APPENDIX E

APPENDIX E-1:

PRELIMINARY REGIONAL DRAINAGE STUDY

OTAY RANCH VILLAGE 7
VILLAGE OF VISTA VERDE

Preliminary Drainage Study

August 2003
Revised October 2003
Revised April 9, 2004
Revised May 24, 2004

Prepared for:

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INTRODUCTION

This preliminary drainage study has been prepared in conjunction with the Specific Planning Area (SPA) and Tentative Map applications for Otay Ranch Village 7. Further details will be provided at the time the McMillin Otay Ranch Village 7 Final Grading Plans are prepared. The intent of this study is to establish general design procedures, preliminary flows and directions, and guidelines to be implemented with the project. This study identifies flows and major drainage facilities, addresses the issue of increased flows due to development, discusses the mitigation for the increased flows, and sets preliminary guidelines for subsequent phase(s) of the project.

PURPOSE

The purpose of this preliminary drainage study is to evaluate the approximate drainage patterns and flows as a result of the development of Village 7. This preliminary drainage study will identify major drainage facilities and identify the issue of increased runoff due to development and the required detention to mitigate these improvements. This study will draw conclusions that may be used in future design phases of this project. This drainage study is preliminary in nature, and therefore does not detail all related drainage facilities (i.e., peak runoff at each inlet, outlet, interceptor, and points of concentration or confluence). Future drainage reports will be prepared as required for the final engineering phase(s) of the project. In final design, the project will comply with all current regulations related to water quality, for best management practices (BMPs) during construction and postconstruction maintenance for the project.

BACKGROUND

General

The SPA area noted above is part of the Otay Ranch project. By reference, several reports have been reviewed in preparation for the Village 7 SPA analysis. These reports are as follows:

The Fogg Report (A Special Study of Storm Drain Facilities) by Lawrence, Fogg, Florer and Smith, on file at the City of Chula Vista; State Route 125 South Offsite Hydrology Report, by California Transportation Ventures (CTV), dated July 1994; Otay Ranch SPA Village 6 Preliminary Drainage Study Major Drainage Patterns and Facilities, by P&D Consultants, dated September 4, 2001; Drainage Study to Size Pipe Between E.U.C and Village 7, prepared by Rick Engineering Company, dated February 11, 2004; and Preliminary Water Quality Technical Report for Otay Ranch Village 7, by Rick Engineering Company, dated March 26, 2004. This preliminary drainage study has been prepared based on a review of these prior studies.

Village 7

Village 7 as part of the Otay Ranch General Development Plan area is located in the southeast portion of the City of Chula Vista. The site is approximately 4 miles north of the border with Mexico and 3.5 miles west of the Lower Otay Reservoir. Refer to Figure 1 for the project vicinity map.

Throughout the planning area, the landscape is predominantly rolling hills with arroyos draining to canyons that flow west and south away from the Otay Reservoir Basin and to the Otay River. The major drainage courses for Village 7 are comprised of several branches of Wolf Canyon, all of which flow to the west and then south to the Otay Valley. Several small unnamed arroyos flow directly south to the Otay River.

HYDROLOGICAL SETTING

The study area consists of rolling hills with arroyos draining into larger canyons flowing to the south and west, away from the Otay Reservoir Basin. The natural drainage basin for this portion of the Otay Ranch is a combination of one sub-basin flowing directly into Wolf Canyon, four sub-basins draining south into unnamed tributaries and eventually draining into Wolf Canyon, and one sub-basin draining north into Poggi Canyon. Wolf Canyon ultimately discharges into the Otay River approximately 1.6 miles southwest of the study area.

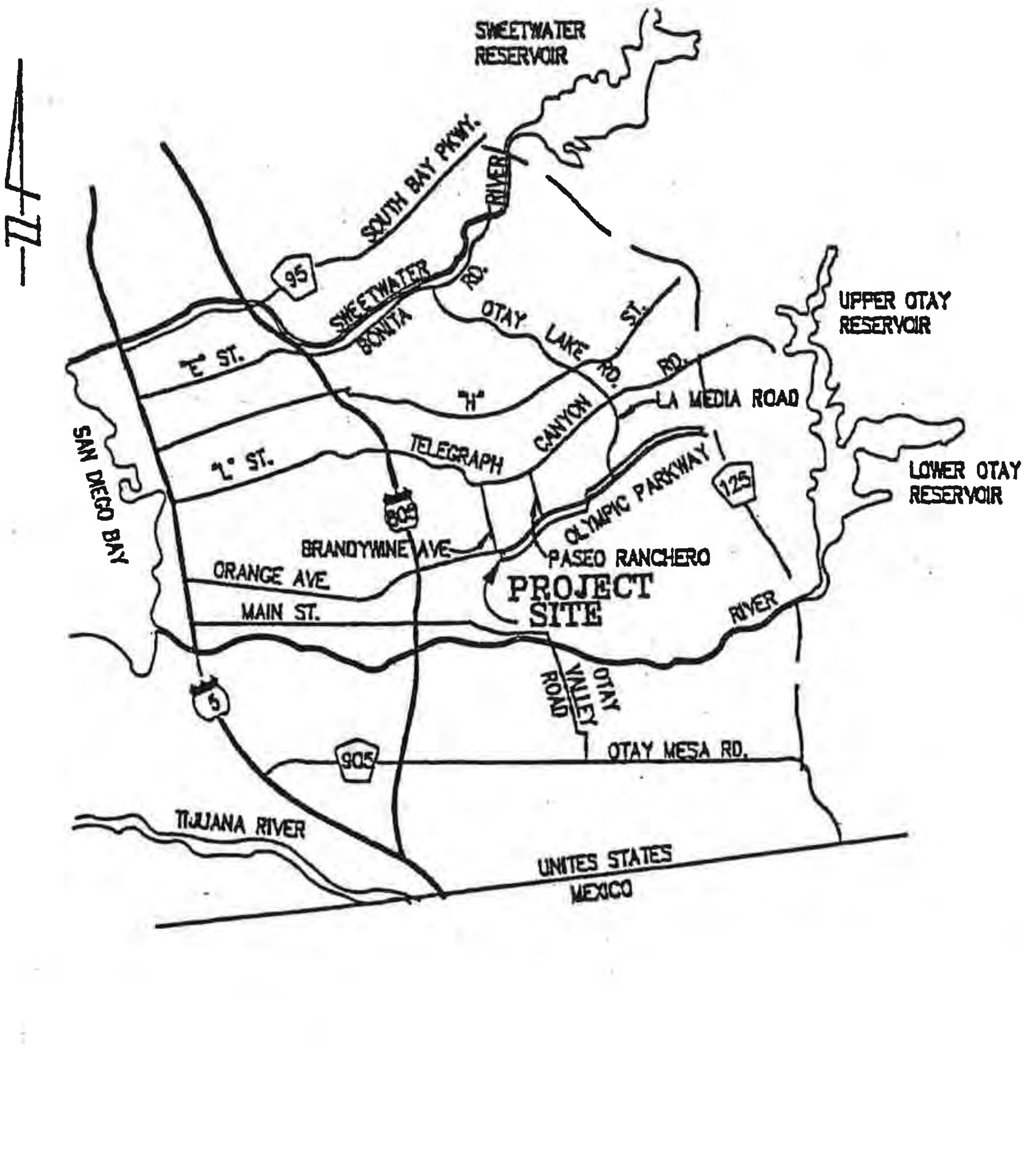

Figure 1. Project Vicinity Map

Immediately east of the Village 7 project and across the planned SR 125 alignment is located the future Eastern Urban Center (EUC), a part of the Village 12 SPA. The majority of the EUC is contained within the natural Wolf Canyon drainage basin, and will ultimately drain through a drainage structure under SR 125 and through Village 7. Refer to Figure 2 for the Otay Ranch GDP Site Utilization Plan.

In its existing condition, a portion of the future SR 125 right-of-way drains directly into the Wolf Canyon drainage basin within Village 7, and is included in the existing condition hydrological calculations. However, after construction of the SR 125, this flow will not be part of the ultimate runoff to the Wolf Canyon basin within Village 7.

Upon development of the Village 7 site, a portion of the existing Wolf Canyon area is planned as a naturalized open basin area. To limit the increased runoff to predevelopment levels, detention basins are proposed to be constructed. The naturalized open basin area will function as a drainage facility providing both detention and water quality treatment areas, integrated into an environmental and scenic feature. To avoid the construction of a series of small detention basins within the EUC, the Village 7 Wolf Canyon basin (between Magdalena Avenue and SR 125) will be sized to retain runoff from the EUC as well as Village 7.

HYDROLOGICAL PROCEDURE

The overall study area is divided into three major basins to analyze the flow patterns and needed drainage facilities. Basin A, the largest of the proposed drainage basins, is in the central portion of Village 7 and the EUC. This drainage basin includes Basin A-1 (EUC), Basin A-2 (Village 7), and the only proposed detention drainage facility for the study area.

For the flows from the EUC, this study accepted the drainage basins and drainage flows from the referenced report <u>Drainage Study to Size Pipe between E.U.C. and Village 7</u>. The developed 100-year storm discharge of 492 cubic feet per second (cfs) per the report was used as the developed flow from the EUC under the SR 125, and into the Village 7 project area.

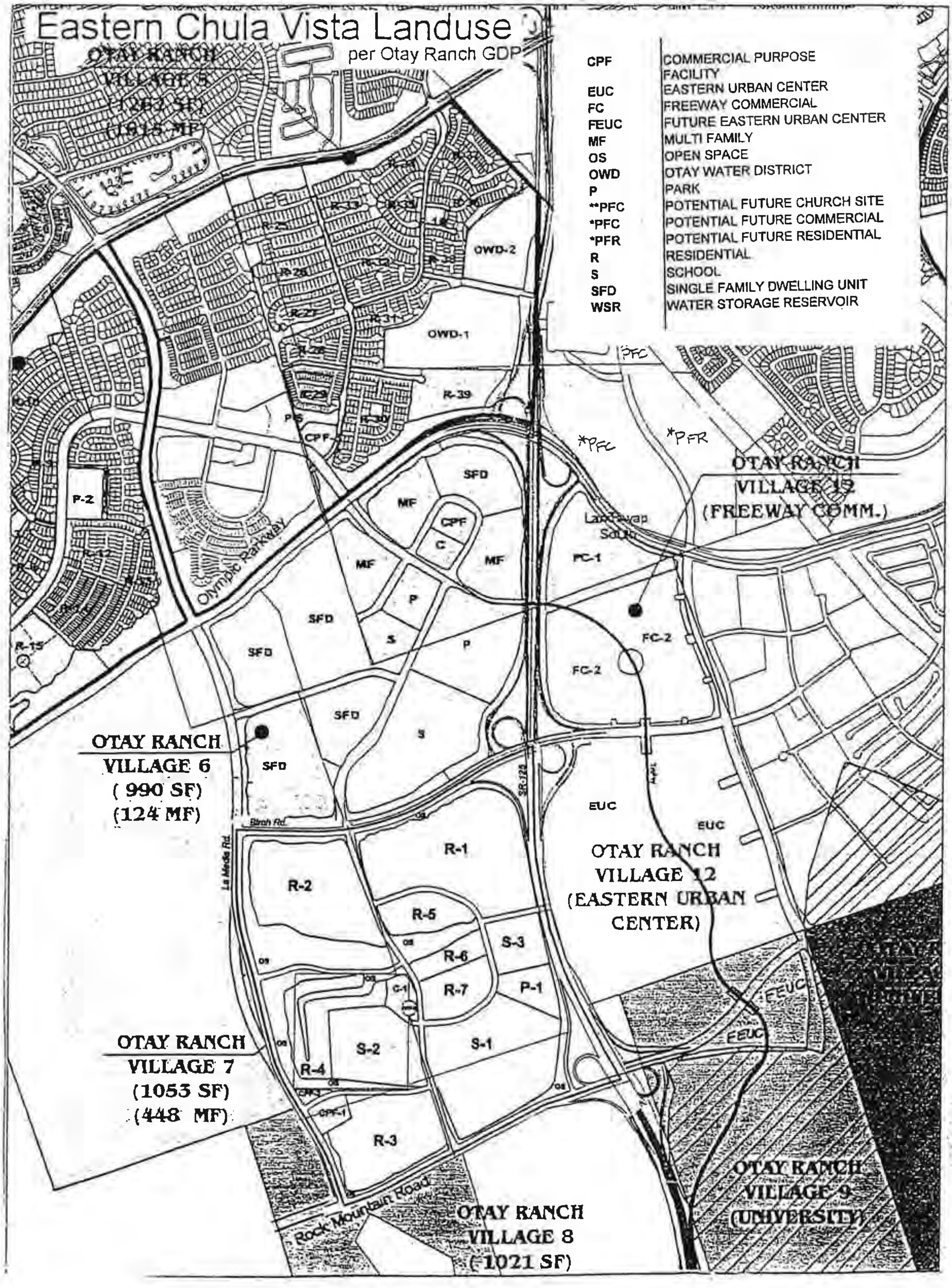

Figure 2. Otay Ranch General Development Plan (GDP) Site Utilization Plan

A small northerly portion of Village 7 drains north toward Birch Road into Poggi Canyon, which was included and identified in the referenced report Otay Ranch SPA Village 6 Preliminary Drainage Study Major Drainage Patterns and Facilities. This area, Basin C-1, is approximately 42 acres of the 300 acres of the drainage basin that drains to Poggi Canyon.

The southerly study area, Basin B-1, is a future school site for the Sweetwater Union High School District, and includes approximately 10 acres of natural area upstream prior to construction of the SR 125. Grading concepts for the Village 7 high school site are general in nature, however Basin B-1 will be modeled based on its intended use.

HYDROLOGICAL METHODOLOGY

The City of Chula Vista Subdivision Manual (Section 3-203) addresses the criteria for hydrologic calculations. This study has been prepared following the guidelines outlined in the City's Subdivision Manual and by the County of San Diego Department of Sanitation and Flood Control. Specifically, this study uses the Rational Method for the determination and routing of storm runoff from the drainage basins being analyzed. For this study, the 100-year storm event was analyzed.

Rational Method

The hydrologic software StormCad Version 5.5, by Haestad Methods, Inc., was used for calculation of developed storm flows. With the user input of the storm definition, current basin area, runoff coefficient, drainage path type, length, and elevation difference, the StormCad program calculates the amount of rainfall that will arrive at the outlet of the basin. Information for subsequent downstream basins is input in the same manner, until the entire basin has been taken into account. The program assesses the points of basin confluence, nodes, and estimates the pipe sizes, which are utilized in the master drainage study. At the time when a final drainage system is designed, a hydraulic grade line analysis will be prepared to accurately determine channel and pipe sizes.

HYDROLOGICAL RESULTS

The hydrological calculations show that there will be an increase in the 100-year storm runoff within all three major drainage basins, with development of Village 7 and the EUC. Refer to Table 1 for a summary of existing and developed runoffs for the various basins.

The main drainage course through Village 7 shall occur through Wolf Canyon. The offsite runoff from the EUC shall enter Village 7 on the east under the proposed SR 125. The total developed 100-year flow for Basin A is 714 cfs, which includes the offsite drainage from the EUC. The runoff from this basin will be routed through a series of detention areas to provide a maximum 100-year discharge of 243 cfs at the discharge point, equal to the existing pre-developed flow at the westerly boundary of the project. All of the storm runoff from Basin A will be routed through the detention areas, with the exception of a minor amount of runoff within Magdalena Avenue. This runoff will be routed directly to the west, bypassing the ponds. The amount of runoff from Magdalena Avenue is not considered significant and will not increase the total discharge above the pre-developed condition.

The northern portion of Village 7, Basin C-1, will drain through pipes in the local streets to Birch Road, westward to La Media Road, and onward to the Poggi Canyon drainage facilities. The total developed 100-year runoff for Basin C-1 is 87 cfs. Basin C-1 is the onsite portion of the larger drainage basin to the north of approximately 300 acres that drains into Poggi Canyon. This developed runoff has already been addressed in the report Otay Ranch Village 6 Preliminary Drainage Study Major Drainage Patterns and Facilities. Detention for the developed runoff will be provided in the Poggi Canyon regional detention facility.

The remaining drainage area, Basin B-1, shall flow from the site to the future southern extension of Magdalena Avenue and to Rock Mountain Road, ultimately flowing to the southern tributary of Wolf Canyon. This is a future school site and will include approximately 10 acres of natural area upstream of the developed school parcel until SR 125 is built. The runoff from the school site was modeled for ultimate conditions. The total 100-year developed runoff is 168 cfs. To mitigate the increase in the storm runoff for Basin B-1, detention within the school parcel will be provided as part of the school site development. A minor amount of runoff within Magdalena Avenue and Rock Mountain Road will drain directly west; however, this flow is not considered significant with regard to overall detention requirements for Basin B-1.

Table 1. Summary of Runoff Estimates for Village 7 – McMillin

Per Appendices A and B of This Report

Location	Existing Condition		Developed Condition	
	Area (Acres)	Q_{100} (cfs)	Area (Acres)	Q_{100} (cfs)
* Basin A-1 (EUC)	153	201	164	491
Basin A-2 North			45	84
Basin A-2 South			36	97
Basin A-2 Ponds			17	42
Subtotal Basin A-2	90	42	98	223
Total Basin A	243	243	262	714
Basin B-1	81	116	71	168
Basin C-1	59	83	42	87

* From the report titled Drainage Study to Size Pipe Between E.U.C. and Village 7.

Since it may be necessary for the developer to construct detention facilities for the future high school site, calculations to size the detention basin are included in this report. The hydrological software PondPack Version 9.0, by Haestad Methods, Inc., was used to estimate the total storage volume required to attenuate the developed 100-year discharge to the existing pre-developed flow. These calculations are included in Appendix C. The existing and developed runoffs (with detention) for Basin B-1 are summarized in Table 2 below.

Table 2. Summary of Basin B-1 Runoff Estimate with Detention

Per Appendix C of This Report

Location	Existing Condition		Developed Condition	
	Area (Acres)	Q_{100} (cfs)	Area (Acres)	Q_{100} (cfs)
Basin B-1	81	116	71	112

CONCLUSION

Based on the findings of this study, the development of Village 7 and the EUC will not adversely impact the existing natural drainage condition. The increased runoff due to the development will be mitigated by the proposed detention basins located within the Village 7 Wolf Canyon area. Refer to the report Preliminary Water Quality Technical Report for Otay Ranch Village 7, dated March 26, 2004 (and revisions thereto), for preliminary design calculations related to the planned detention basins. The future grading design for Village 7 will include preparation of a final drainage report in accordance with this preliminary study and the City of Chula Vista standards and general requirements.

The future onsite drainage facilities for Village 7 are not specifically addressed by this level of study. The onsite facilities consisting of storm drain pipes, inlets, cleanouts, headwalls, drainage ditches, etc. will be sized and designed during the final engineering and permitting phase of the project in accordance with City of Chula Vista standards. A detailed drainage report will be prepared at that time which will size all onsite drainage facilities to accommodate the ultimate storm flows.

Finally, the project will be required to comply with all current City and State regulations related to water quality, including best management practices (BMPs) during construction activity and post-construction maintenance of the project. For a preliminary discussion about proposed water quality treatment for the project, refer to the referenced report Preliminary Water Quality Technical Report for Otay Ranch Village 7. Both the final construction documents and associated reports will include details, notes, and discussions relative to the required or recommended BMPs.

APPENDIX A

EXISTING CONDITIONS
HYDROLOGICAL CALCULATIONS
Q_{100}

P&D Consultants of Nevada, Inc.
3930 Howard Hughes Parkway
Suite 270
Las Vegas, NV 89109
(702) 893-2455 • (702) 893-2495 Fax

P&D

By Date Client Sheet No. Of

Checked Date Job Job No.

EXISTING

BASIN A-1 $A = 153.0\ Ac$ $C = 0.50$

t_i $H_1 = 665$ $H_2 = 635$ $L_i = 650$

$S = \frac{\Delta H}{L} = 4.6\% \Rightarrow V = 2.2\ ft/sec$

$t_i = L_i / V = 4.9\ MIN$

$t_L =$ $H_1 = 635$ $H_2 = 520$ $L = 3250$

$= [11.9 L^3/\Delta H]^{.385} = 14.3\ MIN$

$T_c = t_i + t_L = 19.2\ MIN$

$Q_{100} = CiA$ $i = 7.44(2.38)(T_c^{-.645}) = 2.63$

$= 0.50(153)(2.63) = 201\ cfs$

BASIN A-2 $A = 153\ Ac + 90\ Ac = 243\ Ac$

$T_c = t_i + t_L$ $t_i = 19.2\ MIN$

$t_L = [11.9 L^3/\Delta H]^{.385}$ $H_1 = 520$ $H_2 = 430$

$L = 2247'$

$t_L = 10.3\ MIN$

$T_c = 19.2 + 10.3 = 29.5\ MIN$

$Q_{100} = CiA$ $i = 2.00$

$= 0.5(243\ Ac)(2.0) = 243\ cfs$

P&D Consultants of Nevada, Inc.
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By	Date	Client	Sheet No.	Of
Checked	Date	Job	Job No.	

