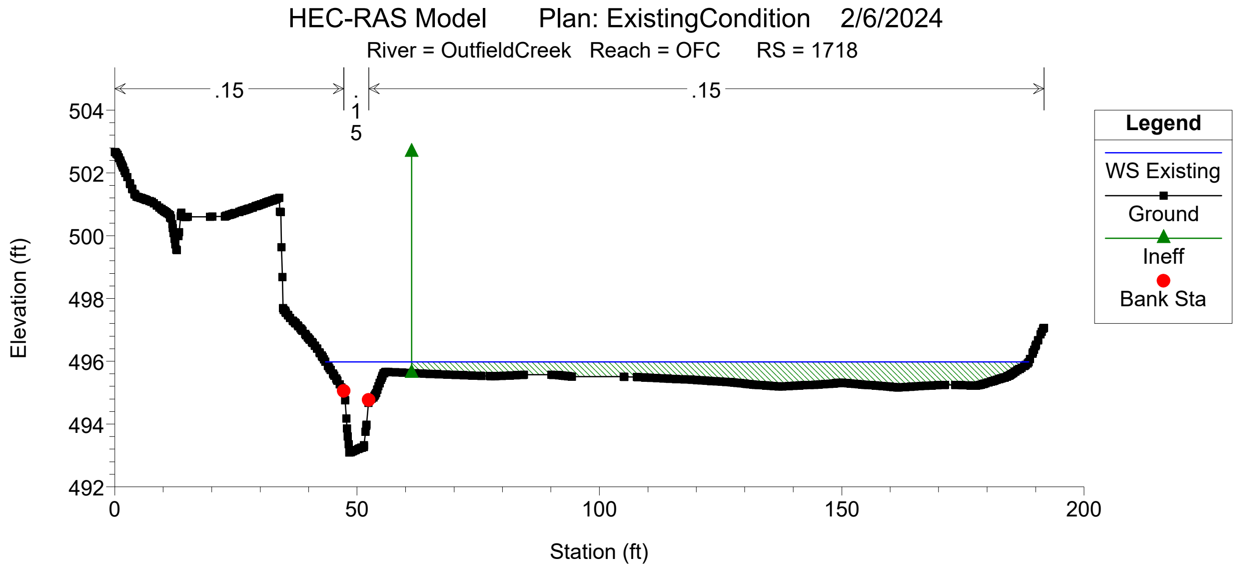

HEC-RAS Model Plan: ExistingCondition 2/6/2024
River = OutfieldCreek Reach = OFC RS = 1923
.15
.15
.15
Elevation (ft)
Station (ft)
Legend
WS Existing
Ground
Ineff
Bank Sta

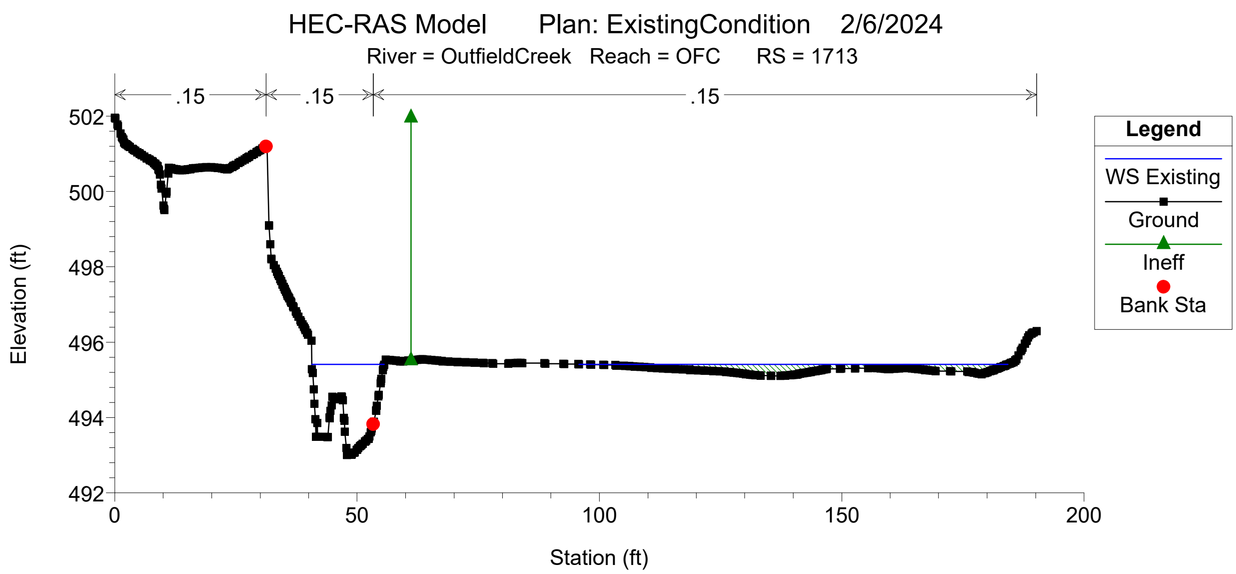

HEC-RAS Model Plan: ExistingCondition 2/6/2024
River = OutfieldCreek Reach = OFC RS = 1898
.15
.15
.15
Elevation (ft)
Station (ft)
Legend
WS Existing
Ground
Ineff
Bank Sta

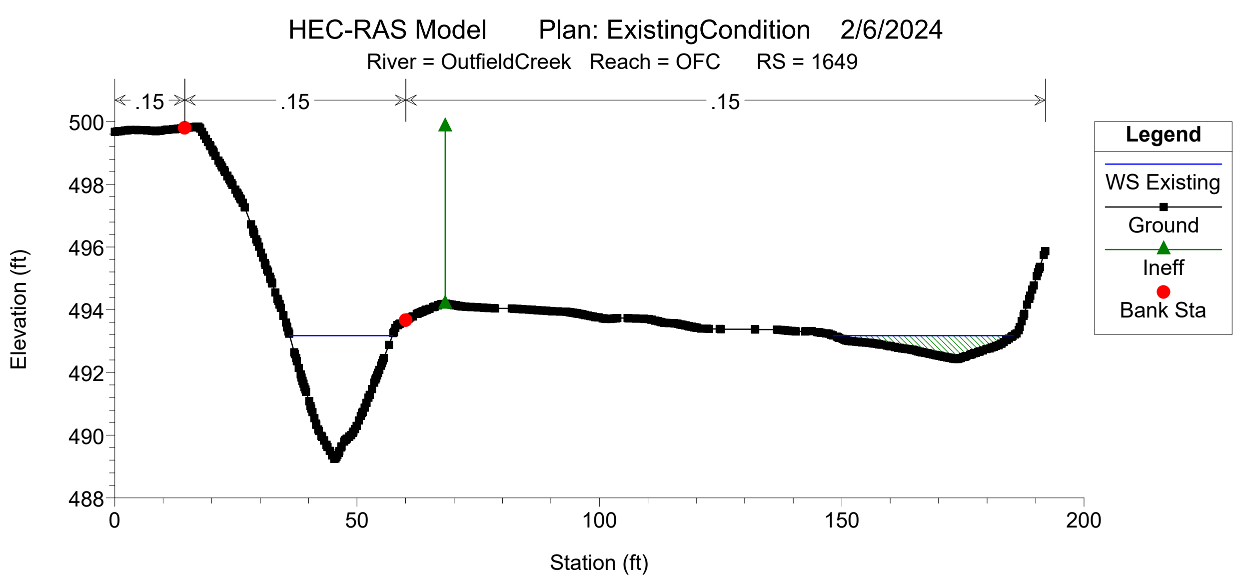

HEC-RAS Model Plan: ExistingCondition 2/6/2024
River = OutfieldCreek Reach = OFC RS = 1867
.15
.15
.15
Elevation (ft)
Station (ft)
Legend
WS Existing
Ground
Ineff
Bank Sta

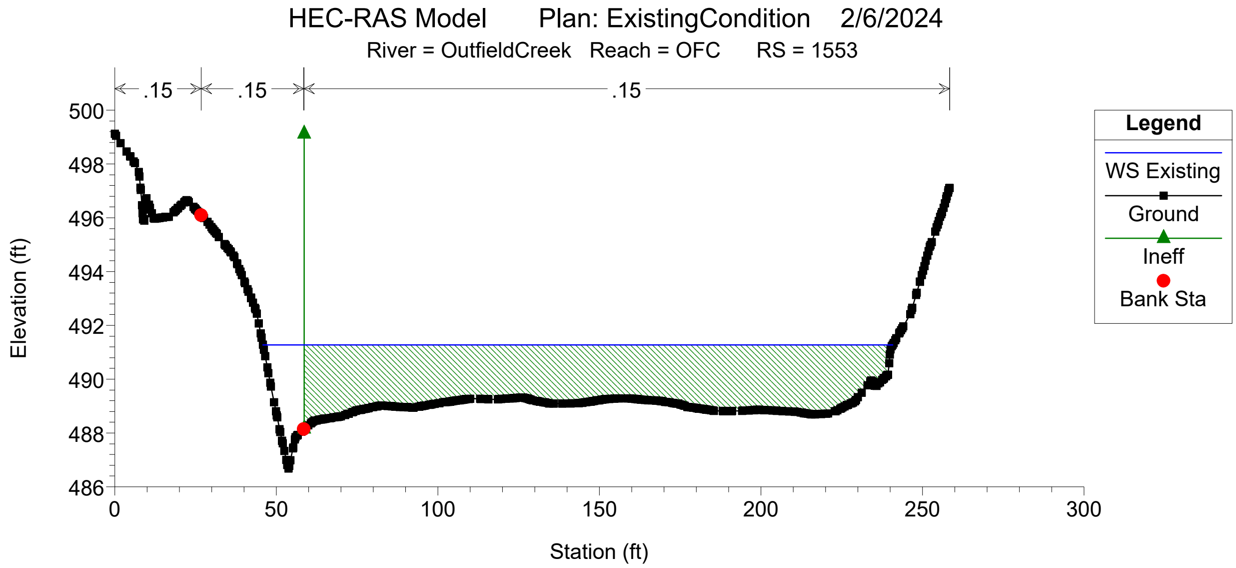

HEC-RAS Model Plan: ExistingCondition 2/6/2024
River = OutfieldCreek Reach = OFC RS = 1863
.15
.15
.15
Elevation (ft)
Station (ft)
Legend
WS Existing
Ground
Ineff
Bank Sta


HEC-RAS Model Plan: ExistingCondition 2/6/2024
River = OutfieldCreek Reach = OFC RS = 1785
.15
.15
.15
Elevation (ft)
Station (ft)
Legend
WS Existing
Ground
Ineff
Bank Sta

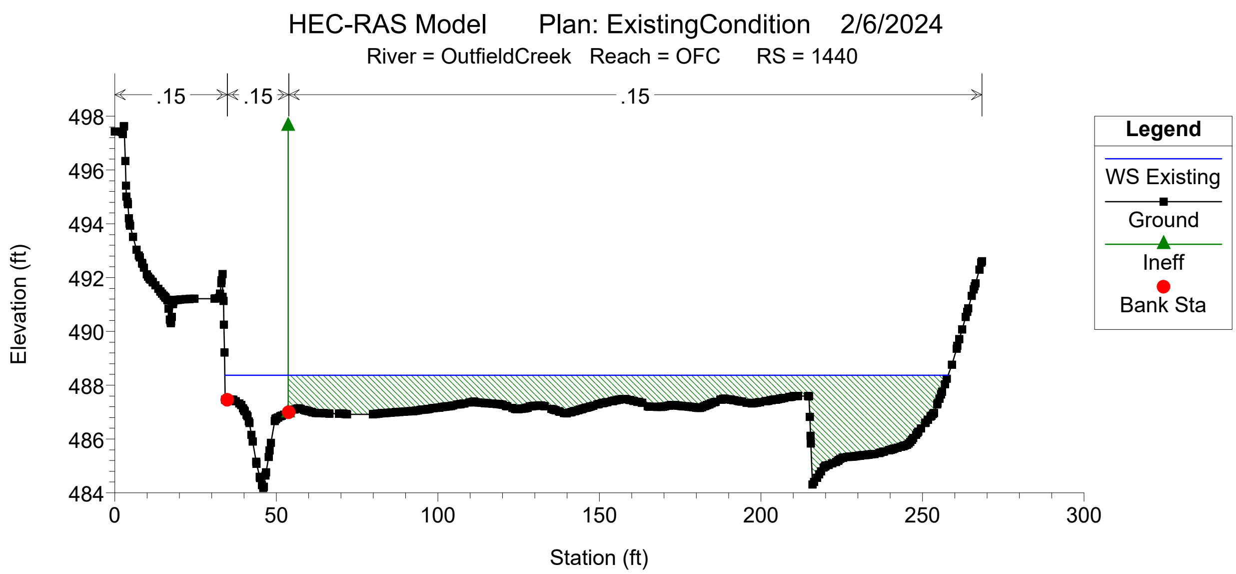

HEC-RAS Model Plan: ExistingCondition 2/6/2024
River = OutfieldCreek Reach = OFC RS = 1718
.15
.15
Elevation (ft)
504
502
500
498
496
494
492
0
50
100
150
200
Station (ft)
Legend
WS Existing
Ground
Ineff
Bank Sta


HEC-RAS Model Plan: ExistingCondition 2/6/2024
River = OutfieldCreek Reach = OFC RS = 1713
.15
.15
.15
Elevation (ft)
502
500
498
496
494
492
0
50
100
150
200
Station (ft)
Legend
WS Existing
Ground
Ineff
Bank Sta

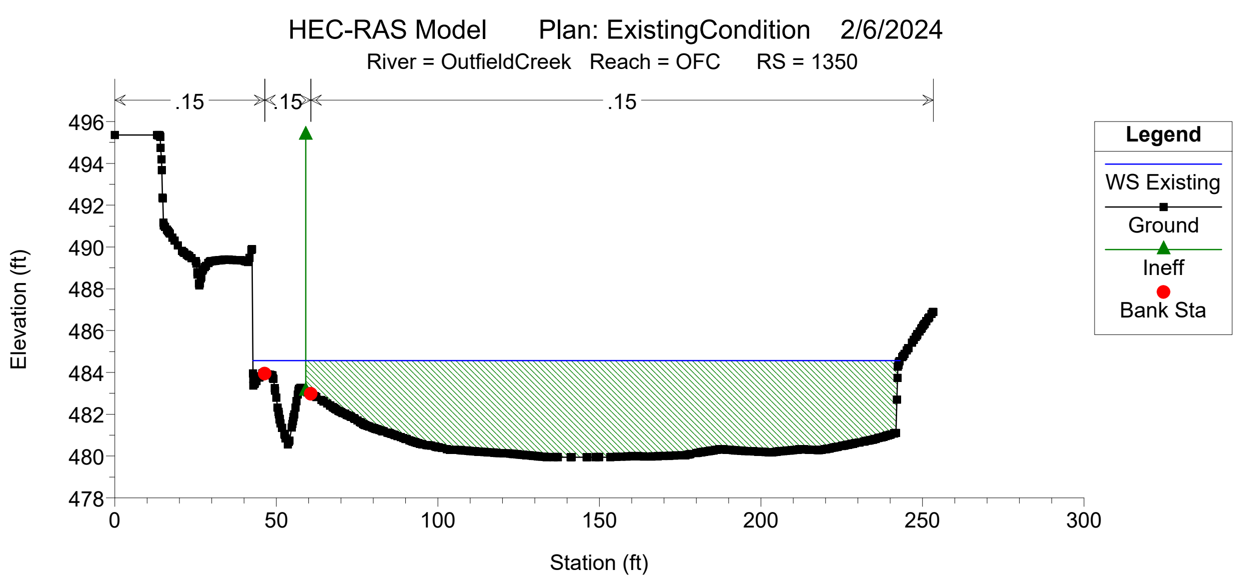

HEC-RAS Model Plan: ExistingCondition 2/6/2024
River = OutfieldCreek Reach = OFC RS = 1649
.15
.15
.15
Elevation (ft)
500
498
496
494
492
490
488
0
50
100
150
200
Station (ft)
Legend
WS Existing
Ground
Ineff
Bank Sta



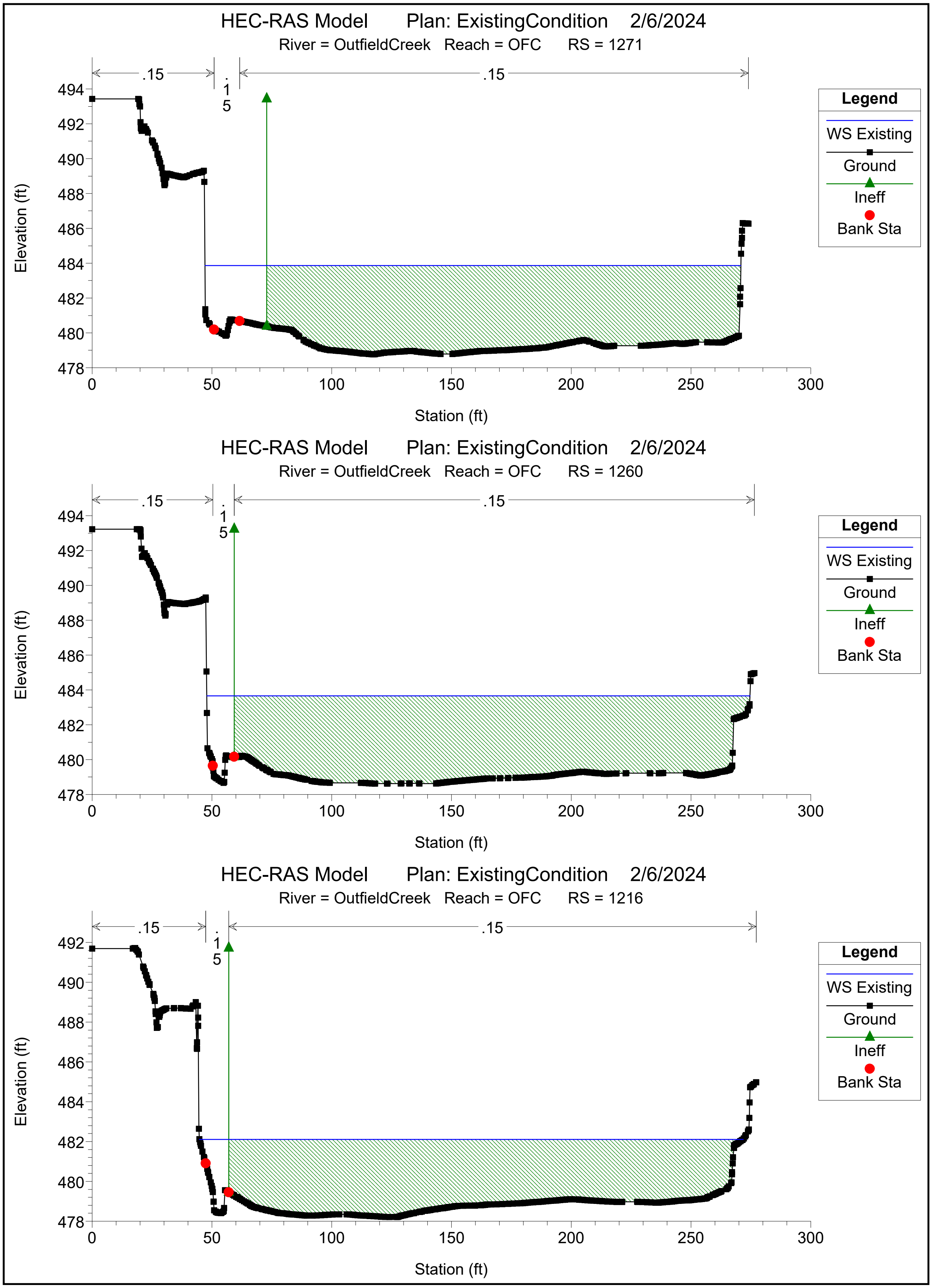
HEC-RAS Model Plan: ExistingCondition 2/6/2024
River = OutfieldCreek Reach = OFC RS = 1553
.15
.15
.15
Elevation (ft)
Station (ft)
Legend
WS Existing
Ground
Ineff
Bank Sta



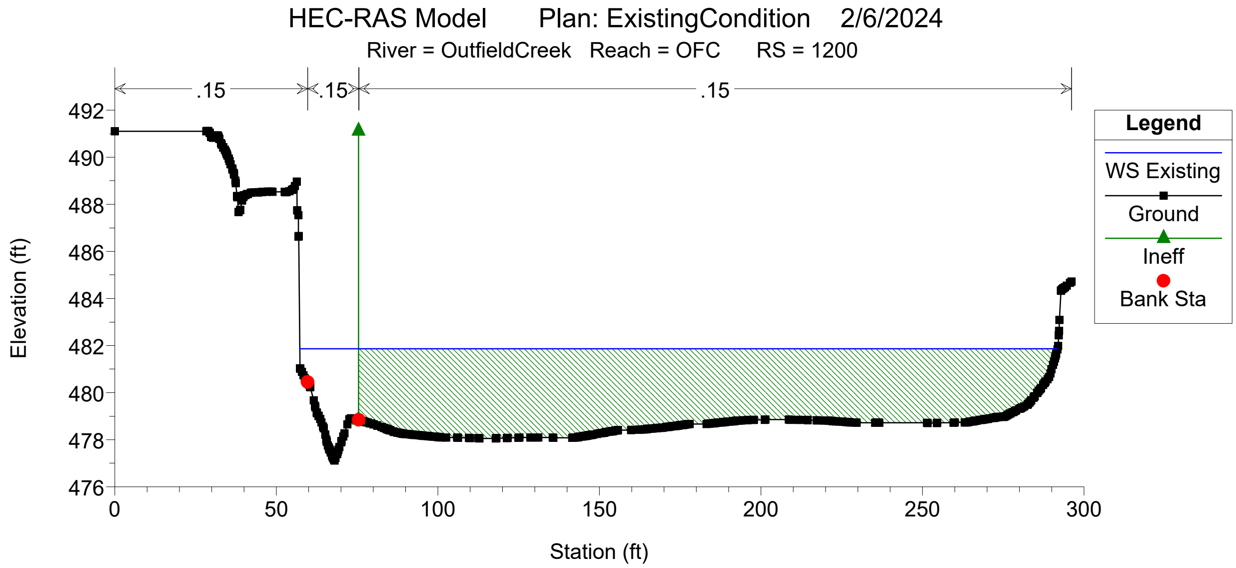
HEC-RAS Model Plan: ExistingCondition 2/6/2024
River = OutfieldCreek Reach = OFC RS = 1440
.15
.15
.15
Elevation (ft)
Station (ft)
Legend
WS Existing
Ground
Ineff
Bank Sta

