TM DRAINAGE STUDY

For

BLUE WAVE

Prepared for**:**

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Report Prepared:

August 24, 2018

Revised:

November 14, 2018

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CHAPTER 1 - EXECUTIVE SUMMARY

1.1 – Introduction

The Blue Wave project site is located to the north of Highway 75 in the City of Imperial Beach, California.

Runoff drains via overland flow in a north westerly direction towards the adjacent development north of the project site, ultimately draining to the salt ponds/San Diego Bay to the north of the project site.

This study analyzes existing and developed condition 100-year peak flowrates from the development to the existing point of discharge.

The project site lies outside any FEMA 100-year floodplain zones. Therefore, no Letters of Map Revision will be required.

Treatment of storm water runoff from the site has been addressed in a separate report - the “Storm Water Quality Management Plan for Blue Wave”, dated November 2018 by REC.

Per County of San Diego drainage criteria, the Modified Rational Method should be used to determine peak design flowrates when the contributing drainage area is less than 1.0 square mile. Since the total watershed area discharging from the site is less than 1.0 square mile, AES computer software was used to model the pre & post developed condition runoff response per the Modified Rational Method.

Methodology used for the computation of design rainfall events, runoff coefficients, and rainfall intensity values are consistent with criteria set forth in the “County of San Diego Drainage Design Manual”. A more detailed explanation of methodology used for this analysis is listed in Chapter 2 of this report.

Developed condition peak flows were calculated using AES. The corresponding hydrographs were generated using the Rat Hydro program by Rick Engineering. Hydraulic Modified-Puls detention basin routing of the AES rational method hydrology was performed using the Army Corps of Engineers HEC-HMS 4.2.1 software.

1.2 – Summary of Existing Conditions

In existing conditions, the Blue Wave project site is currently a vacant lot inclusive of light vegetation with a smaller developed single family residence located to the east of the main vacant lot. Runoff drains via overland flow in a westerly direction towards the adjacent development north of the project site, ultimately draining to the salt ponds/San Diego Bay to the north of the project site.

Per County of San Diego rainfall isopluvial maps, the design 100-year rainfall depth for the project site is 2.4 inches. The project site comprises of hydrologic soil class D soils such that runoff coefficients of 0.35 and 0.52 were used for the vegetated site and single family residence respectively.

Table 1 below summarizes the existing condition design 100-year peak flow from the project site.

Table 1 – SUMMARY OF EXISTING CONDITIONS FLOWS

Discharge Location	Drainage Area (Ac)	Runoff Coefficient (C)	100-Year Peak Flow (cfs)
San Diego Bay	1.31	0.35/0.52[(1)]	1.7

Notes: (1) Initial flow subarea is at the single family residence (0.52), the remainder of the catchment assumes 0.35.

1.3 – Summary of Developed Conditions

The Blue Wave project proposes the construction of a commercial and residential development inclusive of open space, sidewalks and recreational areas. Access to the site will be via an entrance from the adjacent Highway 75 to the south of the project site.

Runoff from the project is drained to a receiving peak flow detention basin located to the northwest corner of the project site. Mitigated peak flows are then drained from the detention facility by a storm drain to connect to the existing storm drain located within the adjacent Highway 75 to the south of the project site. This storm drain discharges to the salt ponds/San Diego Bay.

Runoff draining towards the underground parking lot is intercepted by a trench drain and then pumped to the aforementioned detention basin.

Per County of San Diego rainfall isopluvial maps, the design 100-year rainfall depth for the project site is 2.4 inches. The project site comprises of hydrologic soil class D soils such that a runoff coefficient of 0.85 was used for the development. Table 2 summarizes the unmitigated developed condition design 100-year peak flow from the project site.

Table 2 – SUMMARY OF UNMITIGATED - DEVELOPED CONDITIONS FLOWS

Discharge Location	Drainage Area (Ac)	Runoff Coefficient (C)	100-Year Peak Flow (cfs)
San Diego Bay	1.33	0.85	5.33

Prior to discharging from the project site, first flush runoff will be treated via onsite bio-filtration BMPs in accordance with standards set forth by the Regional Water Quality Control Board and the City of Imperial Beach's BMP Design Manual (see "Storm Water Quality Management Plan for Blue Wave").

The tributary area to the point of discharge from the project site increases by approximately 0.02 Ac due to improvements of the site entranceway.

Runoff from the developed site drains to a single underground peak flow detention vault. Peak flows are mitigated via this facility prior to discharging to the existing point of discharge from the project site. A summary of the detention basin is provided in Table 3 and the corresponding outlet structure is summarized in Table 4.

Table 3 – SUMMARY OF DETENTION BASIN DIMENSIONS

Basin	DIMENSIONS			
	Vault Area (ft^2)	Vault Depth (ft)	Vault Volume (ft^3)	Emergency Spillway Depth (ft)
Basin 1	520	6.0	3,120	5.5

Table 4 – SUMMARY OF DETENTION BASIN OUTLETS

Basin	Lower Slot			Emergency Weir	
	Width (in)	Height (in)	Elev (ft)	Width (ft)	Elev (ft)
1	10.8	2.0	0.00	5.0	5.5

The developed condition peak flows were calculated using the modified rational method. The corresponding hydrographs were generated using the Rat Hydro program by Rick Engineering. This hydrograph was then routed through the proposed on-site detention facility in HEC-HMS. The HMS Modified-Puls results are summarized in Table 5.

Rational method hydrographs, stage-storage, stage-discharge relationships, outlet structure configurations and HEC-HMS model output is provided in Chapter 5 of this report. Table 5 summarizes the peak inflow and discharge from the detention facility.

Table 5 – SUMMARY OF DETENTION BASIN ROUTING

Detention Basin	100-Year Peak Inflow (cfs)	100-Year Peak Outflow (cfs)	Peak Water Surface Elevation[1] (ft)
Basin 1	5.33	1.6	5.0

Notes: (1) Elevation above the basin invert.

1.4 – Summary of Results

Table 6 summarizes developed and existing condition drainage areas and resultant 100-year peak flow rates at the receiving discharge location from the Blue Wave site. Per County of San Diego rainfall isopluvial maps, the design 100-year rainfall depth for the site area is 2.4 inches.

Table 6 – SUMMARY OF PEAK FLOWS

Discharge Location	Area (ac)			100 Year Peak Flow (cfs)		
	Existing	Developed	Difference	Existing	Developed	Difference
San Diego Bay	1.31	1.33	+ 0.02	1.7	1.6	- 0.1

As illustrated in Table 6, the proposed Blue Wave project site will reduce peak flows at the point of discharge from the project site by 0.1 cfs compared to flows experienced in the existing condition, as such there is no increase in peak flow due to the development of the project site.

All developed runoff will receive water quality treatment in accordance with the site specific SWQMP.

Final design details will be provided at the final engineering phase of the development.

1.5 – Conclusions

This report has been prepared in accordance with the County of San Diego Hydrology Manual. This report has evaluated and addressed the potential impacts and proposed mitigation measures. A summary of the facts and findings associated with this project and the measures addressed by this report is as follows:

- The project will not alter drainage patterns on the site or increase runoff after development.
- The ultimate discharge points will not be changed.
- Graded areas and slopes will be hydroseeded to reduce or eliminate sediment discharge.
- Identify and discuss, with appropriate backup/research information, the following question item by item for CEQA purposes. Would the project:

A. Substantially alter the existing drainage patterns of the site or area, including through the alteration if the course of a stream or river, in a manner which would result in substantial erosion or siltation on – or off-site?

The project does not substantially alter the existing drainage pattern of the area and does not alter the course of a stream or river.

The storm drain system for the entire project is designed to route and convey all resulting runoff from developed conditions to existing point of discharge.

B. Substantially alter the existing drainage patterns of the site or area, including through the 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?

The project will not substantially alter the existing drainage pattern of the area as it will not alter the course of a stream or river, and also will not substantially increase the rate or amount of surface runoff in a manner which would result in on- or off-site flooding.

C. Create or contribute runoff water which would exceed the capacity of existing or planned storm water drainage systems?

No. All project discharge points release water at rates less than or equal to existing conditions.

D. Place housing within a 100-year flood hazard area as mapped on a federal Flood Hazard Boundary or Flood insurance Rate Map or other flood hazard delineation map, including County Floodplain Maps? For example; research the foregoing and provide same (to indicate applicability or not) in the study?

The project does not place any housing within a 100-year flood hazard area.

E. Place within a 100-year flood hazard area structures which would impede or redirect flood flows?

There are no structures proposed within a 100-year flood hazard area.

F. Expose people or structures to a significant risk of loss, injury or death involving flooding, including flooding as a result of the failure of a levee or dam on-sit or off-site?

NA

1.6 – References

"*County of San Diego Hydrology Manual*", June 2003

"*San Diego County Hydraulic Design Manual*", September 2014

"*Stormwater Quality Management Plan for Blue Wave*", November 2018, REC Consultants.

1.7 – Declaration of Responsible Charge

THIS PRELIMINARY DRAINAGE STUDY HAS BEEN PREPARED UNDER THE DIRECTION OF THE FOLLOWING REGISTERED CIVIL ENGINEER. THE REGISTERED ENGINEER ATTESTS TO THE TECHNICAL INFORMATION CONTAINED HEREIN AND THE ENGINEERING DATA UPON WHICH RECOMMENDATIONS, CONCLUSIONS, AND DECISIONS ARE BASED.

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CHAPTER 2 - METHODOLOGY

2.1 – County of San Diego Design Criteria

SECTION 3
RATIONAL METHOD AND MODIFIED RATIONAL METHOD

3.1 THE RATIONAL METHOD

The Rational Method (RM) is a mathematical formula used to determine the maximum runoff rate from a given rainfall. It has particular application in urban storm drainage, where it is used to estimate peak runoff rates from small urban and rural watersheds for the design of storm drains and small drainage structures. The RM is recommended for analyzing the runoff response from drainage areas up to approximately 1 square mile in size. It should not be used in instances where there is a junction of independent drainage systems or for drainage areas greater than approximately 1 square mile in size. In these instances, the Modified Rational Method (MRM) should be used for junctions of independent drainage systems in watersheds up to approximately 1 square mile in size (see Section 3.4); or the NRCS Hydrologic Method should be used for watersheds greater than approximately 1 square mile in size (see Section 4).

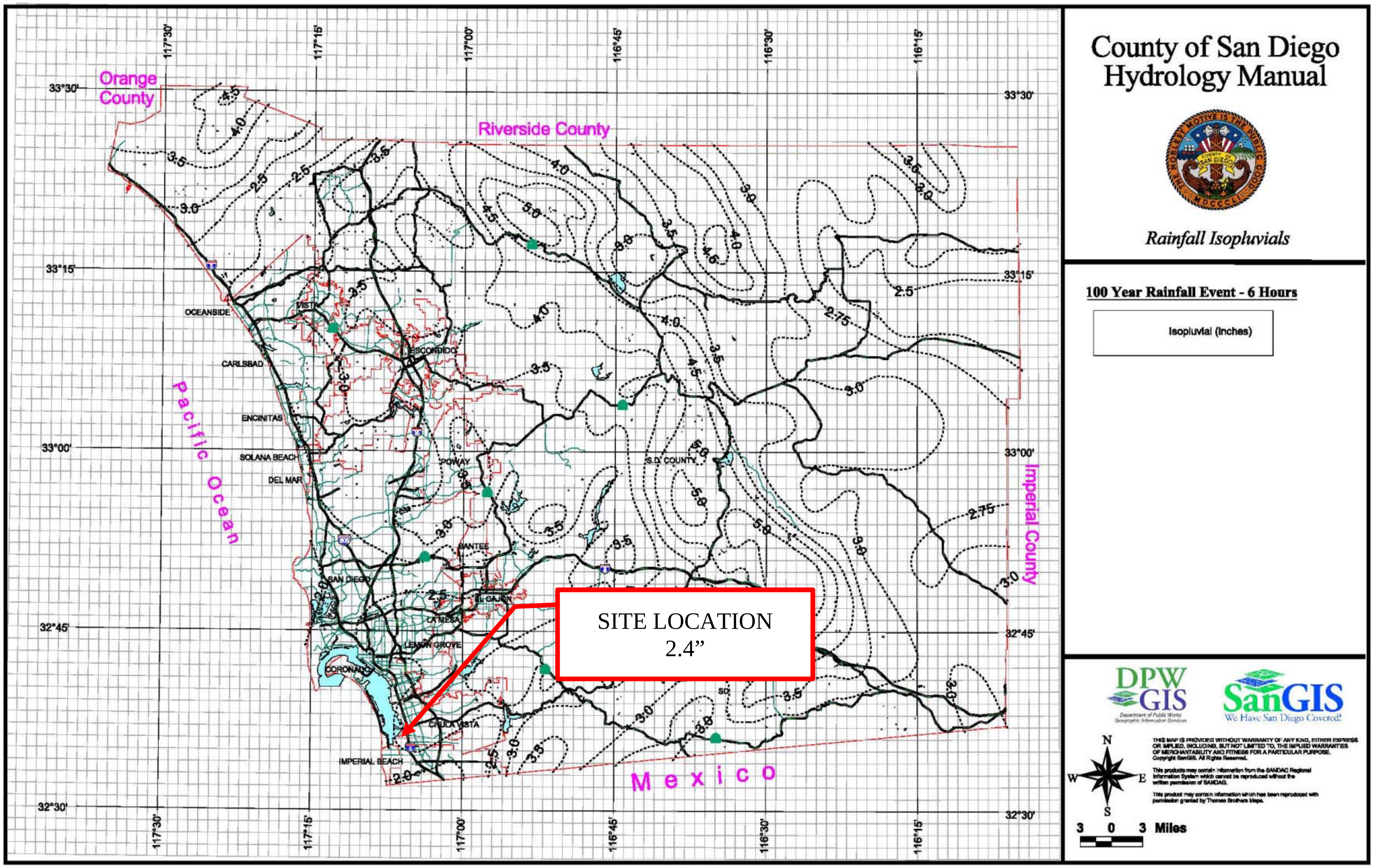
The RM can be applied using any design storm frequency (e.g., 100-year, 50-year, 10-year, etc.). The local agency determines the design storm frequency that must be used based on the type of project and specific local requirements. A discussion of design storm frequency is provided in Section 2.3 of this manual. A procedure has been developed that converts the 6-hour and 24-hour precipitation isopluvial map data to an Intensity-Duration curve that can be used for the rainfall intensity in the RM formula as shown in Figure 3-1. The RM is applicable to a 6-hour storm duration because the procedure uses Intensity-Duration Design Charts that are based on a 6-hour storm duration.