BASIN B-1 $A = 80.5$ Ac $C = 0.50$

t_i : $H_1 = 625'$ $H_2 = 570$ $L_i = 1130$

$S = \frac{\Delta H}{L} = 4.9\%$ ⇒ $V = 2.3$ ft/sec

$t_i = L_i / V = 8.2$ min

$t_L =$ $H_1 = 570'$ $H_2 = 470'$ $L = 1965$

$t_L = (11.9 L^3/\Delta H)^{.385} = 8.4$ min

$T_c = 16.6$ MIN

$Q_{100} = 0.50(80.5)(i)$ $i = 7.44(2.38)(T_c^{-.645}) = 2.8$

$= 116.3$ cfs

BASIN C-1 $A = 59$ Ac $C = 0.50$

t_i $H_1 = 615$ $H_2 = 580$ $L_i = 600'$

$S = \frac{\Delta H}{L} = 5.8\%$ ⇒ $V = 2.5$ ft/sec

$t_i = L_i / V = 4.0$ min

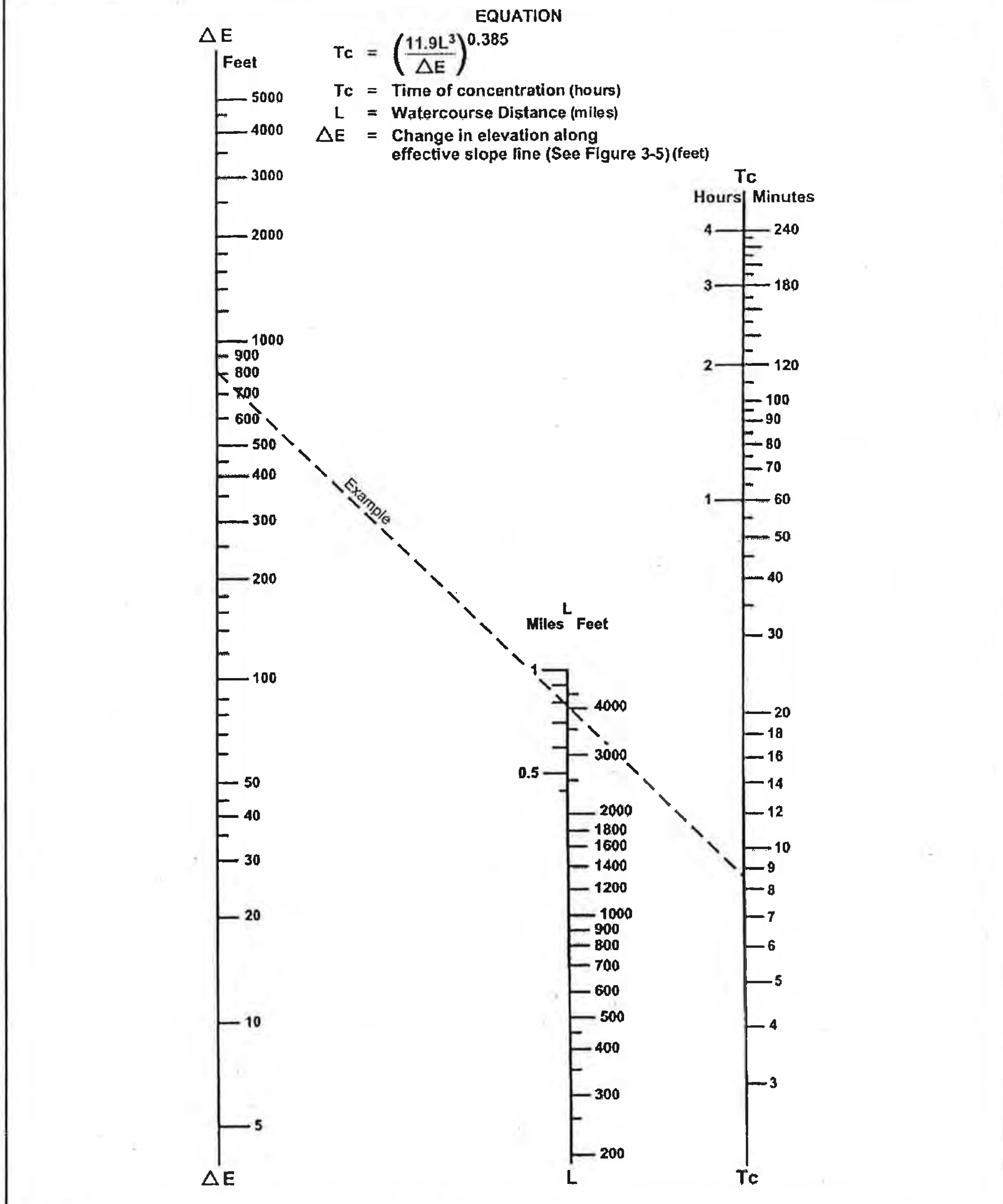
$t_L =$ $H_1 = 580'$ $H_2 = 485$ $L = 2000'$

$t_L = (11.9 L^3/\Delta H)^{.385} = 8.8$ min $+ 5$ min

$T_c = 17.8$ min

$Q_{100} = CIA$ $I = 7.44(2.38)(T_c^{-.645}) = 2.8$

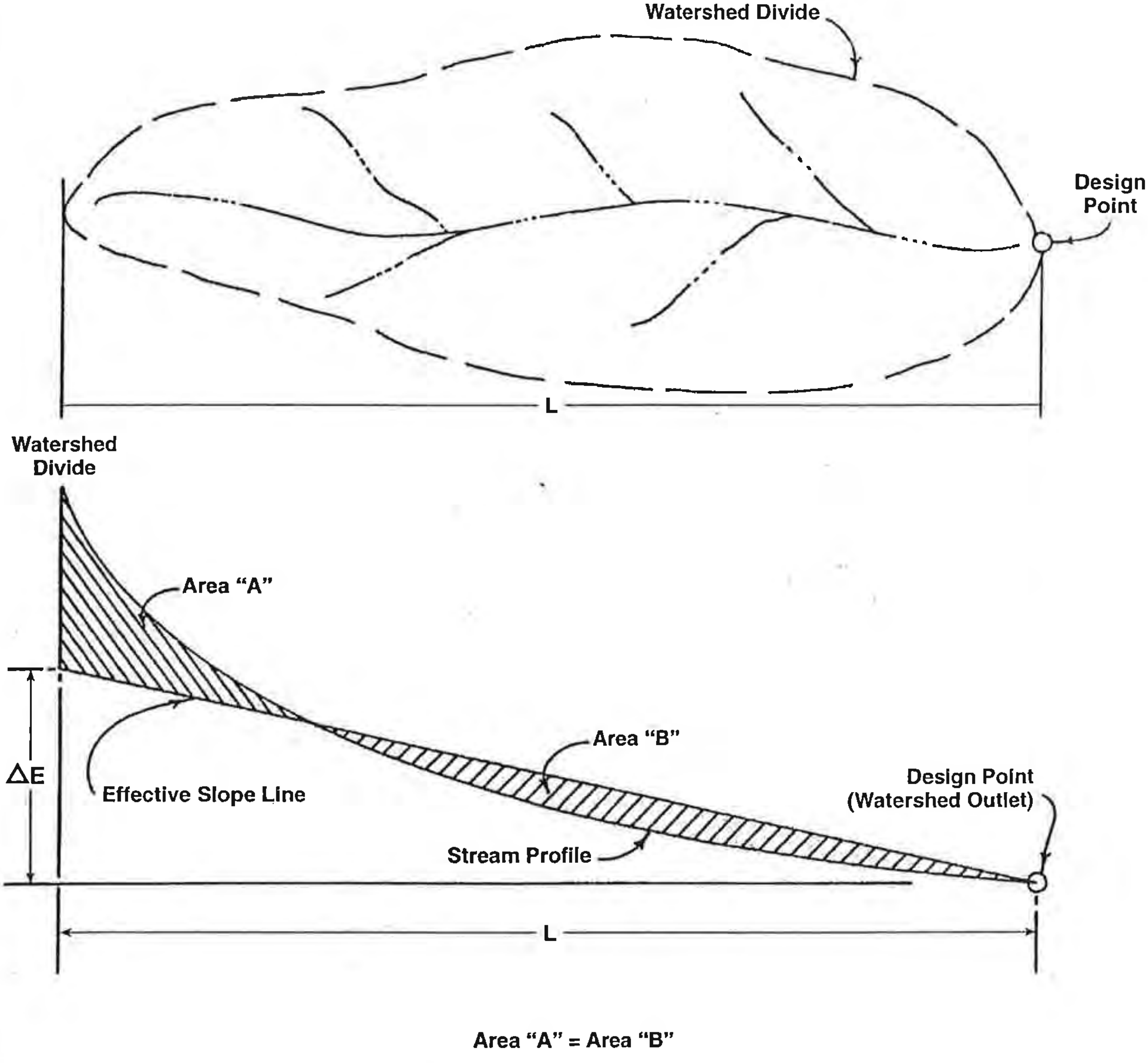
$= .5(59)(2.8) = 83$ cfs



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



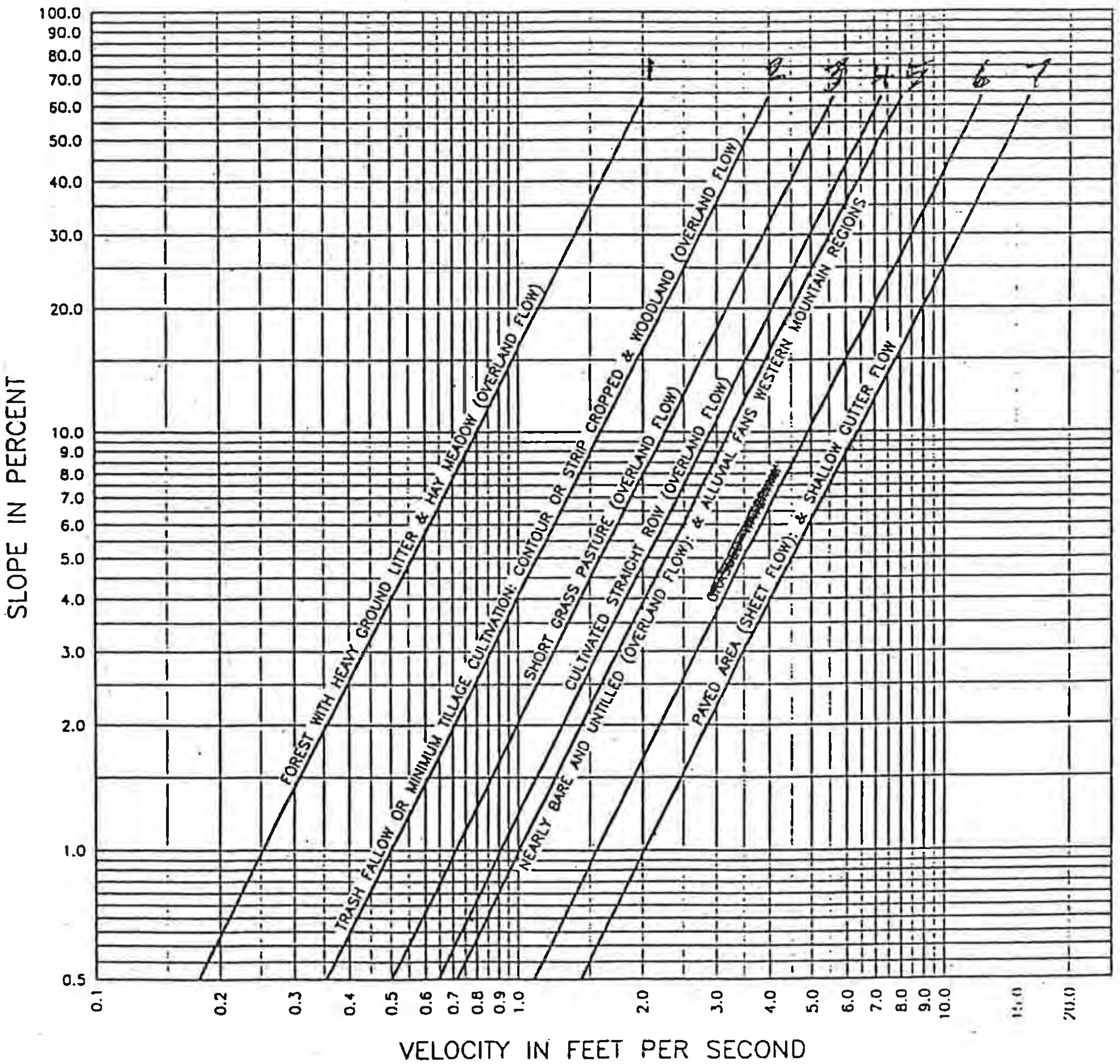
SOURCE: California Division of Highways (1941) and Kirpich (1940)

Computation of Effective Slope for Natural Watersheds

FIGURE

3-5

TRAVEL TIME VELOCITY



VERSION: December 2, 1996	REFERENCE: Soil Conservation Service, 1985 (Modified)	FIGURE 701

Project: McMillin Otay Ranch Village 7 Drainage Study
Job: 175221
Date: 4/9/04

50 Year, 6 hour precitation

$I = 7.44 P_6 D^{-0.645}$

$P_6 = 2.19$

	D	I
1)	5	5.8
2)	10	3.7
3)	15	2.8
4)	20	2.4
5)	25	2.0
6)	30	1.8
7)	40	1.5
8)	50	1.3
9)	60	1.2

100 Year, 6 hour precitation
-Used to SIZE SUMP INLETS and emergency spillways

$P_6 = 2.38$

	D	I
1)	5	6.3
2)	10	4.0
3)	15	3.1
4)	20	2.6
5)	25	2.2
6)	30	2.0
7)	40	1.6
8)	50	1.4
9)	60	1.3

Example calculations:

50 Year, P_6

$$I = 7.44(2.19)(5^{-0.645}) = 5.8$$

APPENDIX B

DEVELOPED CONDITIONS
HYDROLOGICAL CALCULATIONS
Q_{100}

Scenario: Base

Node Report

A-2 NORTH

Label	Area (acres)	Inlet C	Local Rational Flow (cfs)	Total Flow To Inlet (cfs)	Intercepted Rational Flow (cfs)	Diverted Flow Out (cfs)	Bypassed Rational Flow (cfs)	System Flow Time (min)	System Intensity (in/hr)	System CA (acres)	Total System Flow (cfs)
I-2	1.89	0.65	4.36	4.36	1.80	0.00	2.55	12.68	3.52	0.51	1.80
I-1	3.04	0.65	7.01	7.01	2.29	0.00	4.72	12.68	3.52	0.65	2.29
J-1						0.00		12.74	3.51	1.15	4.08
I-16	3.18	0.65	8.33	8.33	4.58	0.00	3.75	10.00	4.00	1.14	4.58
I-15	0.89	0.65	2.33	2.33	1.30	0.00	1.03	10.00	4.00	0.32	1.30
I-5	2.12	0.60	4.43	4.43	1.79	0.00	2.64	13.02	3.46	0.51	1.79
J-2						0.00		13.36	3.40	1.15	3.95
I-21	1.90	0.65	4.53	5.47	2.02	0.00	3.44	12.00	3.64	0.55	2.02
I-22	2.03	0.65	4.84	8.25	4.56	0.00	3.69	12.00	3.64	1.24	4.56
J-12						0.00		10.11	3.98	1.46	5.86
I-4	2.27	0.65	4.92	9.85	6.92	0.00	2.93	13.83	3.31	2.07	6.92
J-3						0.00		13.79	3.32	1.67	5.58
I-3	3.33	0.65	7.21	11.64	5.49	0.00	6.15	13.87	3.30	1.65	5.49
J-18						0.00		12.32	3.58	1.79	6.48
J-13						0.00		10.55	3.90	1.46	5.74
I-6	2.31	0.65	4.92	10.98	5.32	0.00	5.66	14.15	3.25	1.62	5.32
I-7	1.49	0.65	3.19	6.08	3.85	0.00	2.23	14.08	3.27	1.17	3.85
J-4						0.00		13.91	3.30	5.39	17.91
J-19						0.00		13.24	3.42	1.79	6.18
I-24	2.55	0.65	6.08	13.47	13.47	0.00	0.00	12.00	3.64	3.67	13.47
I-23	2.06	0.65	4.91	8.61	4.67	0.00	3.94	12.00	3.64	1.27	4.67
I-19	1.24	0.07	0.30	0.30	0.30	0.00	0.00	12.00	3.64	0.08	0.30
I-20	1.23	0.65	2.93	2.93	1.47	0.00	1.46	12.00	3.64	0.40	1.47
I-17	1.37	0.65	3.27	3.27	1.56	0.00	1.71	12.00	3.64	0.42	1.56
J-14						0.00		12.36	3.57	1.46	5.26
I-18	2.06	0.65	4.91	4.91	1.92	0.00	2.99	12.00	3.64	0.52	1.92
I-8	3.47	0.65	7.22	16.64	9.37	0.00	7.27	14.57	3.18	2.93	9.37
J-5						0.00		14.27	3.23	8.18	26.66
J-20						0.00		14.58	3.18	6.74	21.56
J-17						0.00		12.09	3.62	0.48	1.76
J-15						0.00		14.64	3.16	2.41	7.68
J-6						0.00		14.59	3.17	11.11	35.54
J-16						0.00		15.46	3.05	9.63	29.63
I-13	1.54	0.75	3.66	3.66	1.65	0.00	2.01	14.75	3.15	0.52	1.65
I-14	0.00	0.00	0.00	0.00	0.00	0.00	0.00	14.30	0.00	0.00	491.33
I-10	1.98	0.65	4.08	13.38	13.38	0.00	0.00	14.77	3.14	4.23	13.38
J-7						0.00		16.16	2.98	20.73	62.37
I-11	2.52	0.65	5.16	14.45	8.65	0.00	5.80	14.85	3.13	2.75	8.65
J-10						0.00		14.95	3.11	0.52	492.96
J-8						0.00		16.44	2.96	27.71	82.55
J-11						0.00		16.49	2.95	28.23	575.29
O-1						0.00		16.54	2.95	28.23	575.15

Scenario: Base

Node Report   A-2 SOUTH

Label	Area (acres)	Inlet C	Local Rational Flow (cfs)	Total Flow To Inlet (cfs)	Intercepted Rational Flow (cfs)	Diverted Flow Out (cfs)	Bypassed Rational Flow (cfs)	System Flow Time (min)	System Intensity (in/hr)	System CA (acres)	Total System Flow (cfs)
I-2	7.92	0.70	17.16	17.16	17.16	0.00	0.00	15.30	3.07	5.54	17.16
I-3	10.30	0.65	20.52	20.52	20.52	0.00	0.00	15.60	3.04	6.70	20.52
J-2						0.00		15.46	3.05	5.54	17.07
I-4	12.37	0.70	25.84	25.84	25.84	0.00	0.00	16.40	2.96	8.66	25.84
J-3						0.00		15.80	3.02	12.24	37.25
I-5	18.40	0.65	35.68	35.68	35.68	0.00	0.00	16.40	2.96	11.96	35.68
J-5						0.00		16.53	2.95	32.86	97.59
O-1						0.00		16.62	2.94	32.86	97.30

C

Scenario: Base

Node Report C-1

Label	Area (acres)	Inlet C	Local Rational Flow (cfs)	Total Flow To Inlet (cfs)	Intercepted Rational Flow (cfs)	Diverted Flow Out (cfs)	Bypassed Rational Flow (cfs)	System Flow Time (min)	System Intensity (in/hr)	System CA (acres)	Total System Flow (cfs)
I-12	3.64	0.60	8.01	8.01	4.49	0.00	3.53	12.00	3.64	1.22	4.49
J-9						0.00		12.03	3.63	1.22	4.48
I-8	3.44	0.65	7.60	13.43	8.30	0.00	5.13	13.50	3.37	2.44	8.30
I-9	3.60	0.65	7.95	11.21	5.38	0.00	5.83	13.50	3.37	1.58	5.38
I-14	1.56	0.56	3.21	3.21	1.54	0.00	1.66	12.00	3.64	0.42	1.54
J-10						0.00		12.23	3.60	1.22	4.44
I-13	2.70	0.65	6.44	8.10	4.51	0.00	3.59	12.00	3.64	1.23	4.51
J-5						0.00		13.57	3.36	4.03	13.63
J-11						0.00		12.65	3.52	2.87	10.20
J-6						0.00		13.71	3.33	4.03	13.53
I-11	2.70	0.65	6.44	10.03	5.07	0.00	4.96	12.00	3.64	1.38	5.07
I-10	0.00	0.00	0.00	8.58	4.66	0.00	3.92	0.00	6.30	0.73	4.66
I-5	2.70	0.65	6.35	16.00	8.45	0.00	3.37	12.27	3.59	2.33	11.44
I-4	2.40	0.65	5.70	5.70	2.07	0.00	3.64	12.08	3.63	0.57	2.07
I-16	1.87	0.65	4.02	6.42	2.19	0.00	4.23	14.00	3.28	0.66	2.19
I-15	1.97	0.65	4.00	4.00	1.73	0.00	2.27	15.00	3.10	0.55	1.73
J-12						0.00		13.36	3.40	2.87	9.84
J-7						0.00		14.09	3.26	6.14	20.20
J-3						0.00		12.29	3.59	2.90	13.48
J-14						0.00		15.06	3.09	1.22	3.79
J-8						0.00		14.14	3.25	9.02	29.57
I-2	2.88	0.65	7.14	12.02	7.79	0.00	4.24	11.20	3.78	2.04	7.79
J-16						0.00		12.42	3.56	2.90	13.41
J-15						0.00		15.18	3.08	1.22	3.78
J-13						0.00		14.80	3.14	9.02	28.49
I-3	2.11	0.65	5.23	7.59	4.35	0.00	3.23	11.20	3.78	1.14	4.35
J-2						0.00		16.95	2.91	16.31	50.77
I-18	1.19	0.65	2.56	6.23	2.16	0.00	4.07	14.00	3.28	0.65	2.16
I-17	1.43	0.65	3.07	17.50	11.26	0.00	5.05	14.00	3.28	3.40	12.08
J-17						0.00		17.09	2.89	20.37	63.18
J-18						0.00		17.20	2.88	20.37	62.96
I-19	1.06	0.70	2.72	2.72	2.72	0.00	0.00	12.00	3.64	0.74	2.72
I-20	9.26	0.70	20.25	25.40	25.03	0.00	0.00	15.00	3.10	8.01	25.40
J-19						0.00		17.60	2.84	29.13	87.56
O-1						0.00		17.67	2.83	29.13	87.35

P&D Consultants of Nevada, Inc.
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Suite 270
Las Vegas, NV 89109
(702) 893-2455 • (702) 893-2495 Fax

By Date Client Sheet No. Of

Checked Date Job Job No.

Developed

A-1 PER DRAINAGE STUDY E.V.C.

$A = 164$ Ac $Q_{100} = 491$ cfs

A-2 Prelim. STORMCAD

NORTH

$A = 45.2$ Ac $C = 0.65$

$Q_{100} = 575 - 491 = 84$ cfs

SOUTH

$A = 36$ Ac $Q_{100} = 97$ cfs

Ponds (RATIONAL w/ i determined from STORMCAD)

$A = 17$ Ac $i = 2.95$ $C = 0.85$

$Q_{100} = 42$ cfs

C-1

$A = 42$ Ac $C = 0.65$

$Q_{100} = 87$ cfs

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P&D

By	Date	Client	Sheet No.	Of
Checked	Date	Job	Job No.	