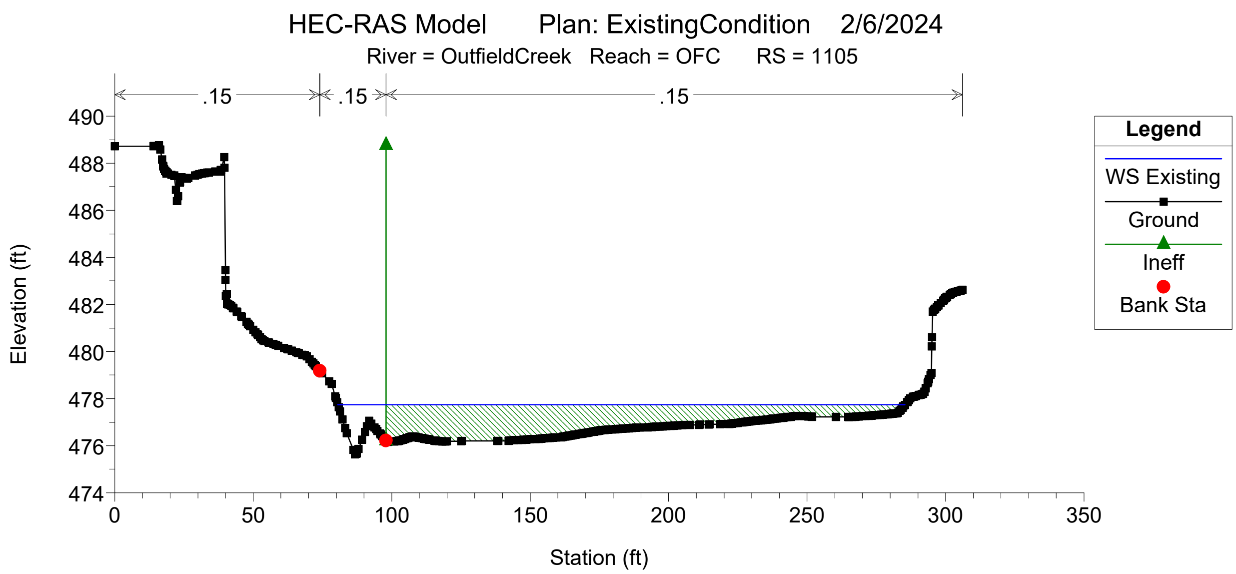


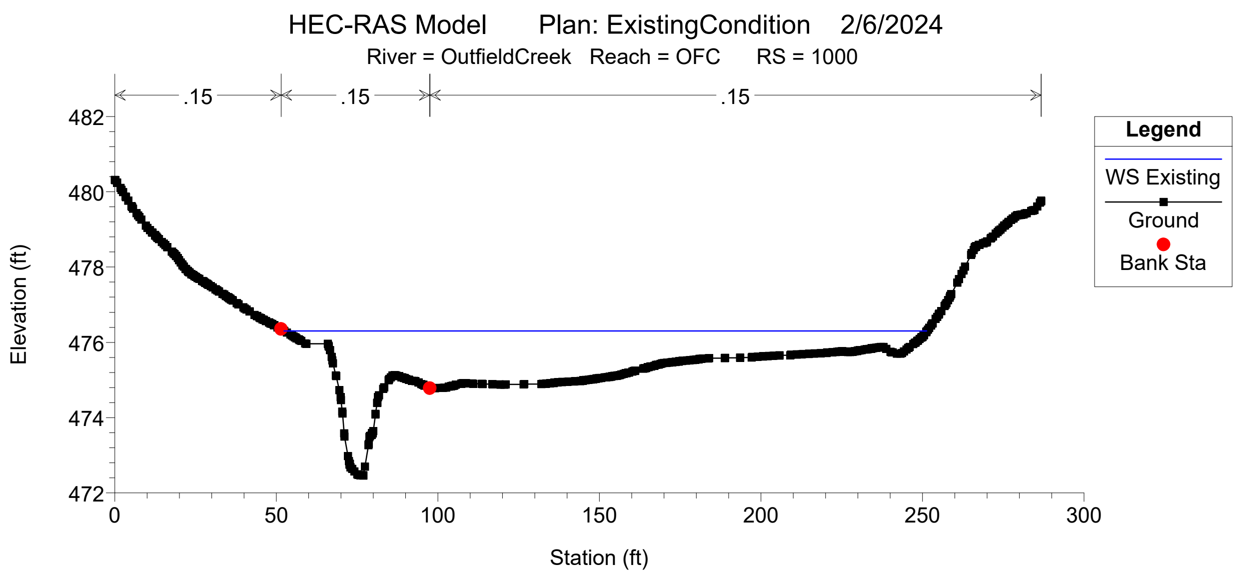
HEC-RAS Model Plan: ExistingCondition 2/6/2024
River = OutfieldCreek Reach = OFC RS = 1350
.15
.15
.15
Elevation (ft)
Station (ft)
Legend
WS Existing
Ground
Ineff
Bank Sta

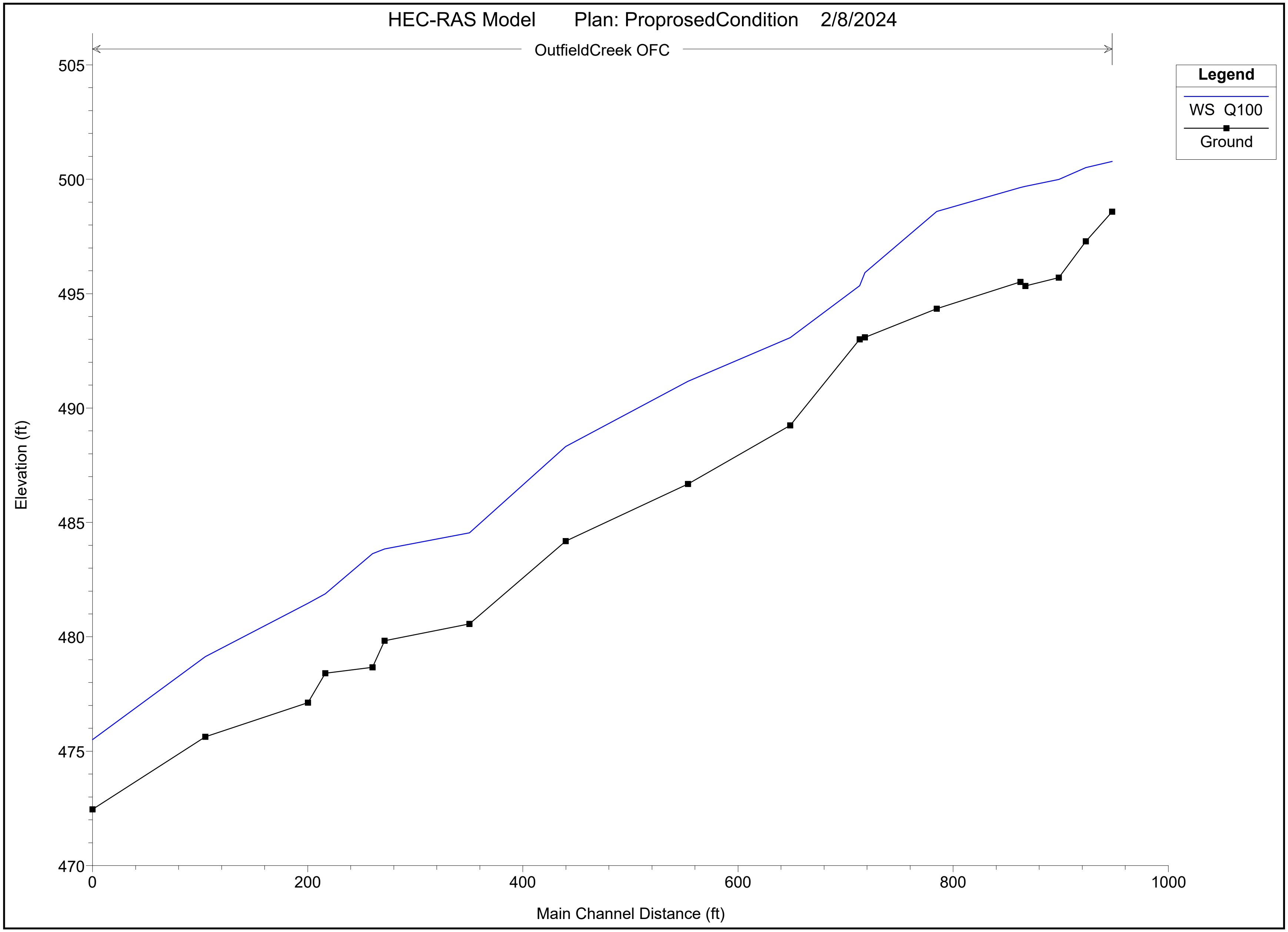


HEC-RAS Model Plan: ExistingCondition 2/6/2024
River = OutfieldCreek Reach = OFC RS = 1271
.15
.15
.15
Elevation (ft)
Station (ft)
Legend
WS Existing
Ground
Ineff
Bank Sta
HEC-RAS Model Plan: ExistingCondition 2/6/2024
River = OutfieldCreek Reach = OFC RS = 1260
.15
.15
.15
Elevation (ft)
Station (ft)
Legend
WS Existing
Ground
Ineff
Bank Sta
HEC-RAS Model Plan: ExistingCondition 2/6/2024
River = OutfieldCreek Reach = OFC RS = 1216
.15
.15
.15
Elevation (ft)
Station (ft)
Legend
WS Existing
Ground
Ineff
Bank Sta

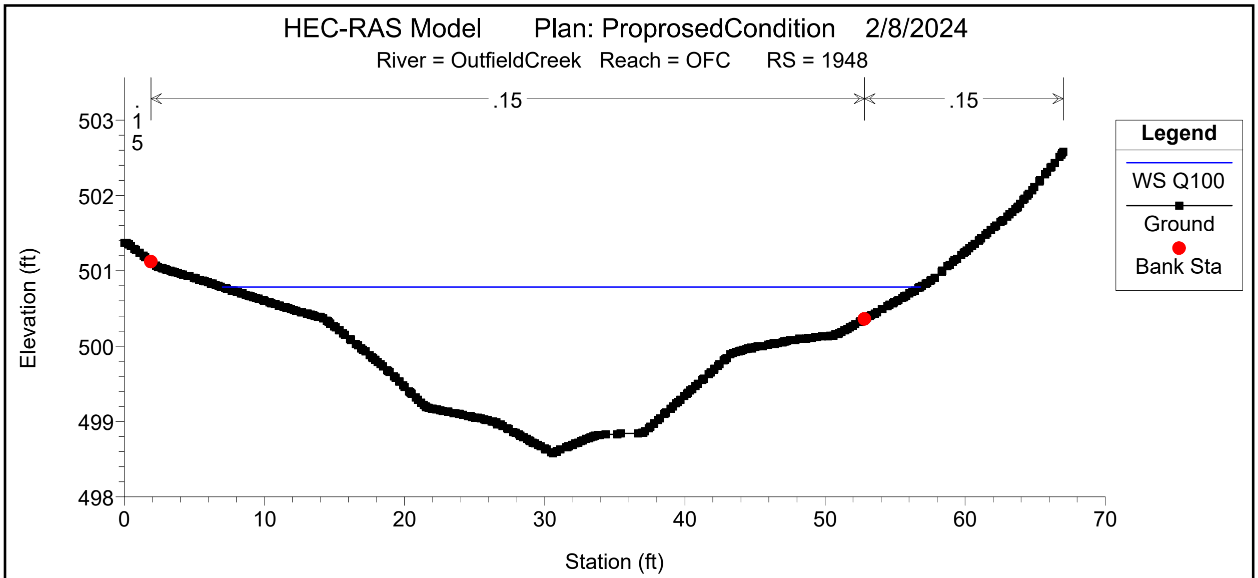

HEC-RAS Model Plan: ExistingCondition 2/6/2024
River = OutfieldCreek Reach = OFC RS = 1200
.15
.15
.15
Legend
WS Existing
Ground
Ineff
Bank Sta
Elevation (ft)
Station (ft)

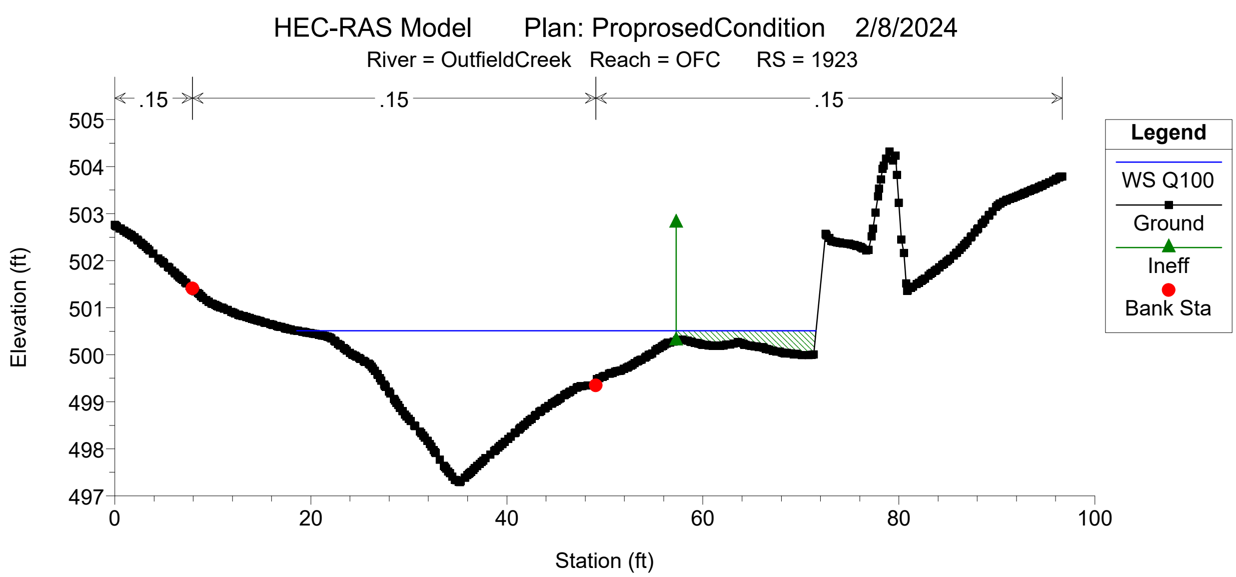

HEC-RAS Model Plan: ExistingCondition 2/6/2024
River = OutfieldCreek Reach = OFC RS = 1105
.15
.15
.15
Legend
WS Existing
Ground
Ineff
Bank Sta
Elevation (ft)
Station (ft)

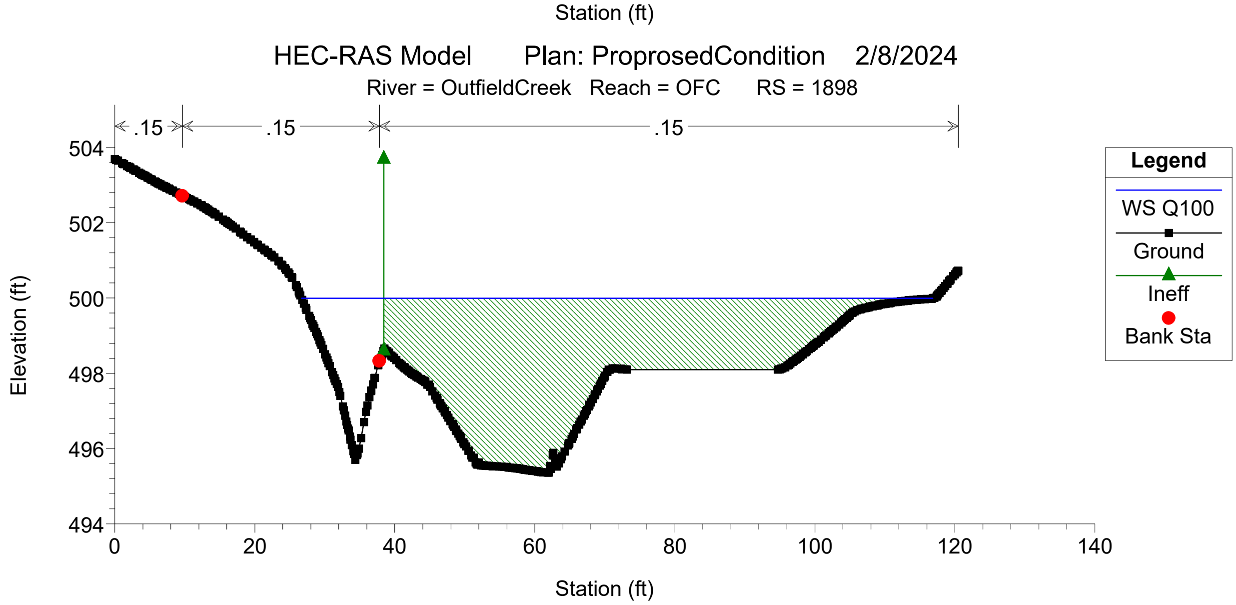

HEC-RAS Model Plan: ExistingCondition 2/6/2024
River = OutfieldCreek Reach = OFC RS = 1000
.15
.15
.15
Legend
WS Existing
Ground
Bank Sta
Elevation (ft)
Station (ft)

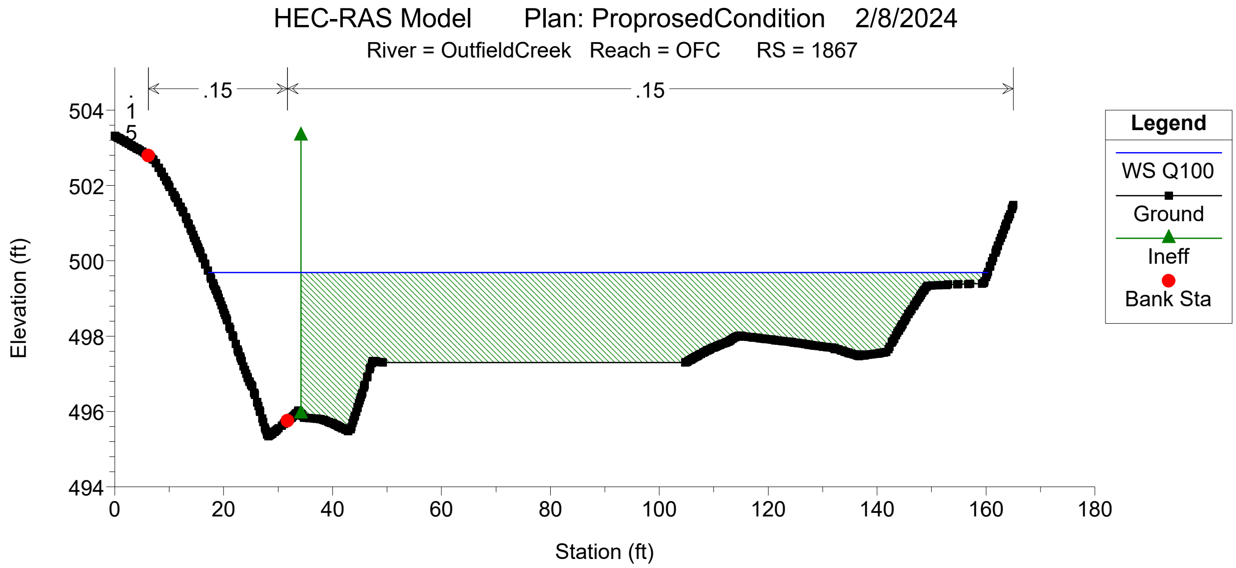


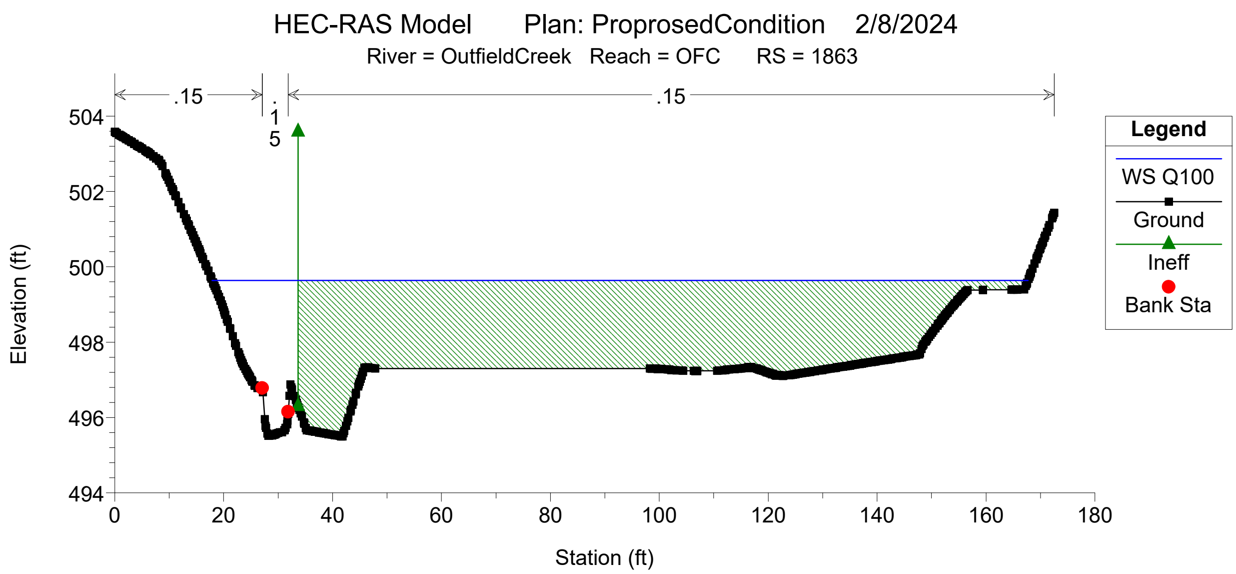
HEC-RAS Model Plan: ProprosedCondition 2/8/2024
OutfieldCreek OFC
Legend
WS Q100
Ground
Elevation (ft)
505
500
495
490
485
480
475
470
0
200
400
600
800
1000
Main Channel Distance (ft)

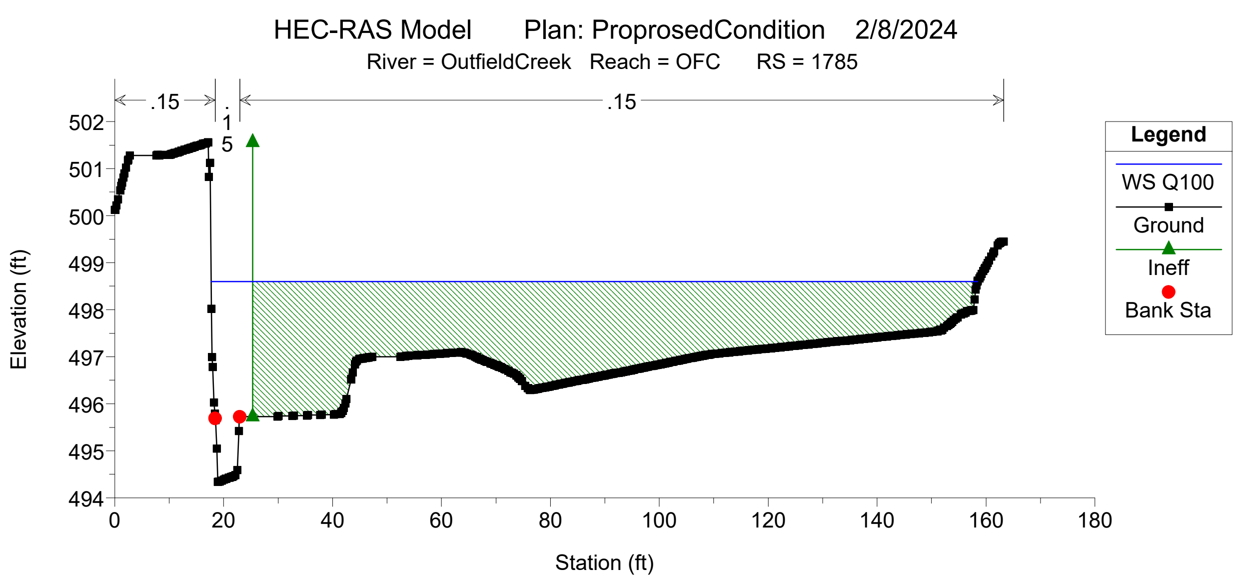

HEC-RAS Model Plan: ProprosedCondition 2/8/2024
River = OutfieldCreek Reach = OFC RS = 1948
.15
.15
.15
Legend
WS Q100
Ground
Bank Sta
Elevation (ft)
Station (ft)
498
499
500
501
502
503
0
10
20
30
40
50
60
70

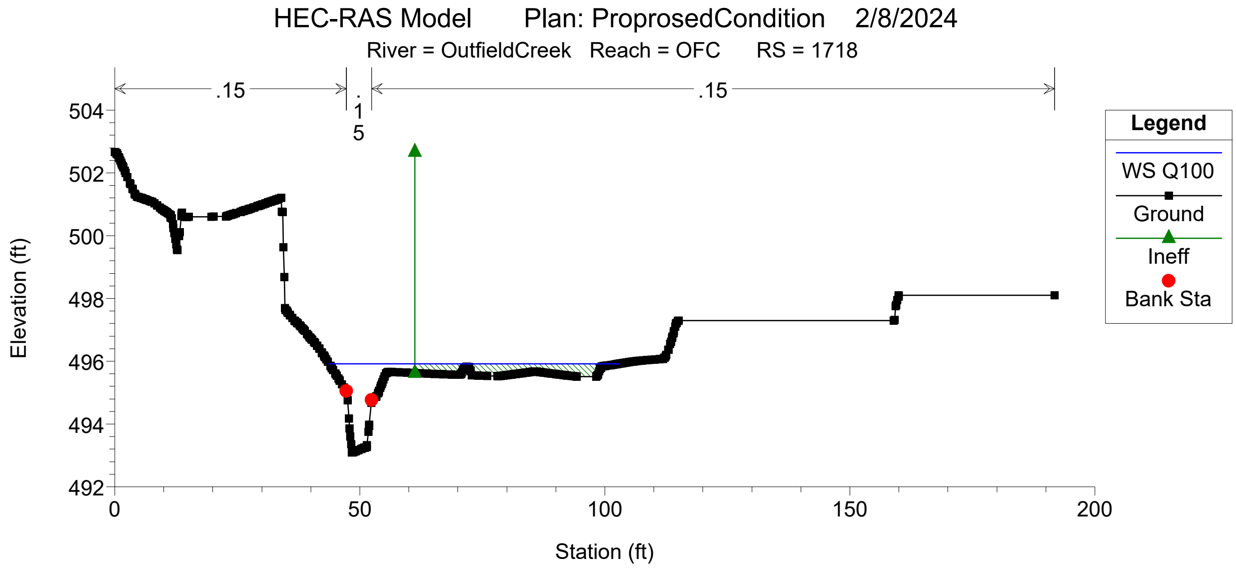

HEC-RAS Model Plan: ProprosedCondition 2/8/2024
River = OutfieldCreek Reach = OFC RS = 1923
.15
.15
.15
Legend
WS Q100
Ground
Ineff
Bank Sta
Elevation (ft)
Station (ft)
497
498
499
500
501
502
503
504
505
0
20
40
60
80
100