3.1.1 Rational Method Formula

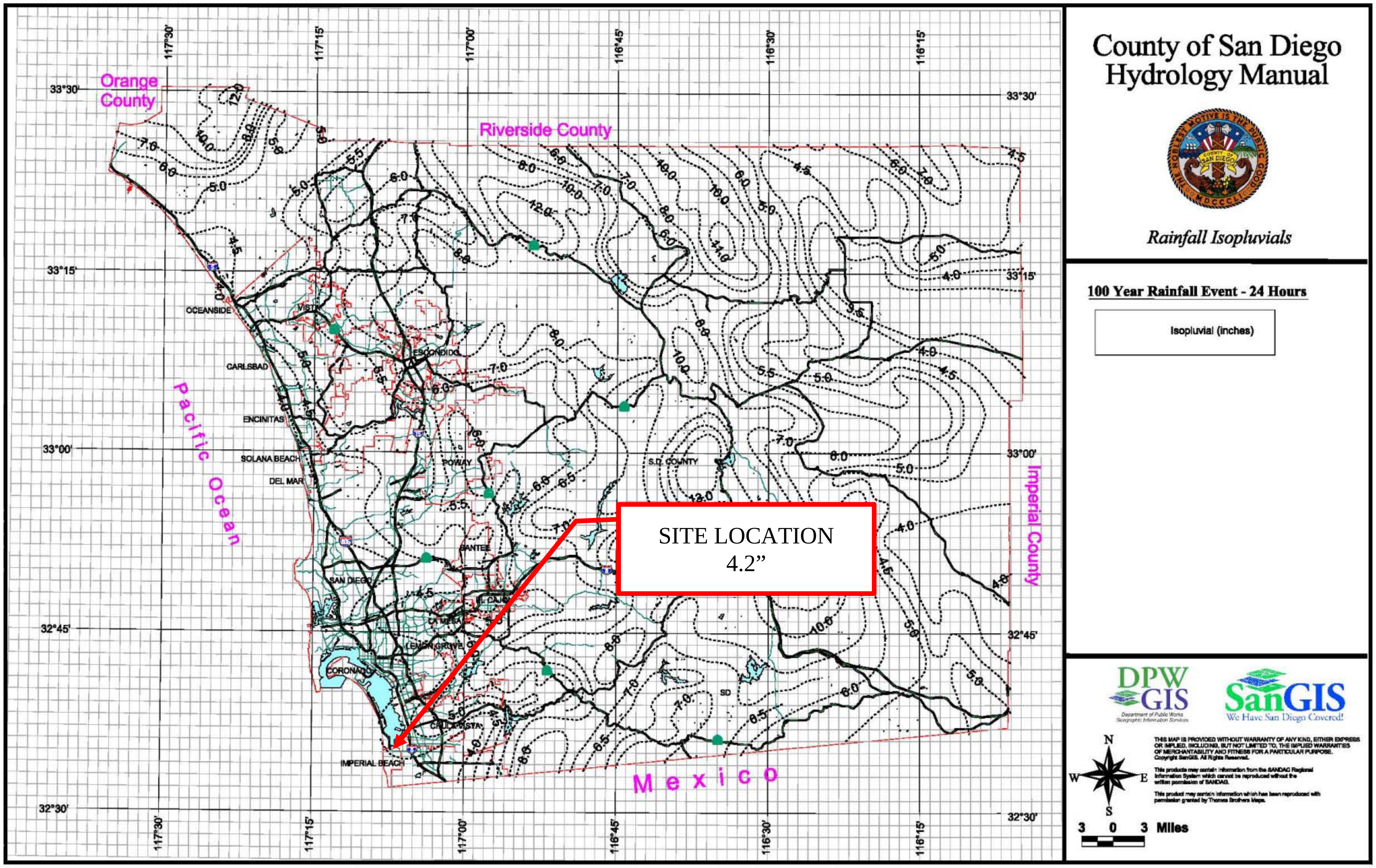
The RM formula estimates the peak rate of runoff at any location in a watershed as a function of the drainage area (A), runoff coefficient (C), and rainfall intensity (I) for a duration equal to the time of concentration (T_c), which is the time required for water to

2.2 – Design Rainfall Determination

2.2.1 – 100-Year, 6-Hour Rainfall Isopluvial Map



2.2.2 – 100-Year, 24-Hour Rainfall Isopluvial Map



2.3 – Runoff Coefficient Determination

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

- The storm frequency of peak discharges is the same as that of I for the given T_c.
- The fraction of rainfall that becomes runoff (or the runoff coefficient, C) is independent of I or precipitation zone number (PZN) condition (PZN Condition is discussed in Section 4.1.2.4).
- The peak rate of runoff is the only information produced by using the RM.

3.1.2 Runoff Coefficient

Table 3-1 lists the estimated runoff coefficients for urban areas. The concepts related to the runoff coefficient were evaluated in a report entitled *Evaluation, Rational Method "C" Values* (Hill, 2002) that was reviewed by the Hydrology Manual Committee. The Report is available at San Diego County Department of Public Works, Flood Control Section and on the San Diego County Department of Public Works web page.

The runoff coefficients are based on land use and soil type. Soil type can be determined from the soil type map provided in Appendix A. An appropriate runoff coefficient (C) for each type of land use in the subarea should be selected from this table and multiplied by the percentage of the total area (A) included in that class. The sum of the products for all land uses is the weighted runoff coefficient (Σ[CA]). Good engineering judgment should be used when applying the values presented in Table 3-1, as adjustments to these values may be appropriate based on site-specific characteristics. In any event, the impervious percentage (% Impervious) as given in the table, for any area, shall govern the selected value for C. The runoff coefficient can also be calculated for an area based on soil type and impervious percentage using the following formula:

$$C = 0.90 \times (\% \text{ Impervious}) + C_p \times (1 - \% \text{ Impervious})$$

Where: C_p = Pervious Coefficient Runoff Value for the soil type (shown in Table 3-1 as Undisturbed Natural Terrain/Permanent Open Space, 0% Impervious). Soil type can be determined from the soil type map provided in Appendix A.

The values in Table 3-1 are typical for most urban areas. However, if the basin contains rural or agricultural land use, parks, golf courses, or other types of nonurban land use that are expected to be permanent, the appropriate value should be selected based upon the soil and cover and approved by the local agency.

2.4 – Urban Watershed Overland Time of Flow Nomograph

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.

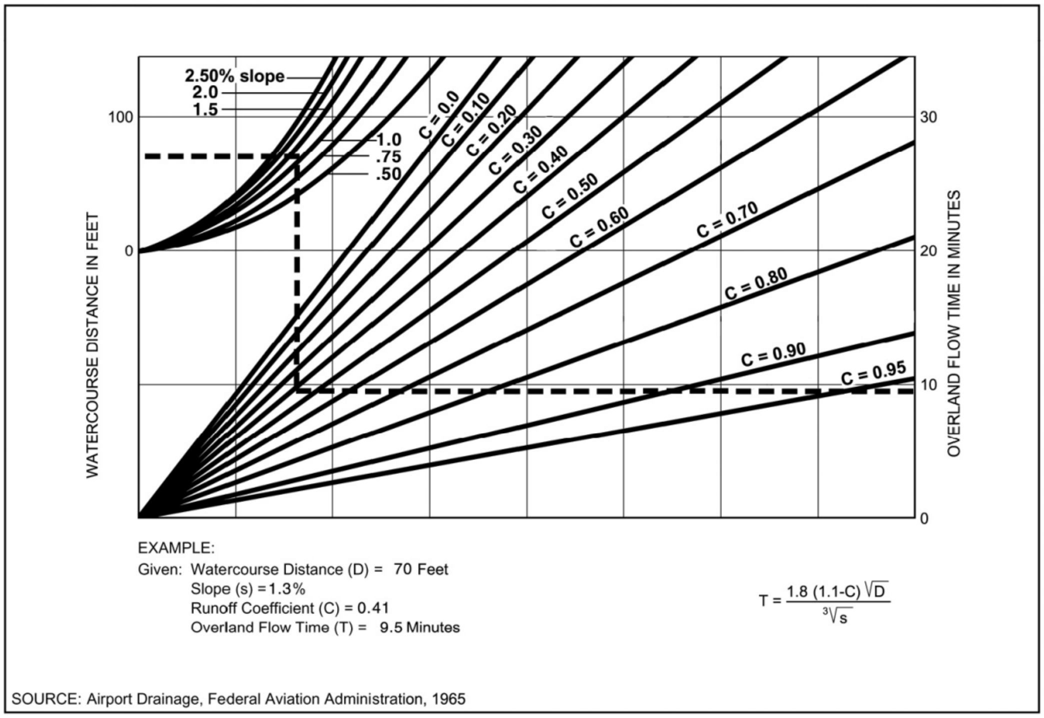
Table 3-2 provides limits of the length (Maximum Length (L_M)) of sheet flow to be used in hydrology studies. Initial T_i values based on average C values for the Land Use Element are also included. These values can be used in planning and design applications as described below. Exceptions may be approved 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/	.5%		1%		2%		3%		5%		10%	
	Acre	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

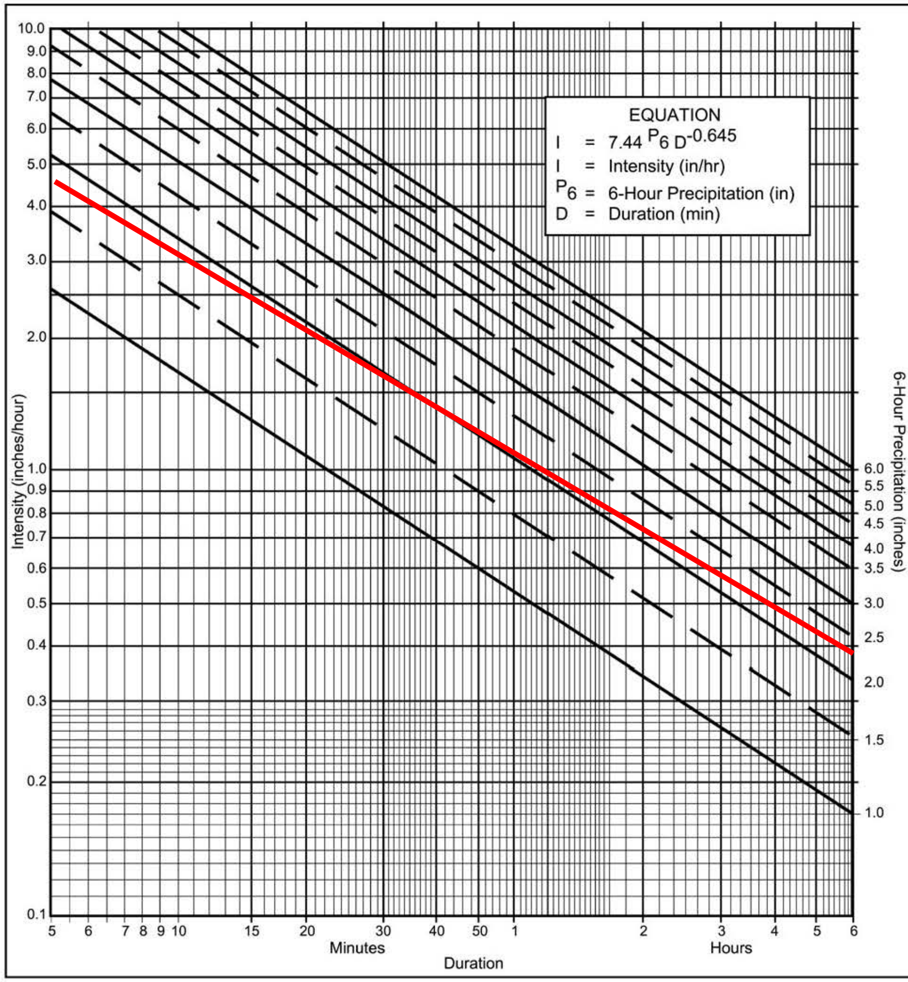
*See Table 3-1 for more detailed description



Rational Formula - Overland Time of Flow Nomograph

FIGURE **3-3**

2.5 – County of San Diego Intensity- Duration Curve



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 applicaple 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 100 year

(b) P_6 = 2.4 in., P_{24} = 4.2, $\frac{P_6}{P_{24}}$ = 57 %(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**

2.6 – Model Development Summary (from County of San Diego Hydrology Manual)

3.2 DEVELOPING INPUT DATA FOR THE RATIONAL METHOD

This section describes the development of the necessary data to perform RM calculations. Section 3.3 describes the RM calculation process. Input data for calculating peak flows and T_c's with the RM should be developed as follows:

1. On a topographic base map, outline the overall drainage area boundary, showing adjacent drains, existing and proposed drains, and overland flow paths.