B-1 School Site $C = .70$

$A = 71$ Ac includes 11 Ac NATURAL UPSTREAM

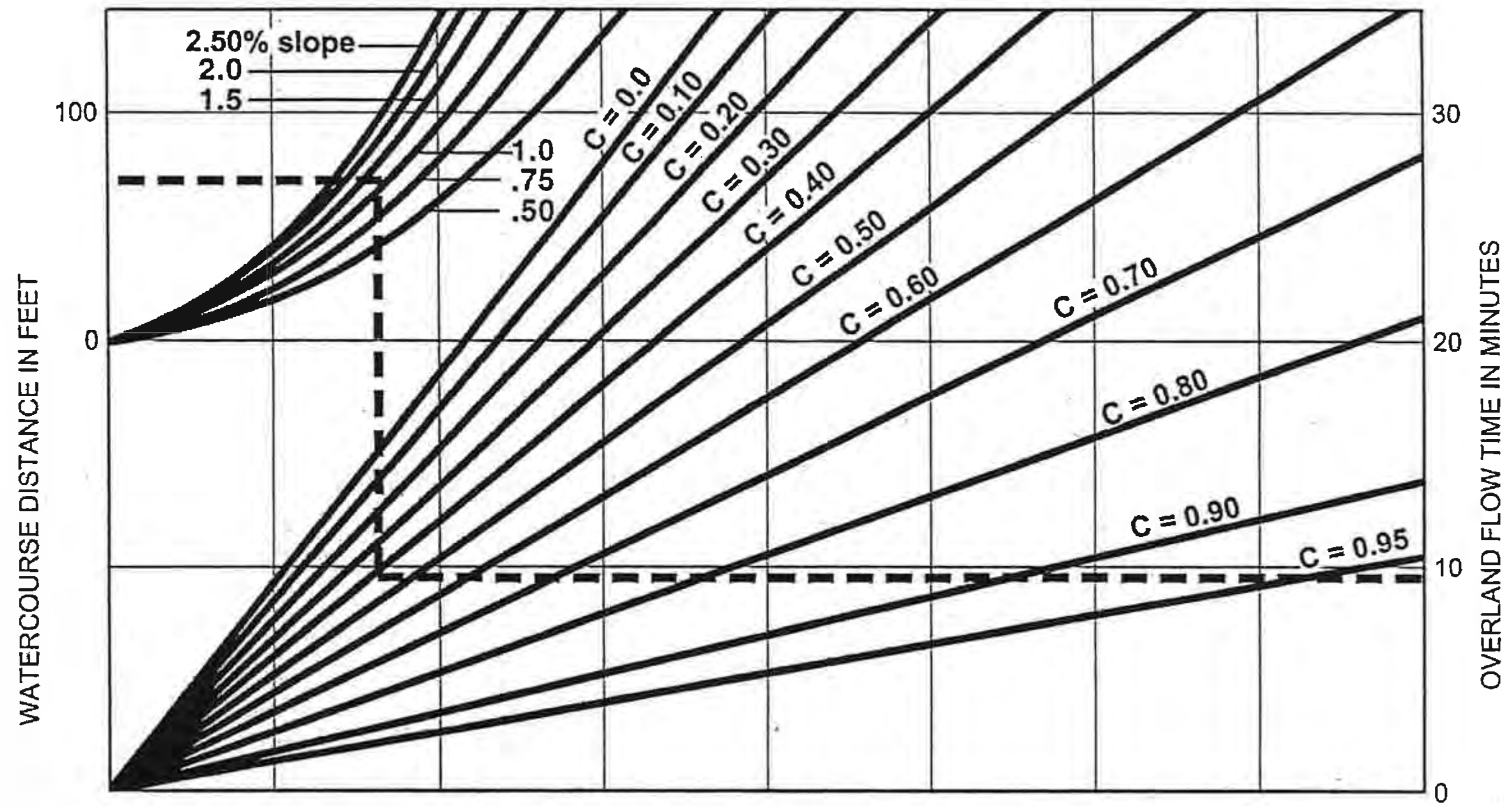
$L_1 = 500'$ $L_2 = 1750'$ $S_1 = 6\%$ $S_2 = 2\%$

$T_1 = \frac{500}{2.5} = 3.3$ min $T_2 = 1750/3.0 = 9.7$ min

$t_c = 13.0$ min

$i = 7.44(2.38)\, t_c^{-.645} = 3.39$

$Q_{100} = (71)(.70)(3.39) = 168.5$ cfs



EXAMPLE:

Given: Watercourse Distance (D) = 70 Feet
Slope (s) = 1.3%
Runoff Coefficient (C) = 0.41
Overland Flow Time (T) = 9.5 Minutes

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

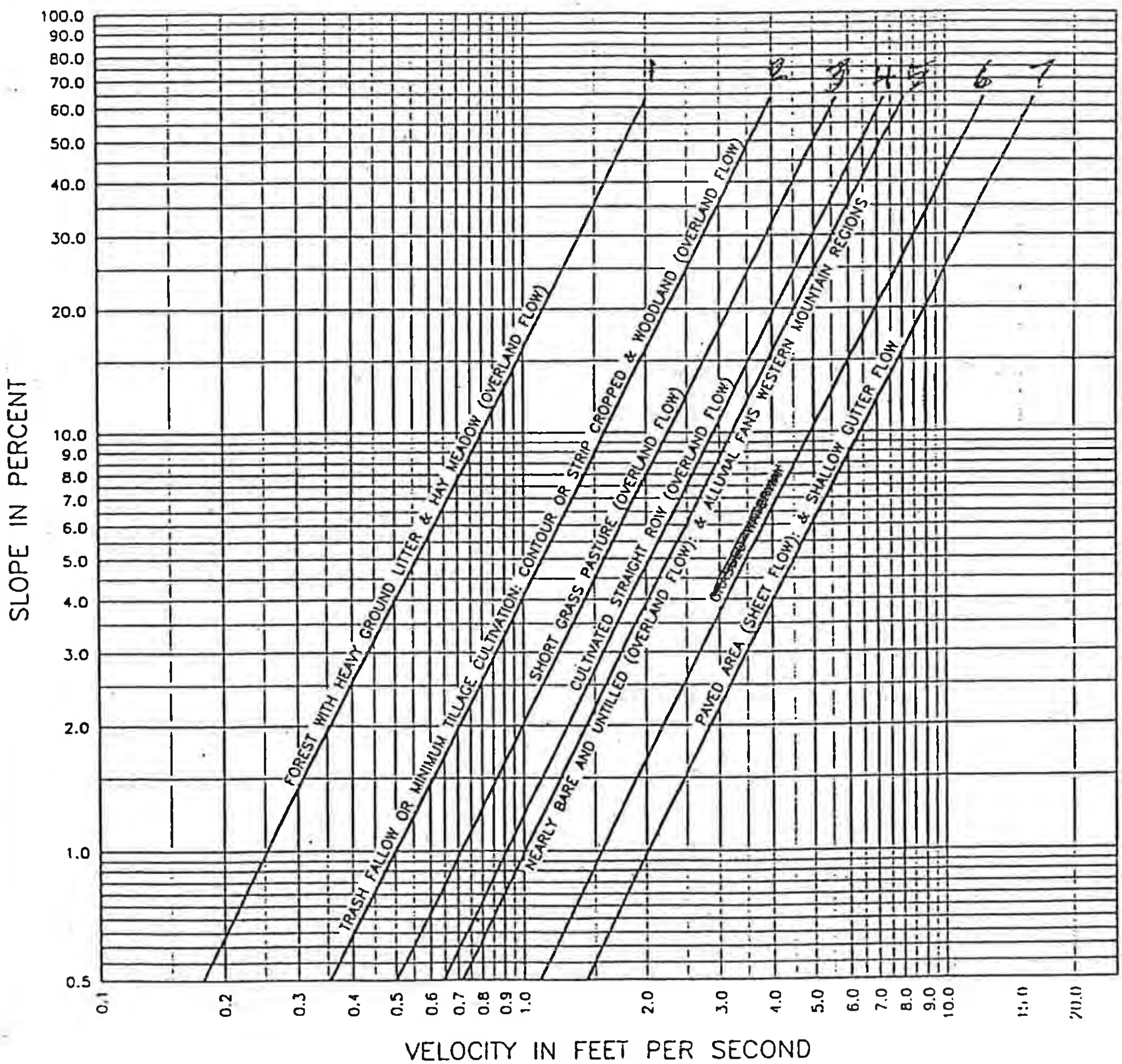
SOURCE: Airport Drainage, Federal Aviation Administration, 1965

Rational Formula - Overland Time of Flow Nomograph

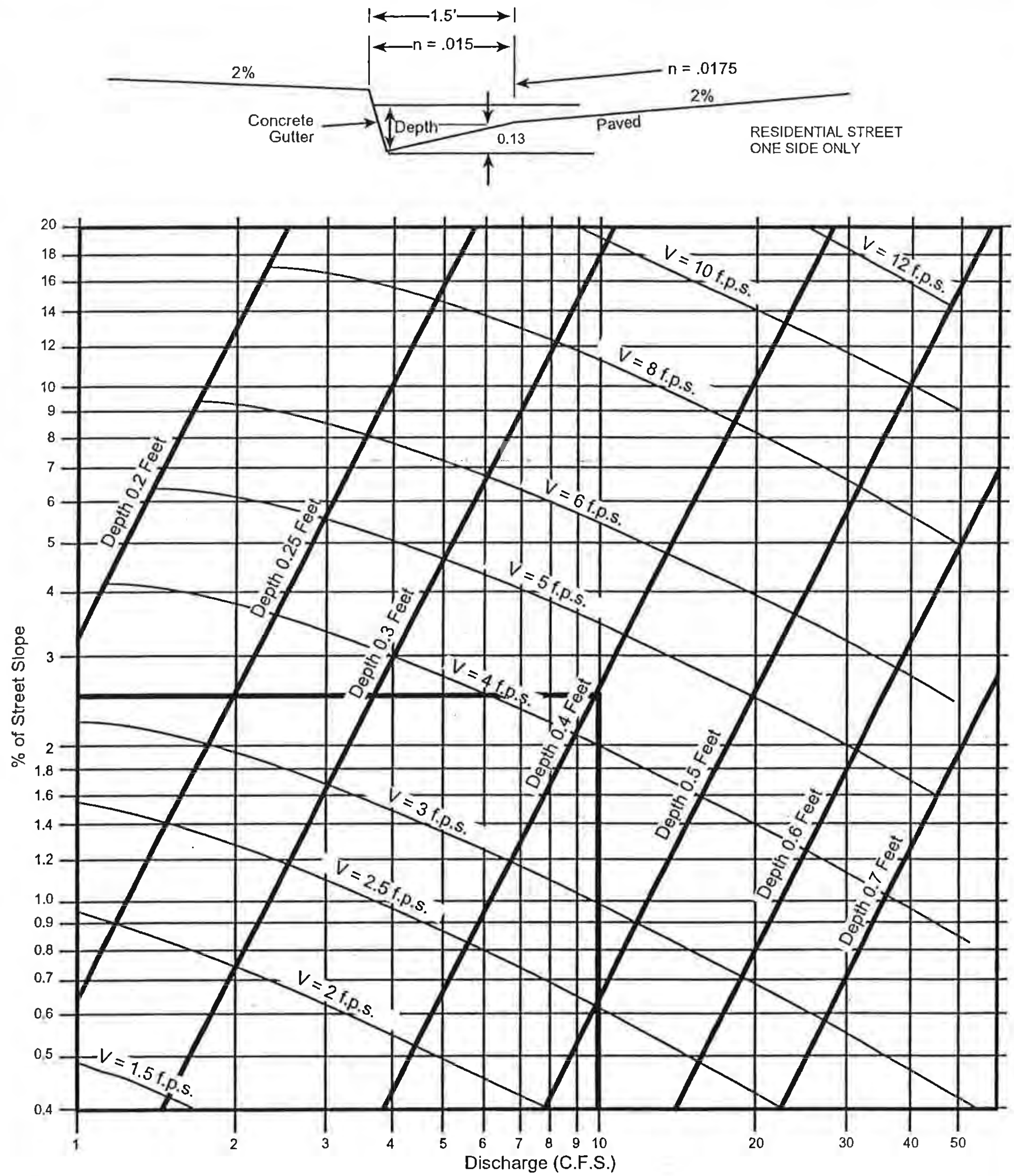
FIGURE

3-3

TRAVEL TIME VELOCITY



VERSION: December 2, 1996	REFERENCE: Soil Conservation Service, 1985 (Modified)	FIGURE 701



EXAMPLE:
Given: Q = 10 S = 2.5%
Chart gives: Depth = 0.4, Velocity = 4.4 f.p.s.

SOURCE: San Diego County Department of Special District Services Design Manual

FIGURE **3-6**

Gutter and Roadway Discharge - Velocity Chart

TABLE 2

RUNOFF COEFFICIENTS (RATIONAL METHOD)

DEVELOPED AREAS (URBAN)

Land Use	Coefficient, C Soil Group (1) A	B	C	D
Residential:				
Single Family	.40	.45	.50	.55
Multi-Units	.45	.50	.60	.70
Mobile homes	.45	.50	.55	.65
Rural (lots greater than 1/2 acre)	.30	.35	.40	.45
Commercial (2) 80% Impervious	.70	.75	.80	.85
Industrial (2) 90% Impervious	.80	.85	.90	.95

NOTES:

(1) Soil Group maps are available at the offices of the Department of Public Works.

(2) Where actual conditions deviate significantly from the tabulated imperviousness values of 80% or 90%, the values given for coefficient C, may be revised by multiplying 80% or 90% by the ratio of actual imperviousness to the tabulated imperviousness. However, in no case shall the final coefficient be less than 0.50. For example: Consider commercial property on D soil group.

Actual imperviousness = 50%

Tabulated imperviousness = 80%

$$\text{Revised } C = \frac{50}{80} \times 0.85 = 0.53$$

Project: McMillin Otay Ranch Village 7 Drainage Study
Job: 175221
Date: 4/9/04

50 Year, 6 hour Precitation

$I = 7.44 P_6 D^{-0.645}$

$P_6 = 2.19$

	D	I
1)	5	5.8
2)	10	3.7
3)	15	2.8
4)	20	2.4
5)	25	2.0
6)	30	1.8
7)	40	1.5
8)	50	1.3
9)	60	1.2

100 Year, 6 hour Precitation
- Used to SIZE SUMP INLETS and emergency spillways

$P_6 = 2.38$

	D	I
1)	5	6.3
2)	10	4.0
3)	15	3.1
4)	20	2.6
5)	25	2.2
6)	30	2.0
7)	40	1.6
8)	50	1.4
9)	60	1.3

Example calculations:

50 Year, P_6

$$I = 7.44(2.19)(5^{-0.645}) = 5.8$$

APPENDIX C

BASIN B-1
DETENTION BASIN CALCULATIONS
Q_{100}

Table of Contents (continued)

Table of Contents (continued)

Village7 school1

	Mod. Rational Storm Calcs	10.04
SUBAREA 1.......	POST	
	C and Area	10.05
SUBAREA 1.......	CV100	
	Rational Predev. Peak Q	10.06
	Mod. Rational Graph	10.07
	Mod. Rational Storm Calcs	10.08

S/N:
PondPack Ver: Compute Time: Date:

Type.... WARNING MESSAGES Page 1.01
Name.... WARNING Event: 100 yr
File.... C:\Program Files\Haestad\PPKW\Sample\VILLAGE7 SCHOOL1.PPW

WARNING: Missed peak when adding hydrograph...
Check output for: Node: Pond Inflow Summary POND 1 IN

WARNING: Missed peak when adding hydrograph...
Check output for: Node: Pond Inflow Summary POND 1 IN

S/N: F21601F07098 AECOM
PondPack Ver: Compute Time: Date:

Type.... Master Network Summary Page 2.01
Name.... Watershed
File.... C:\Program Files\Haestad\PPKW\Sample\VILLAGE7 SCHOOL1.PPW

MASTER DESIGN STORM SUMMARY

Default Network Design Storm File, ID Chula Vista

Return Event	Rainfall Type	IDF ID
CV100	I-D-F Curve	Chula Vista 100y
CV50	I-D-F Curve	Chula Vista 50yr

MASTER NETWORK SUMMARY
Modified Rational Method Network

(*Node=Outfall; +Node=Diversion;)
(Trun= HYG Truncation: Blank=None; L=Left; R=Rt; LR=Left&Rt)

Node ID	Type	Return Event	HYG Vol ac-ft	Trun	Qpeak hrs	Qpeak cfs	Max WSEL ft	Max Pond Storage ac-ft
*OUT 1	JCT	100	3.139		.3060	111.61		
*OUT 1	JCT	50	2.859		.3230	91.20		

```
POND 1        IN  POND    100      3.141          .2210     165.84
POND 1        IN  POND     50      2.861          .2210     151.09

POND 1        OUT POND    100      3.139          .3060     111.61     548.40
1.646
POND 1        OUT POND     50      2.859          .3230      91.20     548.26
1.586

SUBAREA 1         AREA    100      3.142          .2251     168.88
SUBAREA 1         AREA     50      2.863          .2251     153.85
```

S/N: F21601F07098 AECOM
PondPack Ver: Compute Time: Date:

```
Type.... Executive Summary (Nodes)                           Page 3.01
Name.... Watershed                                         Event: 50 yr
File.... C:\Program Files\Haestad\PPKW\Sample\VILLAGE7 SCHOOL1.PPW
Storm... Chula Vista 50yr   Tag: CV50
```

NETWORK SUMMARY -- NODES
(Trun.= HYG Truncation: Blank=None; L=Left; R=Rt; LR=Left & Rt)

```
DEFAULT Design Storm File,ID =


Storm Tag Name       =
------------------------------------------------------------------
Data Type, File, ID =
Total Rainfall Depth=  .0000 in
Duration Multiplier =  0
Resulting Duration  = .0000 hrs
Resulting Start Time= .0000 hrs  Step= .0000 hrs  End= .0000 hrs
```

	Node ID		Type	HYG Vol ac-ft	Trun.	Qpeak hrs	Qpeak cfs	Max WSEL ft
Outfall	OUT 1		JCT	2.859		.3230	91.20	
	POND 1	IN	POND	2.861		.2210	151.09	
	POND 1	OUT	POND	2.859		.3230	91.20	548.26
	SUBAREA 1		AREA	2.863		.2251	153.85	

S/N: F21601F07098 AECOM
PondPack Ver: Compute Time: Date:

```
Type.... Executive Summary (Nodes)                           Page 3.02
Name.... Watershed                                        Event: 100 yr
File.... C:\Program Files\Haestad\PPKW\Sample\VILLAGE7 SCHOOL1.PPW
Storm... Chula Vista 100y   Tag: CV100
```

NETWORK SUMMARY -- NODES
(Trun.= HYG Truncation: Blank=None; L=Left; R=Rt; LR=Left & Rt)

```
DEFAULT Design Storm File,ID =


Storm Tag Name       =
------------------------------------------------------------------
Data Type, File, ID =
Total Rainfall Depth=  .0000 in
Duration Multiplier =  0
Resulting Duration  = .0000 hrs
```

Village7 school1
Resulting Start Time= .0000 hrs Step= .0000 hrs End= .0000 hrs

	Node ID		Type	HYG Vol ac-ft	Trun.	Qpeak hrs	Qpeak cfs	Max WSEL ft
Outfall	OUT 1		JCT	3.139		.3060	111.61	
	POND 1	IN	POND	3.141		.2210	165.84	
	POND 1	OUT	POND	3.139		.3060	111.61	548.40
	SUBAREA 1		AREA	3.142		.2251	168.88	

S/N: F21601F07098 AECOM
PondPack Ver: Compute Time: Date:

Type.... Rational Storms Page 4.01
Name.... Chula Vista

File.... C:\Program Files\Haestad\PPKW\Sample\
Title... Project Date: 4/5/2004
Project Engineer: WOOTEN
Project Title: Rational Pond
Project Comments:

I-D-F DESIGN STORM SUMMARY

Storm Queue File,ID = Chula Vista

Storm Tag Name = CV100

File: Type, ID = : I-D-F Storm... Chula Vista 100y
Storm Frequ. = 100 yr

Storm Tag Name = CV50

File: Type, ID = : I-D-F Storm... Chula Vista 50yr
Storm Frequ. = 50 yr

S/N: F21601F07098 AECOM
PondPack Ver: Compute Time: Date:

Type.... I-D-F Table Page 5.01
Name.... Chula Vista 100y Tag: CV100 Event: 100 yr
File.... C:\Program Files\Haestad\PPKW\Sample\
Storm... Chula Vista 100y Tag: CV100

Rainfall-Intensity-Duration Curve

Time, hrs	Intens., in/hr
.0330	11.3200
.0830	6.3300
.1670	4.0000
.2500	3.1000
.3330	2.6000
.4170	2.2000
.5000	2.0000
.6670	1.6000
.8330	1.4000
1.0000	1.3000
6.0000	.4000

Village7 school1

S/N: F21601F07098 AECOM
PondPack Ver: Compute Time: Date:

Type.... I-D-F Table Page 5.02
Name.... Chula Vista 50yr Tag: CV50 Event: 50 yr
File.... C:\Program Files\Haestad\PPKW\Sample\
Storm... Chula Vista 50yr Tag: CV50

Rainfall-Intensity-Duration Curve

Time, hrs	Intens., in/hr
.0330	10.4200
.0830	5.8000
.1670	3.7000
.2500	2.8000
.3330	2.4000
.4170	2.0000
.5000	1.8000
.6670	1.5000
.8330	1.3000
1.0000	1.2000
6.0000	.3700

S/N: F21601F07098 AECOM
PondPack Ver: Compute Time: Date:

Type.... Tc Calcs Page 6.01
Name.... SUBAREA 1 Tag: PRE

File.... C:\Program Files\Haestad\PPKW\Sample\VILLAGE7 SCHOOL1.PPW

TIME OF CONCENTRATION CALCULATOR

Segment #1: Tc: User Defined

Segment #1 Time: .2900 hrs

Total Tc: .2900 hrs

S/N: F21601F07098 AECOM
PondPack Ver: Compute Time: Date:

Type.... Tc Calcs Page 6.02
Name.... SUBAREA 1 Tag: PRE

File.... C:\Program Files\Haestad\PPKW\Sample\VILLAGE7 SCHOOL1.PPW

Tc Equations used...