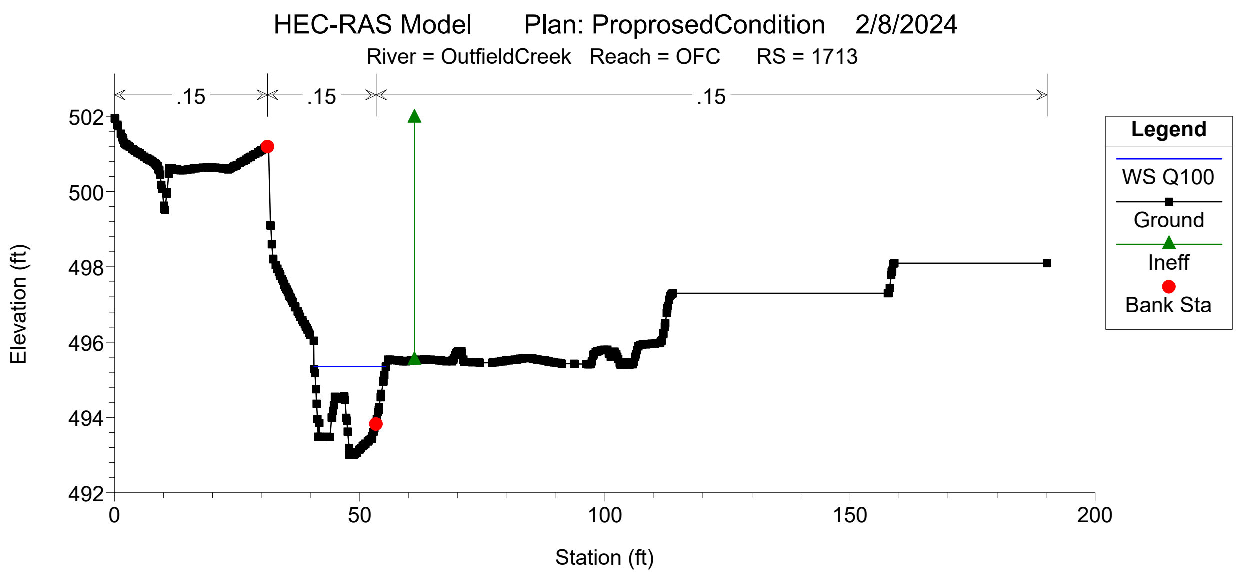

HEC-RAS Model Plan: ProprosedCondition 2/8/2024
River = OutfieldCreek Reach = OFC RS = 1898
.15
.15
.15
Legend
WS Q100
Ground
Ineff
Bank Sta
Elevation (ft)
Station (ft)
494
496
498
500
502
504
0
20
40
60
80
100
120
140

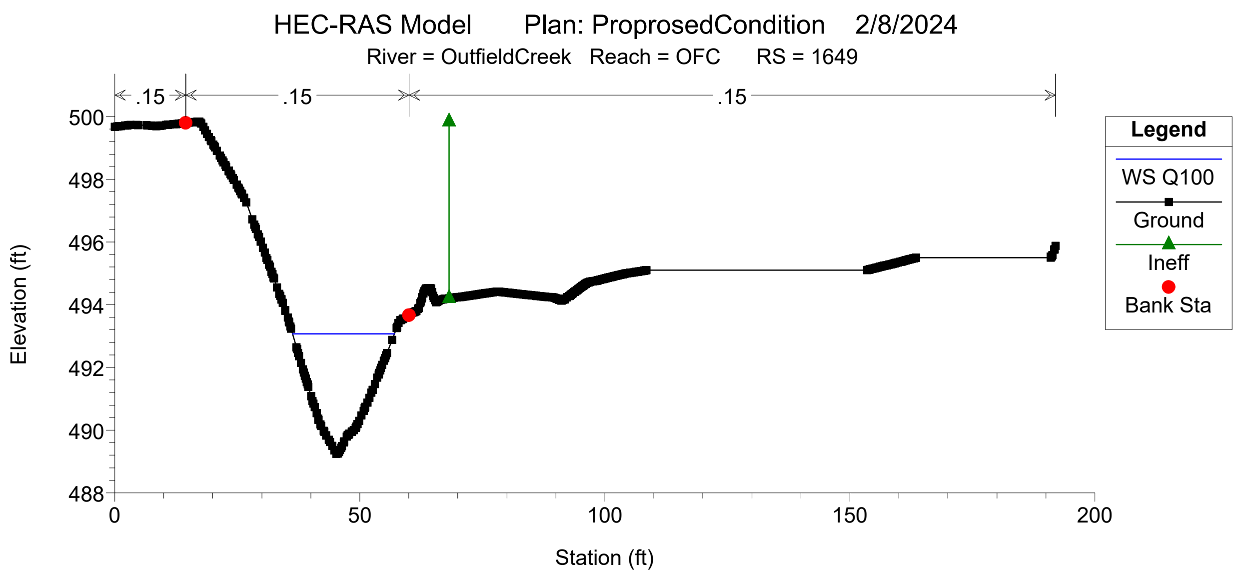

HEC-RAS Model Plan: ProprosedCondition 2/8/2024
River = OutfieldCreek Reach = OFC RS = 1867
.15
.15
Elevation (ft)
Station (ft)
Legend
WS Q100
Ground
Ineff
Bank Sta

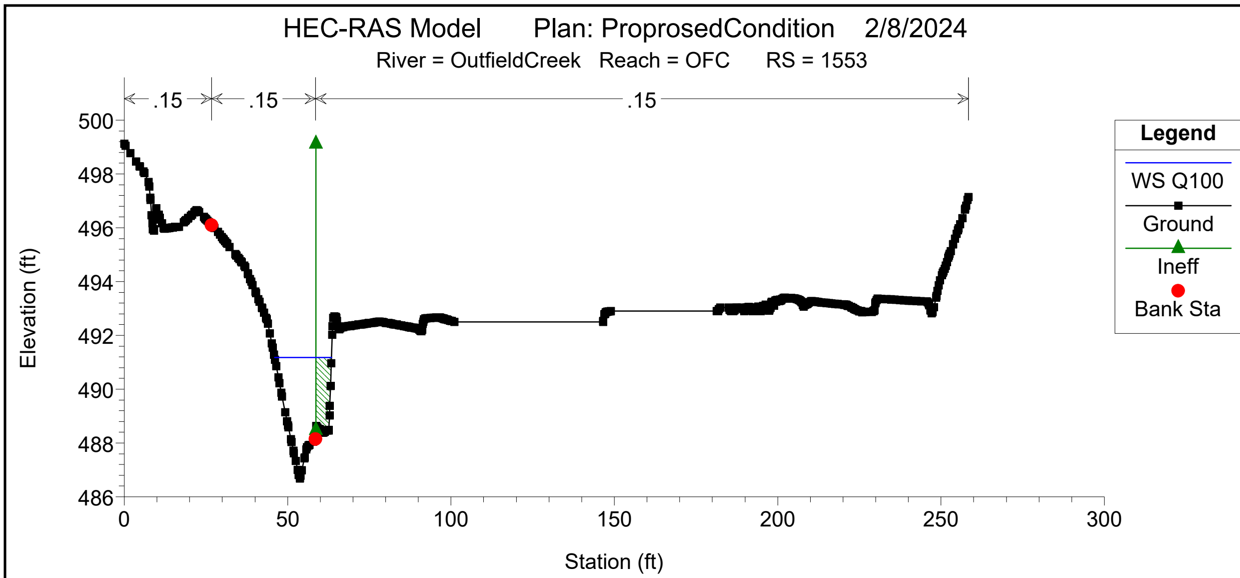

HEC-RAS Model Plan: ProprosedCondition 2/8/2024
River = OutfieldCreek Reach = OFC RS = 1863
.15
.15
Elevation (ft)
Station (ft)
Legend
WS Q100
Ground
Ineff
Bank Sta


HEC-RAS Model Plan: ProprosedCondition 2/8/2024
River = OutfieldCreek Reach = OFC RS = 1785
.15
.15
Elevation (ft)
Station (ft)
Legend
WS Q100
Ground
Ineff
Bank Sta

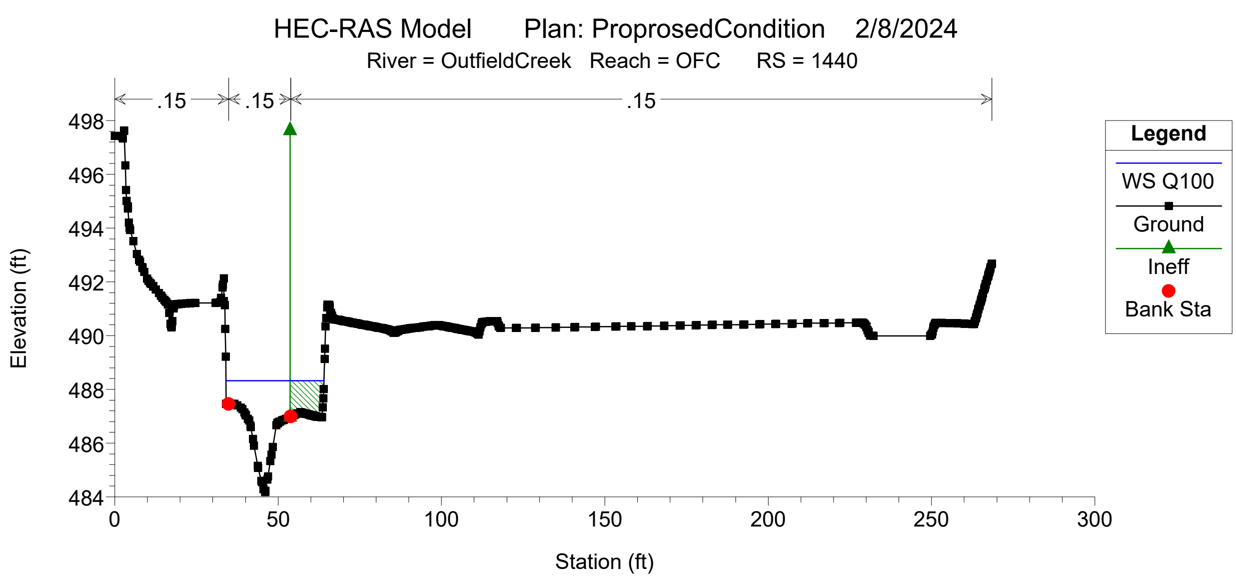

HEC-RAS Model Plan: ProprosedCondition 2/8/2024
River = OutfieldCreek Reach = OFC RS = 1718
.15
.15
Elevation (ft)
Station (ft)
Legend
WS Q100
Ground
Ineff
Bank Sta


HEC-RAS Model Plan: ProprosedCondition 2/8/2024
River = OutfieldCreek Reach = OFC RS = 1713
.15
.15
.15
Elevation (ft)
Station (ft)
Legend
WS Q100
Ground
Ineff
Bank Sta

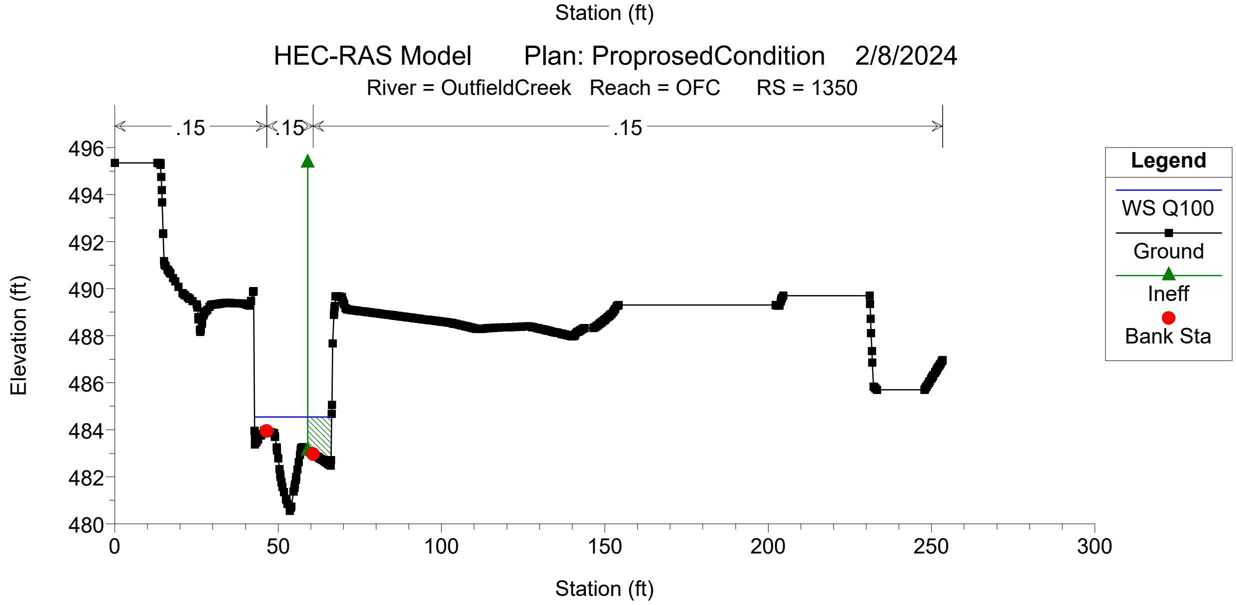

HEC-RAS Model Plan: ProprosedCondition 2/8/2024
River = OutfieldCreek Reach = OFC RS = 1649
.15
.15
.15
Elevation (ft)
Station (ft)
Legend
WS Q100
Ground
Ineff
Bank Sta



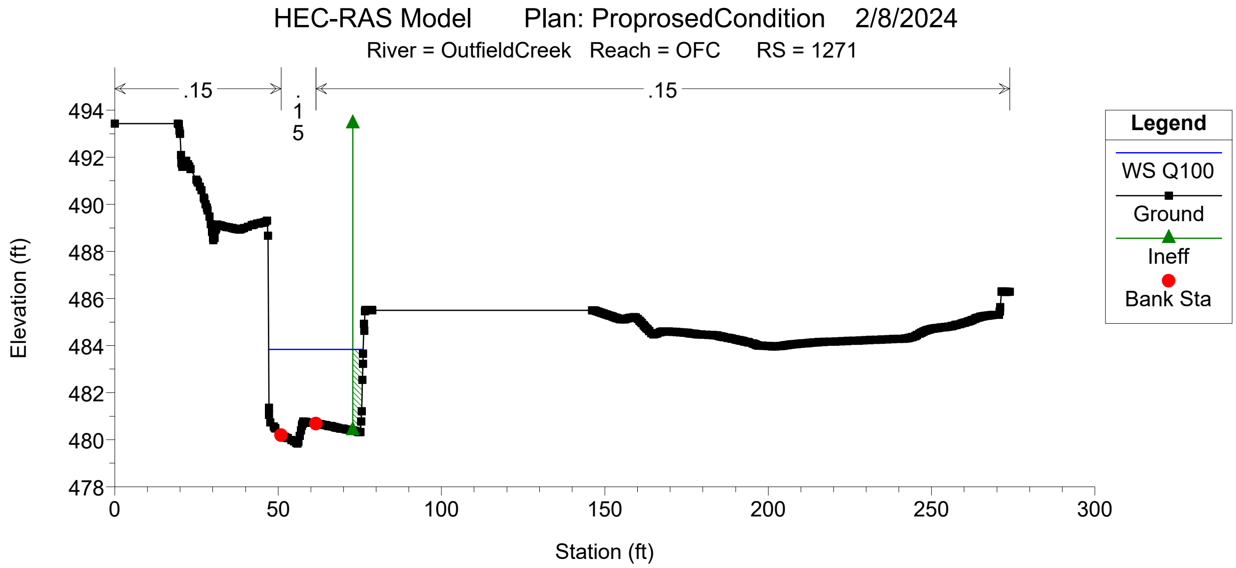
HEC-RAS Model Plan: ProprosedCondition 2/8/2024
River = OutfieldCreek Reach = OFC RS = 1553
.15 .15 .15
Elevation (ft)
500
498
496
494
492
490
488
486
0 50 100 150 200 250 300
Station (ft)
Legend
WS Q100
Ground
Ineff
Bank Sta

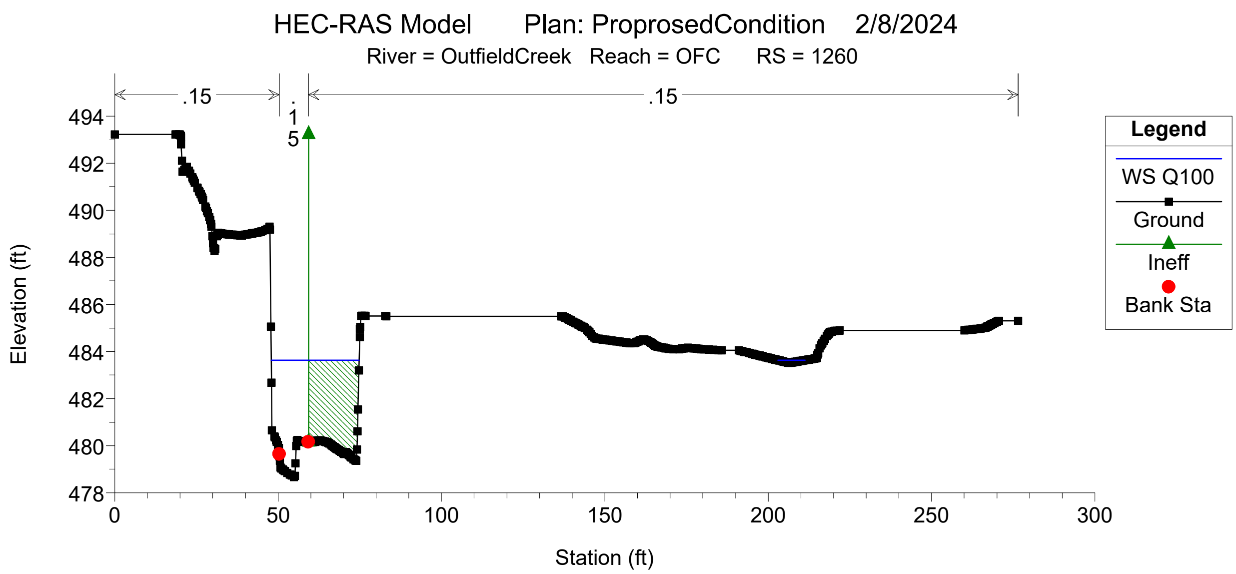


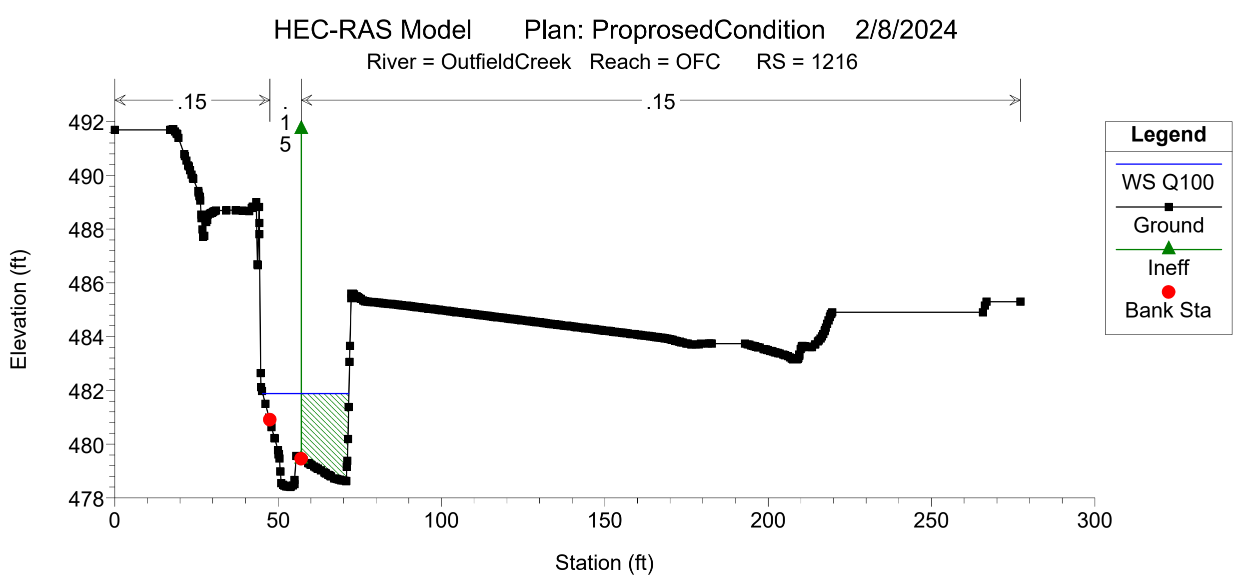
HEC-RAS Model Plan: ProprosedCondition 2/8/2024
River = OutfieldCreek Reach = OFC RS = 1440
.15 .15 .15
Elevation (ft)
498
496
494
492
490
488
486
484
0 50 100 150 200 250 300
Station (ft)
Legend
WS Q100
Ground
Ineff
Bank Sta