2. Verify the accuracy of the drainage map in the field.

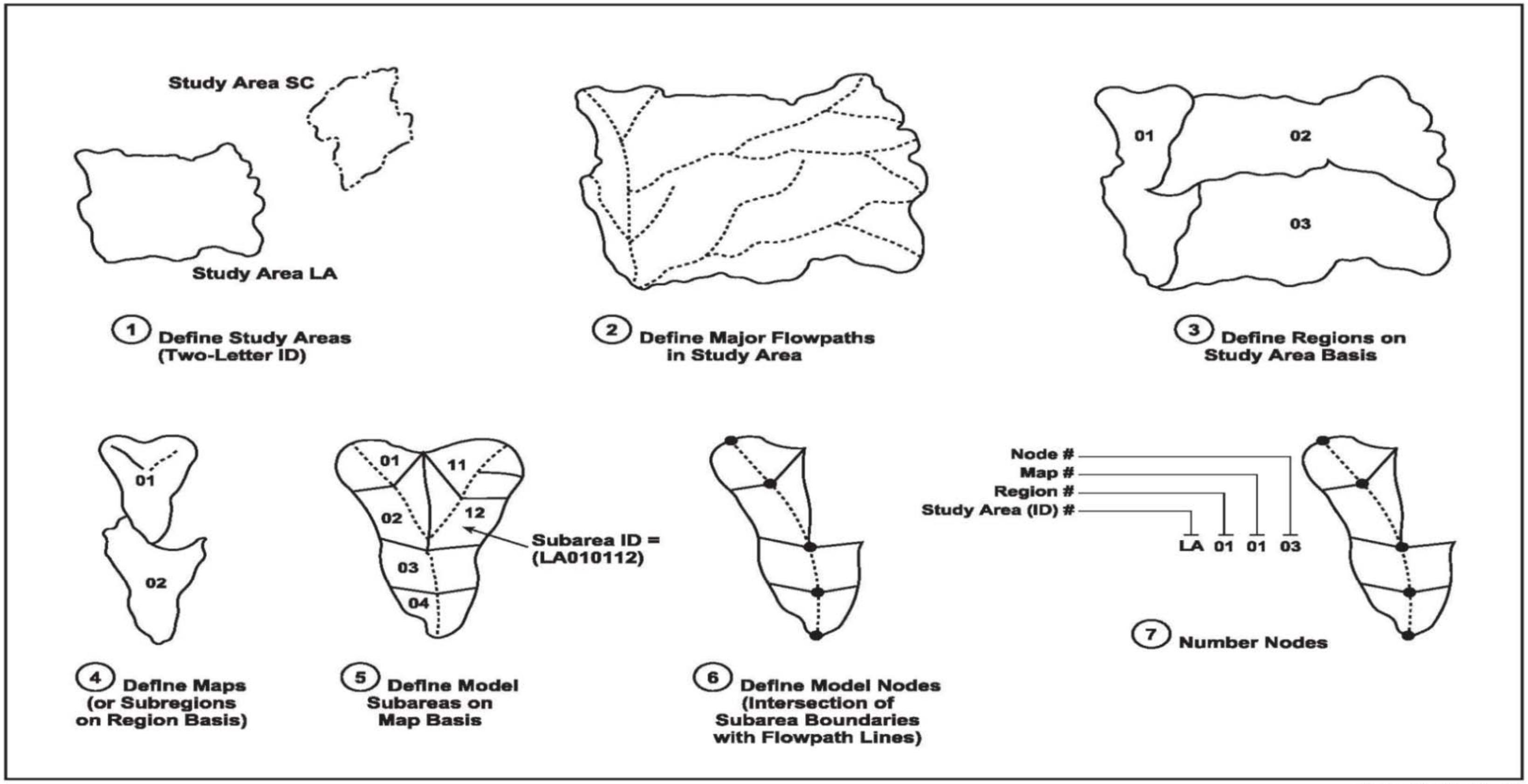
3. Divide the drainage area into subareas by locating significant points of interest. These divisions should be based on topography, soil type, and land use. Ensure that an appropriate first subarea is delineated. For natural areas, the first subarea flow path length should be less than or equal to 4,000 feet plus the overland flow length (Table 3-2). For developed areas, the initial subarea flow path length should be consistent with Table 3-2. The topography and slope within the initial subarea should be generally uniform.

4. Working from upstream to downstream, assign a number representing each subarea in the drainage system to each point of interest. Figure 3-8 provides guidelines for node numbers for geographic information system (GIS)-based studies.

5. Measure each subarea in the drainage area to determine its size in acres (A).

6. Determine the length and effective slope of the flow path in each subarea.

7. Identify the soil type for each subarea.



GIS/Hydrologic Model
Data Base Linkage Setup:
Nodes, Subareas, Links

FIGURE
3-8

8. Determine the runoff coefficient (C) for each subarea based on Table 3-1. If the subarea contains more than one type of development classification, use a proportionate average for C. In determining C for the subarea, use future land use taken from the applicable community plan, Multiple Species Conservation Plan, National Forest land use plan, etc.

9. Calculate the CA value for the subarea.

10. Calculate the Σ(CA) value(s) for the subareas upstream of the point(s) of interest.

11. Determine P_6 and P_{24} for the study using the isopluvial maps provided in Appendix B. If necessary, adjust the value for P_6 to be within 45% to 65% of the value for P_{24}.

See Section 3.3 for a description of the RM calculation process.

3.3 Performing Rational Method Calculations

This section describes the RM calculation process. Using the input data, calculation of peak flows and T_c's should be performed as follows:

1. Determine T_i for the first subarea. Use Table 3-2 or Figure 3-3 as discussed in Section 3.1.4. If the watershed is natural, the travel time to the downstream end of the first subarea can be added to T_i to obtain the T_c. Refer to paragraph 3.1.4.2 (a).

2. Determine I for the subarea using Figure 3-1. If T_i was less than 5 minutes, use the 5 minute time to determine intensity for calculating the flow.

3. Calculate the peak discharge flow rate for the subarea, where $Q_p = \Sigma(CA)\ I$.
   In case that the downstream flow rate is less than the upstream flow rate, due to the long travel time that is not offset by the additional subarea runoff, use the upstream peak flow for design purposes until downstream flows increase again.

4. Estimate the T_t to the next point of interest.

5. Add the T_t to the previous T_c to obtain a new T_c.

6. Continue with step 2, above, until the final point of interest is reached.

Note: The MRM should be used to calculate the peak discharge when there is a junction from independent subareas into the drainage system.

3.4 Modified Rational Method (for Junction Analysis)

The purpose of this section is to describe the steps necessary to develop a hydrology report for a small watershed using the MRM. It is necessary to use the MRM if the watershed contains junctions of independent drainage systems. The process is based on the design manuals of the City/County of San Diego. The general process description for using this method, including an example of the application of this method, is described below.

The engineer should only use the MRM for drainage areas up to approximately 1 square mile in size. If the watershed will significantly exceed 1 square mile then the NRCS method described in Section 4 should be used. The engineer may choose to use either the RM or the MRM for calculations for up to an approximately 1-square-mile area and then transition the study to the NRCS method for additional downstream areas that exceed approximately 1 square mile. The transition process is described in Section 4.

3.4.1 Modified Rational Method General Process Description

The general process for the MRM differs from the RM only when a junction of independent drainage systems is reached. The peak Q, T_c, and I for each of the independent drainage systems at the point of the junction are calculated by the RM. The independent drainage systems are then combined using the MRM procedure described below. The peak Q, T_c, and I for each of the independent drainage systems at the point of the junction must be calculated prior to using the MRM procedure to combine the independent drainage systems, as these values will be used for the MRM calculations. After the independent drainage systems have been combined, RM calculations are continued to the next point of interest.

3.4.2 Procedure for Combining Independent Drainage Systems at a Junction

Calculate the peak Q, T_c, and I for each of the independent drainage systems at the point of the junction. These values will be used for the MRM calculations.

At the junction of two or more independent drainage systems, the respective peak flows are combined to obtain the maximum flow out of the junction at T_c. Based on the approximation that total runoff increases directly in proportion to time, a general equation may be written to determine the maximum Q and its corresponding T_c using the peak Q, T_c, and I for each of the independent drainage systems at the point immediately before the junction. The general equation requires that contributing Q's be numbered in order of increasing T_c.

Let Q_1, T_1, and I_1 correspond to the tributary area with the shortest T_c. Likewise, let Q_2, T_2, and I_2 correspond to the tributary area with the next longer T_c; Q_3, T_3, and I_3 correspond to the tributary area with the next longer T_c; and so on. When only two independent drainage systems are combined, leave Q_3, T_3, and I_3 out of the equation. Combine the independent drainage systems using the junction equation below:

Junction Equation: $\mathbf{T_1 < T_2 < T_3}$

$$Q_{T1} = Q_1 + \frac{T_1}{T_2} Q_2 + \frac{T_1}{T_3} Q_3$$

$$Q_{T2} = Q_2 + \frac{I_2}{I_1} Q_1 + \frac{T_2}{T_3} Q_3$$

$$Q_{T3} = Q_3 + \frac{I_3}{I_1} Q_1 + \frac{I_3}{I_2} Q_2$$

Calculate Q_{T1}, Q_{T2}, and Q_{T3}. Select the largest Q and use the T_c associated with that Q for further calculations (see the three Notes for options). If the largest calculated Q's are equal (e.g., $Q_{T1} = Q_{T2} > Q_{T3}$), use the shorter of the T_c's associated with that Q.

This equation may be expanded for a junction of more than three independent drainage systems using the same concept. The concept is that when Q from a selected subarea (e.g., Q_2) is combined with Q from another subarea with a shorter T_c (e.g., Q_1), the Q from the subarea with the shorter T_c is reduced by the ratio of the I's (I_2/I_1); and when Q from a selected subarea (e.g., Q_2) is combined with Q from another subarea with a longer T_c (e.g., Q_3), the Q from the subarea with the longer T_c is reduced by the ratio of the T_c's (T_2/T_3).

<u>Note #1</u>: At a junction of two independent drainage systems that have the same T_c, the tributary flows may be added to obtain the Q_p.

$$Q_p = Q_1 + Q_2\text{; when } T_1 = T_2\text{; and } T_c = T_1 = T_2$$

This can be verified by using the junction equation above. Let Q_3, T_3, and $I_3 = 0$. When T_1 and T_2 are the same, I_1 and I_2 are also the same, and T_1/T_2 and $I_2/I_1 = 1$. T_1/T_2 and I_2/I_1 are cancelled from the equations. At this point, $Q_{T1} = Q_{T2} = Q_1 + Q_2$.

<u>Note #2</u>: In the upstream part of a watershed, a conservative computation is acceptable. When the times of concentration (T_c's) are relatively close in magnitude (within 10%), use the shorter T_c for the intensity and the equation $Q = \Sigma(CA)I$.

<u>Note #3</u>: . An optional method of determining the T_c is to use the equation
$T_c = [(\Sigma\ (CA)7.44\ P_6)/Q]^{1.55}$

This equation is from $Q = \Sigma(CA)I = \Sigma(CA)(7.44\ P_6/T_c^{.645})$ and solving for T_c. The advantage in this option is that the T_c is consistent with the peak flow Q, and avoids inappropriate fluctuation in downstream flows in some cases.

CHAPTER 3 - 100 YEAR HYDROLOGIC ANALYSIS FOR EXISTING CONDITIONS

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

            RATIONAL METHOD HYDROLOGY COMPUTER PROGRAM PACKAGE
            Reference: SAN DIEGO COUNTY FLOOD CONTROL DISTRICT
                         2003,1985,1981 HYDROLOGY MANUAL
         (c) Copyright 1982-2015 Advanced Engineering Software (aes)
             Ver. 22.0 Release Date: 07/01/2015  License ID 1643

                          Analysis prepared by:




 ************************** DESCRIPTION OF STUDY **************************
* 100 YEAR PRE DEVELOPED CONDITION HYDROLOGIC ANALYSIS FOR                 *
* BLUE WAVE, IMPERIAL BEACH, CA                                            *
*                                                                          *
 **************************************************************************

  FILE NAME: PRE100.DAT
  TIME/DATE OF STUDY: 08:33 08/23/2018
----------------------------------------------------------------------------
  USER SPECIFIED HYDROLOGY AND HYDRAULIC MODEL INFORMATION:
----------------------------------------------------------------------------
  2003 SAN DIEGO MANUAL CRITERIA

  USER SPECIFIED STORM EVENT(YEAR) = 100.00
  6-HOUR DURATION PRECIPITATION (INCHES) =   2.400
  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 0.0312 0.167 0.0150

  GLOBAL STREET FLOW-DEPTH CONSTRAINTS:
    1. Relative Flow-Depth =  0.00 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      1.00 TO NODE      2.00 IS CODE =  21
----------------------------------------------------------------------------
  >>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<
============================================================================
  *USER SPECIFIED(SUBAREA):
  RESIDENTIAL (4.3 DU/AC OR LESS) RUNOFF COEFFICIENT = .5200
  S.C.S. CURVE NUMBER (AMC II) =   0
  INITIAL SUBAREA FLOW-LENGTH(FEET) =    70.00
  UPSTREAM ELEVATION(FEET) =     22.50
  DOWNSTREAM ELEVATION(FEET) =     21.60
  ELEVATION DIFFERENCE(FEET) =      0.90
  SUBAREA OVERLAND TIME OF FLOW(MIN.) =    8.033
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  4.657
  SUBAREA RUNOFF(CFS) =      0.19
  TOTAL AREA(ACRES) =      0.08   TOTAL RUNOFF(CFS) =      0.19