==== User Defined ==

Tc = Value entered by user

Where: Tc = Time of concentration

S/N: F21601F07098   AECOM
PondPack Ver: Compute Time: Date:

Type.... Tc Calcs   Page 6.03
Name.... SUBAREA 1   Tag: POST

File.... C:\Program Files\Haestad\PPKW\Sample\VILLAGE7 SCHOOL1.PPW

::
TIME OF CONCENTRATION CALCULATOR
::

Segment #1: Tc: User Defined

Segment #1 Time: .2250 hrs

========================
Total Tc: .2250 hrs
========================

S/N: F21601F07098   AECOM
PondPack Ver: Compute Time: Date:

Type.... Tc Calcs   Page 6.04
Name.... SUBAREA 1   Tag: POST

File.... C:\Program Files\Haestad\PPKW\Sample\VILLAGE7 SCHOOL1.PPW

Tc Equations used...

==== User Defined ==

Tc = Value entered by user

Where: Tc = Time of concentration

S/N: F21601F07098   AECOM
PondPack Ver: Compute Time: Date:

Type.... Vol: Elev-Area   Page 7.01
Name.... POND 1

File.... C:\Program Files\Haestad\PPKW\Sample\VILLAGE7 SCHOOL1.PPW

Elevation (ft)	Planimeter (sq.in)	Area (acres)	A1+A2+sqr(A1*A2) (acres)	Volume (ac-ft)	Volume Sum (ac-ft)

544.00	-----	.3200	.0000	.000	.000
546.00	-----	.3700	1.0341	.689	.689
550.00	-----	.4700	1.2570	1.676	2.365

POND VOLUME EQUATIONS

* Incremental volume computed by the Conic Method for Reservoir Volumes.

Volume = (1/3) * (EL2-EL1) * (Area1 + Area2 + sq.rt.(Area1*Area2))

where: EL1, EL2 = Lower and upper elevations of the increment
Area1,Area2 = Areas computed for EL1, EL2, respectively
Volume = Incremental volume between EL1 and EL2

S/N: F21601F07098 AECOM
PondPack Ver: Compute Time: Date:

Type.... Outlet Input Data Page 8.01
Name.... Outlet 3

File.... C:\Program Files\Haestad\PPKW\Sample\VILLAGE7 SCHOOL1.PPW

REQUESTED POND WS ELEVATIONS:

Min. Elev.= 544.00 ft
Increment = .10 ft
Max. Elev.= 550.00 ft

****
OUTLET CONNECTIVITY
****

---> Forward Flow Only (UpStream to DnStream)
<--- Reverse Flow Only (DnStream to UpStream)
<---> Forward and Reverse Both Allowed

Structure	No.		Outfall	E1, ft	E2, ft
Weir-Rectangular	W3	--->	TW	548.000	550.000
Culvert-Circular	C3	--->	TW	544.000	550.000
TW SETUP, DS Channel					

S/N: F21601F07098 AECOM
PondPack Ver: Compute Time: Date:

Type.... Outlet Input Data Page 8.02
Name.... Outlet 3

File.... C:\Program Files\Haestad\PPKW\Sample\VILLAGE7 SCHOOL1.PPW

OUTLET STRUCTURE INPUT DATA

Structure ID = W3
Structure Type = Weir-Rectangular

\# of Openings = 1

```
                      Village7 school1
        Crest Elev.         =    548.00 ft
        Weir Length         =     50.00 ft
        Weir Coeff.         =  2.800000

        Weir TW effects       (Use adjustment equation)
```



```
              OUTLET STRUCTURE INPUT DATA


              Structure ID        = C3
              Structure Type      = Culvert-Circular
              ------------------------------------
              No. Barrels         =        1
              Barrel Diameter     =    4.0000 ft
              Upstream Invert     =    544.00 ft
              Dnstream Invert     =    470.00 ft
              Horiz. Length       =    200.00 ft
              Barrel Length       =    213.25 ft
              Barrel Slope        =    .37000 ft/ft

              OUTLET CONTROL DATA...
              Mannings n          =     .0130
              Ke                  =     .5000  (forward entrance loss)
              Kb                  =   .004925  (per ft of full flow)
              Kr                  =     .5000  (reverse entrance loss)
              HW Convergence      =      .010  +/- ft

              INLET CONTROL DATA...
              Equation form       =        1
              Inlet Control K     =     .0098
              Inlet Control M     =     .0398
              Inlet Control c     =    .03980
              Inlet Control Y     =     .6700
              T1 ratio (HW/D)     =      .865
              T2 ratio (HW/D)     =     1.122
              Slope Factor        =     -.500

 Use unsubmerged inlet control Form 1 equ. below T1 elev.
 Use   submerged inlet control Form 1 equ. above T2 elev.

 In transition zone between unsubmerged and submerged inlet control,
 interpolate between flows at T1 & T2...
 At T1 Elev =     547.46 ft  ---> Flow =     87.96 cfs
 At T2 Elev =     548.49 ft  ---> Flow =    100.53 cfs


              Structure ID        = TW
              Structure Type      = TW SETUP, DS Channel
              ------------------------------------
              FREE OUTFALL CONDITIONS SPECIFIED

              CONVERGENCE TOLERANCES...
```

Village7 school1

```
Maximum Iterations=    30
Min. TW tolerance =    .10 ft
Max. TW tolerance =    .10 ft
Min. HW tolerance =    .10 ft
Max. HW tolerance =    .10 ft
Min.  Q tolerance =    .10 cfs
Max.  Q tolerance =    .10 cfs
```

S/N: F21601F07098   AECOM
PondPack Ver: Compute Time: Date:

```
Type.... Node: Pond Inflow Summary                              Page 9.01
Name.... POND 1       IN                                      Event: 50 yr
File.... C:\Program Files\Haestad\PPKW\Sample\VILLAGE7 SCHOOL1.PPW
Storm... Chula Vista 50yr   Tag: CV50
```

SUMMARY FOR HYDROGRAPH ADDITION
at Node: POND 1   IN

HYG Directory: C:\Program Files\Haestad\PPKW\Sample\

Upstream Link ID	Upstream Node ID	HYG file	HYG ID	HYG tag
WARNING: Missed peak when adding hydrograph...				
LINK 1	SUBAREA 1		SUBAREA 1	CV50

INFLOWS TO: POND 1   IN

HYG file	HYG ID	HYG tag	Volume ac-ft	Peak Time hrs	Peak Flow cfs
	SUBAREA 1	CV50	2.863	.2251	153.85

TOTAL FLOW INTO: POND 1   IN

HYG file	HYG ID	HYG tag	Volume ac-ft	Peak Time hrs	Peak Flow cfs
	POND 1 IN	CV50	2.861	.2210	151.09

S/N: F21601F07098   AECOM
PondPack Ver: Compute Time: Date:

```
Type.... Node: Pond Inflow Summary                              Page 9.02
Name.... POND 1       IN                                      Event: 50 yr
File.... C:\Program Files\Haestad\PPKW\Sample\VILLAGE7 SCHOOL1.PPW
Storm... Chula Vista 50yr   Tag: CV50
```

```
TOTAL NODE INFLOW...
HYG file =
HYG ID   = POND 1       IN
HYG Tag  = CV50
-----------------------------------
Peak Discharge =      151.09 cfs
Time to Peak   =       .2210 hrs
HYG Volume     =       2.861 ac-ft
-----------------------------------
```

HYDROGRAPH ORDINATES (cfs)
Time | Output Time increment = .0170 hrs

Village7 school1

hrs	Time on left represents time for first value in each row.				
.0000	.00	11.63	23.26	34.89	46.51
.0850	58.14	69.77	81.40	93.03	104.66
.1700	116.28	127.91	139.54	151.09	145.01
.2550	133.38	121.76	110.13	98.50	86.87
.3400	75.24	63.61	51.99	40.36	28.73
.4250	17.10	5.47	.64		

S/N: F21601F07098 AECOM
PondPack Ver: Compute Time: Date:

Type.... Node: Pond Inflow Summary Page 9.03
Name.... POND 1 IN Event: 100 yr
File.... C:\Program Files\Haestad\PPKW\Sample\VILLAGE7 SCHOOL1.PPW
Storm... Chula Vista 100y Tag: CV100

SUMMARY FOR HYDROGRAPH ADDITION
at Node: POND 1 IN

HYG Directory: C:\Program Files\Haestad\PPKW\Sample\

Upstream Link ID	Upstream Node ID	HYG file	HYG ID	HYG tag
WARNING: Missed peak when adding hydrograph...				
LINK 1	SUBAREA 1		SUBAREA 1	CV100

INFLOWS TO: POND 1 IN

HYG file	HYG ID	HYG tag	Volume ac-ft	Peak Time hrs	Peak Flow cfs
	SUBAREA 1	CV100	3.142	.2251	168.88

TOTAL FLOW INTO: POND 1 IN

HYG file	HYG ID	HYG tag	Volume ac-ft	Peak Time hrs	Peak Flow cfs
	POND 1 IN	CV100	3.141	.2210	165.84

S/N: F21601F07098 AECOM
PondPack Ver: Compute Time: Date:

Type.... Node: Pond Inflow Summary Page 9.04
Name.... POND 1 IN Event: 100 yr
File.... C:\Program Files\Haestad\PPKW\Sample\VILLAGE7 SCHOOL1.PPW
Storm... Chula Vista 100y Tag: CV100

TOTAL NODE INFLOW...
HYG file =
HYG ID = POND 1 IN
HYG Tag = CV100

Peak Discharge = 165.84 cfs
Time to Peak = .2210 hrs
HYG Volume = 3.141 ac-ft

HYDROGRAPH ORDINATES (cfs)

Village7 school1

Time hrs	Output Time increment = .0170 hrs. Time on left represents time for first value in each row.				
.0000	.00	12.76	25.53	38.29	51.06
.0850	63.82	76.59	89.35	102.11	114.88
.1700	127.64	140.41	153.17	165.84	159.18
.2550	146.41	133.65	120.89	108.12	95.36
.3400	82.59	69.83	57.06	44.30	31.54
.4250	18.77	6.01	.71		

Type.... Pond Routing Summary Page 9.05
Name.... POND 1 OUT Tag: CV50 Event: 50 yr
File.... C:\Program Files\Haestad\PPKW\Sample\VILLAGE7 SCHOOL1.PPW
Storm... Chula Vista 50yr Tag: CV50

LEVEL POOL ROUTING SUMMARY

HYG Dir = C:\Program Files\Haestad\PPKW\Sample\
Inflow HYG file = NONE STORED - POND 1 IN CV50
Outflow HYG file = NONE STORED - POND 1 OUT CV50

Pond Node Data = POND 1
Pond Volume Data = POND 1
Pond Outlet Data = Outlet 3

No Infiltration

INITIAL CONDITIONS

Starting WS Elev = 544.00 ft
Starting Volume = .000 ac-ft
Starting Outflow = .00 cfs
Starting Infiltr. = .00 cfs
Starting Total Qout= .00 cfs
Time Increment = .0170 hrs

INFLOW/OUTFLOW HYDROGRAPH SUMMARY

Peak Inflow = 151.09 cfs at .2210 hrs
Peak Outflow = 91.20 cfs at .3230 hrs

Peak Elevation = 548.26 ft
Peak Storage = 1.586 ac-ft

MASS BALANCE (ac-ft)

\+ Initial Vol = .000
\+ HYG Vol IN = 2.861
\- Infiltration = .000
\- HYG Vol OUT = 2.859
\- Retained Vol = .002

Unrouted Vol = -.000 ac-ft (.000% of Inflow Volume)

```
Type.... Pond Routing Summary                                Page 9.06
Name.... POND 1        OUT   Tag: CV100                 Event: 100 yr
File.... C:\Program Files\Haestad\PPKW\Sample\VILLAGE7 SCHOOL1.PPW
Storm... Chula Vista 100y    Tag: CV100

                    LEVEL POOL ROUTING SUMMARY

 HYG Dir          = C:\Program Files\Haestad\PPKW\Sample\
 Inflow  HYG file = NONE STORED - POND 1        IN  CV100
 Outflow HYG file = NONE STORED - POND 1        OUT CV100

 Pond Node   Data = POND 1
 Pond Volume Data = POND 1
 Pond Outlet Data = Outlet 3

 No Infiltration

 INITIAL CONDITIONS
 ----------------------------------
 Starting WS Elev    =   544.00 ft
 Starting Volume     =     .000 ac-ft
 Starting Outflow    =      .00 cfs
 Starting Infiltr.   =      .00 cfs
 Starting Total Qout=       .00 cfs
 Time Increment      =    .0170 hrs


 INFLOW/OUTFLOW HYDROGRAPH SUMMARY
 ======================================================
 Peak Inflow        =    165.84 cfs    at      .2210 hrs
 Peak Outflow       =    111.61 cfs    at      .3060 hrs
 ------------------------------------------------------
 Peak Elevation     =    548.40 ft
 Peak Storage =           1.646 ac-ft
 ======================================================


 MASS BALANCE (ac-ft)
 --------------------------
+ Initial Vol  =        .000
+ HYG Vol IN   =       3.141
- Infiltration =        .000
- HYG Vol OUT  =       3.139
- Retained Vol =        .002
                  ----------
 Unrouted Vol  =       -.000 ac-ft  (.000% of Inflow Volume)
```



```
Type.... C and Area                                          Page 10.01
Name.... SUBAREA 1           Tag: PRE

File.... C:\Program Files\Haestad\PPKW\Sample\VILLAGE7 SCHOOL1.PPW

RATIONAL C COEFFICIENT DATA
::::::::::::::::::::::::::::::::::::::::::::::::::::::::::::::::::::::
```

Soil/Surface Description	C	Area acres	C x Area acres
Undeveloped	.5000	81.000	40.500
WEIGHTED C & TOTAL AREA --->	.5000	81.000	40.500

S/N: F21601F07098 AECOM
PondPack Ver: Compute Time: Date:

Type.... Rational Predev. Peak Q Page 10.02
Name.... SUBAREA 1 Event: 50 yr
File.... C:\Program Files\Haestad\PPKW\Sample\VILLAGE7 SCHOOL1.PPW
Storm... Chula Vista 50yr Tag: CV50

SUMMARY OF RATIONAL METHOD PEAK DISCHARGES
--- PREDEVELOPED CONDITIONS ---

Q = CiA * Units Conversion; Where Conversion = 43560 / (12 * 3600)

Tag	Freq	File	IDF Curve
CV50	50		Chula Vista 50yr

Tc = .2900 hrs

Tag	Freq (years)	C	C adj factor	C final	I in/hr	Area acres	Peak Q cfs
CV50	50	.500	1.000	.500	2.6072	81.000	106.47

S/N: F21601F07098 AECOM
PondPack Ver: Compute Time: Date:

S/N: F21601F07098 AECOM
PondPack Ver: Compute Time: Date:

Type.... Mod. Rational Graph Page 10.03
Name.... SUBAREA 1 Tag: CV50 Event: 50 yr
File.... C:\Program Files\Haestad\PPKW\Sample\VILLAGE7 SCHOOL1.PPW
Storm... Chula Vista 50yr Tag: CV50

MODIFIED RATIONAL METHOD
---- Graphical Summary for Specified Storm Duration ----
Method T

Q = CiA * Units Conversion; Where Conversion = 43560 / (12 * 3600)

****

```
                         Village7 school1
*  RETURN FREQUENCY: 50 yr      |  Allowable Outflow: 106.47 cfs      *
*  'C' Adjustment: 1.000        |  Required Storage: .854 ac-ft       *
*---------------------------------------------------------------------*
*  Peak Inflow: 153.90 cfs                                            *
* .HYG File:   CV50                                                   *
***********************************************************************

   Q |
     |
     |                Tc= .2250 hrs
     |                I = 3.0711 in/hr          Area = 71.000 acres
     |              . Q = 153.90 cfs            Weighted C = .700
     |             .  .                         Adjusted C = .700
     |            .    .
     |           .      .           Required Storage
     |          .        .----------- .854 ac-ft
     |         .         |.
     |        .          | .
     |       .           |  .
     |      .                .
     |     .                  o o o    Q = 106.47 cfs
     |    .              o    |.       (Allow.Outflow)
     |   .           o        | .
     |  .        o            |  .      NOT TO SCALE
     | .   o                  |   .     ============
     | o                      |    .
      ------------------------|----------------------
                   .2943 hrs                      T
```

S/N: F21601F07098   AECOM
PondPack Ver: Compute Time: Date:

Type.... Mod. Rational Storm Calcs   Page 10.04
Name.... SUBAREA 1   Tag: CV50   Event: 50 yr
File.... C:\Program Files\Haestad\PPKW\Sample\VILLAGE7 SCHOOL1.PPW
Storm... Chula Vista 50yr   Tag: CV50

MODIFIED RATIONAL METHOD
---- Summary for Single Storm Frequency ----

Q = CiA * Units Conversion; Where Conversion = 43560 / (12 * 3600)

RETURN FREQUENCY: 50 yr `C' Adjustment = 1.000Allowable Q = 106.47 cfs

Hydrograph Storm Duration, Td = .2250 hrs   Tc = .2250 hrs
Hydrograph File: CV50

Wtd. `C'	Adjusted 'C'	Duration hrs	Intens. in/hr	Area acres	Qpeak cfs	VOLUMES Inflow ac-ft	Storage ac-ft
.700	.700	.2250	3.0711	71.000	153.90	2.862	.854

S/N: F21601F07098   AECOM
PondPack Ver: Compute Time: Date:

Type.... C and Area   Page 10.05
Name.... SUBAREA 1   Tag: POST

File.... C:\Program Files\Haestad\PPKW\Sample\VILLAGE7 SCHOOL1.PPW

RATIONAL C COEFFICIENT DATA

Soil/Surface Description	C	Area acres	C x Area acres
Developed	.7000	71.000	49.700
WEIGHTED C & TOTAL AREA --->	.7000	71.000	49.700

S/N: F21601F07098 AECOM
PondPack Ver: Compute Time: Date:

S/N: F21601F07098 AECOM
PondPack Ver: Compute Time: Date:

Type.... Rational Predev. Peak Q Page 10.06
Name.... SUBAREA 1 Event: 100 yr
File.... C:\Program Files\Haestad\PPKW\Sample\VILLAGE7 SCHOOL1.PPW
Storm... Chula Vista 100y Tag: CV100

SUMMARY OF RATIONAL METHOD PEAK DISCHARGES
--- PREDEVELOPED CONDITIONS ---

Q = CiA * Units Conversion; Where Conversion = 43560 / (12 * 3600)

Tag	Freq	File	IDF Curve
CV100	100		Chula Vista 100y

Tc = .2900 hrs

Tag	Freq (years)	C	C adj factor	C final	I in/hr	Area acres	Peak Q cfs
CV100	100	.500	1.000	.500	2.8590	81.000	116.76

S/N: F21601F07098 AECOM
PondPack Ver: Compute Time: Date:

S/N: F21601F07098 AECOM
PondPack Ver: Compute Time: Date:

Type.... Mod. Rational Graph Page 10.07
Name.... SUBAREA 1 Tag: CV100 Event: 100 yr
File.... C:\Program Files\Haestad\PPKW\Sample\VILLAGE7 SCHOOL1.PPW
Storm... Chula Vista 100y Tag: CV100

MODIFIED RATIONAL METHOD
---- Graphical Summary for Specified Storm Duration ----

Village7 school1
Method T

Q = CiA * Units Conversion; Where Conversion = 43560 / (12 * 3600)

```
**********************************************************************
*  RETURN FREQUENCY: 100 yr     |  Allowable Outflow: 116.76 cfs     *
*  'C' Adjustment: 1.000        |  Required Storage: .983 ac-ft      *
*--------------------------------------------------------------------*
*  Peak Inflow: 168.94 cfs                                           *
* .HYG File:   CV100                                                 *
**********************************************************************
```

```
   Q |
     |
     |                Tc= .2250 hrs
     |                I = 3.3711 in/hr          Area = 71.000 acres
     |              . Q = 168.94 cfs            Weighted C = .700
     |             . .                          Adjusted C = .700
     |            .   .
     |           .     .             Required Storage
     |          .       .-----------  .983 ac-ft
     |         .        |.
     |        .         | .
     |       .          |  .
     |      .              .
     |     .                o o o   Q = 116.76 cfs
     |    .             o  |.       (Allow.Outflow)
     |   .          o      | .
     |  .      o           |  .      NOT TO SCALE
     | .   o               |   .     ============
     | o                   |    .
     ----------------------|------------------------
                      .2945 hrs                    T
```

S/N: F21601F07098   AECOM
PondPack Ver: Compute Time: Date:

Type.... Mod. Rational Storm Calcs   Page 10.08
Name.... SUBAREA 1   Tag: CV100   Event: 100 yr
File.... C:\Program Files\Haestad\PPKW\Sample\VILLAGE7 SCHOOL1.PPW
Storm... Chula Vista 100y   Tag: CV100

MODIFIED RATIONAL METHOD
---- Summary for Single Storm Frequency ----

Q = CiA * Units Conversion; Where Conversion = 43560 / (12 * 3600)

RETURN FREQUENCY: 100 yr   `C' Adjustment = 1.000Allowable Q = 116.76 cfs

Hydrograph Storm Duration, Td = .2250 hrs   Tc = .2250 hrs
Hydrograph File: CV100

Wtd. `C'	Adjusted 'C'	Duration hrs	Intens. in/hr	Area acres	Qpeak cfs	VOLUMES Inflow ac-ft	Storage ac-ft
.700	.700	.2250	3.3711	71.000	168.94	3.141	.983

Appendix A A-1

Index of Starting Page Numbers for ID Names

OVERSIZED EXHIBIT "A"

This exhibit is on file at the City of Chula Vista, Planning Department located at 276 Fourth Avenue, Chula Vista, CA 91910

OVERSIZED EXHIBIT "B"

This exhibit is on file at the City of Chula Vista, Planning Department located at 276 Fourth Avenue, Chula Vista, CA 91910

OVERSIZED EXHIBIT "C"

This exhibit is on file at the City of Chula Vista, Planning Department located at 276 Fourth Avenue, Chula Vista, CA 91910

APPENDIX E-2

PRELIMINARY WATER QUALITY TECHNICAL REPORT FOR OTAY RANCH VILLAGE 7

RICK
ENGINEERING
COMPANY

PRELIMINARY
WATER QUALITY TECHNICAL REPORT
FOR
OTAY RANCH VILLAGE 7

Job Number 14483
September 19, 2003
Revised: October 20, 2003
Revised: February 27, 2004
Revised: March 26, 2004
Revised: May 24, 2004

WO# ______, DWG# ______

PRELIMINARY

WATER QUALITY TECHNICAL REPORT

FOR

OTAY RANCH VILLAGE 7

Job Number 14483

WO# ______, DWG# ______

Dennis C. Bowling, M.S.
R.C.E. #32838,
Exp. 06/06

REGISTERED PROFESSIONAL ENGINEER
DENNIS CALVIN BOWLING
No. 32838
Exp. 6/30/06
CIVIL
STATE OF CALIFORNIA

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Water Resources Department
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San Diego, California 92110-2596
(619) 291-0707

September 19, 2003
Revised: October 20, 2003
Revised: February 27, 2004
Revised: March 26, 2004
Revised: May 24, 2004

WATER QUALITY TECHNICAL REPORT (WQTR)

FOR

OTAY RANCH VILLAGE 7

J – 14483

REVISION PAGE FOR PLANCHECK COMMENTS

May 18, 2004

This Revision Page provides changes to the revised WQTR dated March 26, 2004 to incorporate any revisions resulting from the City of Chula Vista's (herein refereed to as the "City's") Plan Check Review comments dated May 10, 2004. In addition, this report has been modified as a result of previous plan check comments from the San Diego Regional Water Quality Control Board (SDRWQCB).