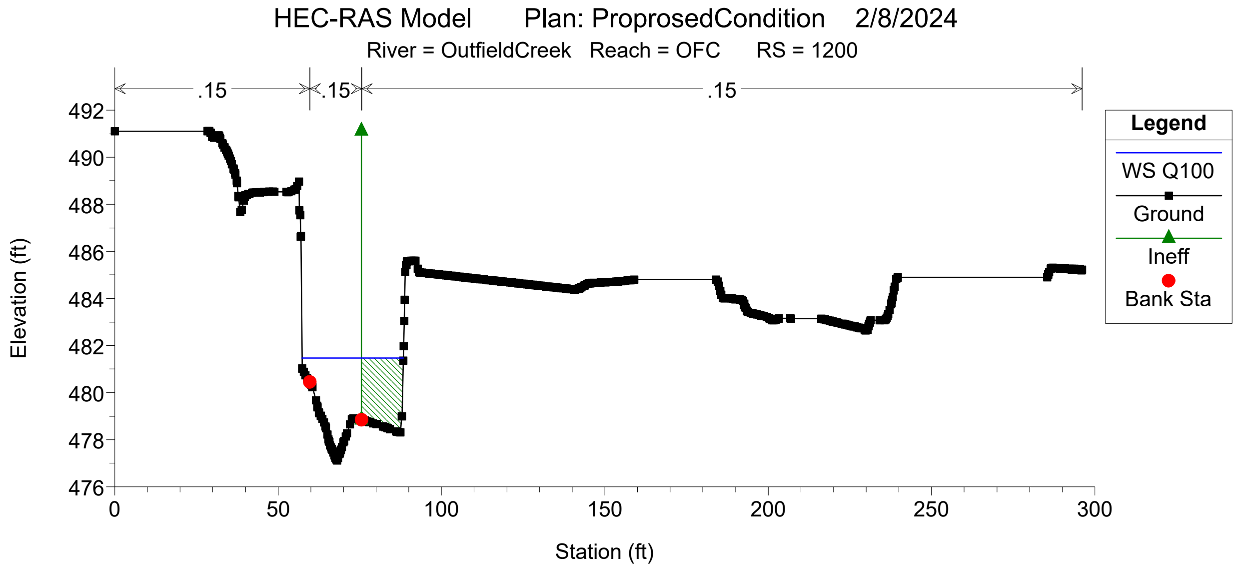


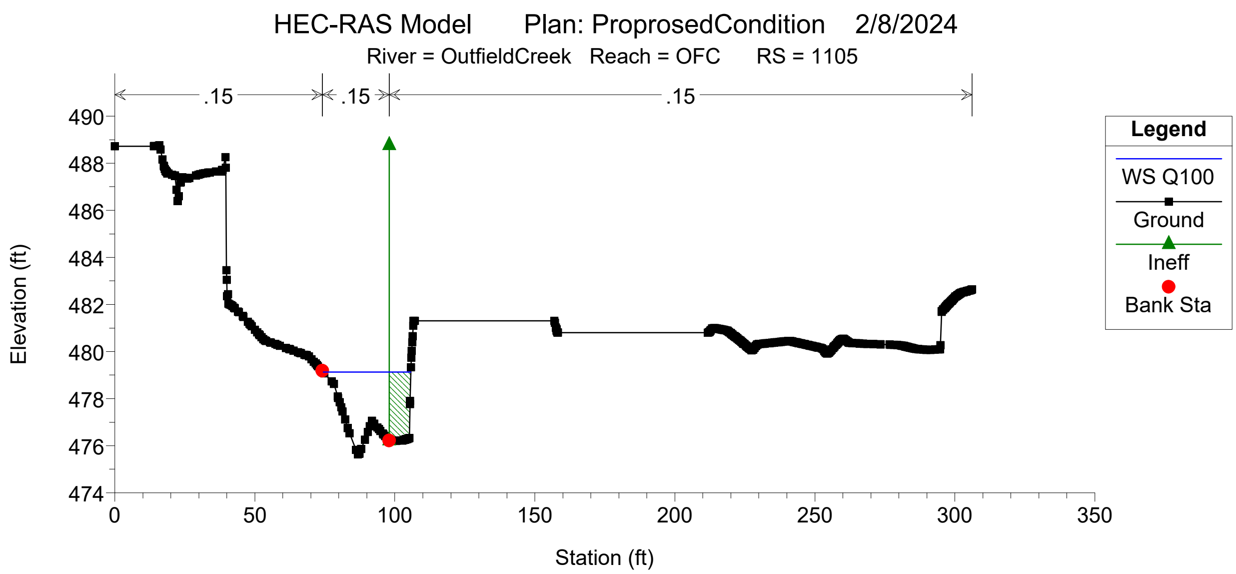
HEC-RAS Model Plan: ProprosedCondition 2/8/2024
River = OutfieldCreek Reach = OFC RS = 1350
.15 .15 .15
Elevation (ft)
496
494
492
490
488
486
484
482
480
0 50 100 150 200 250 300
Station (ft)
Legend
WS Q100
Ground
Ineff
Bank Sta

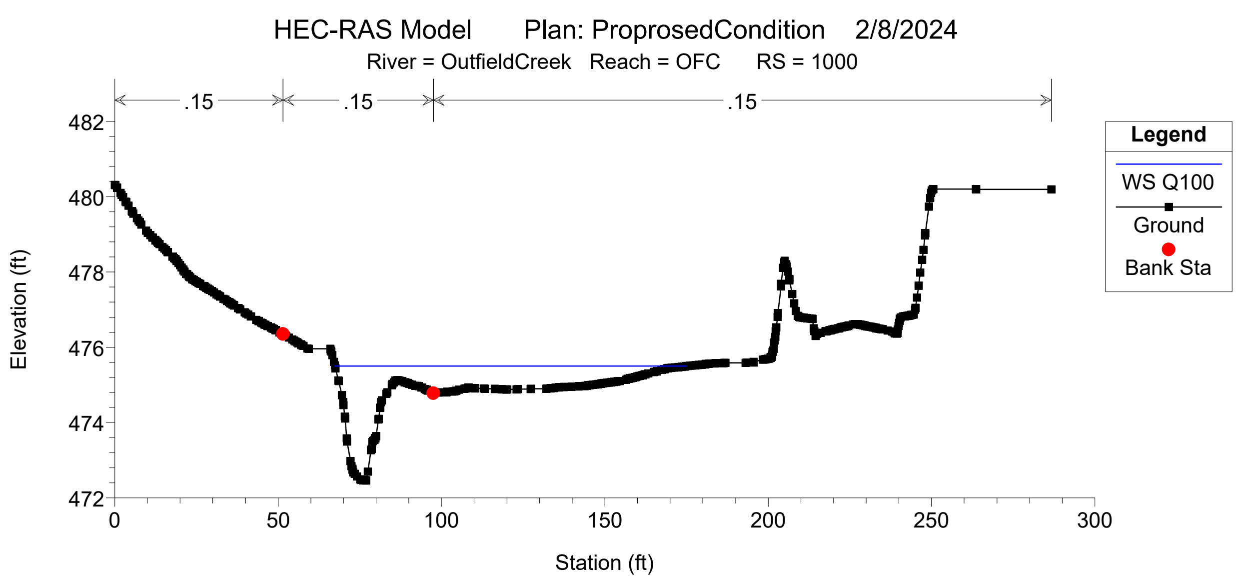

HEC-RAS Model Plan: ProprosedCondition 2/8/2024
River = OutfieldCreek Reach = OFC RS = 1271
.15
.
1
5
.15
Elevation (ft)
494
492
490
488
486
484
482
480
478
0
50
100
150
200
250
300
Station (ft)
Legend
WS Q100
Ground
Ineff
Bank Sta


HEC-RAS Model Plan: ProprosedCondition 2/8/2024
River = OutfieldCreek Reach = OFC RS = 1260
.15
.
1
5
.15
Elevation (ft)
494
492
490
488
486
484
482
480
478
0
50
100
150
200
250
300
Station (ft)
Legend
WS Q100
Ground
Ineff
Bank Sta


HEC-RAS Model Plan: ProprosedCondition 2/8/2024
River = OutfieldCreek Reach = OFC RS = 1216
.15
.
1
5
.15
Elevation (ft)
492
490
488
486
484
482
480
478
0
50
100
150
200
250
300
Station (ft)
Legend
WS Q100
Ground
Ineff
Bank Sta


HEC-RAS Model Plan: ProprosedCondition 2/8/2024
River = OutfieldCreek Reach = OFC RS = 1200
.15
.15
.15
Elevation (ft)
476
478
480
482
484
486
488
490
492
0
50
100
150
200
250
300
Station (ft)
Legend
WS Q100
Ground
Ineff
Bank Sta


HEC-RAS Model Plan: ProprosedCondition 2/8/2024
River = OutfieldCreek Reach = OFC RS = 1105
.15
.15
.15
Elevation (ft)
474
476
478
480
482
484
486
488
490
0
50
100
150
200
250
300
350
Station (ft)
Legend
WS Q100
Ground
Ineff
Bank Sta


HEC-RAS Model Plan: ProprosedCondition 2/8/2024
River = OutfieldCreek Reach = OFC RS = 1000
.15
.15
.15
Elevation (ft)
472
474
476
478
480
482
0
50
100
150
200
250
300
Station (ft)
Legend
WS Q100
Ground
Bank Sta

VISTA BALL FIELDS, HEC RAS MODELING RESULTS

		EXISTING			PROPOSED					
Reach	River Sta	Q Total	W.S. Elev	Velocity	Q Total	W.S. Elev	Velocity	Nearest Pad Elevation	Pad Elevation - Pr. WSEL	Proposed WSEL - Existing WSEL
		(cfs)	(ft)	(ft/s)	(cfs)	(ft)	(ft/s)	(ft)	(ft)	(ft)
OFC	1949	64.3	500.78	1.2	64.3	500.78	1.19	NA	NA	0
OFC	1924	68.6	500.51	1.34	68.6	500.51	1.33	501.6	1.09	0
OFC	1899	68.6	499.98	2.75	68.6	499.99	2.73	501	1.01	0.01
OFC	1868	68.6	499.68	1.38	68.6	499.69	1.37	501	1.31	0.01
OFC	1863	68.6	499.62	2.1	68.6	499.64	2.1	501	1.36	0.02
OFC	1785	68.6	498.56	2.74	68.6	498.6	2.71	501	2.4	0.04
OFC	1718	68.6	495.98	4.44	68.6	495.92	4.7	499.7	3.78	-0.06
OFC	1713	99.7	495.41	4.38	94.4	495.35	4.33	499.7	4.35	-0.06
OFC	1649	99.7	493.17	2.12	94.4	493.07	2.09	495.1	2.03	-0.1
OFC	1554	99.7	491.27	2.72	94.4	491.17	2.68	492.5	1.33	-0.1
OFC	1440	99.7	488.36	2.61	94.4	488.32	2.53	492	3.68	-0.04
OFC	1350	99.7	484.57	3.56	98.7	484.55	3.56	489.3	4.75	-0.02
OFC	1271	131.4	483.87	1.58	130.4	483.84	1.59	485.5	1.66	-0.03
OFC	1260	131.4	483.66	3.14	130.4	483.63	3.13	485.5	1.87	-0.03
OFC	1216	131.4	482.11	4.64	130.4	481.88	5.04	485.5	3.62	-0.23
OFC	1200	131.4	481.86	2.46	130.4	481.46	2.79	485.5	4.04	-0.4
OFC	1105	131.4	477.74	6.23	130.4	479.13	2.69	481.3	2.17	1.39
OFC	1000	131.4	476.3	0.75	130.4	475.5	2.29	480.7	5.2	-0.8

Notes.
1. Existing condition and proposed capacity runs use Manning's N valus of 0.15.
2. Areas that are behind fences or obstructed by buildings are modeled as ineffective flow areas.

NOTES:

1. THERE IS AN INCREASE IN WATER SURFACE ELEVATION AT CROSS-SECTION 1105, HOWEVER, THIS INCREASE IS WITHIN THE LIMITS OF THE PROJECT AND DOES NOT EFFECT OTHER PROPERTIES. OTHER CROSS-SECTIONS AT THE UPSTREAM END OF THE PROJECT (1899, 1868, 1863, AND 1758) SHOW AN INCREASE OF LESS THAN 0.05-FT, WHICH GIVEN THE LIMITATION OF THE MODELING SOFTWARE, SHOULD EFFECTIVELY ROUND TO 0.0-FT. THE TABLE HAS BEEN SHOWN TO TWO DECIMAL PLACES FOR CONSISTENCY WITH THE HEC-RAS OUTPUT.

LEGEND:

- CROSS-SECTION
- INEFFECTIVE FLOW AREA
- 1000 CROSS-SECTION ID
- STREAM CENTERLINE
- AQUATIC RESOURCES 5FT BUFFER
- LOT LINES

HYDRAULIC WORK MAP FOR VISTA 2 (PROPOSED CONDITION)

Date: April 7, 2023
Revised: February 27, 2024
Revised: April 17, 2024

J-19253-B

RICK ENGINEERING COMPANY
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Riverside - Orange - Sacramento - San Luis Obispo - Phoenix - Tucson - Denver

J-19278B

APPENDIX J

INLAND RAIL TRAIL PHASE 2B DRAINAGE REPORT

INLAND RAIL TRAIL PHASE 2B
PRELIMINARY DRAINAGE STUDY

100% SUBMITTAL

DECEMBER, 2014

Prepared For:

COUNTY OF SAN DIEGO - DPW
COUNTY OPERATIONS CENTER
5510 OVERLAND AVENUE, SUITE 410
SAN DIEGO, CA 92123

Prepared By:
PSOMAS
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San Diego, CA 92108

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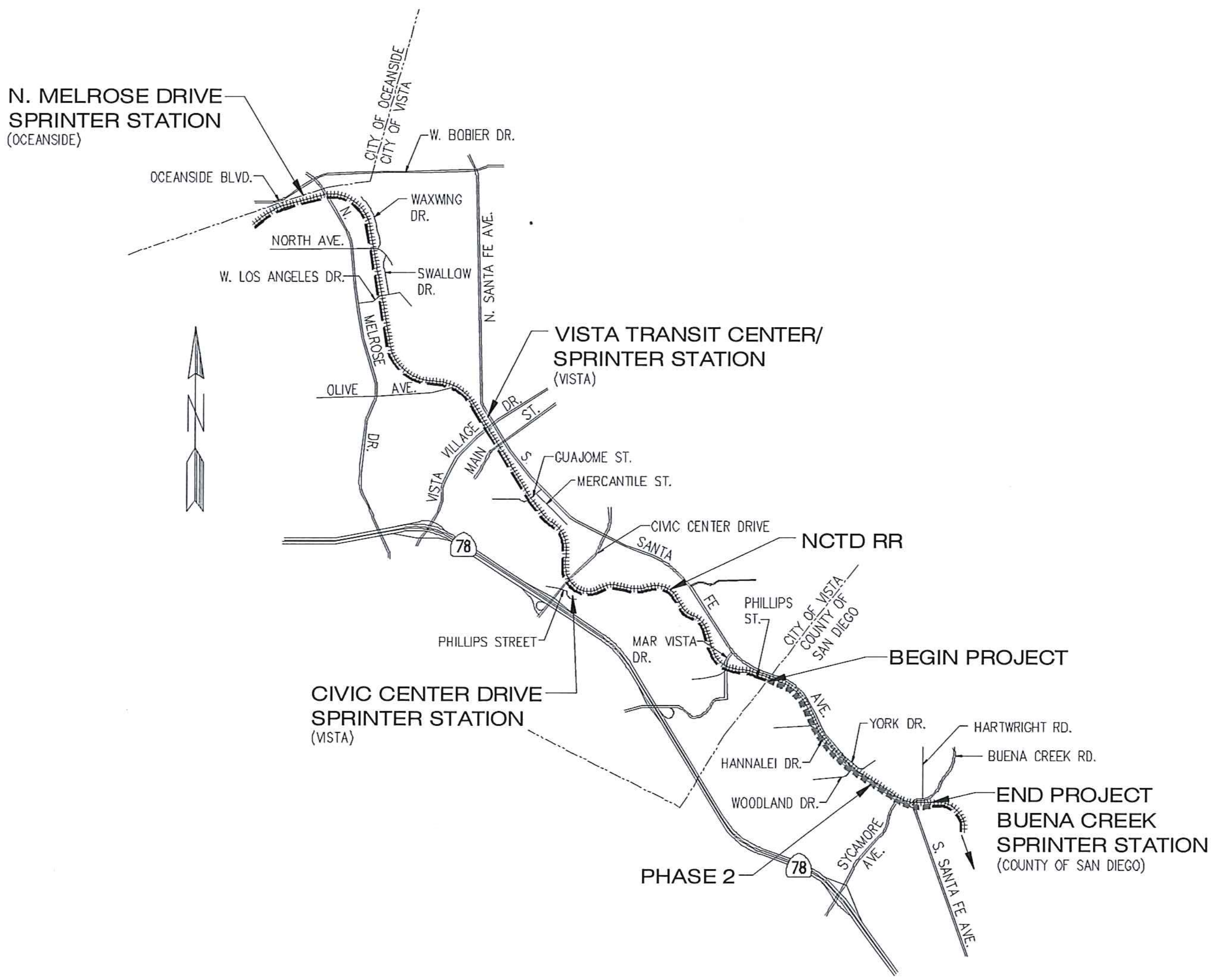
APPENDICES / EXHIBITS

BACKGROUND

The purpose of this study is to analyze the hydrology for the Inland Rail Trail Phase 2B Bike Path project. The project includes approximately 5500 feet of new Bike Path running adjacent to the NCTD railroad tracks within the railroad right of way. The project is located in the County of San Diego, extending from the City of Vista/County of San Diego border on the west end to the Buena Creek Sprinter Station on the east end.



VICINITY MAP

NTS

DESIGN CRITERIA

The drainage design criteria used for this project is per the County of San Diego Hydrology Manual, dated June 2003.

A. DESIGN RUNOFF METHOD

The contributing watersheds are less than one square mile, and therefore, flow rates shall be calculated using the rational method, given as:

$$Q = C \times I \times A$$

Where:
Q = Flow rate in cubic feet per second (cfs)
C = Coefficient of Runoff
I = Rainfall Intensity in inches per hour (in/hr)
A = Area in acres (ac)

Hydrologic characteristics for the project are as follows:

- Soil Group is determined to be Soil Group C & D from the U.S.G.S Hydrologic Soil Group Maps for San Diego County. Soil type D is used for the analysis purposes. See Appendix A.
- The 100-year 6-hour precipitation (P6) is determined to be 3.5 inches using County Isopluvial Maps. See Appendix B.
- The 100-year 24-hour precipitation (P24) is determined to be 6.0 inches using County Isopluvial Maps. See Appendix B.
- The Runoff Coefficient is based on land use which is determined from Table 3-1 from the County of San Diego Hydrology Manual. See Appendix C.

Rainfall intensity shall be determined by the equation given in Figure 3-1 of the Manual, where:

I = 7.44 x P6 x (Tc^-.645)

Where:

I = Rainfall Intensity in inches per hour (in/hr)
P6 = 6-hour Precipitation in inches (in)
Tc = Time of concentration in minutes (min)

Time of concentration is governed by surface characteristics of the watershed. The time of concentration for natural watersheds is based on the Kirpitch formula, given in Figure 3-4 of the Manual, where:

$$Tc = \{60 * (11.9 * L^3/H)^{0.385}\}$$

Where:
Tc = Time of Concentration in minutes (min.)
L = Length of Watershed in miles (mi.)
H = Difference in Elevation in feet (ft.)

For urban watersheds the time of concentration is per the FAA formula given in Figure 3-3 of the manual, where:

$$Tc = [1.8*(1.1-C)*D^{0.5}]/(S^{0.33})$$

Where:
Tc = Time of Concentration in minutes (min.)
C = Runoff coefficient
D = Distance of watercourse
S = Slope in %

See Appendix C for Tables and Nomographs taken from the Sn Diego County Hydrology Manual.

B. HYDRAULIC DESIGN

- Flow in proposed storm drain pipes and open channels shall be analyzed using Bentley FlowMaster software.

HYDROLOGY/HYDRAULICS

A. EXISTING HYRDOLOGY

The existing project site, located in the County of San Diego, runs along the south edge of the NCTD Right-of-Way from Phillips Street to Woodland Drive (Segments A and B) and along the north edge of the NCTD Right-of-Way from York Drive to Buena Creek Road (Segment C).

Segment A:
This section of the project has narrow, linear drainage sub-basins inside the NCTD R/W that collect runoff generated near the railroad tracks and larger overland drainage sub-basins to the south that are comprised of natural undisturbed hillsides and grassy baseball-fields. The railroad runoff is routed through a series of culverts which feed into the existing earthen channel that runs along the south side of the NCDT R/W from west to east. These sub-basins ultimately feed into a catch basin at Hannalei Drive and become part of the public drainage system.

Segment B:
This small drainage basin collects runoff from the NCTD Right-of-Way and feeds into the detention basin that was constructed with the South Santa Fe Widening Project in 2009. Storm runoff is routed in a short series of concrete ditches and collected by catch basins which outlet to the detention basin near the intersection or Hannalei Drive and Woodland Drive. See Appendix D for the "Drainage Report – South Santa Fe Widening Project".

Segment C:
This section of the project also consists of narrow drainage sub-basins created by the NCTD railroad, and some residential sub-basins to the north, located between York Drive and Buena Creek Road just inside the northern NCTD R/W line. The sub-basins in this segment confluence to a 96" culvert that runs under the NCTD R/W (north to south) which outlets into the South Santa Fe Ave. drainage facility, and ultimately to Buena Creek.

B. PROPOSED HYDROLOGY

Hydrology calculations have been tabulated in Appendix C for the 100-year storm event. See also Exhibit 1 for the Drainage Basin Map. Note that time of concentration calculations are based on the formula given in Figure 3-3 and values given in Table 3-2 of the County of San Diego Hydrology Hydrology Manual-2003. To be conservative, values for time of concentration given in Table 3-2 were used as a maximum for an entire sub-basin. This assumption will give Q100 values that are conservative in nature, adding to the factor of safety in the design of downstream rectangular concrete channels.

Segment A:
The construction of the bike path intercepts overland flow form basins A1 and A3 (See Exhibit 1 – Drainage Basin Map) conveying it back to the natural earthen channel just past CD-4. Concrete brow ditch CD-1 captures flow at the top of the 2:1cut slope to the south. Drainage Swale CD-2, a LID Bioretention Trench (See Exhibit 4 for Bioretention Trench Detail), catches runoff from the bike path and confluences with CD-2 to CD-4. The Bioretention Trench runs adjacent to the bike path and treats the runoff that the bike path generates. The Bioretention Trench also serves to slow down the runoff created
by the bike paths impervious material. CD-4 converges with runoff conveyed through an extended existing 36" RCP storm drain culvert (see Exhibit 3 – Hydraulic Calculations for culvert hydraulic data) that delivers runoff from the NCTD R/W from the County / City of Vista line. The impact to the outlet at CD-4 is minimal due to the nature of the Bioretention Trench and the very small change (only 1.0 cfs) in 100-year flow generated by the bike path itself and the rip-rap energy dissipater at the outlet point to the natural channel.

Overland flows from Basin A-5 are conveyed to a rectangular channel (CD-5) just north of the little League baseball field. This rectangular channel (b=4.25', h=1.5') realigns the natural channel around the "pinch point" between the NCTD R/W and the ball field. The channel converges with an extension of an existing 24" RCP conveying runoff from the NCTD R/W (Basin A7) and the two outlet to a riprap energy dissipater to the natural channel to the east. Runoff generated by the bike path in basin A7 is also treated by a Bioretention Trench and the runoff coefficient is conservatively weighted as if no Bioretention Trench were use. With no Bioretention Trench in use, the runoff generated by the bike path adds only 0.7 cfs to the total Q of basin A7. This additional water is easily handled by the 24" RCP culvert as shown in Exhibit 3.

A similar situation occurs with Basin A6 – outletting to a rectangular channel (CD-7, b=4', d=1.5') around the major league field "pinch point" with the NCTD R/W. This channel converges with another 24" RCP culvert extension conveying flows from Basin A8. The additional flows generated by the bike path, not taking into account the Bioretention Trench treatment and percolation, is merely 0.6 cfs and is again dissipated by rip-rap at the entrance to the natural earthen channel. The final flows for this segment continue past the last little League baseball fields and are delivered to a catch basin at Hannalei Drive.

Segment B:
The construction of the bike path to the north of Hannalei Drive intercepts flows from the NCTD R/W (Basin B1) that would have been collected by the Hannalei drainage system. A concrete brow ditch (CD-8), located at the top of a short retaining wall, captures runoff from the Rail R/W and directs it eastward toward an existing concrete ditch where the two ditches converge. This existing concrete ditch then conveys water from an existing culvert and the flows from Basin B1 to a closed-pipe system and eventually outlets into a detention basin. See Appendix D for data regarding the South Santa Fe Widening Project. The runoff placed into the existing system by Basins B1, B2 and B3 is the same water that currently flows to the catch basin at the end of the existing concrete ditch, so no additional calculations for the existing facility were generated. 100-year Q values generated by Basins B1, B2 and B3 are available in Exhibit 2.

Segment C:
Basin C1 functions very similarly to the existing condition with the exception of the impervious area generated by the construction of the bike path. Again, this new bike path runoff is treated by a Bioretention Trench that runs the length of the path. The flow path of this Basin is only slightly modified in ditch alignment (CD-9) and connection via drop inlet to the existing 96" CMP culvert pipe running under the NCTD tracks. The additional flow generated by the path, not including losses from the Bioretention Trench, is only 0.7cfs.