****************************************************************************
  FLOW PROCESS FROM NODE      2.00 TO NODE      3.00 IS CODE =  51
----------------------------------------------------------------------------
  >>>>>COMPUTE TRAPEZOIDAL CHANNEL FLOW<<<<<
  >>>>>TRAVELTIME THRU SUBAREA (EXISTING ELEMENT)<<<<<
```

```
==============================================================================
  ELEVATION DATA: UPSTREAM(FEET) =     21.60  DOWNSTREAM(FEET) =     15.50
  CHANNEL LENGTH THRU SUBAREA(FEET) =   424.00   CHANNEL SLOPE =  0.0144
  CHANNEL BASE(FEET) =    4.00   "Z" FACTOR =   2.000
  MANNING'S FACTOR = 0.030   MAXIMUM DEPTH(FEET) =   1.00
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  3.481
  *USER SPECIFIED(SUBAREA):
  ANNUAL GRASS (DRYLAND) POOR COVER RUNOFF COEFFICIENT = .3500
  S.C.S. CURVE NUMBER (AMC II) =   0
  TRAVEL TIME COMPUTED USING ESTIMATED FLOW(CFS) =       0.98
  TRAVEL TIME THRU SUBAREA BASED ON VELOCITY(FEET/SEC.) =   1.54
  AVERAGE FLOW DEPTH(FEET) =   0.15   TRAVEL TIME(MIN.) =   4.58
  Tc(MIN.) =   12.61
  SUBAREA AREA(ACRES) =     1.23       SUBAREA RUNOFF(CFS) =    1.50
  AREA-AVERAGE RUNOFF COEFFICIENT =  0.360
  TOTAL AREA(ACRES) =        1.3         PEAK FLOW RATE(CFS) =       1.65

  END OF SUBAREA CHANNEL FLOW HYDRAULICS:
  DEPTH(FEET) =  0.20   FLOW VELOCITY(FEET/SEC.) =   1.86
  LONGEST FLOWPATH FROM NODE      1.00 TO NODE      3.00 =     494.00 FEET.
==============================================================================
  END OF STUDY SUMMARY:
  TOTAL AREA(ACRES)     =        1.3  TC(MIN.) =     12.61
  PEAK FLOW RATE(CFS)   =       1.65
==============================================================================
==============================================================================
  END OF RATIONAL METHOD ANALYSIS
```

CHAPTER 4 - 100 YEAR HYDROLOGIC ANALYSIS FOR DEVELOPED CONDITIONS - UNMITIGATED

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             RATIONAL METHOD HYDROLOGY COMPUTER PROGRAM PACKAGE
             Reference: SAN DIEGO COUNTY FLOOD CONTROL DISTRICT
                          2003,1985,1981 HYDROLOGY MANUAL
          (c) Copyright 1982-2015 Advanced Engineering Software (aes)
              Ver. 22.0 Release Date: 07/01/2015  License ID 1643

                           Analysis prepared by:




 ************************** DESCRIPTION OF STUDY **************************
* 100-YEAR DEVELOPED CONDITIONS ANALYSIS FOR                               *
* BLUE WAVE, IMPERIAL BEACH, CA                                            *
*                                                                          *
 **************************************************************************

  FILE NAME: C:\AES\DEV100.DAT
  TIME/DATE OF STUDY: 10:07 11/13/2018
----------------------------------------------------------------------------
  USER SPECIFIED HYDROLOGY AND HYDRAULIC MODEL INFORMATION:
----------------------------------------------------------------------------
  2003 SAN DIEGO MANUAL CRITERIA

  USER SPECIFIED STORM EVENT(YEAR) = 100.00
  6-HOUR DURATION PRECIPITATION (INCHES) =   2.400
  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   12.0      7.0    0.018/0.018/0.020   0.50    1.50 0.0313 0.125 0.0150

  GLOBAL STREET FLOW-DEPTH CONSTRAINTS:
    1. Relative Flow-Depth =  1.00 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      1.00 TO NODE      2.00 IS CODE =  21
----------------------------------------------------------------------------
  >>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<
============================================================================
  *USER SPECIFIED(SUBAREA):
  GENERAL COMMERCIAL RUNOFF COEFFICIENT = .8500
  S.C.S. CURVE NUMBER (AMC II) =   0
  INITIAL SUBAREA FLOW-LENGTH(FEET) =    60.00
  UPSTREAM ELEVATION(FEET) =     23.80
  DOWNSTREAM ELEVATION(FEET) =     23.20
  ELEVATION DIFFERENCE(FEET) =      0.60
  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) =  6.323
  NOTE: RAINFALL INTENSITY IS BASED ON Tc = 5-MINUTE.
  SUBAREA RUNOFF(CFS) =      0.37
  TOTAL AREA(ACRES) =      0.07   TOTAL RUNOFF(CFS) =      0.37
```

```
****************************************************************************
  FLOW PROCESS FROM NODE      2.00 TO NODE    100.00 IS CODE =  62
----------------------------------------------------------------------------
  >>>>>COMPUTE STREET FLOW TRAVEL TIME THRU SUBAREA<<<<<
  >>>>>(STREET TABLE SECTION #  1 USED)<<<<<
============================================================================
  UPSTREAM ELEVATION(FEET) =   23.20  DOWNSTREAM ELEVATION(FEET) =   20.80
  STREET LENGTH(FEET) =   281.00   CURB HEIGHT(INCHES) =  6.0
  STREET HALFWIDTH(FEET) = 12.00

  DISTANCE FROM CROWN TO CROSSFALL GRADEBREAK(FEET) =   7.00
  INSIDE STREET CROSSFALL(DECIMAL) =  0.018
  OUTSIDE STREET CROSSFALL(DECIMAL)  =  0.018

  SPECIFIED NUMBER OF HALFSTREETS CARRYING RUNOFF =  2
  STREET PARKWAY CROSSFALL(DECIMAL)  =  0.020
  Manning's FRICTION FACTOR for Streetflow Section(curb-to-curb) =   0.0150
  Manning's FRICTION FACTOR for Back-of-Walk Flow Section =   0.0200

    **TRAVEL TIME COMPUTED USING ESTIMATED FLOW(CFS) =       2.22
    STREETFLOW MODEL RESULTS USING ESTIMATED FLOW:
    STREET FLOW DEPTH(FEET) =  0.27
    HALFSTREET FLOOD WIDTH(FEET) =    7.68
    AVERAGE FLOW VELOCITY(FEET/SEC.) =    1.72
    PRODUCT OF DEPTH&VELOCITY(FT*FT/SEC.) =    0.46
  STREET FLOW TRAVEL TIME(MIN.) =   2.73   Tc(MIN.) =    6.21
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  5.496
  *USER SPECIFIED(SUBAREA):
  GENERAL COMMERCIAL RUNOFF COEFFICIENT = .8500
  S.C.S. CURVE NUMBER (AMC II) =   0
  AREA-AVERAGE RUNOFF COEFFICIENT =  0.850
  SUBAREA AREA(ACRES) =    0.79      SUBAREA RUNOFF(CFS) =    3.69
  TOTAL AREA(ACRES) =        0.9        PEAK FLOW RATE(CFS) =       4.01

  END OF SUBAREA STREET FLOW HYDRAULICS:
  DEPTH(FEET) = 0.31   HALFSTREET FLOOD WIDTH(FEET) =  10.09
  FLOW VELOCITY(FEET/SEC.) =  1.95   DEPTH*VELOCITY(FT*FT/SEC.) =   0.60
  LONGEST FLOWPATH FROM NODE      1.00 TO NODE    100.00 =     341.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    100.00 TO NODE    200.00 IS CODE =  31
----------------------------------------------------------------------------
  >>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
  >>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<
============================================================================
  ELEVATION DATA: UPSTREAM(FEET) =    15.00  DOWNSTREAM(FEET) =    12.00
  FLOW LENGTH(FEET) =   280.40   MANNING'S N =  0.013
  ESTIMATED PIPE DIAMETER(INCH) INCREASED TO 18.000
  DEPTH OF FLOW IN  18.0 INCH PIPE IS   7.8 INCHES
  PIPE-FLOW VELOCITY(FEET/SEC.) =   5.47
  ESTIMATED PIPE DIAMETER(INCH) =  18.00    NUMBER OF PIPES =   1
  PIPE-FLOW(CFS) =       4.01
  PIPE TRAVEL TIME(MIN.) =   0.85    Tc(MIN.) =    7.07
  LONGEST FLOWPATH FROM NODE      1.00 TO NODE    200.00 =     621.40 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    200.00 TO NODE    200.00 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.) =    7.07
  RAINFALL INTENSITY(INCH/HR) =   5.06
  TOTAL STREAM AREA(ACRES) =     0.86
  PEAK FLOW RATE(CFS) AT CONFLUENCE =      4.01

****************************************************************************
  FLOW PROCESS FROM NODE      3.00 TO NODE      4.00 IS CODE =  21
----------------------------------------------------------------------------
  >>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<
```

```
============================================================================
  *USER SPECIFIED(SUBAREA):
  GENERAL COMMERCIAL RUNOFF COEFFICIENT = .8500
  S.C.S. CURVE NUMBER (AMC II) =   0
  INITIAL SUBAREA FLOW-LENGTH(FEET) =    60.00
  UPSTREAM ELEVATION(FEET) =     22.40
  DOWNSTREAM ELEVATION(FEET) =     21.80
  ELEVATION DIFFERENCE(FEET) =      0.60
  SUBAREA OVERLAND TIME OF FLOW(MIN.) =    3.486
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  6.323
  NOTE: RAINFALL INTENSITY IS BASED ON Tc = 5-MINUTE.
  SUBAREA RUNOFF(CFS) =      0.49
  TOTAL AREA(ACRES) =      0.09   TOTAL RUNOFF(CFS) =      0.49

****************************************************************************
  FLOW PROCESS FROM NODE      4.00 TO NODE    200.00 IS CODE =  62
----------------------------------------------------------------------------
  >>>>>COMPUTE STREET FLOW TRAVEL TIME THRU SUBAREA<<<<<
  >>>>>(STREET TABLE SECTION #  1 USED)<<<<<
============================================================================
  UPSTREAM ELEVATION(FEET) =   21.80  DOWNSTREAM ELEVATION(FEET) =   19.00
  STREET LENGTH(FEET) =   129.00   CURB HEIGHT(INCHES) =  6.0
  STREET HALFWIDTH(FEET) = 12.00

  DISTANCE FROM CROWN TO CROSSFALL GRADEBREAK(FEET) =   7.00
  INSIDE STREET CROSSFALL(DECIMAL) =  0.018
  OUTSIDE STREET CROSSFALL(DECIMAL)  =  0.018

  SPECIFIED NUMBER OF HALFSTREETS CARRYING RUNOFF =  2
  STREET PARKWAY CROSSFALL(DECIMAL)  =  0.020
  Manning's FRICTION FACTOR for Streetflow Section(curb-to-curb) =   0.0150
  Manning's FRICTION FACTOR for Back-of-Walk Flow Section =   0.0200

    **TRAVEL TIME COMPUTED USING ESTIMATED FLOW(CFS) =       0.78
    STREETFLOW MODEL RESULTS USING ESTIMATED FLOW:
    STREET FLOW DEPTH(FEET) =  0.17
    HALFSTREET FLOOD WIDTH(FEET) =    2.10
    AVERAGE FLOW VELOCITY(FEET/SEC.) =    2.45
    PRODUCT OF DEPTH&VELOCITY(FT*FT/SEC.) =    0.41
  STREET FLOW TRAVEL TIME(MIN.) =   0.88   Tc(MIN.) =    4.36
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  6.323
  NOTE: RAINFALL INTENSITY IS BASED ON Tc = 5-MINUTE.
  *USER SPECIFIED(SUBAREA):
  GENERAL COMMERCIAL RUNOFF COEFFICIENT = .8500
  S.C.S. CURVE NUMBER (AMC II) =   0
  AREA-AVERAGE RUNOFF COEFFICIENT =  0.850
  SUBAREA AREA(ACRES) =    0.11      SUBAREA RUNOFF(CFS) =    0.59
  TOTAL AREA(ACRES) =        0.2        PEAK FLOW RATE(CFS) =       1.08

  END OF SUBAREA STREET FLOW HYDRAULICS:
  DEPTH(FEET) = 0.19   HALFSTREET FLOOD WIDTH(FEET) =   3.63
  FLOW VELOCITY(FEET/SEC.) =  2.27   DEPTH*VELOCITY(FT*FT/SEC.) =   0.44
  LONGEST FLOWPATH FROM NODE      3.00 TO NODE    200.00 =     189.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    200.00 TO NODE    200.00 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.) =    4.36
  RAINFALL INTENSITY(INCH/HR) =   6.32
  TOTAL STREAM AREA(ACRES) =     0.20
  PEAK FLOW RATE(CFS) AT CONFLUENCE =      1.08

  ** CONFLUENCE DATA **
  STREAM     RUNOFF       Tc      INTENSITY      AREA
  NUMBER      (CFS)     (MIN.)   (INCH/HOUR)    (ACRE)
      1        4.01     7.07        5.058          0.86
```

```
    2        1.08     4.36        6.323          0.20

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        3.55     4.36       6.323
    2        4.87     7.07       5.058

COMPUTED CONFLUENCE ESTIMATES ARE AS FOLLOWS:
PEAK FLOW RATE(CFS) =       4.87   Tc(MIN.) =    7.07
TOTAL AREA(ACRES) =        1.1
LONGEST FLOWPATH FROM NODE      1.00 TO NODE    200.00 =     621.40 FEET.

****************************************************************************
FLOW PROCESS FROM NODE    200.00 TO NODE    200.00 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.) =    7.07
RAINFALL INTENSITY(INCH/HR) =   5.06
TOTAL STREAM AREA(ACRES) =     1.06
PEAK FLOW RATE(CFS) AT CONFLUENCE =      4.87

****************************************************************************
FLOW PROCESS FROM NODE      5.00 TO NODE      6.00 IS CODE =  21
----------------------------------------------------------------------------
>>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<
============================================================================
*USER SPECIFIED(SUBAREA):
GENERAL COMMERCIAL RUNOFF COEFFICIENT = .8500
S.C.S. CURVE NUMBER (AMC II) =   0
INITIAL SUBAREA FLOW-LENGTH(FEET) =    50.00
UPSTREAM ELEVATION(FEET) =     20.30
DOWNSTREAM ELEVATION(FEET) =     20.05
ELEVATION DIFFERENCE(FEET) =      0.25
SUBAREA OVERLAND TIME OF FLOW(MIN.) =    4.009
WARNING: THE MINIMUM OVERLAND FLOW SLOPE, 0.5%, IS USED IN Tc CALCULATION!
 100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  6.323
NOTE: RAINFALL INTENSITY IS BASED ON Tc = 5-MINUTE.
SUBAREA RUNOFF(CFS) =      0.40
TOTAL AREA(ACRES) =      0.08   TOTAL RUNOFF(CFS) =      0.40

****************************************************************************
FLOW PROCESS FROM NODE      6.00 TO NODE    200.00 IS CODE =  51
----------------------------------------------------------------------------
>>>>>COMPUTE TRAPEZOIDAL CHANNEL FLOW<<<<<
>>>>>TRAVELTIME THRU SUBAREA (EXISTING ELEMENT)<<<<<
============================================================================
ELEVATION DATA: UPSTREAM(FEET) =     20.05  DOWNSTREAM(FEET) =     19.00
CHANNEL LENGTH THRU SUBAREA(FEET) =   251.00   CHANNEL SLOPE =  0.0042
CHANNEL BASE(FEET) =   10.00   "Z" FACTOR =   2.000
MANNING'S FACTOR = 0.045   MAXIMUM DEPTH(FEET) =   1.00
 100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  3.539
*USER SPECIFIED(SUBAREA):
GENERAL COMMERCIAL RUNOFF COEFFICIENT = .8500
S.C.S. CURVE NUMBER (AMC II) =   0
TRAVEL TIME COMPUTED USING ESTIMATED FLOW(CFS) =       0.65
TRAVEL TIME THRU SUBAREA BASED ON VELOCITY(FEET/SEC.) =   0.50
AVERAGE FLOW DEPTH(FEET) =   0.12   TRAVEL TIME(MIN.) =   8.29
Tc(MIN.) =   12.30
SUBAREA AREA(ACRES) =     0.15       SUBAREA RUNOFF(CFS) =    0.46
AREA-AVERAGE RUNOFF COEFFICIENT =  0.850
TOTAL AREA(ACRES) =        0.2         PEAK FLOW RATE(CFS) =       0.69

END OF SUBAREA CHANNEL FLOW HYDRAULICS:
DEPTH(FEET) =  0.13   FLOW VELOCITY(FEET/SEC.) =   0.53
```

```
  LONGEST FLOWPATH FROM NODE      5.00 TO NODE    200.00 =     301.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE      9.00 TO NODE      9.00 IS CODE =  81
----------------------------------------------------------------------------
  >>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
============================================================================
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  3.539
  *USER SPECIFIED(SUBAREA):
  GENERAL COMMERCIAL RUNOFF COEFFICIENT = .8500
  S.C.S. CURVE NUMBER (AMC II) =   0
  AREA-AVERAGE RUNOFF COEFFICIENT = 0.8500
  SUBAREA AREA(ACRES) =    0.04   SUBAREA RUNOFF(CFS) =    0.11
  TOTAL AREA(ACRES) =        0.3   TOTAL RUNOFF(CFS) =       0.80
  TC(MIN.) =   12.30

****************************************************************************
  FLOW PROCESS FROM NODE    200.00 TO NODE    200.00 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.) =   12.30
  RAINFALL INTENSITY(INCH/HR) =   3.54
  TOTAL STREAM AREA(ACRES) =     0.27
  PEAK FLOW RATE(CFS) AT CONFLUENCE =      0.80

  ** CONFLUENCE DATA **
  STREAM     RUNOFF       Tc      INTENSITY      AREA
  NUMBER      (CFS)     (MIN.)   (INCH/HOUR)    (ACRE)
     1        4.87     7.07        5.058          1.06
     2        0.80    12.30        3.539          0.27

  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        5.33     7.07       5.058
     2        4.21    12.30       3.539

  COMPUTED CONFLUENCE ESTIMATES ARE AS FOLLOWS:
  PEAK FLOW RATE(CFS) =       5.33   Tc(MIN.) =    7.07
  TOTAL AREA(ACRES) =        1.3
  LONGEST FLOWPATH FROM NODE      1.00 TO NODE    200.00 =     621.40 FEET.
============================================================================
  END OF STUDY SUMMARY:
  TOTAL AREA(ACRES)     =        1.3  TC(MIN.) =      7.07
  PEAK FLOW RATE(CFS)   =       5.33
============================================================================
============================================================================
  END OF RATIONAL METHOD ANALYSIS
```