The City's plan check comments and Rick Engineering Company's responses are presented below. The City's plan check comments are italicized.

<u>City of Chula Vista (City's) Plan Check Comments dated May 10, 2004:</u>

1. **City's Comment:**
   Any proposed land disturbance activities within the Wolf Canyon Creek requires approval from environmental regulating agencies.

   Response:
   Currently the Environmental Impact Report (EIR) and other supplemental reports (e.g., Rick Engineering Company's WQTR for Otay Ranch Village 7 dated, March 23, 2004) are currently being reviewed by the SDRWQCB. The WQTR shall be amended (as necessary) once comments have been received from the SDRWQCB.

2. **City's Comment:**
   The second paragraph of page 4 states, "At such time the City of Chula Vista will take over the maintenance responsibilities for the public storm drain facilities." However, on page 33 of the report it is stated, "All post-construction structural BMPs will be maintained by the CFD and/or HOA..." Clarification of maintenance responsibilities and funding mechanisms is required. [See comment 9 below]

   Response:
   The City of Chula Vista will perform maintenance on all structural post-construction BMPs proposed for this project. The Community Facility District (CFD) will provide the funding for all post-construction BMPs. The HOAs will assume responsibility for common areas.

Please see the revised WQTR (Section 6.0) dated May 24, 2003 for additional information and clarification.

3. **City's Comment:**
The Water Quality, Detention, and Temporary Desilting Basins shall be fenced for public safety.

Response:
The Wolf Canyon Water Quality and Detention Basin will have fencing for public safety. This shall be detailed on the final design plans for the project.

4. **City's Comment:**
There are certain missing sections in the report (such as Appendix C, Storm Water Costs and Details) that need to be completed with the final submittal of the report. [See comment 19 below]

Response:
It has been determined through a meeting with Dino Serafini and Chester Bautista on May 18, 2004 that Appendix C (Storm Water Costs and Details) could be completed upon final design of this project.

5. **City's Comment:**
Table 3 of the City of Chula Vista SUSMP shows low removal efficiency by Drainage Inserts for the majority of pollutants of concern. Please provide a discussion as to the reason this type of permanent treatment BMP was selected for a considerable portion of the development. In view of high maintenance costs and low efficiency of filter inserts, this type of treatment BMP may not be appropriate for the project.

Response:
For the purposes of this WQTR two alternatives (Alternative A and B) have been provided. Please see Section 4.2.C and Map Pocket 2 for detailed information regarding both alternatives.

6. **City's Comment:**
Commercial and industrial facilities are required to have on-site Treatment Control BMPs appropriate for the potential pollutants generated at the site.

Response:
Please see Section 4.1.D of the revised WQTR dated May 24, 2004.

7. **City's Comment:**
It is acknowledged that <u>peak flow rates</u> have been addressed. Consideration should be given to impact of increased flow <u>volume</u> on downstream erosion. [Also see comment 13 below]

Response:
Conditions of concern for this project were addressed. A field reconnaissance was performed and as a result it was determined that the project shall be designed to detain the 2-year and 10-year post-project peak flow rates back to equal or less than the pre-developed conditions and thus not adversely impacting downstream conditions along Wolf Canyon with respect to erosion as outlined in the City's Standard Urban Stormwater Mitigation Plan (SUSMP). The Wolf Canyon Basin shall also detain the 100-year post-project peak flow rates back to equal or less than pre-project conditions as outlined in the City's Subdivision Design Manual. In addition to attenuating the post-project discharge for the 2, 10-, and 100-year storm events the Wolf Canyon Basin shall also maintain pre-project velocities.

8. **City's Comment:**
The comments from Mike Porter of the Regional Water Quality Control Board e-mailed to Todd Galarneau, of McMillin Land Development, and copied to City staff on 4/22/04 shall be addressed, as appropriate, in the Water Quality Technical Report.

Response:
The revised WQTR dated March 26, 2004 incorporated the original comments and the April 22, 2004 comments from Mike Porter. No additional comments from Mike Porter or any other personnel of the SDRWQCB have been received to date. However, if additional comments are received the report will be amended (if necessary).

9. **City's Comment:**
The second paragraph add: "Funding for the long-term maintenance and monitoring of water quality facilities located within public open space or ROW, will be established by a special tax district formed for that purpose." In the second bullet on this page change the adoption date of the City Storm Water Management Standards Manual to Nov. 26, 2002 and add to the end: "The Storm Water Standards Manual also contains the City of Chula Vista's Standard Urban Storm Water Mitigation Plan (SUSMP) requirements".

Response:
Please see the revised WQTR dated May 24, 2004.

10. **City's Comment:**
Page 5 – In the first full paragraph, combine the first two sentences as follows: "The City of Chula Vista's Storm Water Standards Manual provides guidance for new development and redevelopment projects to achieve compliance with the City of Chula Vista SUSMP".

Response:
Please see the revised WQTR dated May 24, 2004.

11. **City's Comment:**
Page 8 – write out SWSAS for the first time in the first paragraph.

Response:
Please see the revised WQTR dated May 24, 2004.

12. City's Comment:
Page 10 – Describe how the RWQCB review/approval of the PCSWOMP coincides with the implementation of the City's SUSMP requirements. City doesn't address the PCSWOMP process; what, if any, relevance is there of PCSWOMP in the post-construction SWPPP.
Response:
This section has been removed. Please see the revised WQTR dated May 24, 2004.

13. City's Comment:
Chapter 5.0 Permanent BMP Selection Procedure:

A) Section 1.A, page 13 include "Attached Residential Development" as priority project category here on page 16.
B) Section 1 C the SUSMP (Sec. V1.c)) states that a Condition of Concern includes runoff volume and velocity when runoff discharges to natural channels, as is proposed west of La Media Road. The WQTR needs to identify the flow characteristics of the 2yr. and 10yr. return frequency discharge and impacts therefrom on downstream reaches of Wolf Canyon that will remain natural. On Table 1: separate out volumes attributable to the EUC.
C) Section 1.D – page 18 second full paragraph: Explain the usage of the words "proposed" and "selected" in terms of the project BMP's. Several Site Design BMP's and Design Concepts are proposed in Sections 2.A through 2.D actually there is no "D". However, Chapter 6 – "Permanent Storm Water BMP's", focuses on just two selected BMP's: catch basin inserts and the extended detention basin. I'm sure if this report is saying that the Chapter 5 measures will be implemented or not, please clarify.

Response:
A) The Attached Residential Development priority project category has been added. Please see the revised WQTR dated May 24, 2004.
B) The separation of the volume calculations into smaller areas (such as the EUC) does not indicate the post-project condition of the Wolf Canyon Water Quality and Detention Basin, thus it was not included. However, included in the report (Map Pocket 1) are pre- and post-project flow rates for the EUC. In addition, please see response for City Comment Number 13
C) Section 2.D of the revised WQTR dated May 24, 2004 has been removed. Section 4.0 (Section 5.0 in previous report) talks about the site, source, and treatment control BMPs that will implemented for this project. However Section 6.0 and 7.0 further discuss the structural treatment control BMPs in more detail with respect to maintenance, funding, and design.

14. City's Comment:
Section 2.A. page 19 and 22 change "high school" to "Sweetwater Unified High School District".

Response:
Please see the revised WQTR dated May 24, 2004.

15. City's Comment:
Section 2.A. page 19 Provide more specific and applicable implementation to the "Design Concepts" listed; for example: "3 Minimize directly connected impervious areas" be specific as to where and how (types of land-use, dispersed or collected system) roof downspouts shall be directed to landscaping. "4 Construct walkways, trails, patios…with permeable surfaces such as pervious concrete, porous asphalt, unit pavers…" the implementation suggested: "The McMillin Village 7 project will provide landscaping and vegetation wherever possible" doesn't address the use of permeable surfaces as a design feature; "5 Maximizing canopy interception and water conservation by preserving existing native trees and shrubs and planing {sic} additional native or drought tolerant trees and large shrubs" the suggested implementation: "the project will direct runoff away from tops of slopes…" describes ordinary (and a required minimum) erosion control measures; "*6 minimize the use of impervious surfaces, such as decorative concrete, in the landscape design.*" *What does planting native or drought tolerant vegetation on all vegetated slopes have to do with minimizing impervious surfaces? It is a good idea and reduces water use and thus, indirectly, achieves some WQ benefits, but this measure is intended to provide specific alternatives to impervious surfaces. In general, the idea is to propose measures that result in a reduction of runoff depth, thus less reliance on structural treatment measures.*

Response:
Section 4.2.A describes the Site Design BMPs that are associated with this project. Upon final design, more detailed information shall be provided.

16. City's Comment:
Page 21 concept 4 add: The extended detention facilities shall detain the 2, 10, 50 and 100 yr., 6-hour post project flows…"

Response:
It has been determined through a meeting with Dino Serafini and Chester Bautista on May 18, 2004 that 50-year detention is not required. The revised WQTR dated May 24, 2004 has analyzed the 100-year storm event. Upon final design, detailed analyses for the 2- and 10-year storm events shall be performed.

17. City's Comment:
A)Page 30 first paragraph: Propose a more practical system of roof drainage treatment for commercial, public and multi-family projects that can be cost-effectively monitored – by the City if necessary.
B) State on page 30 in the last paragraph that final selection of publicly maintained curb inlet filtration devices shall be shown on street improvements plans and approved by the Director of Public Works.

Response:

A) Please see the revised WQTR dated May 24, 2004.

B) Please see the revised WQTR dated May 24, 2004.

18. City's Comment:

Page 28 Delete the two sentences referring to the letter from City of Chula Vista (final approval of this document will render this statement unnecessary). Table 1: show the 2, 10 and 50 yr flow rates also. On Table 3: show the EUC volumes separately. Provide the extended detention basin's in low and outflow hydrograph for each design storm (the full range required by both the SUSMP and the Submanual) and calcs. showing the dewatering rate and time for both the detention and WQ basin.)

Response:

The two sentences referring to the letter from the City of Chula Vista has been removed from the revised report. Upon final design of this project, calculations for the Wolf Canyon Basin shall be performed for the 2-and 10-year storm events. For the purposes of this preliminary report only the 100-year storm event was analyzed. In addition, the separation of the volume calculations into smaller areas (such as the EUC) does not indicate the post-project condition of the Wolf Canyon Water Quality and Detention Basin, thus it was not included. However, included in the report (Map Pocket 1) are pre- and post-project flow rates for the EUC. For discussion regarding the 50-year flow rates, please see response for City Comment Number 16.

19. City's Comment:

Chapter 7.0 Provide new section on BMP maintenance costs and preface the section with the following: "The following costs are intended only to provide a magnitude of the costs involved in maintaining BMP's. Specific unit costs shall be verified prior to the formation of the respective maintenance CFD."

Include the following cost items for the WQ/EDB:

Slope and bank erosion repair;
Reseed and sod damaged ground cover;
Repair and resurface maintenance access road;
Monitor sediment accumulation rates;
Measuring, analyzing and reporting pollutant production and removal;
Other professional services as required (biologist, landscape architect, ecological engineer, hydrogeologist, etc.)

Response:

It has been determined through a meeting with Dino Serafini and Chester Bautista on May 18, 2004 that BMP maintenance costs can be performed upon final design.

20. **City's Comment:**
Address the potential for permitting requirements, if any, from resource agencies derived from routine or extraordinary maintenance operations in the WQ/EDB.

Response:
Currently all environmental permitting processes are currently being reviewed by the SDRWQCB. In addition, a maintenance plan is being created for the Village 7 project and provided at later date per discussions with Dino Serafini and Chester Bautista.

21. **City's Comment:**
Provide an outline of a runoff quality-monitoring program and sample inspection monitoring forms.

Response:
It has been determined through a meeting with Dino Serafini and Chester Bautista on May 18, 2004 that providing an outline of a runoff quality-monitoring program and sample inspection monitoring forms is unnecessary.

22. **City's Comment:**
Appendix A Form 5500: This Form (and subsequent forms) should also reflect the development of the EUC, since the WQ/EDB substantially provides the water treatment function for the EUC.

Response:
Please see the revised WQTR dated May24, 2004.

TABLE OF CONTENTS

Map Pocket 1:

Preliminary Existing Drainage Basin Boundaries – Otay Ranch Village 7
(from August 2003 report titled, Otay Ranch SPA Village 7 – Preliminary Regional Drainage Study Major Drainage Patterns and Facilities, prepared by P&D Consultants, Inc.)

Developed Drainage Basin Boundaries – Otay Ranch Village 7
(from August 2003 report titled, Otay Ranch SPA Village 7 – Preliminary Regional Drainage Study Major Drainage Patterns and Facilities, prepared by P&D Consultants, Inc.)

Map Pocket 2:

Water Quality Technical Report Exhibit for Otay Ranch Village 7 (Alternative "A")
Water Quality Technical Report Exhibit for Otay Ranch Village 7 (Alternative "B")

1.0 INTRODUCTION

This water quality technical report (WQTR) summarizes storm water protection requirements for a portion of the Otay Ranch Village 7 project. The project is located in the City of Chula Vista, north of Rock Mountain Road, south of Birch Parkway, west of the future State Route 125, and east of La Media Road. This WQTR describes the permanent storm water best management practices (BMPs) that will be incorporated in order to mitigate the impacts of urban runoff due to the Otay Ranch Village 7 project.

McMillin Companies owns the northeast portion of the Otay Ranch Village 7 project site (east of Magdalena Road). This WQTR shall address only the McMillin owned portion of the project for the purposes of this report and the McMillin property will be referred to as McMillin Village 7 herein. See the Vicinity and Site Map, located in Section 2.0 of this report, for the location of the McMillin owned portion. The McMillin Village 7 project consists of the construction of approximately 750 dwelling units, an elementary school, park, a high school site, and a series of extended detention basins along Wolf Canyon on approximately 180 acres.

The project is subject to National Pollutant Discharge Elimination System (NPDES) requirements. NPDES requirements are contained in Section 402(p) of the Federal Clean Water Act. These requirements are implemented through permits issued by the State Water Resources Control Board (SWRCB) or the local San Diego Regional Water Quality Control Board (SDRWQCB) in which the project is located, and the governing municipality where the project is located (City of Chula Vista).

The pre-project conditions consist of rolling hills with arroyos draining into larger canyons flowing southwest, away from the Otay Reservoir Basin. There are three major pre-project watersheds associated with McMillin Village 7. The northern drainage basin flows in a northwesterly direction to the Village 6 development and ultimately to Poggi Canyon. The central drainage basin (which comprises the majority of the project site) flows in a westerly direction into Wolf Canyon. The third drainage basin located in the southern portion of the project site, flows offsite in a southwesterly direction. Preliminary hydrologic analyses for the pre-project condition have been preformed by Rick Engineering Company. In addition, the basin that is tributary to Poggi Canyon has been analyzed for the design of Village 6, located directly north of McMillin Village 7. This basin was included and identified in the December 13, 2001 report titled, *Otay Ranch SPA Village 6 - Preliminary Drainage Study Major Drainage Patterns and Facilities*, prepared P&D Consultants, Inc. Please reference the exhibit titled, *Preliminary Existing Drainage Basin Boundaries – Otay Ranch Village 7*, located in Map Pocket 1 for the locations of the above-described basins.

At this stage in the project development the grading concepts for McMillin Village 7 have not been approved for final engineering. The design scenario for the post-project condition is as follows:

> McMillin Village 7 shall convey post-project flows from the majority of the Eastern Urban Center (EUC), located upstream of Village Seven, east of the State Route 125. The EUC, which is currently undeveloped, is planned for high-density multi family residential, office and commercial land uses and will function as the urban center of the Otay Ranch. The runoff shall be conveyed from the EUC to Village 7 (Wolf Canyon) where it will confluence with the proposed McMillin Village 7 on-site storm drain system. At this confluence the developed flows from the EUC and a large portion of the McMillin Village 7 shall outlet into a forebay. The forebay will be followed by a series of extended detention basins (known as the Wolf Canyon Water Quality and Detention Basin), as shown on the exhibits titled, *Water Quality Technical Report Exhibit for Otay Ranch Village 7 (Alternative "A" and "B")* located in Map Pocket 2. In addition, the project is recreating a receiving water (that will convey a portion of Village 7 treated post-project flows) that will be located adjacent to the series of basins proposed for Wolf Canyon.

Field surveys have been performed by Helix Environmental Planning, Inc. for the upper portion of Wolf Canyon (McMillin Village 7 project site). The surveys, conducted in 2003, determined that the canyon consists of un-vegetated, non-wetland waters. This is consistent with surveys conducted as part of the Otay Ranch General Development Plan Environmental Impact Report. In addition, this area has limited functions and values for wildlife and water quality, as it has been in active agricultural usage up to the present day. Therefore, it was determined that the appropriate location for the extended detention basin would be in Wolf Canyon, just upstream of the proposed Magdalena Road. In the design scenario for McMillin Village 7, the extended detention basin and the forebay shall be equipped to handle, treat, and detain the post-project flow from the EUC and the majority of the residential portion of McMillin Village 7. The project proposes regional benefits with respect to water quality, while maximizing conservation of more significant biological resources offsite in areas with much higher long-term conservation value. The project site is not identified as a Conserved Area in the City of Chula Vista's Multiple Species Conservation Program (MSCP) Subarea Plan.

There are two alternatives proposed for this project. Both alternatives use a combination of structural treatment control BMPs (in addition to the Wolf Canyon Water Quality and Detention Basin) to treat post-project flows, generated from the project, before leaving the site. Located in Map Pocket 2 are two alternatives; Alternative A and B. Alternative A proposes treating post-project flows tributary to the storm drain systems that are located in Magdalena Road and Rock Mountain Road and the storm drain system that serves the northern basin (that drains to Poggi Canyon) with inlet filter inserts. In addition, the project proposes an in-line treatment facility at the downstream portion of the storm drain system that outlets into the receiving water associated with the Wolf Canyon Basin. Alternative B proposes three in-line treatment facilities. One shall be located at the downstream portion of the storm drain system that outlets into the receiving water associated with the Wolf Canyon Basin. Another unit shall be installed at the downstream portion (near the intersection of Magdalena Road and Birch Parkway) of the storm drain system that serves the drainage basin that flows northerly to Poggi Canyon. The last unit shall be associated with the storm drain system that is located on Magdalena in the central portion of the project, just before the confluence with the Wolf Canyon Basin flows. For the remainder portion of the project (storm drain systems that serve the southern portion of the project) inlet filter inserts shall be installed. For location of the structural treatment control BMPs described above please refer to the exhibits located in Map Pocket 2.

Post-construction structural BMPs (either inlet filter inserts or an in-line treatment facility based on the chosen design alternative) shall serve the northerly portion of McMillin Village 7 to treat post-project flows (before discharging through Village 6 and to Poggi Canyon). A regional detention facility further downstream has also been designed by Village 6 to capture and detain the post-project flows associated with this basin.

The drainage basin along the southerly portion of the McMillin Village 7 (high school site) will consist of a mass graded area (high school site) a portion of Magdalena Road and Rock Mountain Road. The high school site shall be served by a temporary desilting basin. In addition proposed storm drain systems shall be installed on Magdalena Road and Rock Mountain Road to convey the flows from the high school site and the runoff from the roads offsite in a westerly direction. Bio Clean Inlet Filter Inserts shall be installed in a portion of the inlets that are located within this portion of McMillin Village 7) as shown on the exhibits located in Map Pocket 2. Therefore, treating the post-construction runoff before discharging off-site.

The post-project condition has been analyzed in an August 2003 report titled, *Otay Ranch SPA Village 7 - Preliminary Regional Drainage Study Major Drainage Patterns and Facilities*, prepared P&D Consultants, Inc. In addition, located in Appendix B, Rick Engineering Company has performed preliminary hydrologic, detention, and water quality analyses for the McMillin Village 7 project and this WQTR.

McMillin Companies will assume the ownership and maintenance responsibilities of the storm drain system within McMillin Village 7 until construction is complete. At such time the City of Chula Vista will take over the maintenance responsibilities for the public storm drain facilities. Funding for long-term maintenance and monitoring of water quality facilities located within public open space or ROW, will be established by a special tax district formed for that purpose.

For the purposes of water quality management, the proposed McMillin Village 7 project will follow the guidelines and requirements set forth in the following documents:

- "*Waste Discharge Requirements (WDRS) for Discharges of Storm Water Runoff Associated with Construction Activity*" as indicated by the State Water Resources Control Board (SWRCB) Order No. 99-08-DWQ National Pollutant Discharge Elimination System (NPDES) General Permit No. CAS000002 (General Construction Permit). The General Construction Permit was adopted by the State Water Resources Control Board on August 19, 1999.