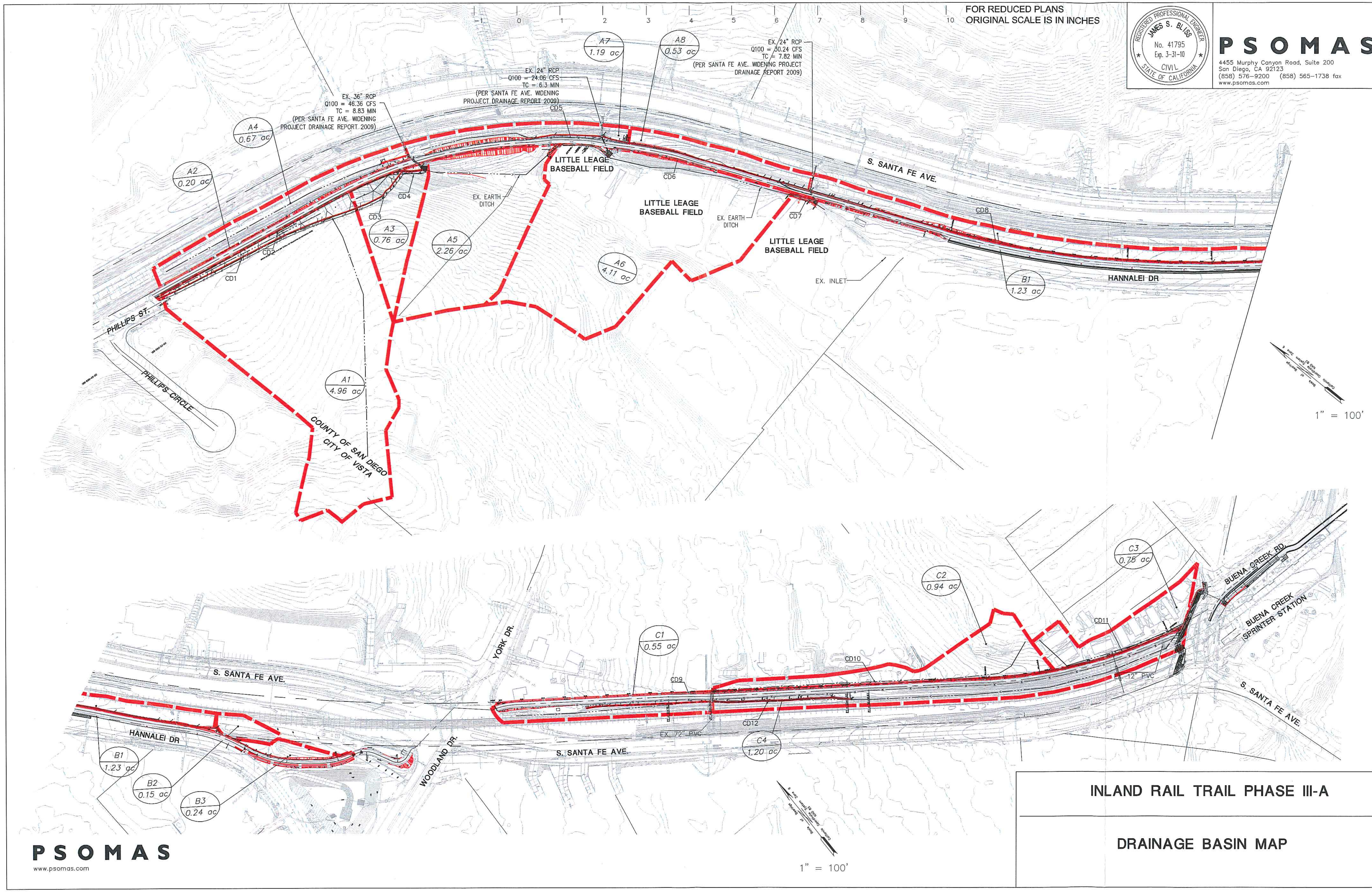
Basin C2 delivers overland flow from a hillside to the north of the bike path to the existing 96" culvert opening. This flow is only impacted by the addition of a concrete ditch at the base of a concrete fill retaining wall. Flow to the 96" culvert is a marginal percentage of the total pipe's capacity at 2.0 cfs and is not different from the existing condition.

Basin C3 conveys flows from an existing mobile home park. The Runoff Coefficient used for this basin is medium density residential based on data taken from Appendix C. Flow in this area is routed to a proposed catch basin and into a 12" PVC pipe which outlets to the south side of the bike path into an existing concrete ditch inside the NCTD R/W at Buena Creek Road. Hydraulic data for the 12" PVC can be found in Exhibit 3.

Drainage Basin C4 is similar to that of C1, collecting runoff from the tracks and the bike path, but flows east to west to the same drop inlet into the existing 96" CMP culvert. The bike path Bioretention Trench treats the new runoff and the additional 100-year Q is only 1.2 cfs. The existing concrete ditch is only slightly realigned to accommodate the new bike path and slopes.

CONCLUSION

The construction of the Inland Rail Trail bike path has very little impact to the existing natural or man-made drainage facilities. The generation of a calculated Q100 2.3 cfs in Segment A and 1.9 cfs in Segment C is less than 2% of the entire flow and capacity. This increase generated by the bike path is mitigated by the use of Bioretention Trenchs and rip-rap energy dissipaters, and, therefore, has negligible impact to the existing facilities.



INLAND RAIL TRAIL PHASE III-A

DRAINAGE BASIN MAP

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EXHIBIT 2

HYDROLOGY CALCULATION DATA TABLE

PSOMAS
3111 Camino Del Rio North, Suite 702
San Diego, CA 92108

INLAND RAIL TRAIL - PHASE 2
5DOK010100
CALCULATED BY: SCO
CHECKED BY: JSB

SEE APPENDIX D OF THIS REFERENCED REPORT FOR "SANTA FE AVE. ROAD WIDENING PROJECT DRAINAGE REPORT" SEPTEMBER 2009, PREPARED BY PROJECT DESIGN CONSULTANTS

EXISTING CHANNEL HYDROLOGY

HYDROLOGY CALCULATIONS-100 YEAR STORM EVENT

DRAINAGE AREA	Land Type	A (ACRES)	Tc (MIN)	C	*I100 (IN/HR) 100 YR P6 = 3.35	ΔQ100 (CFS)	ΣQ100 (CFS)	SLOPE (%)	SECTION	REMARKS
HYDROLOGY CALCULATIONS PER METHODS DESCRIBED IN THE 2003 COUNTY OF SAN DIEGO HYDROLOGY MANUAL										
A1	UNT	5.11	6.9	0.35	7.17	12.8	12.8	10.0%	ND	(Max Ti Used per Table 3-2) Lm=100 Ti=6.9
A2	BP	0.25	7.2	0.90	6.96	1.6	14.4	2.2%	Bio V-Swale	dH=15', s=2.2%(effective) Tc=7.2 min.
A3	UNT	0.76	9.8	0.35	5.72	1.5	**15.9**	2.0%	CD	Flow to CD4 (Exist Natural Channel)
A4	RW	0.72	12.5	0.57	4.89	2.0	17.9	1.0%	CD	Additional Flow generated by Bike Path Initial Run (track side) - Lm=70 Ti=12.5 min.
KNOWN OFFSITE FLOW (EX. 36"RCP)			8.8		6.12	**46.4**			36" RCP	Exist. 36" RCP data from Santa Fe Ave Widening Proj. - Drainage Report 2009
A5	UNT	2.26	10.5	0.35	5.48	4.3	68.6	3.0%	ND	(overland) dH=68', s=17.1%, Tc=10.47min.
							68.6		Rect. Channel b=4.25', h=1.5'	Flow to CD5: Bypass around Little League Field
A6	UNT	4.11	19.1	0.35	3.72	5.3	74.0	2.6%	NC	dH=11.2', s=2.6% Tc=19.1 min.
A7	RW	0.6	12.5	0.57	4.89	1.7	75.6	1.3%	CD	Additional Flow Generated by Bike Path Initial Basin - Lm=70 Ti=12.5 MIN
KNOWN OFFSITE FLOW (EX. 24"RCP)			6.3		7.60	**24.1**			24" RCP	Exist. 24" RCP data from Santa Fe Ave Widening Proj. - Drainage Report 2009 + FLOW from A7
							99.7		Rect. Channel b=4', h<1.5'	Flow to CD7: Bypass around Major League Field
KNOWN OFFSITE FLOW (EX. 24"RCP)			7.8		6.91	**30.2**			24" RCP	Exist. 24" RCP data from Santa Fe Ave Widening Proj. - Drainage Report 2009
A8	RW	0.53	12.5	0.57	4.89	1.5	31.7			Additional Flow to Exist. 24" RCP (Ex 24" + A8)
							131.4			Combined Flow to Exist. Natural Channel
B1	RW	1.23	10.3	0.45	5.54	3.1		3.0%	CD	Additional Flow added to Exist. Conc. Ditch Initial Basin - Lm=100 &3% Ti=10.3 MIN
KNOWN OFFSITE FLOW from "NCTD Culvert 4" to Exist. CD						**35.0**			24" RCP	Exist. 24" RCP data from Santa Fe Ave Widening Proj. - Drainage Report 2009 Flow into Exist. Conc. Ditch
							38.1			Combined Flow in Exist Conc. Ditch
B2	UNT	0.15	10.3	0.35	5.54	0.3	**38.4**	2.0%	CD	Initial Basin - Lm=85 &3% Ti=10.9 MIN
B3	UNT	0.24	9.5	0.57	5.83	0.8	**39.2**	2.6%	CD	Additional Flow droped into Exist. SD System. dH=5', s=2.6% Tc=9.5 min.
C1	RW	0.55	10.6	0.57	5.44	1.7	**1.7**	2.5%	CD	Initial Basin - Natural: Lm=93's=2.5%, Ti = 10.6min.
										Flow Dropped into 96" Culvert
C2	UND	0.94	8.3	0.35	6.37	2.1	**2.1**	6.0%	CD	Initial Basin - Natural: Lm=100', s=6.0%, Ti=8.3min.
										Flow into North end of Culvert
C3	MOBILE H.	0.75	5.4	0.63	8.40	4.0	4.0	4.5%	CD	Initial Basin - MDR 12DU/Ac s=4.5% Lm=100', Tc=5.4mir
										C3 Flow into 12" PVC Line 'G' to C4
C4	RW	1.20	13.2	0.57	4.72	3.2	**7.2**	0.6%	CD	Max based on Table 3-2 Lm=50' s=0.5% Tc=13.2min Dropped into 96" Culvert
							15.0			Total Flow Placed into 96" Culvert from IRT Project - Impacted by the Bike Path

* Intensities are based on the Regression equation from the San Diego County - Hydrology Manual, Figure 3-1 where:

I(t) = 7.44 * P6 * D^-0.645

**Q based on the rational method equation from the County of San Diego Hydrology Manual expressed as:

Q = C * I *A

***Q1,T1,I1 is defined by the tributary with the shortest Tc. Select largest Qt, and the Tc associated with it.

LEGEND:
- C.P. Concentration Point
- LDR Low Density Residential
- UNT Undisturbed Natural Terrain
- ND Natural Ditch
- CD Concrete Brow Ditch Type B
- BP Bike Path
- RW Rail Way = BP + UNT (Weighted C Coeff)

APPENDIX D

DRAINAGE REPORT REFERENCE:
SOUTH SANTA FE AVENUE WIDENING PROJECT

DRAINAGE REPORT
SOUTH SANTA FE AVENUE
WIDENING PROJECT

County of San Diego, CA
SEPTEMBER 2009

Prepared For:
COUNTY OF SAN DIEGO
5555 Overland Avenue (MS 0340)
San Diego, CA 92123-1295

Prepared By:
PROJECT DESIGN CONSULTANTS
701 B Street, Suite 800
San Diego, CA 92101

Project No. 1217.00

Prepared By: Matt Moore
Modified By: Richard Isaac
Modified By: C. Pack, P.E.
Under the supervision of

Debby Reece

Debby Reece RCE C56148
Registration Expires 12/31/10

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FIGURES

TABLES

APPENDICES

EXHIBITS

1. INTRODUCTION

This drainage report has been prepared for the County of San Diego (County) for the design of the storm drain improvements associated with the South Santa Fe Avenue, (SSFA) Widening Project (Project). In addition to the SSFA widening, roadway segments associated with Hannalei, York, and Woodland Drives, will also be improved to accommodate the new SSFA alignment. The aforementioned roadways are located near the southerly terminus of the Project.

The Project is located in the County of San Diego, approximately 0.6 miles northeast of State Highway 78, and consists of widening SSFA from a 2-lane to a 4-lane major arterial. In general, the Project is approximately 4,000 feet long, and is bound by: 1) Robelini Drive and Montgomery Drive to the south and north, respectively, and 2) the North County Transit District (NCTD) railroad tracks to the west between Montgomery Drive and Woodland Drive, and to the east between Woodland Drive and Robelini Drive. See Figure 1 for the Project Vicinity Map.

From a hydrologic land-use perspective, "The North County Metropolitan Planning Area Map, Sheet 1, of the Land Use Element Section II, Part XXV, of the County of San Diego General Plan" was used to determine the projected build-out zoning, or land- uses, for the drainage basins tributary to the Project (See Exhibit A-1). The latter information was used to determine the Rational Method runoff coefficients used for the Project hydrology analyses. In general, the land-uses for the Project consist of residential, office professional commercial, general commercial, and service commercial.

During the previous design phase for the project, it was determined that further drainage analysis and improvements were required south of the proposed project surface improvements near Anna Lane and Woodland Drive to mitigate for the drainage impacts associated with the project. The County contracted with Rick Engineering to evaluate different drainage alternatives. Rick Engineering's preliminary analysis is presented in the *South Santa Fe Avenue (North) Widening Anna Lane Drainage Improvements Preliminary Alternatives Analysis* (dated April 11, 2008), (herein referred to as the "Anna Lane report"). After reviewing the Anna Lane report, the County chose Alternative 5B, which consists of constructing a detention/water quality basin near the proposed Woodland Drive/Hannalei Drive intersection. Therefore, the main purpose of this drainage report revision is to update the drainage calculations to incorporate the new basin, including detention routing and water quality calculations. Note that the original and revisions to the Storm Water Management Plan (SWMP) for the project are being completed by Rick Engineering, but the water quality basin drawdown calculations are contained herein.

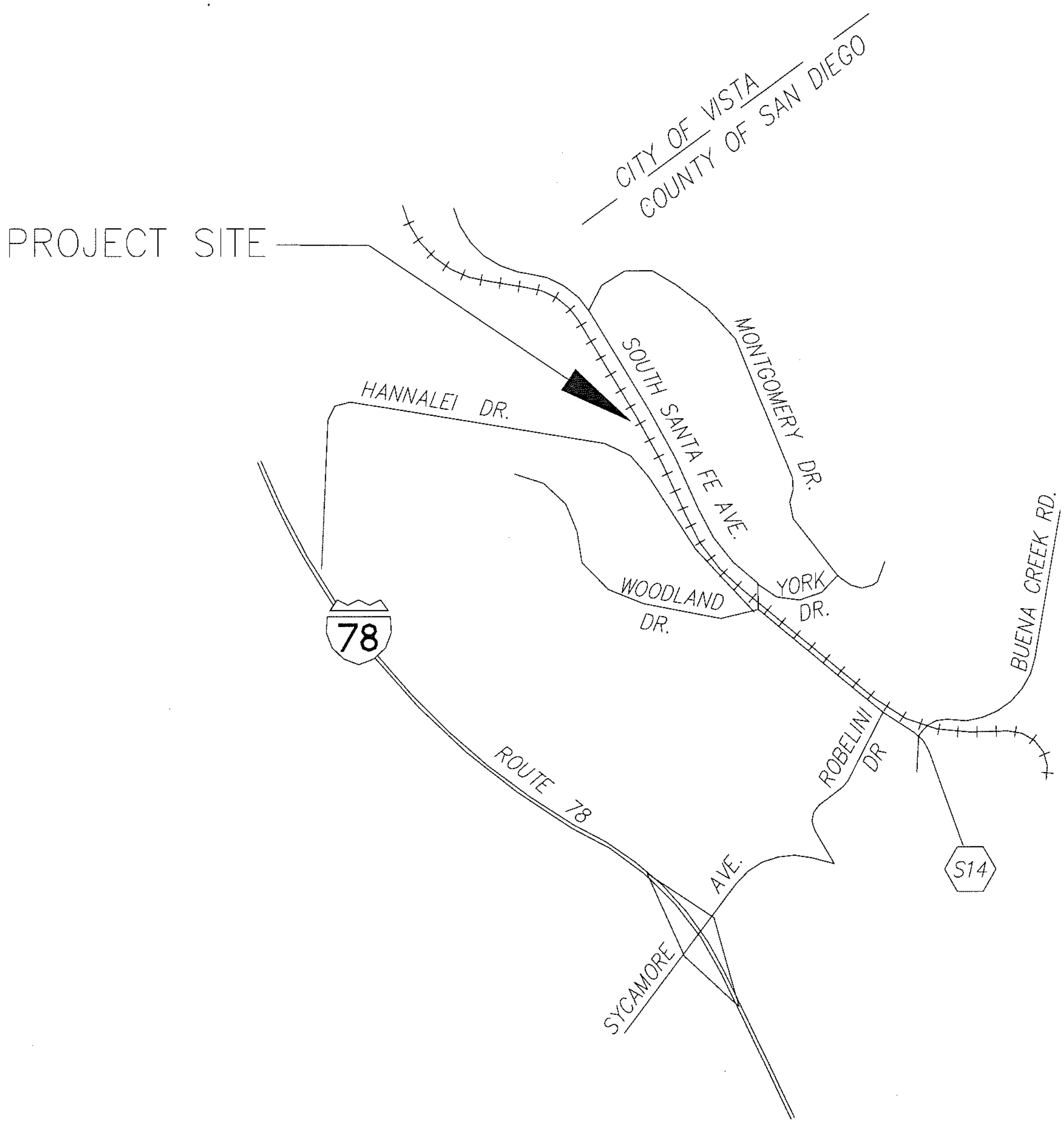

Figure 1: Vicinity Map

2. EXISTING DRAINAGE IMPROVEMENTS

SSFA, within the project limits, is a two-lane crowned paved roadway with no curb, gutter or sidewalk. The section below describes the five existing drainage basins and improvements, relative to the improvement plan roadway stationing and the Rational Method node numbers. The existing condition hydrology node numbers are identified on Exhibit B.

- Systems 100A, 100B, and 200 (Nodes 100-295): Storm flows from the northern most three basins are collected and conveyed from the east side of the roadway to the west side via inlet and pipe systems. The first is located just north of Montgomery Drive (Station 8+75, Node 110); the second is just north of the first driveway immediately south of Montgomery Drive (Station 13+00, Node 155); and the final is located at approximate Station 19+25 SSFA (Node 270). These three systems consist of inlets, pipes, and headwalls. Storm flows are conveyed to an existing ditch located between SSFA and the NCTD railroad (RR) tracks. Existing flows are then conveyed under the RR tracks by three culverts (NCTD Culverts 1, 2 and 3), just downstream of each outlet from SSFA. These three NCTD culverts do not have capacity to convey the existing flows, therefore some bypass occurs. This bypass continues south to a fourth culvert (NCTD Culvert 4).

- System 300A (Nodes 300-395): Storm flows from this basin, which consist of the southern half of the project, are collected and conveyed via roadside swales to the existing York Drive/SSFA intersection. At this location flows from the east side of SSFA cross the road, are combined with the flows from the west side of SSFA and the bypass flows from the other Systems, then subsequently are conveyed under the RR tracks via NCTD Culvert 4. This culvert does not have capacity for the total combined flow to this location. Bypass continues as sheet flow in a southwesterly direction, through the property west of Hannalei Drive.

- System 300D (Nodes 425-435): Storm flows from this final basin drain via sheet flow from the west side of the NCTD RR, combine with the outflow and bypass flows from NCTD Culvert 4, to the intersection of Woodland Drive and Anna Lane. Note that there is a 0.82 acre subarea at the northwest corner of the intersection of Woodland Drive and Anna Lane (Nodes 430-435) that appears to not match the topography. This is because the area represents a portion of the Buena Vista 9 subdivision, which was constructed after the date of topography. The runoff coefficient and flow rate from this 0.82 acre area was copied into System 300D from the results presented in *Rough Grading Drainage Study for Buena Vista 9* (prepared by Hunsaker and Associates, dated September 29, 2006). Pertinent excerpts from the report are included in Appendix 17. At the downstream end of System 300D, there is one small pipe crossing under Woodland Drive which is substantially undersized. This pipe conveys low flows to a small roadside swale/ditch on the west side of Anna Lane. When the capacity of this small swale is reached, this area floods, saturating the front yards of the properties along Anna Lane. Project flows combine with flows from other adjacent tributary areas and continue south to an existing 60-inch RCP culvert located on the north side of the existing commercial center, south of the southerly terminus of Anna Lane. The Anna Lane report contains calculations that show how the total combined existing condition 100-year peak flow to the existing 60-inch headwall is too large to drain into the pipe without causing significant ponding at the surface over the headwall. Therefore, the area surrounding the headwall acts as a detention basin for the 100-year storm.

3. PROPOSED DRAINAGE IMPROVEMENTS

This section describes the proposed drainage improvements for the Project relative to the improvement plan roadway stationing and the Rational Method node numbers. The proposed condition hydrology node numbers are identified on Exhibit C.

Additionally, this section also provides a general description of the drainage improvements that will be used at a number of the steep driveways that tie into SSFA. The driveway drainage and grading improvements are a key component of the overall SSFA roadway design, due to the superelevated section of SSFA for portions of the proposed improvements. In effect, the driveway improvements were designed to minimize driveway runoff from potentially sheeting across the superelevated section of SSFA.

- <u>Systems 1000, 2000, 3000, and 4000</u>: These systems are approximately located between Stations 9+00 and 22+00 of SSFA. Storm runoff from tributary basins that are located on the east side of SSFA is conveyed by overland flow then collected and conveyed by underground piping to the west side of SSFA. Subsequently, the runoff is discharged to a concrete brow ditch located between SSFA and the NCTD RR tracks. Similar to the existing condition, some of the NCTD culverts will have bypass for the peak 100-year flow. Therefore, for each of the culverts with bypass flows, the bypass flows were added to the next downstream system.

- System 1000: This system discharge location is at approximate Station 9+00 of SSFA. The system consists of 18-inch to 36-inch RCP's. Runoff is collected by inlets on SSFA and Montgomery Avenue and a catch basin north of the intersection of SSFA and Montgomery Avenue. Runoff is discharged south of SSFA at approximate station 9+00 onto a D-40 riprap pad and thence into the NCTD swale. Runoff then flows southeasterly via the swale to NCTD Culvert 1 under the RR tracks at approximate Station 10+60 SSFA.

- System 2000: This system discharge location is at approximate Station 13+00 of SSFA. The system consists of 24-inch to 30-inch RCP's. Runoff is collected by grate inlets within a commercial center. Runoff is discharged south of SSFA at approximate Station 13+00 onto D-40 riprap pad and thence into a concrete brow ditch. Runoff then flows southeasterly via the concrete brow ditch to NCTD Culvert 2 under the RR tracks at approximate Station 15+60 SSFA. Bypass from this culvert continues southeasterly in a concrete brow ditch to NCTD Culvert 3 crossing under the RR tracks at approximate Station 20+85 SSFA.

- System 3000: This system discharge location is at approximate Station 18+90 SSFA. The system consists of 18-inch to 36-inch RCP's. Runoff is collected by inlets on SSFA, private driveways, and catch basins. Runoff is discharged south of SSFA at approximate Station 18+90 onto a D-40 riprap pad and thence into a concrete brow ditch. Runoff then flows southeasterly to NCTD Culvert 3 under the RR tracks at approximately Station 20+85 SSFA. Bypass from this culvert continues southeasterly in a concrete brow ditch to NCTD Culvert 4 crossing under the RR tracks at approximate Station 34+25 SSFA.

- System 4000: This system discharge location is at approximate Station 21+75 SSFA. The system consists of 18-inch RCP's. Runoff is collected by an inlet on SSFA and catch basins north of SSFA, and is discharged south of SSFA at approximate Station 21+75 onto a D-40 riprap pad and thence into a concrete brow ditch. Runoff then flows southeasterly via the concrete brow ditch to NCTD Culvert 4 under the RR tracks at approximate Station 34+25 SSFA. NCTD Culvert 4 conveys all proposed condition flows southerly to Woodland Drive and Anna Lane with no bypass.

- System 5000: System 5000 consists of all of the drainage area that drains into the east side of the proposed detention/water quality basin at the Hannalei/Woodland Drive intersection. The system consists of 18-inch to 48-inch RCP's. Runoff is collected by inlets on SSFA, York Drive, Frontage Road, Woodland Drive, and a series of catch basins north of SSFA. Runoff from this system will be combined with System 9000 and detained and treated prior to draining south in Anna Lane via a proposed 42/48-inch pipe to the existing 60-inch storm drain headwall located on the north side of the existing commercial center, at the southerly terminus of Anna Lane.