CHAPTER 5 - MODIFIED-PULS DETENTION ROUTING

5.1 – Rational Method Hydrograph

RATIONAL METHOD HYDROGRAPH PROGRAM

RUN DATE 11/13/2018
HYDROGRAPH FILE NAME Text1
TIME OF CONCENTRATION 7 MIN.
6 HOUR RAINFALL 2.4 INCHES
BASIN AREA 1.33 ACRES
RUNOFF COEFFICIENT 0.85
PEAK DISCHARGE 5.33 CFS

TIME (MIN) = 0 DISCHARGE (CFS) = 0
TIME (MIN) = 7 DISCHARGE (CFS) = 0.2
TIME (MIN) = 14 DISCHARGE (CFS) = 0.2
TIME (MIN) = 21 DISCHARGE (CFS) = 0.2
TIME (MIN) = 28 DISCHARGE (CFS) = 0.2
TIME (MIN) = 35 DISCHARGE (CFS) = 0.2
TIME (MIN) = 42 DISCHARGE (CFS) = 0.2
TIME (MIN) = 49 DISCHARGE (CFS) = 0.2
TIME (MIN) = 56 DISCHARGE (CFS) = 0.2
TIME (MIN) = 63 DISCHARGE (CFS) = 0.2
TIME (MIN) = 70 DISCHARGE (CFS) = 0.2
TIME (MIN) = 77 DISCHARGE (CFS) = 0.2
TIME (MIN) = 84 DISCHARGE (CFS) = 0.2
TIME (MIN) = 91 DISCHARGE (CFS) = 0.2
TIME (MIN) = 98 DISCHARGE (CFS) = 0.2
TIME (MIN) = 105 DISCHARGE (CFS) = 0.2
TIME (MIN) = 112 DISCHARGE (CFS) = 0.2
TIME (MIN) = 119 DISCHARGE (CFS) = 0.2
TIME (MIN) = 126 DISCHARGE (CFS) = 0.3
TIME (MIN) = 133 DISCHARGE (CFS) = 0.3
TIME (MIN) = 140 DISCHARGE (CFS) = 0.3
TIME (MIN) = 147 DISCHARGE (CFS) = 0.3
TIME (MIN) = 154 DISCHARGE (CFS) = 0.3
TIME (MIN) = 161 DISCHARGE (CFS) = 0.3
TIME (MIN) = 168 DISCHARGE (CFS) = 0.3
TIME (MIN) = 175 DISCHARGE (CFS) = 0.4
TIME (MIN) = 182 DISCHARGE (CFS) = 0.4
TIME (MIN) = 189 DISCHARGE (CFS) = 0.4
TIME (MIN) = 196 DISCHARGE (CFS) = 0.4
TIME (MIN) = 203 DISCHARGE (CFS) = 0.5
TIME (MIN) = 210 DISCHARGE (CFS) = 0.6
TIME (MIN) = 217 DISCHARGE (CFS) = 0.7
TIME (MIN) = 224 DISCHARGE (CFS) = 0.8
TIME (MIN) = 231 DISCHARGE (CFS) = 1.1
TIME (MIN) = 238 DISCHARGE (CFS) = 2
TIME (MIN) = 245 DISCHARGE (CFS) = 5.33
TIME (MIN) = 252 DISCHARGE (CFS) = 0.9
TIME (MIN) = 259 DISCHARGE (CFS) = 0.6
TIME (MIN) = 266 DISCHARGE (CFS) = 0.5
TIME (MIN) = 273 DISCHARGE (CFS) = 0.4
TIME (MIN) = 280 DISCHARGE (CFS) = 0.3
TIME (MIN) = 287 DISCHARGE (CFS) = 0.3
TIME (MIN) = 294 DISCHARGE (CFS) = 0.3
TIME (MIN) = 301 DISCHARGE (CFS) = 0.3
TIME (MIN) = 308 DISCHARGE (CFS) = 0.2
TIME (MIN) = 315 DISCHARGE (CFS) = 0.2
TIME (MIN) = 322 DISCHARGE (CFS) = 0.2
TIME (MIN) = 329 DISCHARGE (CFS) = 0.2
TIME (MIN) = 336 DISCHARGE (CFS) = 0.2
TIME (MIN) = 343 DISCHARGE (CFS) = 0.2
TIME (MIN) = 350 DISCHARGE (CFS) = 0.2
TIME (MIN) = 357 DISCHARGE (CFS) = 0.2

5.2 – Stage-Storage & Stage-Discharge Relationships

Detention Vault Stage-Stoarge Calculation

Elevation (ft)	Area (ft^2)	Volume (ft^3)	Volume (Ac-ft)
0	520	0	0
1	520	520	0.011938
2	520	1040	0.023875
3	520	1560	0.035813
4	520	2080	0.04775
5	520	2600	0.059688
6	520	3120	0.071625

Outlet structure for Discharge of Detention Basin 1

Low orifice:	1 "	Lower slot			Emergency Weir		
Number:	0	Invert:	0.00	ft	Invert:	5.500	ft
Cg-low:	0.62	B	0.90	ft	B:	5	ft
Middle orifice:	1 "	h	0.167	ft			
number of orif:	0	Upper slot					
Cg-middle:	0.62	Invert:	0.00	ft			
invert elev:	0.75 ft	B:	0.00	ft			
		h	0.167	ft			

h (ft)	H/D-low -	H/D-mid -	Qlow-orif (cfs)	Qlow-weir (cfs)	Qtot-low (cfs)	Qmid-orif (cfs)	Qmid-weir (cfs)	Qtot-med (cfs)	Qslot-low (cfs)	Qslot-upp (cfs)	Qemer (cfs)	Qtot (cfs)	Total H (ft)	Total Q (cfs)
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
0.200	2.400	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.249	0.000	0.000	0.249	0.200	0.249
0.400	4.800	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.413	0.000	0.000	0.413	0.400	0.413
0.600	7.200	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.528	0.000	0.000	0.528	0.600	0.528
0.800	9.600	0.600	0.000	0.000	0.000	0.000	0.000	0.000	0.622	0.000	0.000	0.622	0.800	0.622
1.000	12.000	3.000	0.000	0.000	0.000	0.000	0.000	0.000	0.703	0.000	0.000	0.703	1.000	0.703
1.200	14.400	5.400	0.000	0.000	0.000	0.000	0.000	0.000	0.776	0.000	0.000	0.776	1.200	0.776
1.400	16.800	7.800	0.000	0.000	0.000	0.000	0.000	0.000	0.843	0.000	0.000	0.843	1.400	0.843
1.600	19.200	10.200	0.000	0.000	0.000	0.000	0.000	0.000	0.904	0.000	0.000	0.904	1.600	0.904
1.800	21.600	12.600	0.000	0.000	0.000	0.000	0.000	0.000	0.962	0.000	0.000	0.962	1.800	0.962
2.000	24.000	15.000	0.000	0.000	0.000	0.000	0.000	0.000	1.017	0.000	0.000	1.017	2.000	1.017
2.200	26.400	17.400	0.000	0.000	0.000	0.000	0.000	0.000	1.068	0.000	0.000	1.068	2.200	1.068
2.400	28.800	19.800	0.000	0.000	0.000	0.000	0.000	0.000	1.118	0.000	0.000	1.118	2.400	1.118
2.600	31.200	22.200	0.000	0.000	0.000	0.000	0.000	0.000	1.165	0.000	0.000	1.165	2.600	1.165
2.800	33.600	24.600	0.000	0.000	0.000	0.000	0.000	0.000	1.210	0.000	0.000	1.210	2.800	1.210
3.000	36.000	27.000	0.000	0.000	0.000	0.000	0.000	0.000	1.254	0.000	0.000	1.254	3.000	1.254
3.200	38.400	29.400	0.000	0.000	0.000	0.000	0.000	0.000	1.296	0.000	0.000	1.296	3.200	1.296
3.400	40.800	31.800	0.000	0.000	0.000	0.000	0.000	0.000	1.337	0.000	0.000	1.337	3.400	1.337
3.600	43.200	34.200	0.000	0.000	0.000	0.000	0.000	0.000	1.377	0.000	0.000	1.377	3.600	1.377
3.800	45.600	36.600	0.000	0.000	0.000	0.000	0.000	0.000	1.416	0.000	0.000	1.416	3.800	1.416
4.000	48.000	39.000	0.000	0.000	0.000	0.000	0.000	0.000	1.453	0.000	0.000	1.453	4.000	1.453
4.200	50.400	41.400	0.000	0.000	0.000	0.000	0.000	0.000	1.490	0.000	0.000	1.490	4.200	1.490
4.400	52.800	43.800	0.000	0.000	0.000	0.000	0.000	0.000	1.526	0.000	0.000	1.526	4.400	1.526
4.600	55.200	46.200	0.000	0.000	0.000	0.000	0.000	0.000	1.561	0.000	0.000	1.561	4.600	1.561
4.800	57.600	48.600	0.000	0.000	0.000	0.000	0.000	0.000	1.595	0.000	0.000	1.595	4.800	1.595
5.000	60.000	51.000	0.000	0.000	0.000	0.000	0.000	0.000	1.628	0.000	0.000	1.628	5.000	1.628
5.200	62.400	53.400	0.000	0.000	0.000	0.000	0.000	0.000	1.661	0.000	0.000	1.661	5.200	1.661
5.400	64.800	55.800	0.000	0.000	0.000	0.000	0.000	0.000	1.693	0.000	0.000	1.693	5.400	1.693
5.600	67.200	58.200	0.000	0.000	0.000	0.000	0.000	0.000	1.725	0.000	0.490	2.215	5.600	2.215
5.800	69.600	60.600	0.000	0.000	0.000	0.000	0.000	0.000	1.756	0.000	2.547	4.303	5.800	4.303
6.000	72.000	63.000	0.000	0.000	0.000	0.000	0.000	0.000	1.786	0.000	5.480	7.266	6.000	7.266