- "*Development and Redevelopment Projects Storm Water Management Standards Requirements Manual: Manual for Permanent Storm Water Management BMPs & Construction Standards Requirements*" adopted on November 26, 2002 (herein referred to as Storm Water Standards Manual). The effective date of the Storm Water Standards Manual is December 9, 2002, and applies to all projects requiring any permit approvals on or after December 9, 2002, regardless if the project is currently under review or if previous approvals have been obtained. The Storm Water Standards Manual also contains the City of Chula Vista's Standard Urban Storm Water Mitigation Plan (SUSMP) requirements.

The General Construction Permit is a statewide permit that requires permittees to implement specific sampling and analytical procedures to determine whether Best Management Practices (BMPs) implemented on a construction site are:

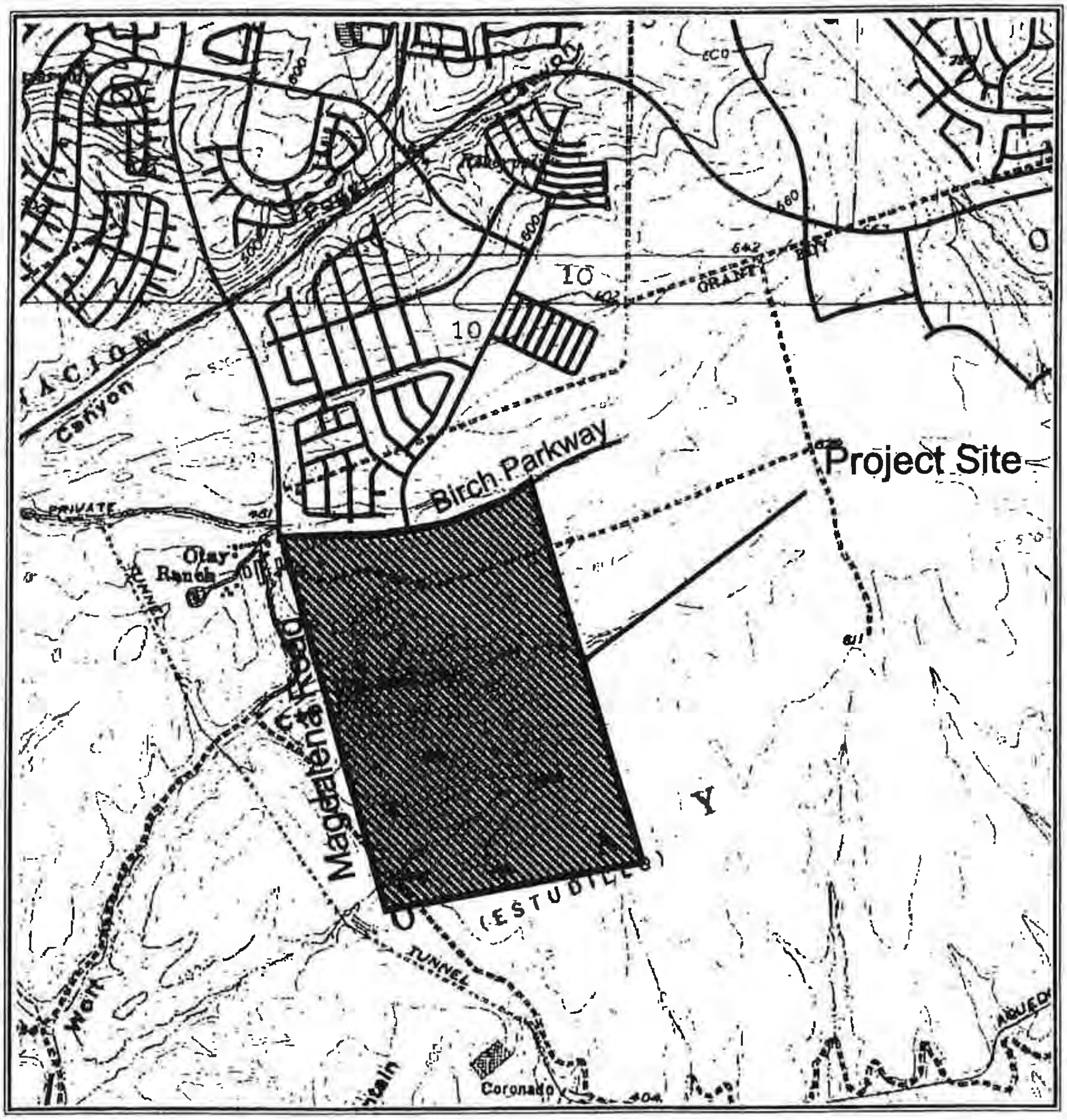
(1) preventing further impairment by sediment in storm waters discharged directly into waters listed as impaired for sediment or silt, and
(2) preventing other pollutants, that are known or should be known by permittees to occur on construction sites and that are not visually detectable in storm water discharges, from causing or contributing to an exceedance of water quality objectives.

The City of Chula Vista's Storm Water Standards Manual (SUSMP) provides guidance for new development and redevelopment projects to achieve compliance with the City of Chula Vista's SUSMP. The City of Chula Vista adopted the SUSMP on November 26, 2002, as required by the Municipal Storm Water Permit (Municipal Permit) adopted by the California Regional Water Quality Control Board, San Diego Region (SDRWQCB), Order No. 2001-01, National Pollutant Discharge Elimination System (NPDES) No. CAS0108758 draining the watersheds of the County of San Diego, the incorporated cities of San Diego County, and the San Diego Unified Port District. The Municipal Permit was adopted by the SDRWQCB on February 21, 2001.

The Storm Water Standards Manual provides checklists for determining applicability of storm water requirements for projects in the City of Chula Vista's project review and permitting process. The "Storm Water Requirements Applicability Checklists and Forms" provided within the Storm Water Standards Manual has been completed and is located in Appendix A of this WQTR. Water quality requirements during construction are discussed in further detail in Section 3.0 of this report.

The McMillin Village 7 project will provide permanent storm water BMPs to ensure that water quality treatment is provided prior to discharge from the project site. Further discussion of permanent storm water BMPs is provided in Section 4.0 of this report. Section 5.0 will discuss the treatment control BMPs that have been chosen for this project in more detail. Section 6.0 will discuss maintenance procedures, costs, and funding for these BMPs.

2.0 VICINITY AND SITE MAP



N
W
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S

SCALE: 1" = 2000'

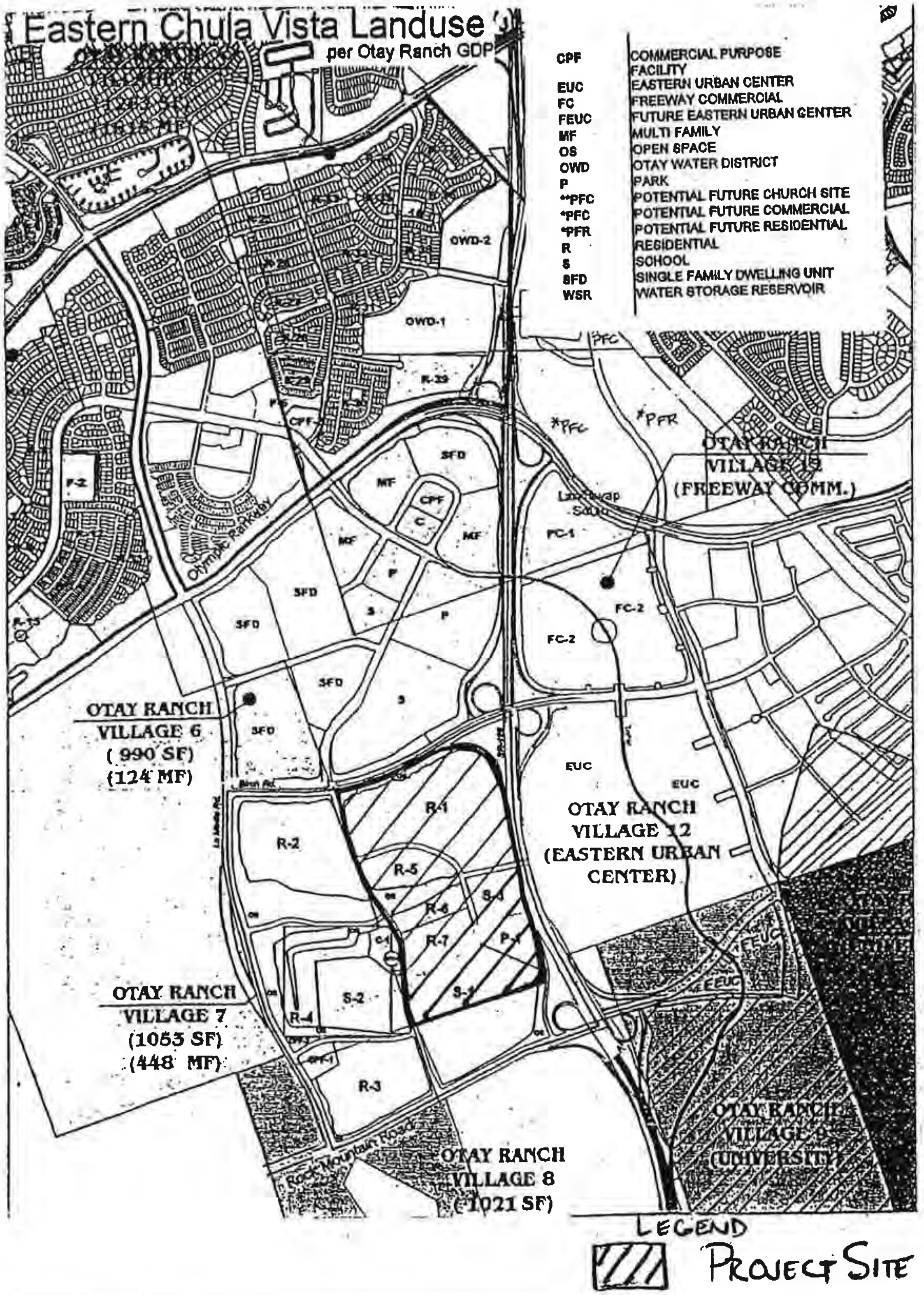
LEGEND

PROJECT SITE AS DESCRIBED IN THE WATER QUALITY TECHNICAL REPORT (WQTR)

VICINITY MAP

Otay Ranch Village 7

Site Map for Village 7



3.0 WATER QUALITY REQUIREMENTS DURING CONSTRUCTION

The Storm Water Standards manual is "intended to generally provide information on how to comply with all of the City's permanent and construction storm water BMP requirements, including the SUSMP requirements, for new private and public development projects in the City of Chula Vista." The effective date of the City's Storm Water Standards manual is December 9, 2002, and applies to all projects requiring any permit approvals on or after December 9, 2002, regardless if the project is currently under review or if previous approvals have been obtained. The City of Chula Vista's Storm Water Standards Manual states the following:

(1) Effectively prohibit non-storm discharges; and
(2) Reduce the discharge of pollutants from storm water conveyance systems to the Maximum Extent Practicable (MEP statutory standard) both during construction and throughout the use of a developed site.

During the construction phase, the project is subject to the requirements of the "Waste Discharge Requirements for Discharges of Storm Water Runoff Associated with Construction Activity" as indicated by the State Water Resources Control Board (SWRCB) Order No. 99-08-DWQ National Pollutant Discharge Elimination System (NPDES) General Permit No. CAS000002 (General Construction Permit). The State Water Resources Control Board adopted the General Construction Permit on August 19, 1999. For coverage by the General Construction Permit, the project owner is required to submit to the SWRCB a Notice of Intent (NOI) to comply with the General Construction Permit, and develop a Storm Water Pollution Prevention Plan (SWPPP) describing best management practices (BMPs) to be used during and after construction to prevent the discharge of sediment and other pollutants in storm water runoff from the project. In order to terminate coverage under the General Construction Permit, the developer must submit a Notice of Termination (NOT) and a Post-Construction Storm Water Operation and Management Plan (PCSWOMP), according to Section A.10 of the General Construction Permit, to the

Regional Water Quality Control Board (RWQCB). Permanent BMPs for the McMillin Village 7 project are discussed in Sections 5.0 and 6.0 of this WQTR.

At this time, the post-project grading concept has not been approved for final engineering and therefore a SWPPP has not been prepared. However, prior to commencement of construction, it is currently anticipated that Rick Engineering Company will develop the SWPPP for the McMillin Village 7 project once the final grading scenario has been determined. As part of the SWPPP, a Storm Water Sampling and Analysis Strategy (SWSAS) will be developed for the construction site and shall be included in the project's SWPPP. For a detailed discussion of the construction storm water BMPs that will be implemented please refer to the SWPPP/SWSAS for the project

In the site's present state the pollutant of concern with the site is primarily sediment. During construction the pollutants of concern on the site are sediment and non-visible pollutants. The site owner is responsible to prevent these pollutants from leaving the site by implementing temporary BMPs. Typical temporary BMPs that may be used during construction include good housekeeping practices and erosion and sediment control measures. Good housekeeping practices include practices such as street sweeping, waste disposal, vehicle and equipment maintenance, materials storage, minimization of hazardous materials and proper handling and storage of hazardous materials. Typical erosion and sediment control measures include silt fence, fiber rolls, gravel bags, temporary desilting basins, velocity check dams, temporary ditches or swales, storm water inlet protection, soil stabilization measures such as erosion control mats, tackifier, or hydroseed, and other measures. The project's SWPPP will be required to identify the specific BMPs to be used on the project site during construction.

As part of the project's SWPPP, a Storm Water Sampling and Analysis Strategy will be developed for the construction site and included in the project's SWPPP. The objectives of the Storm Water Sampling and Analysis Strategy (SWSAS) are to determine whether best management practices (BMPs) implemented on a construction site are:

(1) preventing further impairment by sediment in storm waters discharged directly into waters listed as impaired for sediment or silt [i.e., listed on Attachment 3 of the General Construction Permit, which identifies waters listed as impaired for sediment, silt, or turbidity on the Clean Water Act Section 303(d) List]; and

(2) preventing other pollutants, that are known or should be known by permittees to occur on construction sites and that are not visually detectable in storm water discharges, from causing or contributing to exceedances of water quality objectives.

On February 4, 2003, the SWRCB adopted the 2002 CWA Section 303(d) List to update the previous 1998 Clean Water Act Section 303(d) List. The proposed McMillin Village 7 project is located within the Otay Valley Hydrologic Area, within the Otay Hydrologic Unit, within Region 9. Therefore, the Hydrologic Basin Number for this project is 910.20 (Region Number '9', Hydrologic Unit Number '10', and Hydrologic Area Number '2'). Under both pre-project and post-project conditions, the majority of the project site discharges into Wolf Canyon. The proposed design concept will maintain the pre-project drainage conditions and as a result the basin located north of Wolf Canyon shall discharge into Poggi Canyon (and maintain natural drainage conditions). There are currently no water bodies on the list titled, *2002 CWA Section 303(d) of Water Quality Limited Segments* within the Hydrologic Basin 910.20. The project site does not discharge directly into 303(d) listed water bodies and therefore the first (1) SWSAS objective (listed above) does not apply to the McMillin Village 7 project. However, the second (2) above mentioned SWSAS objective does apply to the McMillin Village 7 project, and the Storm Water Sampling and Analysis Strategy (SWSAS) will be required to meet this objective.

The receiving waters for the project, the pollutants of concern, conditions of concern, and the Hydrologic Unit classification are discussed in further detail within the "Pollutants and Conditions of Concern" portion of this water quality technical report, Section 4.0.

4.0 PERMANENT BEST MANAGEMENT PRACTICES SELECTION PROCEDURE

For the purposes of meeting the City of Chula Vista SUSMP requirements, the guidelines as defined within Appendix B of the Storm Water Standards manual have been followed. In order to follow the procedure guidelines pollutants and conditions of concern must be identified. Next, the appropriate BMPs must be selected and incorporated. These steps will be discussed in this section of the WQTR. The specifics regarding design of the selected BMPs (as applicable) will be discussed in Section 5.0 of this WQTR.

According to Section I – Priority Projects (in Appendix B of the Storm Water Standards manual), the McMillin Village 7 project applies to the following four priority project categories: (1) Home subdivisions of 100 housing units or more, (2) Attached Residential Development, (3) Streets, roads, highways, and freeways, and (4) Commercial Development >100,000 ft^2. Table 1 – *Anticipated and Potential Pollutants Generated by Land Use Type* (Section V.1.a of Storm Water Standards) indicates General Pollutant Categories that are either anticipated or potential pollutants for specific priority project categories. Anticipated and potential pollutants for the four different priority project categories of the McMillin Village 7 project are as follows:

Identify Pollutants and Conditions of Concern

1.A Pollutants from the Project Area

Table 1 of the Storm Water Standards Manual, *Anticipated and Potential Pollutants Generated by Land Use Type*, indicates general pollutant categories that are either anticipated or potential pollutants for specific project categories. Based on Table 1 of the Storm Water Standards Manual, anticipated and potential pollutants for the priority project categories "Attached Residential Development", "Detached Residential Development", "Streets, Freeways & Highways", and "Commercial Development >100,000 ft^2 are as follows:

Attached Residential Development –

a) Anticipated Pollutants: Sediments, Nutrients, Trash and Debris, Oxygen Demanding Substances, Oil and Grease, Bacteria and Viruses, and Pesticides.

Detached Residential Development –

b) Anticipated Pollutants: Sediments, Nutrients, Trash and Debris, Oxygen Demanding Substances, Oil and Grease, Bacteria and Viruses, and Pesticides.

There are no pollutants listed on Table 1 of the Storm Water Standards Manual for "Detached Residential Development" that are categorized as "potential pollutants." However Bacteria and Viruses are categorized as "potential pollutants" for "Attached Residential Development."

Streets, Highways & Freeways –

a) Anticipated Pollutants: Sediments, Heavy Metals, Organic Compounds (including petroleum hydrocarbons), Trash and Debris, Oil and Grease.
b) Potential Pollutants: Nutrients, Oxygen Demanding Substances (including solvents).

Commercial Development >100,00 ft^2 –

a) Anticipated Pollutants: Trash and Debris, Oil and Grease.
b) Potential Pollutants: Sediments, Nutrients, Organic Compounds, Oxygen Demanding Substances (including solvents), Bacteria and Viruses, Pesticides.

Nutrients are a potential pollutant generated by the "Streets, Highways & Freeways" and "Commercial Development >100, 000 ft^2" land use category for the McMillin Village 7 project because landscaping exists on-site. The "Commercial Development >100, 000 ft^2" priority project category was used for this project because of the elementary and high school site and the EUC.

While the site is not expected to generate a large volume of sediment once buildout has been completed and landscaping has been established, some sediment will be tracked in by cars and a small amount may be generated on site. This sediment is defined as a pollutant, and may also contain attached pollutants such as heavy metals. The majority of anticipated and potential pollutants will be transported by low flows that typically occur during the initial stage of a storm event.

1.B Pollutants of Concern in Receiving Waters

According to the September 8, 1994 report titled, *Water Quality Control Plan for the San Diego Basin (9),* the proposed McMillin Village 7 project is located within the Otay Valley Hydrologic Area within the Otay Hydrologic Unit. The corresponding number designation is 910.20 (Region '9', Hydrologic Unit '10', Hydrologic Area '2'). An exhibit has been provided in Appendix D of this report titled, *Location Map for Village 7 in Hydrologic Basin 910.20*, which shows the project location found in the north central region of the Otay Hydrologic Unit (Hydrologic Basin 910.20).

On February 4, 2003, the SWRCB adopted the 2002 CWA Section 303(d) List to update the previous 1998 Clean Water Act Section 303(d) List. There are currently no water bodies on the list titled, *2002 CWA Section 303(d) of Water Quality Limited Segments* within the Hydrologic Basin 910.20 listed as impaired. Therefore, the McMillin Village 7 project does not discharge directly into any 303(d) listed impaired water body and is not subject to the requirements of a 303(d) listing.

In the post-project condition, runoff from a portion of the EUC (located upstream of the project site) and runoff from a large portion of the project shall be collected within a proposed on-site storm drain system that will flow into an In-line Treatment System then will discharge into the forebay and detention basin located on-site within Wolf Canyon, referred to as the Wolf Canyon Water Quality and Detention Basin. The Wolf Canyon Basin shall be designed to handle, treat, and detain the post-project flows for the 2-, 10-, and 100-year storm event. The 2- and 10-year detention is to mitigate for downstream erosion (City of Chula Vista SUSMP) and the 100-year detention is to attenuate post-project flow rates to pre-project levels (City of Chula Vista Subdivision Design Manual). Structural BMPs will treat the remaining flows before leaving the project site. Wolf Canyon flows in a westerly direction and eventually confluence with the Otay River. The remaining northern portion of the project that generates runoff (the northerly drainage basin that flows to Poggi Canyon), shall be collected in a storm drain system, that shall be served by structural BMPs. In addition, a temporary desilting basin will treat the southerly portion of the project and BioClean inserts installed at all catch basin/inlet locations, for locations and additional information see the exhibits located in Map Pocket 2.

1.C Conditions of Concern

Conditions of concern for the project are related to any relevant hydrologic and environmental factors that are to be protected specific to the project area's watershed.

There are several reports and analyses that are relevant to this project site. They are titled, *Otay Ranch SPA Village 7 – Preliminary Regional Drainage Study Major Drainage Patterns and Facilities (dated August 2003)* and *Otay Ranch SPA Village 6 – Preliminary Regional Drainage Study Major Drainage Patterns and Facilities (dated December 13, 2001),* both reports have been prepared by P&D Consultants, Inc. A copy of the drainage study map titled, *Developed Drainage Basin Boundaries – Otay Ranch Village 7,* has been included in Map Pocket 1 of this WQTR. In addition to the above referenced reports, Rick Engineering Company has performed preliminary hydrologic, detention, and water quality analyses for the pre- and post-project conditions for the McMillin Village 7 project, located in Appendix B.

As discussed in Section 1.0 of this WQTR the grading concept for McMillin Village 7 has not been approved for final engineering. McMillin Village 7 shall convey post-project flows from the majority of the Eastern Urban Center (EUC), located upstream of Village Seven, east of the State Route 125. The EUC, which is currently undeveloped, is planned for high-density multi family residential, office and commercial land uses and will function as the urban center of the Otay Ranch. The runoff shall be conveyed from the EUC to Village 7 (Wolf Canyon) where it will confluence with the proposed McMillin Village 7 on-site storm drain system. At this confluence the developed flows from the EUC and a large portion of the McMillin Village 7 shall outlet into a forebay. The forebay will be followed by a series of extended detention basins (known as the Wolf Canyon Water Quality and Detention Basin), as shown on the exhibits titled, *Water Quality Technical Report Exhibit for Otay Ranch Village 7 (Alternative "A" and "B")* located in Map Pocket 2. In addition, the project is recreating receiving water (that will convey a portion of Village 7 treated post-project flows) that will be located adjacent to the series of basins proposed for Wolf Canyon.