- System 9000: System 9000 consists of all of the drainage area that drains into the west side of the proposed detention/water quality basin at the Hannalei/Woodland Drive intersection. The system consists of 18-inch to 36-inch RCP's. Runoff is collected by inlets on Hannalei Drive and an F-type catch basin that collects the discharge from NCTD Culvert 4 (which includes upstream bypass from NCTD Culvert 3 and the peak flow from System 4000).

- System 7000: System 7000 consists of the small drainage area on Hannalei Drive and Woodland Drive that drains into the pipe that outlets from the basin. The system consists of 18- and 24-inch pipes.

- Driveway Improvements: A key element in the design of the driveway drainage improvements is to prevent the sheeting of flow across segments of superelevated roadway and to maintain 12-foot dry lanes during the peak 100-year storm.

- Type "F" Catch Basins/Ditches: Ditches will convey runoff to Type 'F' catch basins to: 1) protect the newly manufactured slopes along the easterly side of SSFA, 2) prevent the sheeting of runoff across superelevated segments of the roadway, and 3) direct runoff to the driveway drainage improvements. PDC and the County agreed to use Type F catch basins, in lieu of CMP risers (STD DWG D-16), to minimize maintenance.

4. HYDROLOGIC CRITERIA, METHODOLOGY, AND RESULTS

4.1 Hydrology Analysis Criteria

Table 1 below summarizes the drainage criteria for the project. Appendix 1 includes the 100-year, 6-hour and 24-hour Isopluvials, Intensity-Duration Chart and the Soil Group Map.

Table 1: Hydrology Analysis Criteria

Design Storm:	100-year
Land Use:	Mostly Residential, Commercial, and Streets
Runoff Coefficients:	Based on criteria presented in the 2003 County of San Diego Hydrology Manual. (See Section 4.3)
Hydrologic Soil Group:	C and D per the County of San Diego Soil Group Maps. Soil type D was used for the analysis.
Intensity:	Based on criteria presented in the 2003 County of San Diego Hydrology Manual.

4.2 Hydrology Analysis Methodology

The Modified Rational Method was used to determine the 100-year storm runoff for the design of the Project storm drainpipes, curb inlets, concrete brow ditches and erosion protection.

This section of the report addresses the Project hydrology methodology relative to: 1) the selection of the runoff coefficients, 2) the hydrology methodology used, and 3) the AES Rational Method software model used for the hydrologic analysis.

4.3 Project Runoff Coefficients

From a hydrologic land-use perspective, "The North County Metropolitan Planning Area Map, Sheet 1, of the Land Use Element Section II, Part XXV, of the County of San Diego General Plan" (Land Use Map) was used to determine the build-out zoning, i.e. land-uses, for the drainage basins tributary to SSFA. For the north-western project drainage areas within City of Vista jurisdiction, the City of Vista Zoning Map was used as a reference to determine C values. (See Exhibit A-2). The zoning maps were used to determine the runoff coefficients (C values) used in the Rational Method. See Appendix 2 for the calculation of the weighted C values for each System, and Exhibits A-1 and A-2 for the Land Use Maps. For drainage subareas with multiple land uses, a composite runoff coefficient was calculated. See calculations in Appendix 2.

4.4 Explanation of AES Rational Method Software

Advanced Engineering Software (AES) Rational Method Program was used to perform the hydrologic calculations. The following section provides a brief explanation of the computational procedure used in the computer model.

The AES Modified Rational Method Hydrology Program is a computer-aided design program where the user develops a node link model of the watershed. Developing independent node link models for each interior watershed and linking these sub-models together at confluence points creates the node link model. The intensity-duration-frequency relationships are applied to each of the drainage areas in the model to get the peak flow rates at each point of interest.

4.5 100-year Hydrology Analysis Results

The results of the existing and proposed conditions Rational Method computer output are located in Appendices 3 and 4. A summary of the 100-year peak flows are shown in Table 2 on the following page.

As Table 2 indicates, if detention was not provided, the proposed condition peak flow would be greater than the existing conditions flow at the Anna Lane - Woodland Drive intersection. In addition, there is a small area (approximately 3 acres) at the intersection of York and SSFA that in the existing condition drains south along SSFA to the localized low point near the existing RV Storage Center. In the proposed condition, flows from this area are collected via inlets and conveyed to the Anna Lane – Woodland Drive intersection. The increase of flow will be detained with the proposed basin so that the ponding depth at the existing 60-inch pipe at the southern end of Anna Lane under proposed conditions will be less than or equal to the existing condition ponding depth, per Rick Engineering's detention analysis in the Anna Lane report.

Table 2. Summary of Hydrology Results for 100-year Storm

	EXISTING CONDITIONS						PROPOSED CONDITIONS					
Point of Interest (Description)	System	Node(s)	Q_{100} IN (cfs)	Q_{100} OUT (cfs)	Contrib. Area (acres)	Adjust. Contrib. Area (acres)*	System	Node(s)	Q_{100} IN (cfs)	Q_{100} OUT (cfs)	Contrib. Area (acres)	Adjust. Contrib. Area (acres)*
NCTD Culvert 1	System 100A	120	49.5	49.5	13.8	N/A	System 1000	1090	46.4	46.4	13.76	N/A
NCTD Culvert 2	System 100B	170	32.4	25.9	6.64	N/A	System 2000	2040	25.7	24.1	5.55	N/A
NCTD Culvert 3	System 200	290	55.3	29.9	13.9	15.22	System 3000	3090	57.5	30.2	12.69	13.04
Total outfalls towards school=			**137.2**	**105.3**	**34.34**	**35.66**			**129.6**	**100.7**	**32.0**	**32.35**
NCTD Culvert 4	System 300A	370	66.0	53.3	15.64	22.63	Portion of System 9000	9082	35.0	35.0	3.61	9.78
Anna Lane - Woodland Drive Intersection	System 300D	435	97.8	97.8	25.39	32.38	System 5000	5098	74.8	-	17.98	N/A
							System 9000	9090	51.2	-	9.11	15.29
							Basin Subtotal:	N/A	117.5	75.5	27.09	33.27
							System 7000		20.6	20.6	3.24	N/A
Total at outfall=			**97.8**	**97.8**	**25.39**	**32.38**			**138.1**	**77.9**	**30.33**	**36.51**

Note: Adjusted contributing areas include adjusted area to account for upstream bypass during 100-year peak event, if applicable. See split flow calculations in Appendix 5.

4.6 2 and 10-year Hydrology Analysis Results

As shown in Table 2, there is a minor increase in flow between existing and proposed conditions for Culvert 3. The minor increase in flow and the split flow situation at the culverts prompted the County to request additional hydrology calculations for the 2 and 10 year storms in order to evaluate whether or not a hydrologic condition of concern exists for the unlined channel near the ball fields. This channel conveys the runoff from Culverts 1, 2, and 3 in a southerly direction. The following tables summarize the results of the 2 and 10 year analysis. Refer to Appendix 18 for the calculations.

Table 3. Summary of Hydrology Results for 2-year Storm

	EXISTING CONDITIONS							PROPOSED CONDITIONS						
Point of Interest (Description)	*System*	*Node(s)*	*Q_2 IN (cfs)*	*Q_2 OUT (cfs)*	*Tc (min)*	*Contrib. Area (acres)*	*Adjust. Contrib. Area (acres)**	*System*	*Node(s)*	*Q_2 IN (cfs)*	*Q_2 OUT (cfs)*	*Tc (min)*	*Contrib. Area (acres)*	*Adjust. Contrib. Area (acres)**
NCTD Culvert 1	System 100A	125	19.9	19.9	8.58	13.8	N/A	System 1000	1096	18.1	18.1	10.11	13.76	N/A
NCTD Culvert 2	System 100B	170	13.2	13.2	6.83	6.64	N/A	System 2000	2040	10.5	10.5	6.83	5.55	N/A
NCTD Culvert 3	System 200	290	19.8	19.8	10.01	13.9	N/A	System 3000	3090	17.6	17.6	12.76	12.69	N/A
Total outfalls towards school=			**52.9**	**52.9**		**34.34**	**N/A**			**46.2**	**46.2**		**32.00**	**N/A**

Note: Adjusted contributing areas include adjusted area to account for upstream bypass during peak event, if applicable.

Table 4. Summary of Hydrology Results for 10-year Storm

	EXISTING CONDITIONS							PROPOSED CONDITIONS						
Point of Interest (Description)	*System*	*Node(s)*	*Q_{10} IN (cfs)*	*Q_{10} OUT (cfs)*	*Tc (min)*	*Contrib. Area (acres)*	*Adjust. Contrib. Area (acres)**	*System*	*Node(s)*	*Q_{10} IN (cfs)*	*Q_{10} OUT (cfs)*	*Tc (min)*	*Contrib. Area (acres)*	*Adjust. Contrib. Area (acres)**
NCTD Culvert 1	System 100A	125	30.2	30.2	8.16	13.8	N/A	System 1000	1096	27.7	27.7	9.5	13.76	N/A
NCTD Culvert 2	System 100B	170	19.9	19.9	6.53	6.64	N/A	System 2000	2040	15.7	15.7	6.56	5.55	N/A
NCTD Culvert 3	System 200	290	30.1	25.2	9.61	13.9	N/A	System 3000	3090	33.9	26.2	8.32	12.69	N/A
Total outfalls towards school=			**80.2**	**75.3**		**34.34**	**N/A**			**77.3**	**69.6**		**32.00**	**N/A**

Note: Adjusted contributing areas include adjusted area to account for upstream bypass during peak event, if applicable.

As indicated in the tables, for the 2-year storm, there are no bypass flows for any of the culverts and the proposed flows are less than the existing flows for each of the culverts. For the 10-year storm, Culvert 3 is the only culvert with bypass at the headwall. Culvert 3 is also the only culvert with an increase in flow between existing and proposed conditions. However, the increase is only 1 cfs, and the combined flow between Culverts 1, 2, and 3 shows that there is a reduction in flow to the channel between existing and proposed conditions. Therefore, there is no hydrologic condition of concern for the unlined channel near the ball fields on the southwest side of the railroad tracks due to the proposed project.

4.7 Generating Hydrographs for Systems with Upstream Bypass

As Table 2 indicates, due to the split flow situation at some of the NCTD culverts for the 100-year storm, the tributary area at any point downstream of the culverts with bypass is not a straight-forward calculation. This makes calculating a hydrograph for the basin routing somewhat difficult. Due to the split flow at NCTD Culvert 3 in the proposed condition during the 100-year peak flow, the peak bypass flow drains to System 9000 and becomes tributary to the proposed detention basin. The flow rate and time of concentration for this bypass flow is added to System 9000 in the hydrology model as user-specified hydrology information at a point. Because of the split flow situation (which creates a non-linear relationship between the bypass flow rate and total upstream area), the flow was added with a proportional bypass area in System 9000 in the AES model so that any downstream confluences or flows calculated from sub-area additions would not be skewed by an incorrect upstream area. In order to generate a hydrograph for System 9000, the area flowing downstream of NCTD Culvert 3 was added as a "representative" area of the upstream bypass. The "representative" area was calculated based on the proportion of the area tributary to the culvert and the ratio of the bypass and total peak flow at the culvert headwall. For example, if 10% of the peak flow bypassed the culvert, then 10% of the total area draining to the culvert was added to the next downstream system for the adjusted area calculation. This was needed for System 9000 so that the hydrograph generator would not under-estimate the total hydrograph volume. For these calculations, refer to Appendix 5.

5. HYDRAULIC ANALYSIS CRITERIA AND METHODOLOGY

The following sections describe the criteria and methodology used in the hydraulic analysis of the proposed drainage improvements.

5.1 Hydraulic Analysis Criteria

Table 3 below presents the hydraulic criteria used in the design of the storm drain improvements.

Table 3: Hydraulic Analysis Criteria

FACILITY	CRITERIA
Underground storm drain systems	100-year storm HGL 1-ft below the inlet opening and 1-ft below cleanout top-of-rim elevations.
Inlets	Minimum 85% capture for continuous grade inlets
Brow Ditches	100-year storm capacity.
Riprap Protection	Per Regional Std. Dwg. D-40
Dry Roadway Width	12-foot wide dry lane within the roadway traveled way
Detention Basin Routing	Detain the peak 100-year flow rate in the proposed basin such that the combination of the detained basin flow and the flow from the rest of the drainage area to the existing 60-inch pipe produces a maximum ponded water surface elevation at the southerly end of Anna Lane no worse than the existing condition, per Rick Engineering's offsite analysis.
Water Quality Basin Routing	Drain time for Water Quality volume between 24 and 72 hours, per County BMP worksheet guidance.
Emergency Spillway	Design spillway for undetained peak 100-year flow with 1 foot of freeboard. (Worst case situation assuming basin outlet structure is clogged.)

5.2 Hydraulic Analysis Methodology

Several computer models were employed to analyze the capacity of the proposed storm drain system. The AES Pipeflow and WSPG software hydraulic models were used to determine the hydraulic grade lines for the storm drain system improvements. FLOWMASTER, a proprietary software program by Haestad Methods, was used for the brow ditches, and to determine the roadway gutter depth and flood width necessary to determine the roadway dry-lane width required for emergency vehicle access. Additionally, CULVERTMASTER, proprietary software by Haestad Methods, was used in calculating the capacity and bypass of the NCTD culverts (see Appendix 5 for calculations). PondPack, another proprietary software by Haestad Methods, was used to do the detention and water quality routing for the proposed detention/water quality basin. The following sections provide a brief description of the analytical procedures used in each model.

5.3 Explanation of the AES Pipeflow Software

The AES Pipeflow model was used to determine the hydraulic grade line for the storm drain pipe improvements for this project. The AES computational procedure is based on solving Bernoulli's equation for the total energy at each section; and Manning's formula for the friction loss between the sections in each computational reach. Confluences are analyzed using pressure and momentum theory. In addition, the program uses basic mathematical and hydraulic principals to calculate data such as cross sectional area, velocity, wetted perimeter, normal depth, critical depth, and pressure and momentum. Model input basically includes storm drain facility geometry, inverts, lengths, confluence angles, and downstream/upstream boundary conditions, i.e., initial water surface elevations.

5.4 Explanation of FLOWMASTER Software

The FLOWMASTER model computes flows, water velocities, depths and pressures based on several well-known formulas such as Darcy-Weisbach, Manning's, Kutter's, and Hazen-Williams. For this project, Manning's equation was used in the design of the brow ditches, gutter depths and in the dry lane width calculations.

5.5 Explanation of CULVERTMASTER Software

The CULVERTMASTER model analyzes culvert hydraulics based on methodologies set forth in Hydraulic Design Series No. 5 (HDS 5), Hydraulic Design of Highway Culverts (1985) as prepared by the Federal Highway Administration. CULVERTMASTER was used to model split flow at the NCTD culverts. Based on the spot elevations provided by the County's survey crew, the configuration at each culvert headwall was approximated by the culvert and a generic weir set equal to the elevation of the surrounding ground where bypass starts to occur. The model solved for the ponded water surface elevation that passes the design discharge, and reported the flow rate in the culvert and the bypass flow over the weir. The bypass flow was added to the downstream hydrology model as user-specified information so that the bypass flows could be added downstream.

5.6 Explanation of PondPack Software

The PondPack model was used in the detention and water quality routing for the proposed detention basin. The program inputs include the elevation-area relationship, the hydrographs, and the design for the outlet structures. The level-pool routing routine is used to generate the detained outflow hydrograph. The hydrograph for System 7000 was added to the outflow hydrograph from the basin for the final project hydrograph at Node 7100 (the pipe junction underneath the intersection of Woodland Drive and Anna Lane).

6. HYDRAULIC ANALYSIS RESULTS

In general, the storm drain improvements for this project consist of:

- A system of underground drainpipes.
- Inlets and brow ditches.
- One water quality/detention basin.
- Riprap outlet protection.

The following sections present the results of the SSFA storm drain system hydraulic analyses.

6.1 Storm Drainpipe Analysis

In general, the drainpipe system was designed to flow open channel for the 100-year storm event. However, some segments of the drainpipe are under pressure. As a result, watertight joints will be used at locations where deemed appropriate. See Appendices 6-12 for hydraulic analysis output and Exhibit D for the AES Pipeflow node number locations. Note that the AES Rational Method node numbers are used in the AES Pipeflow computer outputs wherever possible to facilitate the plan check. The storm drain analysis results for the Anna Lane storm drain show that the HGL is above the rim elevations for the two most downstream cleanouts. The recommended limit on the HGL elevation could not be met due to the ponding at the 60-inch pipe and so pressure manholes will be used at those locations. The starting water surface elevation used for the analysis was taken from the calculations from the revised Anna Lane report. Rick Engineering refined their preliminary model with the revised final engineering data presented herein, and the starting water surface elevation at the 60-inch pipe was revised per their results so that the HGL could be finalized for the proposed Anna Lane storm drain.

6.2 Curb Inlet, Type F Catch Basin and Ditch Analyses

The Project storm drain inlets, Type F catch basins, and ditches were designed for the 100-year storm event. The inlets were designed to minimize or prohibit bypass, especially at the driveways where SSFA is superelevated. The County of San Diego inlet design formulas, and gutter depths per the FLOWMASTER computer output, were used to design the required curb opening lengths of the inlets on a continuous grade. See Appendices 13-14 for the inlet, catch basin, and ditch analyses. The continuous grade inlets were sized to capture the vast majority of the 100-year peak flow. For any inlets with bypass flow, the bypass amount was added to the next downstream inlet for inlet sizing purposes. For the pipe hydraulic analysis, the bypass amount was ignored and 100% capture of the full flow to the upstream inlet was assumed. The only inlet that did not meet the 85% capture criteria was Node 3095 (0+50 Driveway 15+15 SSF). A maximum 20-foot opening was used, with an efficiency of only 82% due to the steepness of the driveway.

6.3 Detention Basin Routing

The peak-flow attenuation of the proposed detention basin was modeled with Pondpack, a Haestad Methods detention basin routing software package. The 100-year hydrographs for System 5000 and 9000 were input into the model and the elevation-discharge and elevation-area relationships were used to produce a routed hydrograph. The hydrograph for System 7000 was added to the routed hydrograph, and the peak flow out was used to model the flows in Storm Drain Line G down Anna Lane. Rick Engineering updated the offsite analysis in the Anna Lane report to show that the project peak flow rate at the end of Line G is small enough so that the combined peak flow rate (from the project site plus offsite area) is such that it does not raise the ponded water surface elevation at that location, above existing 100-year conditions. The emergency spillway was designed to pass the peak undetained flow rate with a minimum 1 foot of freeboard.

6.4 Water Quality Volume Routing

The permanent basin will be a combined detention/water quality basin, and therefore the lower part of the basin will empty at a much slower rate to allow the water quality volume sufficient time to empty out of the basin, thereby enhancing pollutant removal. The water quality volume routing was performed in PondPack by starting the model with the full water quality volume in the basin at the initial starting time, per standard convention. The draw-down curve and the results are included in Appendix 16.

6.5 D-40 Energy Dissipater Protection

For the energy dissipation at the pipe outfalls, three different approaches were utilized:

1) For Storm Drain Lines A, B, C, and D discharging into NCTD right-of-way, a riprap pad per Standard Drawing D-40 was specified. For Lines A, B, and C, the soffit tailwater condition was eliminated and the hydraulics were re-run to find the outlet velocity with no tailwater. The pipe discharge velocity from the hydraulic model output was used to determine the riprap class and thickness per the design velocity chart on Standard Drawing D-40. The riprap rock class for Line D was downgraded, based on the criteria on page 7-2 of the County Drainage Design Manual (d_{50} shall not exceed outlet diameter of pipe). At the request of the County, the first five feet of riprap will be grouted to protect against high velocities at the outlet.

2) For Storm Drain Lines E and F discharging into the proposed detention/water quality basin, the pipe hydraulic models were copied and then revised to eliminate the tailwater condition. The velocity at the outlet with no tailwater condition was used to size the riprap energy dissipation per Standard Drawing D-40. This method produced higher velocities and represents the worst

case scenario for the outlets. Note that the first few pages of the revised pipe models are included in Appendix 14 for riprap sizing only and are not the hydraulic models used to plot the HGLs on the plans.

3) For Storm Drain Line G, the outfall apron will tie into an existing concrete pad near the existing 60" headwall location. Because of this, a D-40 dissipater is not needed. However, a splashwall will protect the slope directly across from the new outlet.

7. CONCLUSION

The goal of this project is to widen South Santa Fe Avenue to a 4-lane arterial from the current 2-lane collector. This project consists of approximately 4,000 feet of roadway located 0.6 miles Northeast of State Highway 78. This document analyses the drainage issues that are intrinsic due to the construction of this project. From the analysis the following conclusions can be made:

- According the hydrology and hydraulic models, storm flows can be safely conveyed from the eastern side of SSFA to the western side during the 100-year storm without any detrimental flooding.
- A key element in the design of the driveway drainage improvements is to prevent sheet flow from the driveways to continue onto SSFA and to maintain 12-foot dry lanes.
- The detention/water quality basin will mitigate the small diversion of flow and an increase in peak 100-year flow rate with development of the project and will also provide treatment for approximately 27.1 acres (SSFA between Stations 22+00 and 42+00 and corresponding upstream tributaries).
- The project will maintain existing drainage patterns. The drainage areas and peak flow rates to the NCTD Culverts 1, 2, and 3 are closely matched to the existing conditions. Therefore, the project should not cause significant downstream impacts downstream of those culverts.
- The offsite analysis performed by Rick Engineering shows how the post-project flow affects the existing flooding situation at the existing 60-inch headwall located at the southerly terminus of Anna Lane. This analysis confirmed that the proposed project detention basin at the Hannalei/Woodland intersection does not exacerbate the existing

100-year flooding situation at the downstream 60-inch headwall. Refer to Rick Engineering's final report dated August 19, 2009 entitled *South Santa Fe Avenue Road Widening Project (Northern Segment) Anna Lane Detention Modeling Final Design.*

LEGEND

AES RATIONAL METHOD NODE NUMBER	5150
APPROX. ZONING BOUNDARY	
DRAINAGE SUBAREA	
FLOW DIRECTION	
DRAINAGE SUBBASIN AREA (NOTE: * MEANS COMPOSITE C VALUE WAS USED)	1.00ac
EX. NCTD CULVERT #	

0 20 40 60 120 180

GRAPHICAL SCALE

SCALE: 1"=60'

JOB #: 1217.00

CREATED: 3/1/07

PREPARED BY:

PROJECT DESIGN CONSULTANTS

Planning | Landscape Architecture | Environmental | Engineering | Survey

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COUNTY OF SAN DIEGO

SOUTH SANTA FE AVENUE

HYDROLOGY MAP

PROPOSED CONDITIONS

EXHIBIT C – 1 OF 3

APPENDIX K

DRAINAGE STUDY FOR VISTA HANNALEI

DRAINAGE STUDY
FOR
VISTA HANNALEI
(FINAL ENGINEERING)

Job Number 19253-A

October 14, 2020
Revised: January 18, 2021
Revised: August 23, 2021
Revised: October 19, 2021
Revised: December 16, 2021
Revised: February 22, 2023
Revised: April 14, 2023

RICK ENGINEERING COMPANY

DRAINAGE STUDY

FOR

VISTA HANNALEI

(FINAL ENGINEERING)