5.3 – HEC-HMS Modified-Puls Routing Results

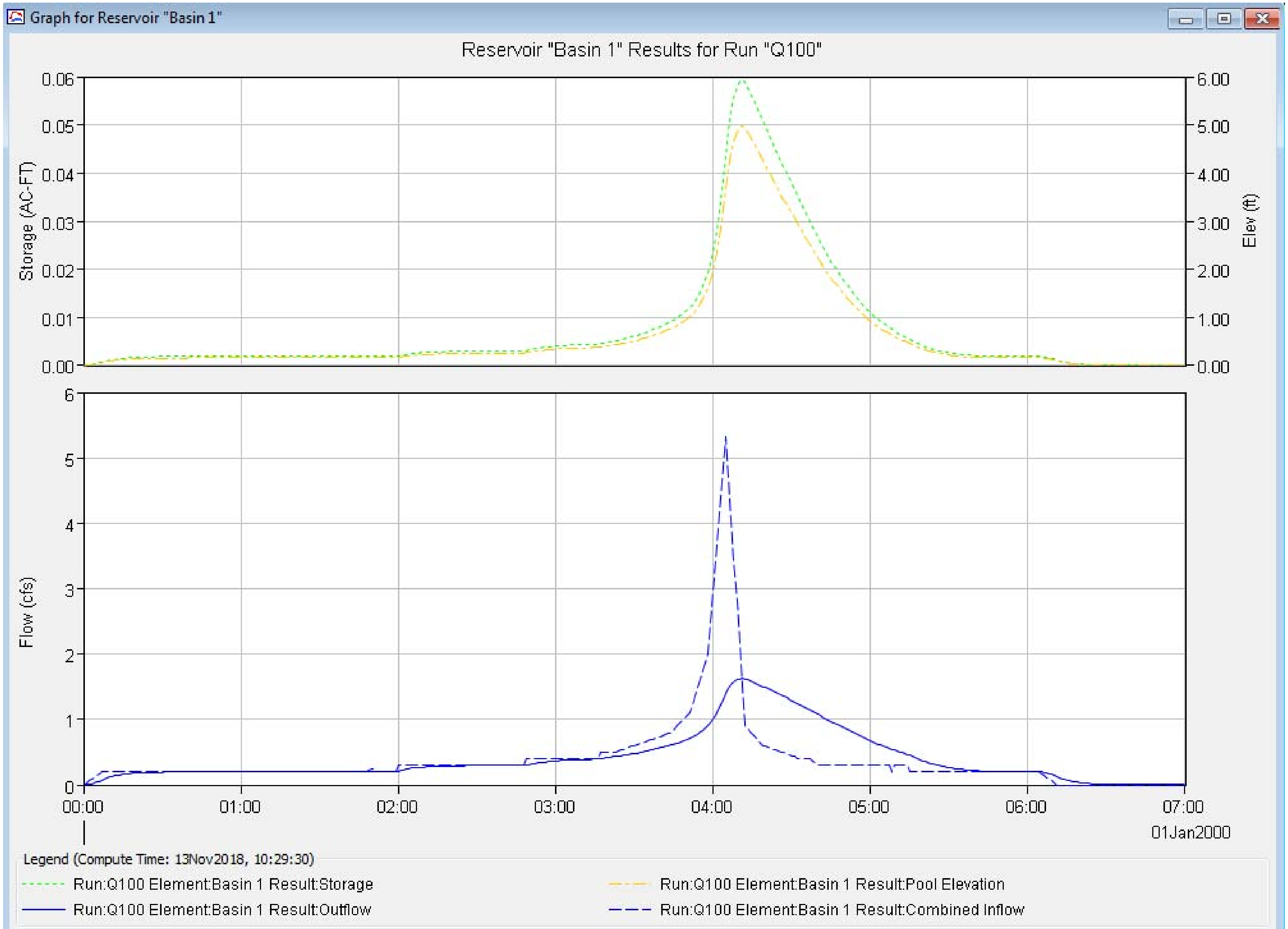
Project: BlueWave Simulation Run: Q100
Reservoir: Basin 1

Start of Run:	01Jan2000, 00:00	Basin Model:	Post_Dev
End of Run:	01Jan2000, 07:00	Meteorologic Model:	Met 1
Compute Time:	13Nov2018, 10:29:30	Control Specifications:	Control 1

Volume Units: IN

Computed Results

Peak Inflow:	5.3 (CFS)	Date/Time of Peak Inflow:	01Jan2000, 04:05
Peak Discharge:	1.6 (CFS)	Date/Time of Peak Discharge:	01Jan2000, 04:11
Inflow Volume:	n/a	Peak Storage:	0.1 (AC-FT)
Discharge Volume	n/a	Peak Elevation:	5.0 (FT)



Graph for Reservoir "Basin 1"
Reservoir "Basin 1" Results for Run "Q100"
Storage (AC-FT)
Elev (ft)
Flow (cfs)
0.06
0.05
0.04
0.03
0.02
0.01
0.00
6.00
5.00
4.00
3.00
2.00
1.00
0.00
6
5
4
3
2
1
0
00:00
01:00
02:00
03:00
04:00
05:00
06:00
07:00
01Jan2000
Legend (Compute Time: 13Nov2018, 10:29:30)
Run:Q100 Element:Basin 1 Result:Storage
Run:Q100 Element:Basin 1 Result:Pool Elevation
Run:Q100 Element:Basin 1 Result:Outflow
Run:Q100 Element:Basin 1 Result:Combined Inflow

Project: BlueWave Simulation Run: Q100
Reservoir: Basin 1

Start of Run: 01Jan2000, 00:00 Basin Model: Post_Dev
End of Run: 01Jan2000, 07:00 Meteorologic Model: Met 1
Compute Time: 13Nov2018, 10:29:30 Control Specifications: Control 1

Date	Time	Inflow (CFS)	Storage (AC-FT)	Elevation (FT)	Outflow (CFS)
01Jan2000	00:00	0.0	0.0	0.0	0.0
01Jan2000	00:01	0.0	0.0	0.0	0.0
01Jan2000	00:02	0.1	0.0	0.0	0.0
01Jan2000	00:03	0.1	0.0	0.0	0.0
01Jan2000	00:04	0.1	0.0	0.0	0.0
01Jan2000	00:05	0.1	0.0	0.0	0.0
01Jan2000	00:06	0.2	0.0	0.0	0.1
01Jan2000	00:07	0.2	0.0	0.1	0.1
01Jan2000	00:08	0.2	0.0	0.1	0.1
01Jan2000	00:09	0.2	0.0	0.1	0.1
01Jan2000	00:10	0.2	0.0	0.1	0.1
01Jan2000	00:11	0.2	0.0	0.1	0.1
01Jan2000	00:12	0.2	0.0	0.1	0.1
01Jan2000	00:13	0.2	0.0	0.1	0.1
01Jan2000	00:14	0.2	0.0	0.1	0.2
01Jan2000	00:15	0.2	0.0	0.1	0.2
01Jan2000	00:16	0.2	0.0	0.1	0.2
01Jan2000	00:17	0.2	0.0	0.1	0.2
01Jan2000	00:18	0.2	0.0	0.1	0.2
01Jan2000	00:19	0.2	0.0	0.1	0.2
01Jan2000	00:20	0.2	0.0	0.1	0.2
01Jan2000	00:21	0.2	0.0	0.1	0.2
01Jan2000	00:22	0.2	0.0	0.1	0.2
01Jan2000	00:23	0.2	0.0	0.2	0.2
01Jan2000	00:24	0.2	0.0	0.2	0.2
01Jan2000	00:25	0.2	0.0	0.2	0.2
01Jan2000	00:26	0.2	0.0	0.2	0.2
01Jan2000	00:27	0.2	0.0	0.2	0.2
01Jan2000	00:28	0.2	0.0	0.2	0.2
01Jan2000	00:29	0.2	0.0	0.2	0.2

Date	Time	Inflow (CFS)	Storage (AC-FT)	Elevation (FT)	Outflow (CFS)
01Jan2000	00:30	0.2	0.0	0.2	0.2
01Jan2000	00:31	0.2	0.0	0.2	0.2
01Jan2000	00:32	0.2	0.0	0.2	0.2
01Jan2000	00:33	0.2	0.0	0.2	0.2
01Jan2000	00:34	0.2	0.0	0.2	0.2
01Jan2000	00:35	0.2	0.0	0.2	0.2
01Jan2000	00:36	0.2	0.0	0.2	0.2
01Jan2000	00:37	0.2	0.0	0.2	0.2
01Jan2000	00:38	0.2	0.0	0.2	0.2
01Jan2000	00:39	0.2	0.0	0.2	0.2
01Jan2000	00:40	0.2	0.0	0.2	0.2
01Jan2000	00:41	0.2	0.0	0.2	0.2
01Jan2000	00:42	0.2	0.0	0.2	0.2
01Jan2000	00:43	0.2	0.0	0.2	0.2
01Jan2000	00:44	0.2	0.0	0.2	0.2
01Jan2000	00:45	0.2	0.0	0.2	0.2
01Jan2000	00:46	0.2	0.0	0.2	0.2
01Jan2000	00:47	0.2	0.0	0.2	0.2
01Jan2000	00:48	0.2	0.0	0.2	0.2
01Jan2000	00:49	0.2	0.0	0.2	0.2
01Jan2000	00:50	0.2	0.0	0.2	0.2
01Jan2000	00:51	0.2	0.0	0.2	0.2
01Jan2000	00:52	0.2	0.0	0.2	0.2
01Jan2000	00:53	0.2	0.0	0.2	0.2
01Jan2000	00:54	0.2	0.0	0.2	0.2
01Jan2000	00:55	0.2	0.0	0.2	0.2
01Jan2000	00:56	0.2	0.0	0.2	0.2
01Jan2000	00:57	0.2	0.0	0.2	0.2
01Jan2000	00:58	0.2	0.0	0.2	0.2
01Jan2000	00:59	0.2	0.0	0.2	0.2
01Jan2000	01:00	0.2	0.0	0.2	0.2
01Jan2000	01:01	0.2	0.0	0.2	0.2
01Jan2000	01:02	0.2	0.0	0.2	0.2
01Jan2000	01:03	0.2	0.0	0.2	0.2
01Jan2000	01:04	0.2	0.0	0.2	0.2

Date	Time	Inflow (CFS)	Storage (AC-FT)	Elevation (FT)	Outflow (CFS)
01Jan2000	01:05	0.2	0.0	0.2	0.2
01Jan2000	01:06	0.2	0.0	0.2	0.2
01Jan2000	01:07	0.2	0.0	0.2	0.2
01Jan2000	01:08	0.2	0.0	0.2	0.2
01Jan2000	01:09	0.2	0.0	0.2	0.2
01Jan2000	01:10	0.2	0.0	0.2	0.2
01Jan2000	01:11	0.2	0.0	0.2	0.2
01Jan2000	01:12	0.2	0.0	0.2	0.2
01Jan2000	01:13	0.2	0.0	0.2	0.2
01Jan2000	01:14	0.2	0.0	0.2	0.2
01Jan2000	01:15	0.2	0.0	0.2	0.2
01Jan2000	01:16	0.2	0.0	0.2	0.2
01Jan2000	01:17	0.2	0.0	0.2	0.2
01Jan2000	01:18	0.2	0.0	0.2	0.2
01Jan2000	01:19	0.2	0.0	0.2	0.2
01Jan2000	01:20	0.2	0.0	0.2	0.2
01Jan2000	01:21	0.2	0.0	0.2	0.2
01Jan2000	01:22	0.2	0.0	0.2	0.2
01Jan2000	01:23	0.2	0.0	0.2	0.2
01Jan2000	01:24	0.2	0.0	0.2	0.2
01Jan2000	01:25	0.2	0.0	0.2	0.2
01Jan2000	01:26	0.2	0.0	0.2	0.2
01Jan2000	01:27	0.2	0.0	0.2	0.2
01Jan2000	01:28	0.2	0.0	0.2	0.2
01Jan2000	01:29	0.2	0.0	0.2	0.2
01Jan2000	01:30	0.2	0.0	0.2	0.2
01Jan2000	01:31	0.2	0.0	0.2	0.2
01Jan2000	01:32	0.2	0.0	0.2	0.2
01Jan2000	01:33	0.2	0.0	0.2	0.2
01Jan2000	01:34	0.2	0.0	0.2	0.2
01Jan2000	01:35	0.2	0.0	0.2	0.2
01Jan2000	01:36	0.2	0.0	0.2	0.2
01Jan2000	01:37	0.2	0.0	0.2	0.2
01Jan2000	01:38	0.2	0.0	0.2	0.2
01Jan2000	01:39	0.2	0.0	0.2	0.2