In addition to the Wolf Canyon Basin, McMillin Village 7 is also proposing treating the post-project flows (that do not enter the extended detention basin) by using structural BMPs. Two alternatives are proposed for this project. Please refer to Section 4.2.C and Map Pocket 2 for a detailed description of both alternatives.

The northerly portion of McMillin Village 7 shall be served by a proposed storm drain system that will be conveyed to the Village 6 development and ultimately to Poggi Canyon. Post-construction structural BMPs shall be installed within this portion of McMillin Village 7 (before discharging through Village 6 and to Poggi Canyon) to treat post-project flows (please refer to Map Pocket 2 for both design alternatives). A regional detention facility further downstream has also been designed by Village 6 to capture and detain the post-project flows associated with this basin.

The drainage basin along the southerly portion of the McMillin Village 7 (high school site) will consist of a mass graded area (high school site) a portion of Magdalena Road and Rock Mountain Road. The high school site shall be served by a temporary desilting basin. In addition proposed storm drain systems shall be installed on Magdalena Road and Rock Mountain Road to convey the flows from the high school site and the runoff from the roads offsite in a westerly direction. Bio Clean Inlet Filter Inserts shall be installed in a portion of the inlets that are located within this portion of McMillin Village 7) as shown on the exhibits located in Map Pocket 2. Therefore, treating the post-construction runoff before discharging off-site.

Rick Engineering Company has performed preliminary analyses for detention and water quality volumes for the grading design scenario for the Wolf Canyon Water Quality and Detention Basin. Water quality calculations, prepared by Rick Engineering Company, have also been performed to design and the inlet filter inserts (BioClean) and in-line treatment facilities (CDS units) for the McMillin Village 7 project. The City of Chula Vista requires numeric sizing criteria be implemented for treatment control BMPs.

In addition, the July 2002 City of Chula Vista Subdivision Manual states, "the maximum allowable release rate after development shall not exceed predevelopment flow rates". Therefore, detention analyses have been performed to analyze the 2-year, 10-year, 100-year, 6-hour peak discharge back to equal or less than the pre-developed conditions and thus not adversely impacting downstream conditions along Wolf Canyon. The Wolf Canyon Basin shall be designed to handle, treat, and detain the post-project flows for the 2-, 10-, and 100-year storm event. The 2- and 10-year detention is to mitigate for downstream erosion (City of Chula Vista SUSMP) and the 100-year detention is to attenuate post-project flow rates to pre-project levels (City of Chula Vista Subdivision Design Manual). The drainage basin along the northerly portion of McMillin Village 7 shall be served by a storm drain system that will discharge through the Village 6 development and ultimately to Poggi Canyon. A regional detention facility further downstream has also been designed and built to capture and detain the post-project flows associated with this basin and Village 6.

1.D. Establish Permanent Storm Water Best Management Practices

As stated in the Storm Water Standards Manual (Section V.2), site design BMPs reduce the need for source and/or treatment control BMPs, and source control BMPs may reduce the amount of treatment control BMPs needed. Commercial and industrial facilities are required to have on-site Treatment Control BMPs appropriate for the potential pollutants generated at the site. As described below, all priority projects shall consider, incorporate, and implement where determined applicable and feasible by the City of Chula Vista, storm water BMPs into the project design, in the following progression: (1) Site Design BMPs, (2) Source Control BMPs, and (3) Treatment Control BMPs. Based on the Form 5500 titled, *Project Permanent Storm Water BMPs (SUSMP) Requirements*, located in Appendix A of this report, the McMillin Village 7 project is subject to "Priority Project Permanent Storm Water BMPs (SUSMP) Requirements." Based on Table 1 of the Storm Water Standards Manual titled, *Anticipated and Potential Pollutants Generated by Land Use Type*, and Form 5500, located in Appendix A the McMillin Village 7 project applies to the following priority project categories: "Attached Residential

Development", "Detached Residential Development", "Streets, Highways & Freeways", and "Commercial Development >100,000 ft^2".

The project will provide permanent storm water BMPs to ensure that water quality treatment is provided prior to storm water runoff discharging from the project site. The following sections 2.A through 2.C of this storm water quality technical report will discuss the permanent storm water BMPs proposed for the project. The specifics regarding design of the selected *structural treatment control* BMPs (as applicable) will be discussed in Section 5.0 of this WQTR. Underlined text and italicized text in the following discussion represents headings and line items from Section V.2 of the Storm Water Standards Manual. Portions of the italicized text are condensed from the Storm Water Standards Manual.

2.A Site Design BMPs

"Site design BMP" means any project design feature that reduces the creation or severity of potential pollutant sources, reduces the alteration of the project site's natural flow regime, or maintains or reduces pre-development downstream erosion and protects stream habitat. The following discussion identifies the site design BMPs from Section V.2.a of the Storm Water Standards Manual that are proposed for the McMillin Village 7 (residential development, elementary school, and park). However, for the portion of the high school site that is located within the McMillin Companies property, the permanent site design BMPs shall be the responsibility of the Sweetwater Unified High School District. For the interim condition, a temporary desilting basin shall be installed and maintained by McMillin Companies until ownership has been transferred.

Maintain Pre-Development Rainfall Runoff Characteristics

Design Concept 1: Minimize Project's Impervious Footprint and Conserve Natural Areas.

1. Minimize impervious footprint.

Landscaping surrounding the individual lots, elementary school, park site, and the extended detention basins shall be installed.

2. Conserve natural areas where feasible.

The McMillin Village 7 project will minimize adverse impacts to the downstream waterbodies and preserve the natural areas with the installation of a riprap outfall or some other device that shall dissipate energy at the downstream outlet of the extended detention facility located in Wolf Canyon. In addition, the extended detention facilities (Wolf Canyon Water Quality and Detention Basin) shall detain the 2-year, 10-year, 100-year, 6-hour post-project flows to pre-project levels to reduce adverse impacts downstream.

3. *Minimize directly connected impervious areas.*

   Where possible, rooftop downspouts will drain through adjacent landscaping prior to discharging to the storm water conveyance system.

4. *Construct walkways, trails, patios, overflow parking lots and alleys and other low-traffic areas with permeable surfaces, such as pervious concrete, porous asphalt, unit pavers, and granular materials.*

   The McMillin Village 7 project will provide landscaping and vegetation to the maximum extent practical.

5. *Maximize canopy interception and water conservation by preserving existing native trees and shrubs, and planing additional native or drought tolerant trees and large shrubs.*

   The project will direct runoff away from the tops of slopes, and will safely collect runoff through a network of swales, area drains, brow ditches, and the proposed underground storm drain systems.

6. *Minimize the use of impervious surfaces, such as decorative concrete, in the landscape design.*

   The project will provide native or drought tolerant vegetation for all vegetated slopes wherever possible.

7. *Use natural drainage systems to the maximum extent practicable.*

   In the preliminary design grading scenario, the majority of the project shall drain into the Wolf Canyon Water Quality and Detention Basin via a proposed storm drain that serves the central portion of the project. The Wolf Canyon Water Quality and Detention Basin will have one forebay (directly downstream of the storm drain outlet that serves the central portion of the project) followed by a series of extended detention basins that will treat and detain the post-project developed flows. In addition, a CDS unit installed at the downstream portion of the storm drain that outlets into the receiving water that is recreated (adjacent to the Wolf Canyon Basin). Depending on the design alternative chosen Bio Clean Inlet Filter Inserts and/or CDS units will treat the northern drainage basin and a combination of a temporary desilting basin and BioClean inserts will treat the southern drainage basin.

8. *Install energy dissipators, such as riprap, at the outlets of new storm drains, culverts, conduits, or channels that enter unlined channels in accordance with applicable specifications to minimize erosion.*

   At each storm drain outlet, an energy dissipater shall be installed.

Design Concept 2: Minimize directly connected Impervious Areas (DCIAs)

1. *Where landscaping is proposed, drain rooftops into adjacent landscaping prior to discharging to the storm drain.*

   Rooftop downspouts will drain through adjacent landscaping prior to discharging to the storm water conveyance system.
2. *Where landscaping is proposed, drain impervious sidewalks, walkways, trails, and patios into adjacent landscaping.*

   The McMillin Village 7 project will provide landscaping and vegetation to drain sidewalks, walkways, trails, and patios into adjacent landscaping to the maximum extent practical.
3. *Other design characteristics, which are comparable and equally effective.*

   See Above.

Protect Slopes and Channels

1. *Convey runoff safely from the tops of slopes.*

   The project will direct runoff away from the tops of slopes, and will safely collect runoff through a network of swales, area drains, brow ditches, and proposed underground storm drain systems.
2. *Vegetate slopes with native or drought tolerant vegetation.*

   The McMillin Village 7 project will provide native or drought tolerant landscaping and vegetation wherever possible.
3. *Control and treat flows in landscaping and/or other control prior to reaching existing natural drainage systems.*

In the preliminary design grading scenario, the majority of the project shall drain into the Wolf Canyon Water Quality and Detention Basin via a proposed storm drain that serves the central portion of the project. The Wolf Canyon Water Quality and Detention Basin will have one forebay (directly downstream of the storm drain outlet that serves the central portion of the project) followed by a series of extended detention basins that will treat and detain the post-project developed flows. In addition, a CDS unit installed at the downstream portion of the storm drain that outlets into the receiving water that is recreated (adjacent to the Wolf Canyon Basin). Depending on the design alternative chosen Bio Clean Inlet Filter Inserts and/or CDS units will treat the northern drainage basin and a combination of a temporary desilting basin and BioClean inserts will treat the southern drainage basin.

Stabilize permanent channel crossings.

The McMillin Village 7 shall stabilize all permanent channel crossings.

4. *Install energy dissipater to minimize erosion.*

   The McMillin Village 7 project will minimize adverse impacts to the downstream waterbodies and preserve the natural areas with the installation of a riprap outfall or some other device that shall dissipate energy at the downstream outlet of the extended detention facility (Wolf Canyon Water Quality and Detention Basin). The extended detention facilities (Wolf Canyon Water Quality and Detention Basin) shall detain the 2-year, 10-year, 100-year, 6-hour post-project flows to pre-project levels. In addition, at the storm drain outlet, an energy dissipater shall be installed within the forebay.

5. *Other design principles that are comparable and equally effective.*

   See Above.

2.B Source Control BMPs

"Source control BMPs (both structural and non-structural)" means land use or site planning practices, or structures that aim to prevent urban runoff pollution by reducing the potential for contamination at the source of pollution. Source control BMPs minimize the contact between pollutants and urban runoff. Examples include roof structures over trash or material storage areas, and berms around fuel dispensing areas.

The following discussion identifies the site design BMPs from Section V.2.b of the Storm Water Standards Manual that are proposed for the McMillin Village 7 project (residential development, elementary school, and park). However, for the portion of the high school site that is located within the McMillin Companies property, the permanent source control BMPs shall be the responsibility of the Sweetwater Unified High School District. For the interim condition, a temporary desilting basin shall be installed and maintained by McMillin Companies until ownership has been transferred.

Provide Storm Water System Stenciling and Signage

Typical concrete stamping procedures will be provided with respect to the needs of the project. In addition, the educational material shall be provided to the home owners, residents, employees and others in order to aid in the prevention of pollutants entering directly into the storm drain system. A standard of maintenance will also be established for the project site (including post-construction BMPs) as well as any drainage improvements that may be installed within any part of the project's property boundaries. These responsibilities will transfer over to the individual property owners when the project is completed and new ownership is in place. The structural BMPs and the maintenance responsibilities for them will be discussed in further detail later within this storm water quality technical report.

Design Outdoor Materials Storage Areas to Reduce Pollution Introduction

Following construction, no hazardous materials are anticipated to be stored within the McMillin Village 7 project site. In the case that hazardous materials are located within the project, the hazardous materials will be stored in sheds, which will be built within secondary containment structures.

Design Trash Storage Areas to Reduce Pollution Introduction

Trash storage areas will be enclosed and covered in approved storage bins.

Use Efficient Irrigation Systems & Landscape Design and Employ Integrated Pest Management Principles (*Requirements 13-15 Limited exclusion: detached residential homes*)

The design of irrigation systems to each landscape area's specific water requirements and the requirement that specifies the use of flow reducers or shutoff valves triggered by a pressure drop to control water loss in the event of broken sprinkler heads or lines. Specific methodologies on how to eliminate or reduce the need for pesticide use are yet to be determined. In addition, the following materials will be distributed to future site residents/tenants:

- Keeping pests out of buildings
- Physical pest elimination techniques
- Understanding natural pest predators
- Proper use of pesticides

Incorporate Requirement Applicable to Individual Priority Project Categories

Table 2 of the Storm Water Standards Manual identifies additional BMPs that are required for specific priority project categories. The McMillin Village 7 project is associated with four priority project categories: detached residential development, attached residential development, streets, highways, and freeways, and commercial development >100,000 ft^2. The following discussion identifies the category specific BMPs from Section V.2.b of the Storm Water Standards Manual that are applicable to the McMillin Village 7 project.

Private Roads

The design of private roadway drainage shall use at least one of the following (for further guidance, see Start at the Source [1999]): (1) rural swale system – street sheet flows to vegetated swale or gravel shoulder, curbs at street corners, culverts under driveways and street crossings; (2) urban curb/swale system – street slopes to curb, periodic swale inlets drain to vegetated swale/biofilter; or (3) dual *drainage system – first flush captured in street catch basins and discharged to adjacent vegetated swale or gravel shoulder.*

<u>*Residential Driveways & Guest Parking*</u>

Driveways shall have one of the following: (1) shared access; (2) flared entrance (single lane at street); (3) wheelstrips (paving only under tires); (4) porous paving; or (5) designed to drain into landscaping prior to discharging to the storm water conveyance system. Uncovered temporary or guest parking on private residential lots shall be: (1) paved with permeable surface; or (2) designed to drain into landscaping prior to discharging to the storm water conveyance system.

<u>*Hillside Landscaping*</u>

Hillside areas, as defined in this SUSMP, that are disturbed by project development shall be landscaped with deep-rooted, drought tolerant plant species selected for erosion control, satisfactory to the City of Chula Vista

<u>*Roadways*</u>

Priority roadway projects shall select treatment control BMPs following the enhanced treatment control selection procedure identified in Section V.2 of the Storm Water Standards Manual titled, Establish Storm Water BMPs.

<u>*Dock Areas*</u>

Loading/unloading dock areas shall include the following: (1) cover loading dock areas, or design drainage to preclude urban run-on and runoff; and (2) An acceptable method of containment and pollutant removal, such as a shut-off valve and containment area. Direct connections to storm drains from depressed loading docks (truck wells) are prohibited.

<u>*Maintenance Bays*</u>

Maintenance bays shall include at least one of the following: (1) repair/ maintenance bays shall be indoors; or, (2) designed to preclude urban run-on and runoff.

In order to address the specific storm water BMP requirements listed above, all areas of the proposed McMillin Village 7 project will be treated through the proposed structural treatment control BMPs discussed below.

2.C Treatment Control BMPs

"Treatment Control (Structural) BMP" means any engineered system designed and constructed to remove pollutants from urban runoff. Pollutant removal is achieved by simple gravity settling of particulate pollutants, filtration, biological uptake, media adsorption or any other physical, biological, or chemical process.

The following discussion identifies the treatment control BMPs from Section V.2.c of the Storm Water Standards Manual that are proposed for the McMillin Village 7 project (residential development, elementary school, and park). However, for the portion of the high school site that is located within the McMillin Companies property, the permanent treatment control BMPs shall be the responsibility of the Sweetwater Unified High School District. For the interim condition, a temporary desilting basin shall be installed and maintained by McMillin Companies until ownership has been transferred.

There are two alternatives proposed for this project. Both alternatives use a combination of structural treatment control BMPs (in addition to the Wolf Canyon Water Quality and Detention Basin) to treat post-project flows, generated from the project, before leaving the site. Located in Map Pocket 2 are two alternatives; Alternative A and B. Alternative A proposes treating post-project flows tributary to the storm drain systems that are located in Magdalena Road and Rock Mountain Road and the storm drain system that serves the northern basin (that drains to Poggi

Canyon) with inlet filter inserts. In addition, the project proposes an in-line treatment facility at the downstream portion of the storm drain system that outlets into the receiving water associated with the Wolf Canyon Basin. Alternative B proposes three in-line treatment facilities. One shall be located at the downstream portion of the storm drain system that outlets into the receiving water associated with the Wolf Canyon Basin. Another unit shall be installed at the downstream portion (near the intersection of Magdalena Road and Birch Parkway) of the storm drain system that serves the drainage basin that flows northerly to Poggi Canyon. The last unit shall be associated with the storm drain system that is located on Magdalena in the central portion of the project, just before the confluence with the Wolf Canyon Basin flows. For the remainder portion of the project (storm drain systems that serve the southern portion of the project) inlet filter inserts shall be installed. For location of the structural treatment control BMPs described above please refer to the exhibits located in Map Pocket 2.

Design to Treatment Control BMP Standards

Numeric sizing criteria was implemented to design the post-construction BMPs associated with McMillin Village 7. These BMPs include the Water Quality portion of the Wolf Canyon Water Quality and Detention Basin, BioClean inserts, and the in-line treatment facilities. In addition, a temporary desilting basin was designed at the high school site within the southern drainage basin based on the General Construction Permit. The specifics regarding the design of the post-construction BMPs and the temporary desilting basin will be discussed in Section 5.0 of this WQTR.

Locate BMPs Near Pollutant Sources

All post-construction BMPs shall be on-site and treat the post-developed flows according to the design criteria stated in the Storm Water Standards Manual.

Restrictions on Use of Infiltration BMPs

There are no infiltration devices associated or proposed for the McMillin Village 7 project.

5.0 PERMANENT STORM WATER BEST MANAGEMENT PRACTICES (BMPs)

Structural BMPs will be designed pursuant to the drainage areas shown on the exhibit titled, *Preliminary Existing Drainage Basin Boundaries – Otay Ranch Village 7* and *Developed Drainage Basin Boundaries – Otay Ranch Village 7* along with, the proposed location of each post-construction BMP as shown on the exhibit titled, *Water Quality Technical Report Exhibit for Otay Ranch Village 7*, located in Map Pockets 1 and 2, respectively. As discussed in Section 4.0 of this report, site and source control BMPs will be implement for this project. However, the following is a discussion of the on-site post-construction structural treatment control BMPs that will be used for this project:

Water Quality and Detention Basin

McMillin Village 7 shall convey post-project flows from the majority of the Eastern Urban Center (EUC), located upstream of Village Seven, east of the State Route 125. The EUC, which is currently undeveloped, is planned for high-density multi family residential, office and commercial land uses and will function as the urban center of the Otay Ranch. The runoff shall be conveyed from the EUC to Village 7 (Wolf Canyon) where it will confluence with the proposed McMillin Village 7 on-site storm drain system. At this confluence the developed flows from the EUC and a large portion of the McMillin Village 7 shall outlet into a forebay. The forebay will be followed by a series of extended detention basins (known as the Wolf Canyon Water Quality and Detention Basin), as shown on the exhibit located in Map Pocket 2.

Field surveys have been performed by Helix Environmental Planning, Inc. for the upper portion of Wolf Canyon (McMillin Village 7 project site). The surveys, conducted in 2003, determined that the canyon consists of un-vegetated, non-wetland waters. This is consistent with surveys conducted as part of the Otay Ranch General Development Plan Environmental Impact Report. In addition, this area has limited functions and values for wildlife and water quality, as it has been in active agricultural usage up to the present day. Therefore, it was determined that the appropriate location for the extended detention basin would be in Wolf Canyon, just upstream of the proposed Magdalena Road. In the design scenario for McMillin Village 7, the extended detention basin and the forebay shall be equipped to handle, treat, and detain the post-project flow from the EUC and the majority of the residential portion of McMillin Village 7. The project proposes regional benefits with respect to water quality, while maximizing conservation of more significant biological resources offsite in areas with much higher long-term conservation value. The project site is not identified as a Conserved Area in the City of Chula Vista's Multiple Species Conservation Program (MSCP) Subarea Plan. The basin proposes regional benefits with respect to water quality, with minimal impact to biology.

The Wolf Canyon Water Quality and Detention Basin, has been designed in such a way that the forebay, located at the upstream portion of the basin, will capture the trash and debris. The extended detention basins, located downstream of the forebay, have been designed and sized to allow particles and associated pollutants to settle. The extended detention basins shall detain the 2-year, 10-year, 100-year, 6-hour flow rates to pre-project levels and treat the required flow volumes. For calculations associated with the basin, please see Appendix B. Table 1 and 2 is a summary of the results. Upon final design the 2-year and 10-year analyses shall be performed.

Table 1: Summary of the Hydrologic Analyses for the Extended Detention Facilities (Wolf Canyon Water Quality and Detention Basin)

Pre-Project		Post-Project		
Drainage Area (ac)	100-Year, 6-Hour Flow Rate (cfs)	Drainage Area (ac)	100-Year, 6-Hour Flow Rate (cfs)	Detained 100-Year Flow Rate (cfs)
240.4	254.4	281.6	716.5	254.4

Table 2: Summary of the Volume Results for the Extended Detention Facilities (Wolf Canyon Water Quality and Detention Basin)

Water Quality Basin (Volume in ac-ft)	100-Year, 6-Hour Detention (Volume in ac-ft)
11.3	20.0

** Includes mitigation for the portion of the EUC tributary to the extended detention basin*

The drainage basin along the northerly portion of McMillin Village 7 shall be served by a storm drain system that will install inlet filter inserts to treat the post-project runoff, before discharging through the Village 6 development and ultimately to Poggi Canyon. A regional detention facility further downstream has also been designed and built to capture and detain the post-project flows associated with this basin and Village 6. The analysis for the regional detention facility located in the December 13, 2001 report titled, *Otay Ranch SPA Village 6 - Preliminary Drainage Study Major Drainage Patterns and Facilities*, prepared P&D Consultants, Inc. Upon final design the approved treatment control BMPs shall be shown on he improvement plans and approved by the director of Public Works.