Job Number 19253-A

REGISTERED PROFESSIONAL ENGINEER
BRENDAN CRAIG HASTIE
NO. 65809
EXP.
CIVIL
STATE OF CALIFORNIA

Brendan Hastie
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October 14, 2020
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Revised: August 23, 2021
Revised: October 19, 2021
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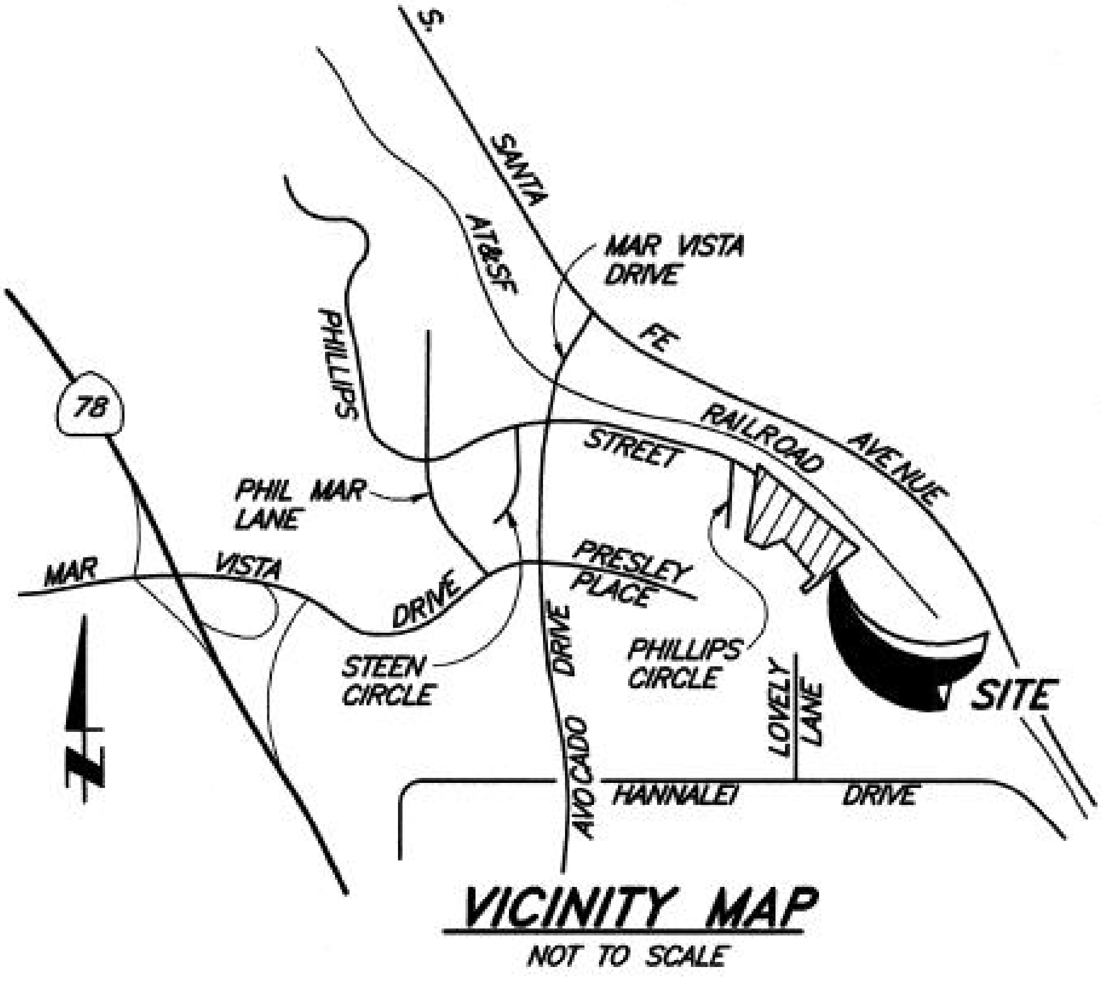
1.0 INTRODUCTION

Vista Hannalei is a residential development located in the City of Vista, approximately 800 linear feet north of Hannalei Drive and approximately 0.6 miles east of State Route 78. The project proposes 44 dwelling units, roadway features, and an emergency access road. This emergency access road connects to an existing private driveway that extends south to intersect Hannalei Drive. This private driveway will also be widened and refinished. This residential development will be constructed on varying slopes and therefore, soil will need to be imported and compacted to stabilize the foundation. Two biofiltration with partial retention BMPs (i.e. BMP-B & BMP-C) and one underground vault and Modular Wetland System (MWS) combination (i.e. BMP-A) will be used for storm water pollutant control, hydromodification management control, and flood control detention for drainage from the project site. The purpose of this report is to evaluate the impacts to any existing drainage conveyance networks as a result of the proposed project development.

The existing project site is comprised of a short grass field with a single residential property located within the site at the top of the slope. Approximately 2.5 acres of land off-site drains westerly onto a 0.8 acre private driveway that extends south and intersects Hannalei Drive. Runoff from Basin B (2.2 acres) of the project site flows north overland and over a cut fill slope into an earthen swale adjacent to the railroad. It then flows easterly until it is intercepted by a catch basin and conveyed into the existing storm drain network. Runoff from Basin A (4.2 acres) also flows north overland and down the cut fill slope where it is then conveyed westerly via earthen swale adjacent to the railroad and into an earthen ditch. An existing 36-inch Reinforced Concrete Pipe (RCP) conveys run-on from an off-site area across the railroad tracks and also discharges into the earthen ditch. This ditch flows approximately 900 linear feet easterly around the existing baseball field and into the existing storm drain system via catch basin. Drainage from the private driveway flows southerly by concrete ditch and continues easterly once it is intercepted by Hannalei Drive until it flows into the existing storm drain network via curb inlet. This is the same storm drain network that receives flow from the earthen ditch to the north.

The proposed drainage characteristics are similar to the existing drainage characteristics in that a majority of the runoff still flows northeasterly and into the earthen ditch, while a similar amount of runoff still flows north and into the earthen swale flowing west. Post-project Basin's A & E reflect the area of pre-project Basin A, while post-project Basin's B, C, & D reflect the area of pre-project Basin B. Runoff from Basin A (2.8 acres) is first treated within BMP-A, an underground vault and MWS combination. Basin E (0.9 acres) consists of pervious slope draining in a north easterly direction and ultimately drains to the earthen ditch. At the western end of the site, runoff from Basin B (1.3 acres) and Basin C (1.1 acres) is first treated through BMP-B and BMP-C, respectively, before being discharged onto Phillips Street. Both BMPs are biofiltration with partial retention and flood control detention basins. The remaining runoff from Basin D (0.2 acres) will not be treated for pollutant control requirements and is offset by BMP-F, a Modular Wetlands System. This Modular Wetlands System is sized to treat an existing upstream, off-site impervious area that is greater than the impervious area located in Basin D. Basin E contains no impervious area and is self-mitigating per section 5.2.1 of the City of Vista BMP Design Manual.

Figure 1-1: Vicinity Map



DECLARATION OF RESPONSIBLE CHARGE

I hereby declare that I am the engineer of work for this project. That I have exercised responsible charge over the design of the project as defined in Section 6703 of the Business and Professions Code, and that the design is consistent with current standards.

I understand that the check of the project drawings and specifications by the County of San Diego is confined to a review only and does not relieve me, as engineer of work, of my responsibilities for project design.

Brendan Hastie

Brendan Hastie R.C.E #65809, Exp. 9/23

2.0 HYDROLOGY

2.1 CRITERIA

The hydrologic conditions were analyzed in accordance with the County of San Diego's design criteria.

Design Storm:	100-year, 6-hour
100-Year 6-Hour Precip (inches):	P = 3.25 inches

June 2003 San Diego County *Hydrology Manual* Criteria (unit-less) (See Appendix C)

Soil Type:	C and D (See Map Pocket 1)

Intensity-Duration-Frequency (I-D-F) Curves within the June 2003 County of San Diego *Hydrology Manual* (inches per hour)

2.2 RATIONAL METHOD

To calculate the flow rates for post-project Basins D and E, the Rational Method equation was used. It should be noted that The Modified Rational Method was used to calculate an accurate Time of Concentration for Basin A through the street and into BMP-A, Basin B through the street and into BMP-B and Basin C through the street and into BMP-C. For each of the other basins, a 5 minute time of concentration was assumed because the areas are significantly smaller compared to Basin A, B and C (post-project). The Rational Method equation is defined by the following equation.

$$Q_{\text{peak, x-year event}} = C * I_{\text{x-year event}} * A$$

Where:

$Q_{\text{peak, x-year event}}$ = peak flow rate for a design storm event (i.e. 10-year, 50-year, etc.) (cfs)
C = the area-weighted runoff coefficient (see runoff coefficient criteria in section 2.1)
$I_{\text{x-year event}}$ = rainfall intensity (see intensity criteria in section 2.1)
A = tributary area to a point of interest (acres)

Weighted runoff coefficients are calculated, where appropriate, based on a percentage of the runoff coefficients for 100% Impervious Area (0.90) and 0% Impervious area (0.35) for Type D Soils, which is calculated based on the following equation:

$$C_{\text{weighted}} = 0.90 * (\text{\% Impervious Area}) + 0.35 * (1 - \text{\% Impervious Area})$$

2.3 MODIFIED RATIONAL METHOD

To calculate the flow rates for Basins A & B (pre-project) and Basin A, B and C (post-project), a Modified Rational Method analysis was performed in accordance with the methodology presented in the June 2003 County of San Diego *Hydrology Manual* to determine pre- and post-project 100-year peak discharge rates for watersheds less than 1 square-mile. The Advanced Engineering Software (AES) Rational Method computer program was used to perform these calculations. The hydrologic model is developed by creating independent node-link models of each interior drainage basin and linking these sub-models together at confluence points. The program has the capability to perform calculations for 15 hydrologic processes. These processes are assigned code numbers that appear in the results. The code numbers and their significance are as follows:

Code 1: Confluence analysis at a node
Code 2: Initial subarea analysis
Code 3: Pipe flow travel time (computer-estimated pipe sizes)
Code 4: Pipe flow travel time (user-specified pipe size)
Code 5: Trapezoidal channel travel time
Code 6: Street flow analysis through a subarea
Code 7: User-specified information at a node
Code 8: Addition of the subarea runoff to mainline
Code 9: V-Gutter flow thru subarea
Code 10: Copy main-stream data onto a memory bank
Code 11: Confluence a memory bank with the main-stream memory
Code 12: Clear a memory bank
Code 13: Clear the main-stream memory
Code 14: Copy a memory bank onto the main-stream memory
Code 15: Hydrologic data bank storage functions

In order for the program to perform the hydrologic analysis; base information for the study area is required. This information includes the land uses, drainage facility locations, flow patterns, drainage basin boundaries, and topographic elevations. The rainfall data, runoff coefficients, and soils information were obtained from the June 2003, County of San Diego *Hydrology Manual*.

2.4 RESULTS

The 100-year Modified Rational Method and Rational Method calculations for pre- and post-project conditions are provided in Appendix A and Appendix B, respectively, and the associated hydrologic drainage exhibits are located in Map Pockets 1 and 2. A summary of the results for contributing areas are listed in the following tables:

Table 1- Hydrologic Summary Table (Pre-project)

Basin	Watershed Area (acres)	Time of Concentration (min)	100-Year Peak Flow Rate (cfs)
A	4.2	11.0	7.4
B	2.2	8.0	4.9
F[1]	2.1	12.8	4.9
TOTAL:	**8.5**		**17.2**

Table 2- Hydrologic Summary Table (Post-project)

Basin	Watershed Area (acres)	Time of Concentration (min)	Undetained 100-Year Peak Flow Rate (cfs)	Detained 100-Year Peak Flow Rate (cfs)
A	2.8	10.0	11.3	6.7
E	0.9	5.0	2.7	
B	1.3	7.8	4.9	2.7
C	1.1	8.0	4.4	
D	0.2	5.0	1.7	
F[1]	2.3	12.7	5.7	5.7
TOTAL:	**8.6**			**15.1**

100-YEAR DETAINED PEAK FLOW TO EXISTING CHANNEL

Table 3 – Peak Flowrate for Confluence of Outfall A and Outfall D at POC-1

	Pre-Project	Post-Project
Area (ac)	16.1	15.6
Time of Concentration (min)	14.6	14.6
100-Year Peak Flow Rate	37.8	36.9[2]

([1]): Flow Rate calculated using the Modified Rational Method to Outfall C

([2]): Flow Rate from Outlet A uses mitigated peak flows when conferencing with Outlet D. See Appendix A & B for confluence calculations.

3.0 HYDRAULICS

The 100-year post-project peak flow rates determined using the methodology described in Section 2 were used to evaluate the hydraulic capacity for the six inlets leading into each of the three BMPs. Hydraulic analysis of the proposed outfall pipe from BMP-A, size of the proposed energy dissipater (ie. Rip-rap) for that outfall pipe discharging into the earthen ditch has been provided in Appendix G.

3.1 INLET SIZING

An inlet design calculation was completed using a computer program based on Equation 1 for inlets on grade and Equation 2 inlets in sump:

Type A Inlets on a Grade

$$Q = 0.7\ L\ (a + y)^{3/2} \qquad \text{(Equation 1)}$$

Where:
y = depth of flow approaching the curb inlet, in feet (ft)
a = depth of depression of curb at inlet, in feet (ft)
L = length of clear opening of inlet for total interception, in feet (ft)
Q = interception capacity of the curb inlet, in cubic feet per second (cfs)

Type B Inlets in a Sump

$$Q/L = 1.5\ \text{cfs/ft} \qquad \text{(Equation 2)}$$

Where:
Q = inlet capacity, in cubic feet per second (cfs)
L = length of clear opening of inlet for total interception, in feet (ft)

3.2 PIPE HYDRAULICS

Proposed storm drain improvements for Vista Hannalei will be analyzed using the AES Pipe Flow Hydraulics Computer Program as a part of next submittal. The program performs gradually varied flow and pressure flow profile computations and is used to calculate the hydraulic and energy grade lines. The results will be provided in an incremental and summarized form and indicate reaches of open channel and pressure flow within a given reach of pipe. The program also accounts for losses that may occur due to friction, junction structures, pipe bends, etc. The codes and an explanation of their function are as follows:

Code 1: Friction Losses
Code 2: Manhole Losses
Code 3: Pipe-Bend Losses
Code 4: Sudden Pipe-Enlargement
Code 5: Junction Losses
Code 6: Angle-Point Losses
Code 7: Sudden Pipe-Reduction
Code 8: Catch Basin Entrance Losses

The results for this AES Pipe Flow hydraulic analysis have been provided in Appendix E.

3.3 ENERGY DISSIPATER DESIGN

Energy dissipaters (i.e. riprap) at the single storm drain outfall and inflow into BMP-A will be specified using the 2014 County of San Diego *Hydraulic Design Manual*, which provides rock classifications for design velocities entering riprap outfalls.

The design velocities were determined from both the AES Pipe Flow hydraulic analyses for flow in the final reach of storm drain leading to the outfall and HEC-RAS hydraulic analysis for flow across the riprap pad immediately downstream of the outfall.

HEC-RAS cross sections will be taken at 1-foot intervals across the riprap pad in order to determine the location of the hydraulic jump that is expected to occur on the riprap pad. The flow regime after the hydraulic jump is subcritical flow at normal depth, and the flow velocity after the hydraulic jump is expected to be less than 6 feet per second. The riprap pad length was then specified based on the location of the hydraulic jump, in order to provide 5 feet (or twice the pipe diameter, whichever is greater) of length beyond the hydraulic jump. The riprap pad width is based on the 2012 Edition of the San Diego Regional Standard Drawings Book Riprap Energy Dissipation, drawing number D-40.

The dimensions and size of riprap have been provided in Appendix G.

3.4 SCOUR ANALYSIS

Normal depth analysis has been performed using Hydraulic Toolbox at sections A, B and C. The results showed that depth of flow is contained within the natural channel and would not scour the walls along the easterly edge of the property. Hence, mitigation is not required.

Refer to Appendix G for cross-section locations and detailed calculations.

4.0 DETENTION

For the detention system design, a rational method hydrologic analysis was performed to determine the 100-year peak discharge rates for the post-project condition. Detention will be provided within BMP-A, B & C to detain back flows for the 100-year storm event to pre-project conditions.

The sizing of a detention facility requires an inflow hydrograph to obtain the necessary storage volume. The modified rational method only yields a peak discharge and time of concentration and does not yield a hydrograph. In order to convert the peak discharge and time of concentration into a hydrograph, a modified rational method hydrograph synthesizing procedure was used. The modified rational method hydrograph synthesizing procedure methodology and criteria that were used are based on the Rational Method Hydrograph Procedure and Detention Basin Design, of the *San Diego County Hydrology Manual 2003*.

The 100-year hydrograph and elevation-storage-outflow rating curves were used in the HEC-1 hydrologic model to perform routing calculations for three detention systems in the project site to determine the 100-year detention volumes required for the systems to reduce the post-project peak discharge rate back to the pre-project peak discharge rate for the storm event.

The 100-year, 6-hour post-project peak discharge rates were routed using the HEC-1 hydrologic model to determine the detention volume required for the basins to reduce the post-project peak discharge rates back to the pre-project peak discharge rates. The HEC-1 detention analyses is provided in Appendix H.

5.0 CONCLUSION

Post-project runoff will be treated for water quality by the two biofiltration with partial retention BMPs, underground vault and Modular Wetland System combination and single Modular Wetland System. The two biofiltration basins and underground will also detain the 100-year 6-hour peak flows back to less than pre-project conditions while also satisfying the hydromodification management requirements per the City of Vista BMP Design Manual. Based on the results of this drainage analysis, it has been determined that the proposed residential development and driveway widening will not adversely impact the existing watershed or drainage patterns. Please refer to the report titled, "Priority Development Project Storm Water Quality Management Plan for Vista Hannalei," dated October 14, 2020 prepared by Rick Engineering Company (Job No. 19253-A), for more information on water quality and hydromodification management.

APPENDIX B

HYDROLOGY ANALYSIS
100 YEAR, RATIONAL METHOD AND MODIFIED RATIONAL METHOD
(POST-PROJECT)

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            RATIONAL METHOD HYDROLOGY COMPUTER PROGRAM PACKAGE
            Reference: SAN DIEGO COUNTY FLOOD CONTROL DISTRICT
                         2003,1985,1981 HYDROLOGY MANUAL
         (c) Copyright 1982-2014 Advanced Engineering Software (aes)
             Ver. 21.0 Release Date: 06/01/2014  License ID 1261

                           Analysis prepared by:

                          RICK ENGINEERING COMPANY
                              5620 Friars Road
                        San Diego, California 92110
                      619-291-0707   Fax 619-291-4165

 ************************** DESCRIPTION OF STUDY **************************
* J-19253A VISTA HANNALEI                                                *
* 100-YR, 6-HR POST-PROJECT CONDITION FOR BASIN A                           *
*                                                                           *
 **************************************************************************

  FILE NAME: VHAPST.RAT
  TIME/DATE OF STUDY: 14:10 01/12/2021
----------------------------------------------------------------------------
  USER SPECIFIED HYDROLOGY AND HYDRAULIC MODEL INFORMATION:
----------------------------------------------------------------------------
  2003 SAN DIEGO MANUAL CRITERIA

  USER SPECIFIED STORM EVENT(YEAR) = 100.00
  6-HOUR DURATION PRECIPITATION (INCHES) =   3.250
  SPECIFIED MINIMUM PIPE SIZE(INCH) =  18.00
  SPECIFIED PERCENT OF GRADIENTS(DECIMAL) TO USE FOR FRICTION SLOPE = 0.90
  SAN DIEGO HYDROLOGY MANUAL "C"-VALUES USED FOR RATIONAL METHOD
  NOTE: USE MODIFIED RATIONAL METHOD PROCEDURES FOR CONFLUENCE ANALYSIS
  *USER-DEFINED STREET-SECTIONS FOR COUPLED PIPEFLOW AND STREETFLOW MODEL*
     HALF-  CROWN TO   STREET-CROSSFALL:   CURB  GUTTER-GEOMETRIES:  MANNING
     WIDTH  CROSSFALL  IN-  / OUT-/PARK-  HEIGHT  WIDTH  LIP   HIKE  FACTOR
NO.   (FT)     (FT)    SIDE / SIDE/ WAY    (FT)    (FT)  (FT)  (FT)    (n)
===  =====  =========  ================  ======   ===== ====== ===== =======
  1   30.0     20.0    0.018/0.018/0.020   0.67    2.00 0.0313 0.167 0.0150
  2   13.0      8.0    0.020/0.020/0.020   0.50    1.50 0.0100 0.125 0.0180

  GLOBAL STREET FLOW-DEPTH CONSTRAINTS:
    1. Relative Flow-Depth =  0.10 FEET
       as (Maximum Allowable Street Flow Depth) - (Top-of-Curb)
    2. (Depth)*(Velocity) Constraint = 10.0 (FT*FT/S)
  *SIZE PIPE WITH A FLOW CAPACITY GREATER THAN
   OR EQUAL TO THE UPSTREAM TRIBUTARY PIPE.*
```

```
****************************************************************************
  FLOW PROCESS FROM NODE    100.00 TO NODE    105.00 IS CODE =  21
----------------------------------------------------------------------------
  >>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<
============================================================================
  USER-SPECIFIED RUNOFF COEFFICIENT = .7100
  S.C.S. CURVE NUMBER (AMC II) =  92
  INITIAL SUBAREA FLOW-LENGTH(FEET) =    86.00
  UPSTREAM ELEVATION(FEET) =    546.00
  DOWNSTREAM ELEVATION(FEET) =    545.50
  ELEVATION DIFFERENCE(FEET) =      0.50
  SUBAREA OVERLAND TIME OF FLOW(MIN.) =    6.091
  WARNING: INITIAL SUBAREA FLOW PATH LENGTH IS GREATER THAN
           THE MAXIMUM OVERLAND FLOW LENGTH =    52.44
           (Reference: Table 3-1B of Hydrology Manual)
           THE MAXIMUM OVERLAND FLOW LENGTH IS USED IN Tc CALCULATION!
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  7.539
  SUBAREA RUNOFF(CFS) =      0.54
  TOTAL AREA(ACRES) =      0.10   TOTAL RUNOFF(CFS) =      0.54

****************************************************************************
  FLOW PROCESS FROM NODE    105.00 TO NODE    110.00 IS CODE =  62
----------------------------------------------------------------------------
  >>>>>COMPUTE STREET FLOW TRAVEL TIME THRU SUBAREA<<<<<
  >>>>>(STREET TABLE SECTION #  2 USED)<<<<<
============================================================================
  UPSTREAM ELEVATION(FEET) =  545.50  DOWNSTREAM ELEVATION(FEET) =  541.00
  STREET LENGTH(FEET) =   468.00   CURB HEIGHT(INCHES) =  6.0
  STREET HALFWIDTH(FEET) = 13.00

  DISTANCE FROM CROWN TO CROSSFALL GRADEBREAK(FEET) =   8.00
  INSIDE STREET CROSSFALL(DECIMAL) =  0.020
  OUTSIDE STREET CROSSFALL(DECIMAL)  =  0.020

  SPECIFIED NUMBER OF HALFSTREETS CARRYING RUNOFF =  1
  STREET PARKWAY CROSSFALL(DECIMAL)  =  0.020
  Manning's FRICTION FACTOR for Streetflow Section(curb-to-curb) =   0.0180
  Manning's FRICTION FACTOR for Back-of-Walk Flow Section =   0.0180

    **TRAVEL TIME COMPUTED USING ESTIMATED FLOW(CFS) =       3.19
    STREETFLOW MODEL RESULTS USING ESTIMATED FLOW:
    STREET FLOW DEPTH(FEET) =  0.35
    HALFSTREET FLOOD WIDTH(FEET) =   12.22
    AVERAGE FLOW VELOCITY(FEET/SEC.) =    2.02
    PRODUCT OF DEPTH&VELOCITY(FT*FT/SEC.) =    0.70
  STREET FLOW TRAVEL TIME(MIN.) =   3.87   Tc(MIN.) =    9.96
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  5.491
  *USER SPECIFIED(SUBAREA):
  USER-SPECIFIED RUNOFF COEFFICIENT = .7350
  S.C.S. CURVE NUMBER (AMC II) =  92
```

```
  AREA-AVERAGE RUNOFF COEFFICIENT =  0.733
  SUBAREA AREA(ACRES) =    1.30      SUBAREA RUNOFF(CFS) =    5.25
  TOTAL AREA(ACRES) =        1.4        PEAK FLOW RATE(CFS) =       5.64

  END OF SUBAREA STREET FLOW HYDRAULICS:
  DEPTH(FEET) = 0.37   HALFSTREET FLOOD WIDTH(FEET) =  13.00
  FLOW VELOCITY(FEET/SEC.) =  2.11   DEPTH*VELOCITY(FT*FT/SEC.) =   0.77
  LONGEST FLOWPATH FROM NODE    100.00 TO NODE    110.00 =     554.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    110.00 TO NODE    110.00 IS CODE =  81
----------------------------------------------------------------------------
  >>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
============================================================================
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  5.491
  *USER SPECIFIED(SUBAREA):
  USER-SPECIFIED RUNOFF COEFFICIENT = .7350
  S.C.S. CURVE NUMBER (AMC II) =  92
  AREA-AVERAGE RUNOFF COEFFICIENT = 0.7341
  SUBAREA AREA(ACRES) =    1.40   SUBAREA RUNOFF(CFS) =    5.65
  TOTAL AREA(ACRES) =        2.8   TOTAL RUNOFF(CFS) =      11.29
  TC(MIN.) =    9.96
============================================================================
  END OF STUDY SUMMARY:
  TOTAL AREA(ACRES)     =        2.8  TC(MIN.) =      9.96
  PEAK FLOW RATE(CFS)   =      11.29
============================================================================
============================================================================
  END OF RATIONAL METHOD ANALYSIS
```

APPENDIX H

HEC-1 DETENTION ANALYSIS

```
1***************************************
**************************************
 *                                      *                                                   *
 *
 *   FLOOD HYDROGRAPH PACKAGE  (HEC-1)   *                                                   *    U.S. ARMY CORPS OF ENGINEERS
 *
 *              JUN   1998              *                                                   *    HYDROLOGIC ENGINEERING CENTER
 *
 *           VERSION 4.1                *                                                   *          609 SECOND STREET
 *
 *                                      *                                                   *       DAVIS, CALIFORNIA 95616
 *
 *  RUN DATE   14JAN21  TIME  07:43:45   *                                                   *           (916) 756-1104
 *
 *                                      *                                                   *
 *
 ***************************************
**************************************


                                  X     X  XXXXXXX   XXXXX          X
                                  X     X  X        X     X        XX
                                  X     X  X        X               X
                                  XXXXXXX  XXXX     X       XXXXX   X
                                  X     X  X        X               X
                                  X     X  X        X     X         X
                                  X     X  XXXXXXX   XXXXX         XXX


          THIS PROGRAM REPLACES ALL PREVIOUS VERSIONS OF HEC-1 KNOWN AS HEC1 (JAN 73), HEC1GS, HEC1DB, AND HEC1KW.