Date	Time	Inflow (CFS)	Storage (AC-FT)	Elevation (FT)	Outflow (CFS)
01Jan2000	01:40	0.2	0.0	0.2	0.2
01Jan2000	01:41	0.2	0.0	0.2	0.2
01Jan2000	01:42	0.2	0.0	0.2	0.2
01Jan2000	01:43	0.2	0.0	0.2	0.2
01Jan2000	01:44	0.2	0.0	0.2	0.2
01Jan2000	01:45	0.2	0.0	0.2	0.2
01Jan2000	01:46	0.2	0.0	0.2	0.2
01Jan2000	01:47	0.2	0.0	0.2	0.2
01Jan2000	01:48	0.2	0.0	0.2	0.2
01Jan2000	01:49	0.2	0.0	0.2	0.2
01Jan2000	01:50	0.2	0.0	0.2	0.2
01Jan2000	01:51	0.2	0.0	0.2	0.2
01Jan2000	01:52	0.2	0.0	0.2	0.2
01Jan2000	01:53	0.2	0.0	0.2	0.2
01Jan2000	01:54	0.2	0.0	0.2	0.2
01Jan2000	01:55	0.2	0.0	0.2	0.2
01Jan2000	01:56	0.2	0.0	0.2	0.2
01Jan2000	01:57	0.2	0.0	0.2	0.2
01Jan2000	01:58	0.2	0.0	0.2	0.2
01Jan2000	01:59	0.2	0.0	0.2	0.2
01Jan2000	02:00	0.3	0.0	0.2	0.2
01Jan2000	02:01	0.3	0.0	0.2	0.2
01Jan2000	02:02	0.3	0.0	0.2	0.2
01Jan2000	02:03	0.3	0.0	0.2	0.2
01Jan2000	02:04	0.3	0.0	0.2	0.2
01Jan2000	02:05	0.3	0.0	0.2	0.3
01Jan2000	02:06	0.3	0.0	0.2	0.3
01Jan2000	02:07	0.3	0.0	0.2	0.3
01Jan2000	02:08	0.3	0.0	0.2	0.3
01Jan2000	02:09	0.3	0.0	0.2	0.3
01Jan2000	02:10	0.3	0.0	0.2	0.3
01Jan2000	02:11	0.3	0.0	0.2	0.3
01Jan2000	02:12	0.3	0.0	0.2	0.3
01Jan2000	02:13	0.3	0.0	0.2	0.3
01Jan2000	02:14	0.3	0.0	0.2	0.3

Date	Time	Inflow (CFS)	Storage (AC-FT)	Elevation (FT)	Outflow (CFS)
01Jan2000	02:15	0.3	0.0	0.2	0.3
01Jan2000	02:16	0.3	0.0	0.2	0.3
01Jan2000	02:17	0.3	0.0	0.2	0.3
01Jan2000	02:18	0.3	0.0	0.2	0.3
01Jan2000	02:19	0.3	0.0	0.2	0.3
01Jan2000	02:20	0.3	0.0	0.2	0.3
01Jan2000	02:21	0.3	0.0	0.2	0.3
01Jan2000	02:22	0.3	0.0	0.3	0.3
01Jan2000	02:23	0.3	0.0	0.3	0.3
01Jan2000	02:24	0.3	0.0	0.3	0.3
01Jan2000	02:25	0.3	0.0	0.3	0.3
01Jan2000	02:26	0.3	0.0	0.3	0.3
01Jan2000	02:27	0.3	0.0	0.3	0.3
01Jan2000	02:28	0.3	0.0	0.3	0.3
01Jan2000	02:29	0.3	0.0	0.3	0.3
01Jan2000	02:30	0.3	0.0	0.3	0.3
01Jan2000	02:31	0.3	0.0	0.3	0.3
01Jan2000	02:32	0.3	0.0	0.3	0.3
01Jan2000	02:33	0.3	0.0	0.3	0.3
01Jan2000	02:34	0.3	0.0	0.3	0.3
01Jan2000	02:35	0.3	0.0	0.3	0.3
01Jan2000	02:36	0.3	0.0	0.3	0.3
01Jan2000	02:37	0.3	0.0	0.3	0.3
01Jan2000	02:38	0.3	0.0	0.3	0.3
01Jan2000	02:39	0.3	0.0	0.3	0.3
01Jan2000	02:40	0.3	0.0	0.3	0.3
01Jan2000	02:41	0.3	0.0	0.3	0.3
01Jan2000	02:42	0.3	0.0	0.3	0.3
01Jan2000	02:43	0.3	0.0	0.3	0.3
01Jan2000	02:44	0.3	0.0	0.3	0.3
01Jan2000	02:45	0.3	0.0	0.3	0.3
01Jan2000	02:46	0.3	0.0	0.3	0.3
01Jan2000	02:47	0.3	0.0	0.3	0.3
01Jan2000	02:48	0.3	0.0	0.3	0.3
01Jan2000	02:49	0.4	0.0	0.3	0.3

Date	Time	Inflow (CFS)	Storage (AC-FT)	Elevation (FT)	Outflow (CFS)
01Jan2000	02:50	0.4	0.0	0.3	0.3
01Jan2000	02:51	0.4	0.0	0.3	0.3
01Jan2000	02:52	0.4	0.0	0.3	0.3
01Jan2000	02:53	0.4	0.0	0.3	0.3
01Jan2000	02:54	0.4	0.0	0.3	0.3
01Jan2000	02:55	0.4	0.0	0.3	0.3
01Jan2000	02:56	0.4	0.0	0.3	0.4
01Jan2000	02:57	0.4	0.0	0.3	0.4
01Jan2000	02:58	0.4	0.0	0.3	0.4
01Jan2000	02:59	0.4	0.0	0.3	0.4
01Jan2000	03:00	0.4	0.0	0.3	0.4
01Jan2000	03:01	0.4	0.0	0.3	0.4
01Jan2000	03:02	0.4	0.0	0.3	0.4
01Jan2000	03:03	0.4	0.0	0.4	0.4
01Jan2000	03:04	0.4	0.0	0.4	0.4
01Jan2000	03:05	0.4	0.0	0.4	0.4
01Jan2000	03:06	0.4	0.0	0.4	0.4
01Jan2000	03:07	0.4	0.0	0.4	0.4
01Jan2000	03:08	0.4	0.0	0.4	0.4
01Jan2000	03:09	0.4	0.0	0.4	0.4
01Jan2000	03:10	0.4	0.0	0.4	0.4
01Jan2000	03:11	0.4	0.0	0.4	0.4
01Jan2000	03:12	0.4	0.0	0.4	0.4
01Jan2000	03:13	0.4	0.0	0.4	0.4
01Jan2000	03:14	0.4	0.0	0.4	0.4
01Jan2000	03:15	0.4	0.0	0.4	0.4
01Jan2000	03:16	0.4	0.0	0.4	0.4
01Jan2000	03:17	0.5	0.0	0.4	0.4
01Jan2000	03:18	0.5	0.0	0.4	0.4
01Jan2000	03:19	0.5	0.0	0.4	0.4
01Jan2000	03:20	0.5	0.0	0.4	0.4
01Jan2000	03:21	0.5	0.0	0.4	0.4
01Jan2000	03:22	0.5	0.0	0.4	0.4
01Jan2000	03:23	0.5	0.0	0.4	0.4
01Jan2000	03:24	0.5	0.0	0.4	0.4

Date	Time	Inflow (CFS)	Storage (AC-FT)	Elevation (FT)	Outflow (CFS)
01Jan2000	03:25	0.5	0.0	0.5	0.4
01Jan2000	03:26	0.5	0.0	0.5	0.5
01Jan2000	03:27	0.6	0.0	0.5	0.5
01Jan2000	03:28	0.6	0.0	0.5	0.5
01Jan2000	03:29	0.6	0.0	0.5	0.5
01Jan2000	03:30	0.6	0.0	0.5	0.5
01Jan2000	03:31	0.6	0.0	0.5	0.5
01Jan2000	03:32	0.6	0.0	0.5	0.5
01Jan2000	03:33	0.6	0.0	0.6	0.5
01Jan2000	03:34	0.7	0.0	0.6	0.5
01Jan2000	03:35	0.7	0.0	0.6	0.5
01Jan2000	03:36	0.7	0.0	0.6	0.5
01Jan2000	03:37	0.7	0.0	0.6	0.5
01Jan2000	03:38	0.7	0.0	0.6	0.6
01Jan2000	03:39	0.7	0.0	0.7	0.6
01Jan2000	03:40	0.7	0.0	0.7	0.6
01Jan2000	03:41	0.8	0.0	0.7	0.6
01Jan2000	03:42	0.8	0.0	0.7	0.6
01Jan2000	03:43	0.8	0.0	0.7	0.6
01Jan2000	03:44	0.8	0.0	0.8	0.6
01Jan2000	03:45	0.8	0.0	0.8	0.6
01Jan2000	03:46	0.9	0.0	0.8	0.6
01Jan2000	03:47	0.9	0.0	0.9	0.6
01Jan2000	03:48	1.0	0.0	0.9	0.7
01Jan2000	03:49	1.0	0.0	0.9	0.7
01Jan2000	03:50	1.1	0.0	1.0	0.7
01Jan2000	03:51	1.1	0.0	1.0	0.7
01Jan2000	03:52	1.2	0.0	1.1	0.7
01Jan2000	03:53	1.4	0.0	1.1	0.7
01Jan2000	03:54	1.5	0.0	1.2	0.8
01Jan2000	03:55	1.6	0.0	1.3	0.8
01Jan2000	03:56	1.7	0.0	1.4	0.8
01Jan2000	03:57	1.9	0.0	1.5	0.9
01Jan2000	03:58	2.0	0.0	1.6	0.9
01Jan2000	03:59	2.5	0.0	1.8	1.0

Date	Time	Inflow (CFS)	Storage (AC-FT)	Elevation (FT)	Outflow (CFS)
01Jan2000	04:00	3.0	0.0	2.0	1.0
01Jan2000	04:01	3.4	0.0	2.2	1.1
01Jan2000	04:02	3.9	0.0	2.5	1.1
01Jan2000	04:03	4.4	0.0	2.9	1.2
01Jan2000	04:04	4.8	0.0	3.2	1.3
01Jan2000	04:05	5.3	0.0	3.7	1.4
01Jan2000	04:06	4.7	0.0	4.1	1.5
01Jan2000	04:07	4.1	0.1	4.4	1.5
01Jan2000	04:08	3.4	0.1	4.7	1.6
01Jan2000	04:09	2.8	0.1	4.8	1.6
01Jan2000	04:10	2.2	0.1	4.9	1.6
01Jan2000	04:11	1.5	0.1	5.0	1.6
01Jan2000	04:12	0.9	0.1	4.9	1.6
01Jan2000	04:13	0.9	0.1	4.8	1.6
01Jan2000	04:14	0.8	0.1	4.8	1.6
01Jan2000	04:15	0.8	0.1	4.7	1.6
01Jan2000	04:16	0.7	0.1	4.6	1.6
01Jan2000	04:17	0.7	0.1	4.5	1.5
01Jan2000	04:18	0.6	0.1	4.4	1.5
01Jan2000	04:19	0.6	0.1	4.3	1.5
01Jan2000	04:20	0.6	0.0	4.2	1.5
01Jan2000	04:21	0.6	0.0	4.1	1.5
01Jan2000	04:22	0.6	0.0	4.0	1.4
01Jan2000	04:23	0.5	0.0	3.9	1.4
01Jan2000	04:24	0.5	0.0	3.8	1.4
01Jan2000	04:25	0.5	0.0	3.7	1.4
01Jan2000	04:26	0.5	0.0	3.6	1.4
01Jan2000	04:27	0.5	0.0	3.5	1.3
01Jan2000	04:28	0.5	0.0	3.4	1.3
01Jan2000	04:29	0.5	0.0	3.3	1.3
01Jan2000	04:30	0.4	0.0	3.2	1.3
01Jan2000	04:31	0.4	0.0	3.1	1.3
01Jan2000	04:32	0.4	0.0	3.0	1.2
01Jan2000	04:33	0.4	0.0	2.9	1.2
01Jan2000	04:34	0.4	0.0	2.8	1.2

Date	Time	Inflow (CFS)	Storage (AC-FT)	Elevation (FT)	Outflow (CFS)
01Jan2000	04:35	0.4	0.0	2.7	1.2
01Jan2000	04:36	0.4	0.0	2.6	1.2
01Jan2000	04:37	0.4	0.0	2.5	1.1
01Jan2000	04:38	0.4	0.0	2.4	1.1
01Jan2000	04:39	0.3	0.0	2.3	1.1
01Jan2000	04:40	0.3	0.0	2.2	1.1
01Jan2000	04:41	0.3	0.0	2.2	1.1
01Jan2000	04:42	0.3	0.0	2.1	1.0
01Jan2000	04:43	0.3	0.0	2.0	1.0
01Jan2000	04:44	0.3	0.0	1.9	1.0
01Jan2000	04:45	0.3	0.0	1.8	1.0
01Jan2000	04:46	0.3	0.0	1.7	0.9
01Jan2000	04:47	0.3	0.0	1.7	0.9
01Jan2000	04:48	0.3	0.0	1.6	0.9
01Jan2000	04:49	0.3	0.0	1.5	0.9
01Jan2000	04:50	0.3	0.0	1.5	0.9
01Jan2000	04:51	0.3	0.0	1.4	0.8
01Jan2000	04:52	0.3	0.0	1.3	0.8
01Jan2000	04:53	0.3	0.0	1.3	0.8
01Jan2000	04:54	0.3	0.0	1.2	0.8
01Jan2000	04:55	0.3	0.0	1.2	0.8
01Jan2000	04:56	0.3	0.0	1.1	0.7
01Jan2000	04:57	0.3	0.0	1.1	0.7
01Jan2000	04:58	0.3	0.0	1.0	0.7
01Jan2000	04:59	0.3	0.0	1.0	0.7
01Jan2000	05:00	0.3	0.0	0.9	0.7
01Jan2000	05:01	0.3	0.0	0.9	0.7
01Jan2000	05:02	0.3	0.0	0.8	0.6
01Jan2000	05:03	0.3	0.0	0.8	0.6
01Jan2000	05:04	0.3	0.0	0.8	0.6
01Jan2000	05:05	0.3	0.0	0.7	0.6
01Jan2000	05:06	0.3	0.0	0.7	0.6
01Jan2000	05:07	0.3	0.0	0.7	0.6
01Jan2000	05:08	0.2	0.0	0.6	0.5
01Jan2000	05:09	0.3	0.0	0.6	0.5

Date	Time	Inflow (CFS)	Storage (AC-FT)	Elevation (FT)	Outflow (CFS)
01Jan2000	05:10	0.3	0.0	0.6	0.5
01Jan2000	05:11	0.3	0.0	0.6	0.5
01Jan2000	05:12	0.3	0.0	0.5	0.5
01Jan2000	05:13	0.3	0.0	0.5	0.5
01Jan2000	05:14	0.3	0.0	0.5	0.5
01Jan2000	05:15	0.2	0.0	0.5	0.5
01Jan2000	05:16	0.2	0.0	0.4	0.4
01Jan2000	05:17	0.2	0.0	0.4	0.4
01Jan2000	05:18	0.2	0.0	0.4	0.4
01Jan2000	05:19	0.2	0.0	0.4	0.4
01Jan2000	05:20	0.2	0.0	0.3	0.4
01Jan2000	05:21	0.2	0.0	0.3	0.4
01Jan2000	05:22	0.2	0.0	0.3	0.3
01Jan2000	05:23	0.2	0.0	0.3	0.3
01Jan2000	05:24	0.2	0.0	0.3	0.3
01Jan2000	05:25	0.2	0.0	0.3	0.3
01Jan2000	05:26	0.2	0.0	0.3	0.3
01Jan2000	05:27	0.2	0.0	0.2	0.3
01Jan2000	05:28	0.2	0.0	0.2	0.3
01Jan2000	05:29	0.2	0.0	0.2	0.3
01Jan2000	05:30	0.2	0.0	0.2	0.3
01Jan2000	05:31	0.2	0.0	0.2	0.3
01Jan2000	05:32	0.2	0.0	0.2	0.3
01Jan2000	05:33	0.2	0.0	0.2	0.2
01Jan2000	05:34	0.2	0.0	0.2	0.2
01Jan2000	05:35	0.2	0.0	0.2	0.2
01Jan2000	05:36	0.2	0.0	0.2	0.2
01Jan2000	05:37	0.2	0.0	0.2	0.2
01Jan2000	05:38	0.2	0.0	0.2	0.2
01Jan2000	05:39	0.2	0.0	0.2	0.2
01Jan2000	05:40	0.2	0.0	0.2	0.2
01Jan2000	05:41	0.2	0.0	0.2	0.2
01Jan2000	05:42	0.2	0.0	0.2	0.2
01Jan2000	05:43	0.2	0.0	0.2	0.2
01Jan2000	05:44	0.2	0.0	0.2	0.2

Date	Time	Inflow (CFS)	Storage (AC-FT)	Elevation (FT)	Outflow (CFS)
01Jan2000	05:45	0.2	0.0	0.2	0.2
01Jan2000	05:46	0.2	0.0	0.2	0.2
01Jan2000	05:47	0.2	0.0	0.2	0.2
01Jan2000	05:48	0.2	0.0	0.2	0.2
01Jan2000	05:49	0.2	0.0	0.2	0.2
01Jan2000	05:50	0.2	0.0	0.2	0.2
01Jan2000	05:51	0.2	0.0	0.2	0.2
01Jan2000	05:52	0.2	0.0	0.2	0.2
01Jan2000	05:53	0.2	0.0	0.2	0.2
01Jan2000	05:54	0.2	0.0	0.2	0.2
01Jan2000	05:55	0.2	0.0	0.2	0.2
01Jan2000	05:56	0.2	0.0	0.2	0.2
01Jan2000	05:57	0.2	0.0	0.2	0.2
01Jan2000	05:58	0.2	0.0	0.2	0.2
01Jan2000	05:59	0.2	0.0	0.2	0.2
01Jan2000	06:00	0.2	0.0	0.2	0.2
01Jan2000	06:01	0.2	0.0	0.2	0.2
01Jan2000	06:02	0.2	0.0	0.2	0.2
01Jan2000	06:03	0.2	0.0	0.2	0.2
01Jan2000	06:04	0.2	0.0	0.2	0.2
01Jan2000	06:05	0.2	0.0	0.2	0.2
01Jan2000	06:06	0.1	0.0	0.2	0.2
01Jan2000	06:07	0.1	0.0	0.1	0.2
01Jan2000	06:08	0.1	0.0	0.1	0.2
01Jan2000	06:09	0.1	0.0	0.1	0.2
01Jan2000	06:10	0.0	0.0	0.1	0.1
01Jan2000	06:11	0.0	0.0	0.1	0.1
01Jan2000	06:12	0.0	0.0	0.1	0.1
01Jan2000	06:13	0.0	0.0	0.1	0.1
01Jan2000	06:14	0.0	0.0	0.1	0.1
01Jan2000	06:15	0.0	0.0	0.1	0.1
01Jan2000	06:16	0.0	0.0	0.0	0.1
01Jan2000	06:17	0.0	0.0	0.0	0.1
01Jan2000	06:18	0.0	0.0	0.0	0.0
01Jan2000	06:19	0.0	0.0	0.0	0.0

Date	Time	Inflow (CFS)	Storage (AC-FT)	Elevation (FT)	Outflow (CFS)
01Jan2000	06:20	0.0	0.0	0.0	0.0
01Jan2000	06:21	0.0	0.0	0.0	0.0
01Jan2000	06:22	0.0	0.0	0.0	0.0
01Jan2000	06:23	0.0	0.0	0.0	0.0
01Jan2000	06:24	0.0	0.0	0.0	0.0
01Jan2000	06:25	0.0	0.0	0.0	0.0
01Jan2000	06:26	0.0	0.0	0.0	0.0
01Jan2000	06:27	0.0	0.0	0.0	0.0
01Jan2000	06:28	0.0	0.0	0.0	0.0
01Jan2000	06:29	0.0	0.0	0.0	0.0
01Jan2000	06:30	0.0	0.0	0.0	0.0
01Jan2000	06:31	0.0	0.0	0.0	0.0
01Jan2000	06:32	0.0	0.0	0.0	0.0
01Jan2000	06:33	0.0	0.0	0.0	0.0
01Jan2000	06:34	0.0	0.0	0.0	0.0
01Jan2000	06:35	0.0	0.0	0.0	0.0
01Jan2000	06:36	0.0	0.0	0.0	0.0
01Jan2000	06:37	0.0	0.0	0.0	0.0
01Jan2000	06:38	0.0	0.0	0.0	0.0
01Jan2000	06:39	0.0	0.0	0.0	0.0
01Jan2000	06:40	0.0	0.0	0.0	0.0
01Jan2000	06:41	0.0	0.0	0.0	0.0
01Jan2000	06:42	0.0	0.0	0.0	0.0
01Jan2000	06:43	0.0	0.0	0.0	0.0
01Jan2000	06:44	0.0	0.0	0.0	0.0
01Jan2000	06:45	0.0	0.0	0.0	0.0
01Jan2000	06:46	0.0	0.0	0.0	0.0
01Jan2000	06:47	0.0	0.0	0.0	0.0
01Jan2000	06:48	0.0	0.0	0.0	0.0
01Jan2000	06:49	0.0	0.0	0.0	0.0
01Jan2000	06:50	0.0	0.0	0.0	0.0
01Jan2000	06:51	0.0	0.0	0.0	0.0
01Jan2000	06:52	0.0	0.0	0.0	0.0
01Jan2000	06:53	0.0	0.0	0.0	0.0
01Jan2000	06:54	0.0	0.0	0.0	0.0

Date	Time	Inflow (CFS)	Storage (AC-FT)	Elevation (FT)	Outflow (CFS)
01Jan2000	06:55	0.0	0.0	0.0	0.0
01Jan2000	06:56	0.0	0.0	0.0	0.0
01Jan2000	06:57	0.0	0.0	0.0	0.0
01Jan2000	06:58	0.0	0.0	0.0	0.0
01Jan2000	06:59	0.0	0.0	0.0	0.0
01Jan2000	07:00	0.0	0.0	0.0	0.0

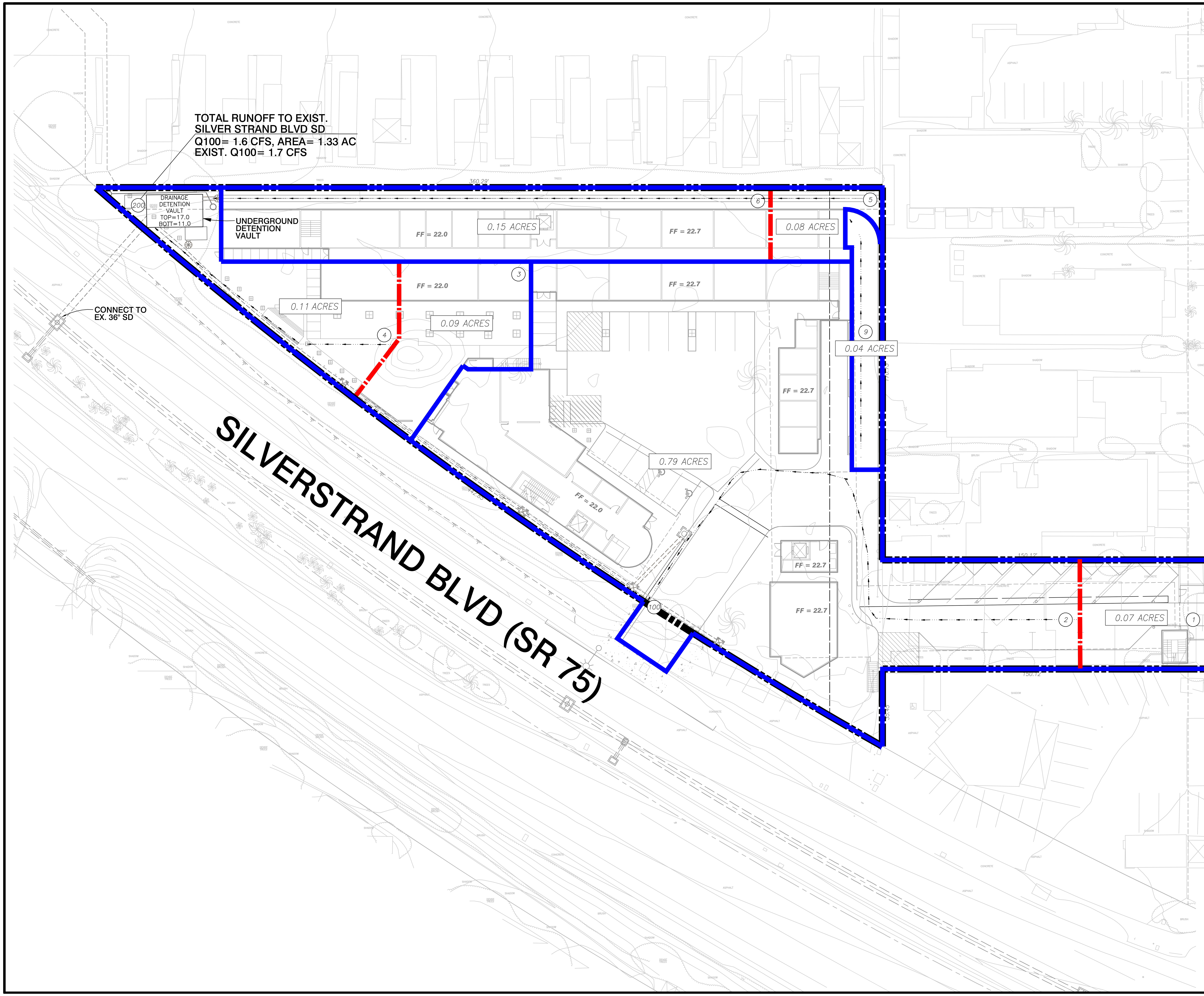
CHAPTER 6 - HYDROLOGY MAPS

EXISTING CONDITION HYDROLOGY EXHIBIT FOR:

BLUE WAVE MIXED USE

550 HIGHWAY 75
CITY OF IMPERIAL BEACH, CALIFORNIA

SHEET 1 OF 2



PROPOSED CONDITION HYDROLOGY EXHIBIT FOR:

BLUE WAVE MIXED USE

550 HIGHWAY 75
CITY OF IMPERIAL BEACH, CALIFORNIA

SHEET 2 OF 2