          THE DEFINITIONS OF VARIABLES -RTIMP- AND -RTIOR- HAVE CHANGED FROM THOSE USED WITH THE 1973-STYLE INPUT STRUCTURE.
          THE DEFINITION OF -AMSKK- ON RM-CARD WAS CHANGED WITH REVISIONS DATED 28 SEP 81. THIS IS THE FORTRAN77 VERSION
          NEW OPTIONS: DAMBREAK OUTFLOW SUBMERGENCE , SINGLE EVENT DAMAGE CALCULATION, DSS:WRITE STAGE FREQUENCY,
          DSS:READ TIME SERIES AT DESIRED CALCULATION INTERVAL   LOSS RATE:GREEN AND AMPT INFILTRATION
          KINEMATIC WAVE: NEW FINITE DIFFERENCE ALGORITHM

1                                                HEC-1 INPUT                                              PAGE  1

         LINE          ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10

 *** FREE ***
                       *DIAGRAM
            1          ID   VISTA HANNALEI, J-19253-A BIOFILTRATION BASIN A
            2          ID   100-YEAR DETENTION ANALYSES - PRELIMINARY ENGINEERING
            3          ID   JANUARY 12, 2020 - FILE NAME: VHA00.HC1
            4          IT       1 01JAN90    1200    1000
            5          IO       5       0

            6          KKBasinA.hc1
            7          KM   RUN DATE   1/12/2021
            8          KM   RATIONAL METHOD HYDROGRAPH PROGRAM
            9          KM   COPYRIGHT 1992, 2014, RICK ENGINEERING COMPANY
           10          KM   6HR RAINFALL IS 3.25 INCHES
           11          KM   RATIONAL METHOD RUNOFF COEFFICIENT IS 0.735
           12          KM   RATIONAL METHOD TIME OF CONCENTRATION IS 10 MIN.
           13          KM   FOR THIS DATA TO RUN PROPERLY THIS IT CARD MUST BE ADDED TO YOUR HEC-1
           14          KM   IT 2 01JAN90 1200 200
           15          BA  0.0044
           16          IN      10 01JAN90    1155
           17          QI       0     0.4     0.4     0.4     0.4     0.5     0.5     0.5     0.5     0.5
           18          QI     0.5     0.6     0.6     0.6     0.7     0.7     0.7     0.8     0.9       1
           19          QI     1.1     1.3     1.5     2.2     3.1    11.3     1.8     1.2     0.9     0.8
           20          QI     0.7     0.6     0.6     0.5     0.5     0.4     0.4       0       0       0
           21          QI       0       0       0       0       0       0       0       0
                       *

           22          KK   DET-A
           23          KO       0       0       0       0      21
           24          RS       1    STOR      -1
           25          SV       0   0.044   0.087   0.131   0.175   0.218   0.262   0.306   0.349   0.393
```

```
           26          SV   0.437   0.480   0.524    0.5
           27          SQ       0   0.015   0.021   0.025   0.029   0.033   0.036   0.065   0.078   0.089
           28          SQ   0.894   2.980   4.063    32.
           29          SE       0     1.0     2.0     3.0     4.0     5.0     6.0     7.0     8.0     9.0
           30          SE    10.0    11.0    12.0    13.0
                       *
           31          ZZ
1
                SCHEMATIC DIAGRAM OF STREAM NETWORK
 INPUT
  LINE      (V) ROUTING         (--->) DIVERSION OR PUMP FLOW

   NO.      (.) CONNECTOR       (<---) RETURN OF DIVERTED OR PUMPED FLOW

     6    BasinA.h
                 V
                 V
    22       DET-A

 (***) RUNOFF ALSO COMPUTED AT THIS LOCATION
1*****************************************
***************************************
 *                                       *                                                    *
 *
 *   FLOOD HYDROGRAPH PACKAGE  (HEC-1)   *                                                    *    U.S. ARMY CORPS OF ENGINEERS
 *
 *              JUN   1998               *                                                    *    HYDROLOGIC ENGINEERING CENTER
 *
 *           VERSION 4.1                 *                                                    *          609 SECOND STREET
 *
 *                                       *                                                    *       DAVIS, CALIFORNIA 95616
 *
 *  RUN DATE   14JAN21  TIME  07:43:45   *                                                    *           (916) 756-1104
 *
 *                                       *                                                    *
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 *****************************************
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                           VISTA HANNALEI, J-19253-A BIOFILTRATION BASIN A
                           100-YEAR DETENTION ANALYSES - PRELIMINARY ENGINEERING
                           JANUARY 12, 2020 - FILE NAME: VHA00.HC1

    5 IO         OUTPUT CONTROL VARIABLES
                       IPRNT          5  PRINT CONTROL
                       IPLOT          0  PLOT CONTROL
                       QSCAL         0.  HYDROGRAPH PLOT SCALE

      IT         HYDROGRAPH TIME DATA
                        NMIN          1  MINUTES IN COMPUTATION INTERVAL
                       IDATE      1JAN90  STARTING DATE
                       ITIME       1200  STARTING TIME
                          NQ       1000  NUMBER OF HYDROGRAPH ORDINATES
                      NDDATE      2JAN90  ENDING DATE
                      NDTIME       0439  ENDING TIME
                       ICENT         19  CENTURY MARK

                  COMPUTATION INTERVAL     .02 HOURS
                       TOTAL TIME BASE   16.65 HOURS

          ENGLISH UNITS
               DRAINAGE AREA         SQUARE MILES
               PRECIPITATION DEPTH   INCHES
               LENGTH, ELEVATION     FEET
               FLOW                  CUBIC FEET PER SECOND
               STORAGE VOLUME        ACRE-FEET
               SURFACE AREA          ACRES
               TEMPERATURE           DEGREES FAHRENHEIT


 *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** ***
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          *            *
   22 KK  *     DET-A  *
          *            *
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   23 KO         OUTPUT CONTROL VARIABLES
                       IPRNT          5  PRINT CONTROL
                       IPLOT          0  PLOT CONTROL
                       QSCAL         0.  HYDROGRAPH PLOT SCALE
                       IPNCH          0  PUNCH COMPUTED HYDROGRAPH
                        IOUT         21  SAVE HYDROGRAPH ON THIS UNIT
                       ISAV1          1  FIRST ORDINATE PUNCHED OR SAVED
                       ISAV2       1000  LAST ORDINATE PUNCHED OR SAVED
                      TIMINT       .017  TIME INTERVAL IN HOURS

1
                                                          RUNOFF SUMMARY
                                                  FLOW IN CUBIC FEET PER SECOND
                                               TIME IN HOURS,  AREA IN SQUARE MILES

                                            PEAK   TIME OF      AVERAGE FLOW FOR MAXIMUM PERIOD      BASIN     MAXIMUM     TIME OF
          OPERATION       STATION           FLOW     PEAK                                             AREA      STAGE     MAX STAGE
+                                                                6-HOUR     24-HOUR     72-HOUR

          HYDROGRAPH AT
+                        BasinA.h          11.    4.08             1.          0.          0.         .00

          ROUTED TO
+                           DET-A           4.    4.08             0.          0.          0.         .00
+                                                                                                              12.00       4.10


 *** NORMAL END OF HEC-1 ***
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    DET-A    11200  1JAN90 0 1 11000      .004
         .200         .200         .202         .204         .208         .212         .217         .221         .226
.230
         .234         .238         .242         .246         .250         .254         .258         .261         .265
.268
         .271         .274         .278         .281         .284         .286         .289         .292         .295
.297
         .300         .302         .305         .307         .310         .312         .314         .317         .319
.322
         .325         .328         .332         .335         .339         .343         .347         .351         .354
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         .362         .365         .368         .372         .375         .378         .381         .384         .387
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         .392         .395         .398         .400         .403         .405         .408         .410         .412
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         .625         .628         .632         .635         .639         .643         .647         .651         .656
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         .717         .723         .729         .736         .742         .748         .754         .761         .767
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1.272
        1.307        1.344        1.384        1.425        1.469        1.514        1.561        1.611        1.664
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        1.776        1.835        1.897        1.960        2.025        2.092        2.183        2.322        2.505
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MAP POCKET 1

DRAINAGE STUDY MAP FOR VISTA HANNALEI

PRE-PROJECT

DRAINAGE STUDY MAP FOR VISTA HANNALEI (PRE-PROJECT)

Date: October 14, 2020
Revised: April 14, 2023

J-19253-A

RICK ENGINEERING COMPANY
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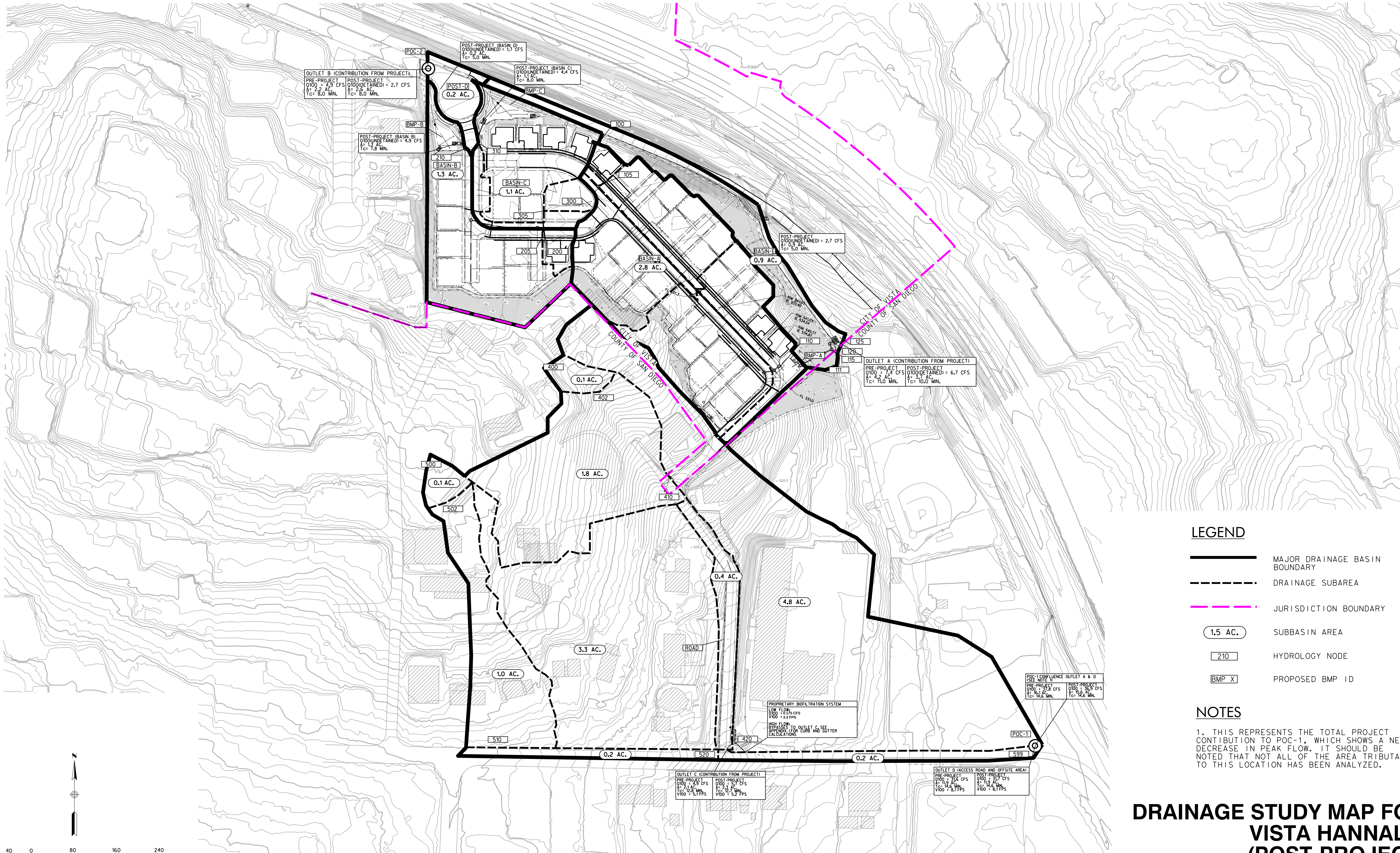
rickengineering.com
Riverside - Orange - Sacramento - San Luis Obispo - Phoenix - Tucson - Denver

J-19253-A

NOT FOR CONSTRUCTION - EXHIBIT FOR DRAINAGE STUDY ONLY

MAP POCKET 2

DRAINAGE STUDY MAP FOR VISTA HANNALEI
POST-PROJECT



LEGEND

(line)	MAJOR DRAINAGE BASIN BOUNDARY
(dashed line)	DRAINAGE SUBAREA
(dashed line)	JURISDICTION BOUNDARY
1.5 AC.	SUBBASIN AREA
210	HYDROLOGY NODE
BMP X	PROPOSED BMP ID

NOTES

1. THIS REPRESENTS THE TOTAL PROJECT CONTIBUTION TO POC-1, WHICH SHOWS A NET DECREASE IN PEAK FLOW. IT SHOULD BE NOTED THAT NOT ALL OF THE AREA TRIBUTARY TO THIS LOCATION HAS BEEN ANALYZED.

80 40 0 80 160 240

GRAPHIC SCALE 1" = 80'

DRAINAGE STUDY MAP FOR VISTA HANNALEI (POST-PROJECT)

Date: October 14, 2020
Revised: April 14, 2023

J-19253-A

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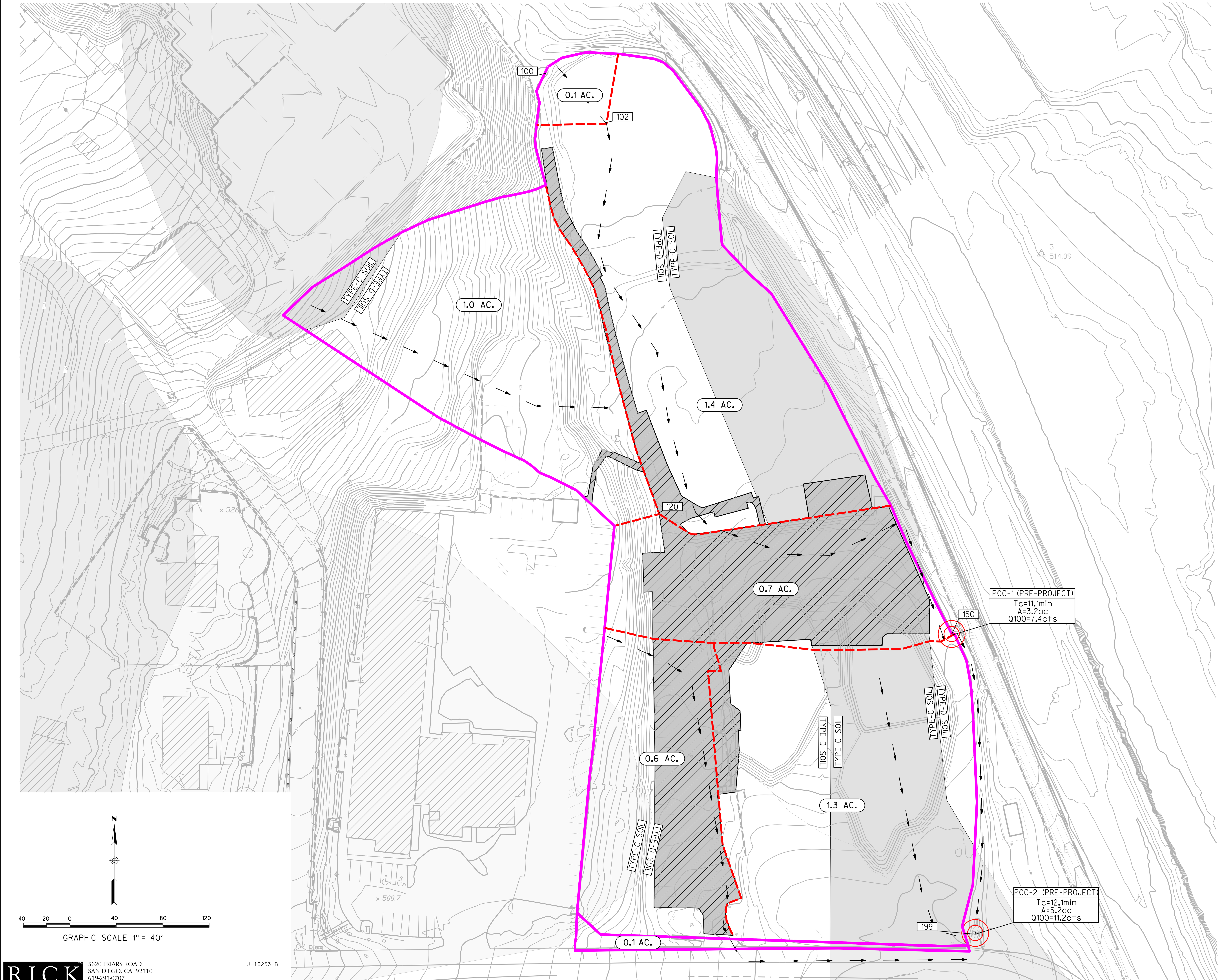
J-19253-A

NOT FOR CONSTRUCTION - EXHIBIT FOR DRAINAGE STUDY ONLY

MAP POCKET 1

DRAINAGE STUDY MAP FOR VISTA II BALLFIELDS
PRE-PROJECT



LEGEND

(magenta line)	MAJOR DRAINAGE BASIN BOUNDARY
(red dashed line)	DRAINAGE SUBAREA
X.X AC.	SUBBASIN AREA
XXX	HYDROLOGY NODE
(hatched)	EXISTING IMPERVIOUS AREA
(circles)	POINT OF COMPLIANCE (POC)
(shaded)	TYPE 'C' HYDROLOGIC SOIL GROUP

DRAINAGE STUDY MAP FOR VISTA II BALLFIELDS (PRE-PROJECT)

Date: April 7, 2023
Revised: October 13, 2023
Revised: February 14, 2024

J-19253-B

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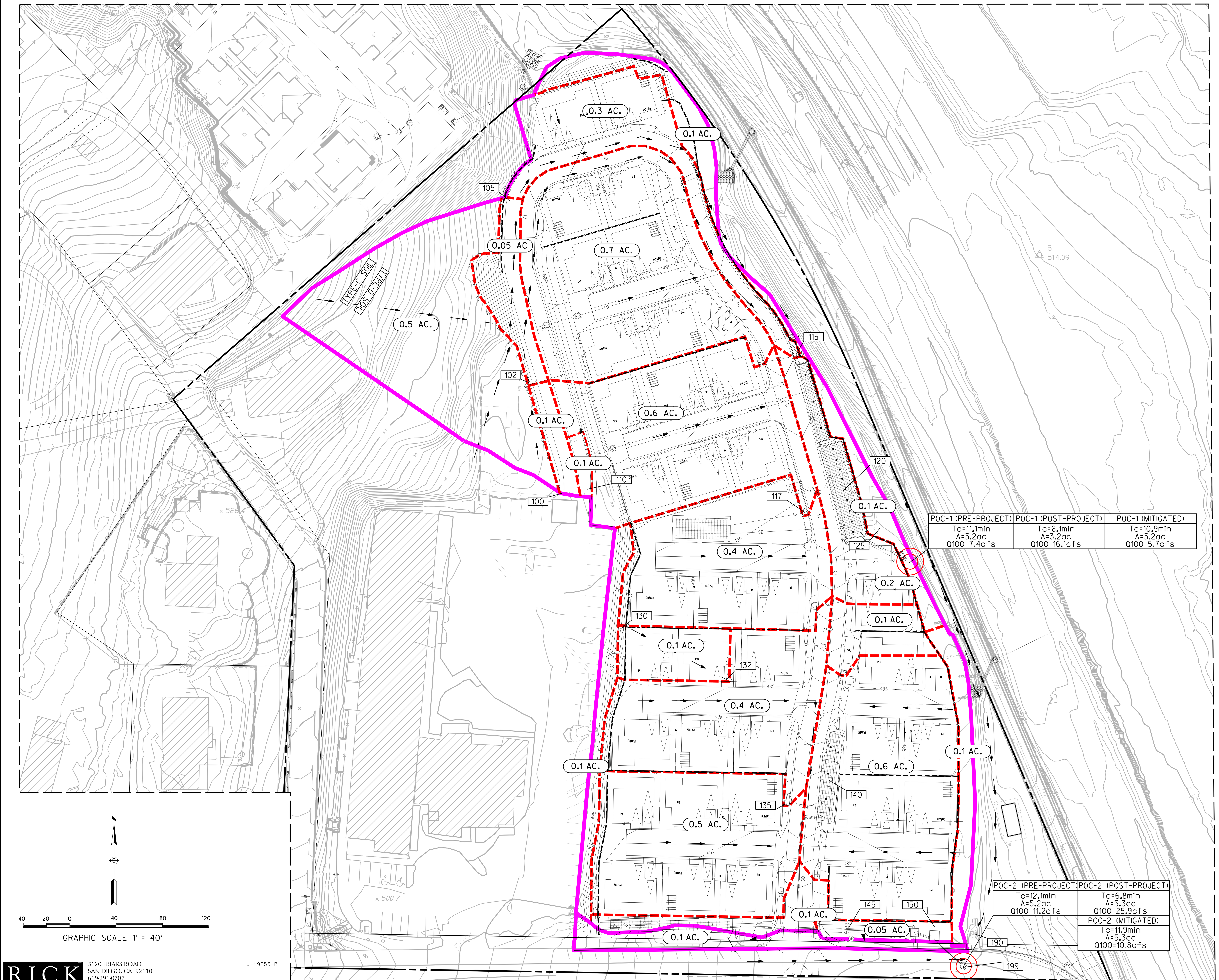
J-19253-B

NOT FOR CONSTRUCTION - EXHIBIT FOR DRAINAGE STUDY ONLY

MAP POCKET 2

DRAINAGE STUDY MAP FOR VISTA II BALLFIELDS
POST-PROJECT



LEGEND

- MAJOR DRAINAGE BASIN BOUNDARY
- DRAINAGE SUBAREA
- X.X AC. — SUBBASIN AREA
- XXX — HYDROLOGY NODE
- POINT OF COMPLIANCE (POC)

DRAINAGE STUDY MAP FOR VISTA II BALLFIELDS (POST-PROJECT)

Date: April 7, 2023
Revised: October 13, 2023
Revised: February 27, 2024

J-19253-B

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