<u>BioClean Inlet Filter Inserts</u>

BioClean inserts (with sorbent material added) have been proposed for the McMillin Village 7 project to meet the storm water quality requirements set forth in the City of Chula Vista Storm Water Standards Manual. As described in Section 4.2.C, two alternatives are proposed for this project (with respect to structural treatment control BMPs). Please refer to the exhibits titled, , *Water Quality Technical Report Exhibit for Otay Ranch Village 7 (Alternative A and B)*, located in Map Pocket 2 for the location of the inlet filter inserts with respect to the Design Alternative.

BioClean inserts (with sorbent material added) are flow-based BMPs. BioClean inserts reduce sediment, trash and debris, oil and grease from the flow and pesticides that attach to sediment. BioClean inserts must be capable of treating the required treatment flow for the area of the project site draining to either type of inlet. These BMPs will be incorporated to meet the requirements of the Storm Water Standards manual and will be sized using a flow-based numeric sizing criteria.

It is important to note, that no private storm drain systems will be tied into the municipal storm drain system unless the water has been treated by some other post-construction BMP. As a result, all drainage (including runoff collected in the area drains) will be directed through the BioClean inserts, therefore meeting the requirements for water quality treatment.

Calculations for water quality treatment flow requirements, water quality treatment capacities, details, and approximate costs for the BioClean inserts have been prepared and are found in Appendix B and C, where applicable. In addition, a summary of the approximate material, installation, and maintenance costs shall be included in Appendix C. Final selection of publicly maintained curb inlet filtration devices shall be shown on the street improvement plans and approved by the Director of Public Works. See table 4 for a summary of the results.

Table 3: Summary of Treatment Flow Calculations for the BioClean Inlet Filter Inserts

Location of Inlet Filter Insert*	Composite Runoff Coefficient**	Intensity (in/hr)***	Area (acre)	Treatment Flow Rate (cubic feet/second)
1	0.7	0.2	0.9	0.1
2	0.8	0.2	1.1	0.2
3	1.0	0.2	0.3	0.1
4	0.8	0.2	1.4	0.2
5	0.8	0.2	0.9	0.1
6	0.7	0.2	2.3	0.3
7	0.7	0.2	2.0	0.3
8	0.7	0.2	1.0	0.1
9	0.7	0.2	1.0	0.1
10	0.7	0.2	1.3	0.2
11	0.7	0.2	1.3	0.2
12	0.7	0.2	1.0	0.1

13	0.7	0.2	1.7	0.2
14	0.8	0.2	0.7	0.1
15	0.7	0.2	0.8	0.1
16	1.0	0.2	0.1	0.0
17	1.0	0.2	0.1	0.0
18	0.7	0.2	1.1	0.2
19	0.8	0.2	0.9	0.1
20	0.7	0.2	2.0	0.3
21	0.7	0.2	1.1	0.2
22	0.7	0.2	3.7	0.5
23	0.7	0.2	2.8	0.4
24	0.8	0.2	0.5	0.1
25	0.8	0.2	0.9	0.1
26	0.7	0.2	1.2	0.2
27	0.8	0.2	5.8	0.9
28	0.8	0.2	1.3	0.2
29	1.0	0.2	0.2	0.0
30	1.0	0.2	0.3	0.1
31	1.0	0.2	0.5	0.1
32	0.9	0.2	1.1	0.2
33	1.0	0.2	0.9	0.2
34	0.9	0.2	0.9	0.2
35	0.9	0.2	1.1	0.2
36	0.7	0.2	1.3	0.2
37	0.9	0.2	0.6	0.1
38	0.9	0.2	0.7	0.1
39	0.8	0.2	1.1	0.2
40	0.9	0.2	0.8	0.1
41	1.0	0.2	0.9	0.2
42	0.8	0.2	1.1	0.2
43	0.8	0.2	1.5	0.2
44	0.7	0.2	1.7	0.2
45	0.8	0.2	0.2	0.0
46	1.0	0.2	0.1	0.0
47	0.8	0.2	1.3	0.2
48	0.8	0.2	1.2	0.2
49	0.9	0.2	0.8	0.1

* Refer to Water Quality Technical Report Exhibit for Otay Ranch Village 7 (Alternative A and B), located in Map Pocket 1.

** Calculations for Composite Runoff Coefficient are found in Appendix B: Inlet Filter Inserts.

*** Pursuant to the City of Chula Vista Development and Redevelopment Project Storm Water Management Standards Requirements Manual the Numeric Sizing Criteria requires the intensity for flow based BMPs to be 0.2 inches/hour.

CDS In-Line Treatment Facility

CDS Units have been proposed for the McMillin Village 7 project to meet the storm water quality requirements set forth in the City of Chula Vista Storm Water Standards Manual. The CDS Units shall be installed at the locations as indicated in the exhibit titled, *Water Quality Technical Report Exhibit for Village 7 (Alternative A and B),* located in Map Pocket 2.

The CDS Unit is a flow-based BMP. A CDS Unit reduces sediment, trash and debris, oil and grease from the flow and pesticides that attach to sediment. The CDS Unit must be capable of treating the required treatment flow for the area of the project site draining to the facility. This BMP will be incorporated to meet the requirements of the Storm Water Standards manual and will be sized using a flow-based numeric sizing criteria.

By utilizing the CDS Unit to provide water quality treatment for the northerly drainage basin, it is important to ensure that there will be no roof drains, area drains, or any other type of local underground drainage systems conveying untreated runoff into the underground storm drain system without passing through the CDS Unit. At this time, it is anticipated that all roof drains will be discharging to adjacent landscaping and area drains, which discharge to the curb and gutter. As a result, all drainage (including runoff collected in the area drains) will be directed through the storm drain system and directly into the CDS Unit, therefore meeting the requirements for water quality treatment.

Calculations for water quality treatment flow requirements, water quality treatment capacities, details, and approximate costs for the CDS Unit have been prepared and are found in Appendix B and C, where applicable. In addition, a summary of the approximate material, installation, and maintenance costs shall be included in Appendix C. See table 3 for a summary of the results.

Table 3: Summary of Treatment Flow Calculations for the CDS Unit

Location of CDS Unit*	Composite Runoff Coefficient**	Intensity (in/hr)***	Area (acre)	Treatment Flow Rate (cubic feet/second)
1	0.8	0.2	41.3	6.6
2	0.95	0.2	9.0	1.7
3	0.8	0.2	16.9	2.7

* Refer to Water Quality Technical Report Exhibit for Otay Ranch Village 7, located in Map Pocket 2.
** Calculations for Composite Runoff Coefficient are found in Appendix B: CDS Unit.
*** Pursuant to the City of Chula Vista Development and Redevelopment Project Storm Water Management Standards Requirements Manual the Numeric Sizing Criteria requires the intensity for flow based BMPs to be 0.2 inches/hour.

As described in Section 4.2.C, two alternatives are proposed for this project (with respect to structural treatment control BMPs). Please refer to the exhibits titled, , *Water Quality Technical Report Exhibit for Otay Ranch Village 7 (Alternative A and B)*, located in Map Pocket 2 for the location of the in-line treatment facilities with respect to the Design Alternative.

Temporary Desilting Basin

The drainage basin located at the southern portion of the McMillin Village 7, in the post-project condition, consists of a mass graded pad. This is the future location for the proposed high school (Sweetwater Unified High School District). The mass graded site shall be served by a temporary desilting basin with an approximate volume of 192,000 cubic-feet and 3:1 side slopes. The proposed dimensions of the bottom of the basin is approximately 380 feet in length, a proposed width of 160 feet and a proposed depth of 6.5 feet (including freeboard). The proposed basin will also have a 60-inch outlet riser with two feet of freeboard. A dual 60-inch riser will also be installed to act as an emergency outlet/spillway. Once ownership for the high school site has been transferred to the Sweetwater Unified High School District, and construction is complete, permanent post-construction BMPs shall be designed and installed by the new owner.

The temporary desilting basin is a volume-based BMP. A temporary desilting basin reduces sediment and debris from runoff. The temporary desilting basin must be capable of treating the required treatment volume for the area of the project site draining to the basin. This BMP will be incorporated to meet the requirements of the General Construction Permit and will be sized using a volume-based numeric sizing criteria.

Calculations for water quality treatment volume requirements and water quality treatment capacities for the temporary desilting basin have been prepared and are found in Appendix B. Due to the nature of the project the installation costs and maintenance costs could not be provided.

6.0 ANTICIPATED MAINTENANCE CONDITION(S)

Following the completion of the project and the transferring of ownership, the maintenance responsibilities shall have to be appointed. A Home Owners Association (HOA) will be responsible for all common areas (such as streets and open space) not transferred over to the private individual property owners. The HOA shall be responsible for properly disposing of waste material from their assumed areas within the project site, maintaining landscaping throughout those areas in a manner that will prevent soil erosion and minimize sediment transport, and shall maintain drainage facilities located throughout the project area in a clean manner and in good repair. All post-construction structural BMPs will be maintained by the City of Chula Vista (which includes the maintenance associated with the forebay, water quality and detention basin, in-line treatment facilities, and the inlet filter inserts). In addition, maintenance of the temporary desilting basin (future high school site) shall be the responsibility of the City of Chula Vista until ownership is transferred. Upon final design, the maintenance responsibilities shall be appointed. As discussed in Section 4.0 of this report, site and source control BMPs will be implement for this project. However, the following is a discussion of the anticipated maintenance condition(s) for the on-site post-construction structural treatment control BMPs:

Water Quality and Detention Basin

The following section discusses the maintenance of the post-construction BMPs. The extended detention and water quality basins shall be inspected regularly during the rainy season between October 1 and April 30. The extended detention basin inlet and outlet shall be inspected for trash, litter, debris, or other solid materials that may hinder the intended function of extended detention basin. Any eroded areas shall be restored and re-vegetated. Invasive species shall be removed as needed.

The maintenance procedure for servicing the extended detention basin consist of mostly sediment, trash, and debris removal. In addition, regards to vegetation management shall have to be taken into consideration. Typical actives and frequencies include:

1) Schedule semiannual inspection for the beginning and end of the wet season for standing water, slope stability, sediment accumulation, trash and debris, and presence of burrows.
2) Remove accumulated trash and debris in the basin and around the riser pipe/outlet during the semiannual inspections. The frequency of the activity may be altered to meet specific site conditions.
3) Trim vegetation at the beginning and end of the wet season and inspect monthly to prevent establishment of woody vegetation and for aesthetic and vector reasons.
4) Remove accumulated sediment and re-grade about every 10-year or when the accumulated sediment volume exceeds 10 percent of the basin volume. Inspect the basin each year for accumulated sediment volume.

BioClean Inserts

The frequency of maintenance required for the BioClean inserts is site and drainage area specific. The inserts should be inspected periodically to assure its condition is adequate to handle anticipated runoff. Initially following the installation of the BioClean inserts, it is important to check that the insert is functioning properly and measure the amount of deposition occurring from specific storm events. At a minimum, inspections should be made on a monthly basis and after every storm event to check that the unit is functioning properly and whether the insert requires servicing at that time. Based on these inspections, it may be necessary to adjust the frequency of scheduled inspections and maintenance cleanings.

The BioClean insert service procedures include the removal of the manhole cover, properly disposing of the waste, replacing the hydrocarbon pouches as necessary (sorbent material), inspecting for needed repairs and/or replacement of the filter medium, closing the manhole cover, properly disposing of the waste, and recording the maintenance service for future reference.

CDS In-Line Treatment Facilities

The maintenance frequency or schedules of the CDS Unit are site specific and depend upon particular land use activities and the amount of gross pollutants and sediment generated within the drainage area. CDS Technologies Incorporated (manufactures of the CDS Unit) recommend that CDS units typically need to be cleaned out approximately 2 to 4 times per year.

Maintenance of a CDS unit consists of cleaning out the sump via a vactor trunk on a seasonal basis and an annual inspection of the screen surface. After removal of the trash and debris the CDS screen and sump can then be inspected visually to determine overall condition of the CDS Unit.

Maintenance costs provided by CDS Technologies Incorporated are based on a typical four hour minimum retail clean-out charge at $100 to $125 per hour, resulting in a minimum cost of $400.

Temporary Desilting Basin

The temporary desilting basin shall be inspected prior to forecast rain, daily during extended rain events, after rain events, weekly during the rainy season, and at two-week intervals during the non-rainy season. Maintenance should include checking the inlet and outlet structures and spillway for any damage, obstructions, and erosion. Sediment accumulation greater than one-half of the designated storage volume within the desilting basin shall be removed periodically. Maintenance shall also include minimizing vector production by removing accumulated live and dead floating vegetation in addition to removing excessive perimeter vegetation.

Responsible Party for Maintenance and Funding of Structural BMPs

The owner of the McMillin Village 7 project will be responsible for compliance with the NPDES Construction permit and the City of Chula Vista Development and Redevelopment Projects

Storm Water Management Standards Requirements Manual (which covers compliance for the City of Chula Vista SUSMP requirements for this project) until ownership is transferred to individual lot property owners. Specifically, the owner of the project will be responsible for BMPs during construction while this project is under construction. However, when construction of this project is completed and a change of ownership occurs, the City of Chula Vista and/or HOA will assume responsibility for the funding of the future maintenance for all post-construction BMPs as described above.

7.0 SUMMARY

The McMillin Village 7 project will conform to applicable NPDES requirements during and after construction. During the construction phase, the project will be subject to the requirements of the General Construction Permit. The project will meet the requirements of the General Construction Permit by implementing a site-specific SWPPP and incorporating temporary BMPs for control of sediment and non-visible pollutants. The site inspection requirements and site-specific storm water sampling and analysis strategy (SWSAS) required in the SWPPP will provide an evaluation of the effectiveness of the BMPs. Adjustments to the BMPs will be made as necessary to maintain or improve effectiveness. The completed project will incorporate a PCSWOMP as a requirement for termination of coverage under the General Construction Permit. The completed project will also require an amount of runoff to be treated, infiltrated, or filtered based on numeric sizing criteria established in the City of Chula Vista SUSMP and the City of Chula Vista Development and Redevelopment Projects Storm Water Management Standards Requirements Manual.

The Wolf Canyon Water Quality and Detention Basin shall convey the developed flows from a portion of the McMillin Village 7 and a portion of the EUC to Wolf Canyon and outlet into a forebay, located at the upstream portion of the basin. The forebay will be responsible to remove sediment and trash. The northern and southern drainage basins shall treat runoff with BioClean inserts. In addition, the project is recreating a receiving water (that will convey a portion of Village 7 treated post-project flows) that will be located adjacent to the series of basins proposed for Wolf Canyon. Also, a temporary desilting basin shall serve the proposed high school site.

The completed project will incorporate a treatment train of non-structural and structural BMPs that may include property owner education, stenciled inlets, street sweeping, landscaping, forebay, extended detention basins, water quality basins, in-line treatment facilities, and inlet filter inserts in order to meet the applicable requirements of the General Construction Permit and the Storm Water Standards manual. Based on the information and analyses provided within this water quality technical report, the McMillin Village 7 project will incorporate post-construction BMPs as shown on the exhibit titled, *Water Quality Technical Report Exhibit for Otay Ranch Village 7 (Alternative A and B),* located in Map Pocket 2.

APPENDIX A

City of Chula Vista Form 5500 and 5501

Project Permanent Storm Water BMPs (SUSMP) Requirements

CITY OF CHULA VISTA	ENGINEERING 276 Fourth Avenue, Chula Vista, CA 91910 619-691-5021 619-691-5171 FAX	PROJECT PERMANENT STORM WATER BMPs (SUSMP) REQUIREMENTS
	FORM 5500	

Appendix A

Complete the following checklist to determine the project's per manent and construction best management practices requirements. This form must be completed and submitted with the permit application.

If one or more questions in the checklist are answered "Yes," the project is subject to the "Priority Project Permanent Storm Water BMPS (SUSMP)" requirements in Appendix B. If all answers are "No", please complete Form 5501 to determine if the project is subject to the "Standard Permanent Storm Water BMP" requirements.

Does the project meet the definition of one or mor e of the priority project categories? Also, refer to the definition in Appendix F for expanded definition of the Significant Redevelopment priority project

		Yes	No
1.	Detached residential development of 10 or more units	X	
2.	Attached residential devel opment of 10 or more units	X	
3.	Commercial development greater than 100,000 square feet	X	
4.	Automotive repair shop		X
5.	Restaurant		X
6.	Steep hillside development greater than 5,000 square feet		X
7.	Project discharging to receiving waters within Envi ronmentally Sensitive Areas		X
8.	Parking lots greater than or equal to 5,000 square feet or with at least 15 parking spaces, and potentially exposed to urban runoff		X
9.	Streets, roads, highways, and freeways which create a new paved surface that is 5,00 0 square feet or greater	X	

* Refer to the definitions in Appendix F for expanded definitions of the priority project categories.

Limited Exclusion: Trenching and resurfacing work associated with utility projects are not considered priority projects. Parking lots, buildings and other structures associated with utility projects are priority projects if one or more of the criteria is met.

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Permanent Storm Water BMPs Requirements

CITY OF CHULA VISTA	ENGINEERING 276 Fourth Avenue, Chula Vista, CA. 91910 619-691-5021 619-691-5171 FAX	PERMANENT STANDARD STORM WATER BMPs REQUIREMENTS
	FORM 5501	

Appendix A

Section 1
Complete the following checklist to determine if the project is subject to "Permanent Standard Storm Water BMPs" requirements.

If one or more questions in the following checklist are answered "Yes", the project is subject to the applicable "Permanent Standard Storm Water BMPs" requirements identified in Section 2 of this Form 5501. If all answers are "No", the project is exempt from permanent storm water BMPs requirements.

	Does the project propose:	Yes	No	Applicable BMP (refer to Section 2 of this Form 5501)
1.	New impervious areas, such as rooftops, ro ads, parking lots, driveways, paths, and sidewalks?	X		A.1, A.2, B1, C.1, C.2, C.8, C11
2.	New pervious landscape areas and irrigation systems?	X		A.1, A.2, B.4, C.10
3.	Permanent structures within 100 feet of any natural water body?		X	A.1, A.2, A.3
4.	Trash storage areas?	X		B.3
5.	Liquid or solid material loading and unloading areas?		X	B.2, C.3
6.	Vehicle or equipment fueling, washing, or maintenance areas?		X	C.4, C.5, C6, C.7, C.9
7.	Require a General NPDES permit for Storm Water Discharges Associated with Industrial Activities (except Construction)? *		X	Applicable BMPs
8.	Commercial or industrial waste handling or storage, excluding typical office or household waste?		X	B2, B3, C.3, C.6
9.	Any grading or ground disturbance during construction?	X		A.1, A.2, A.3, C10
10	Any new storm drains, or alteration to existing storm drains?	X		A.3, B.1, C11

*To find out if the project is required to obtain an individual General NPDES Permit for Storm Water Discharges Associated with Industrial Activities, visit the State Water Resources Control Board web site at, www.swrcb.ca.gov/stormwtr/industrial.html. Applicable BMPs shall be selected from Section 2 of this Form 5501.

APPENDIX B

Calculations for Water Quality Treatment Flow Requirements

Wolf Canyon Water Quality and Detention Details

Prepared By:
Rick Engineering Company – Water Resources Division

DCB:JW:nd/Report/14483.005
9-19-03
Revised: 10-20-03
Revised: 02-27-04
Revised: 03-26-04
Revised: 05-24-04

Engineer's Computation Pad

OTAY RANCH VILLAGE 7 WOLF CANYON
J-14483
OCTOBER 17, 2003

WATER QUALITY CALCULATION FOR
EUC & MCMILLIN OWNED PORTION:

VOLUME BASED -

PRECIPITATION, P = 0.6 IN
RUNOFF COEFFICIENT, C = 0.80
AREA, A = 281.6 AC

$$V = PCA = 0.6\text{ in}\left(\frac{1\text{ ft}}{12\text{ in}}\right)(0.80)(281.6\text{ AC})$$

$$V = 11.3\text{ AC-FT}$$

DETENTION BASIN DESIGN METHODOLOGY AND CRITERIA

Criteria

Design Storm

The detention basin will be designed to attenuate the 100-year storm event for the post-project condition so that the maximum allowable release rate after development shall not exceed the pre-project condition flow rates. This detention criteria is described in more detail in the July 1, 2002 City of Chula Vista *Subdivision Manual.*

Methodology

Planning and design of drainage facilities requires analyses of the undetained peak runoff of the watersheds for the pre- and post-project condition. As a result, regional hydrologic analyses were performed using the modified rational method. The modified rational method, as presented in the July 1, 2002 City of Chula Vista *Subdivision Manual*, was used for estimating peak discharge for the drainage basin for the pre- and post-project conditions. The United States Army Corps of Engineers' HEC-1 computer program was used to analyze the detention volume required for each basin.

A major component of the project is limiting post-development discharge to no greater than the pre-development peak discharge through the use of a detention facility for a required storm event. The sizing of a detention facility requires an inflow hydrograph to obtain the necessary storage volume. The Rational Method only yields a peak discharge. In order to convert peak discharge into a hydrograph, a Rational Method hydrograph synthesizing procedure was developed by San Diego County Flood Control.

The Rational Method hydrograph synthesizing procedure is as follows: The design storm pattern is based on the July 1, 2002 City of Chula Vista *Subdivision Manual.* This criteria uses the following equation to relate the intensity (I) of the storm to time of concentration (Tc):

Equation 1

$I = 7.44\ P_6 D^{-0.645}$

I = Intensity (inches/hour)

P_6 = 6-hour precipitation (inches)

D = Duration, assumed to equal the time of concentration (minutes)

The intensity at any given multiple of the time of concentration can be calculated by the following equation:

Equation 2

$I = ((I_{Tcn})(Tcn/60)-(I_{Tc(n-1)})(Tc(n-1)/60))/Tc$

n = Number of Hydrograph Ordinates

Tcn = Time of Concentration at Ordinate n (minutes)

I_n = Rainfall Intensity at Hydrograph Ordinate n

I_{Tcn} = Rainfall Intensity at Time of Concentration Tcn (inches/hour)

Figure 1 shows the rainfall distribution used for the rational method hydrograph. Rainfall is computed at multiples of time of concentration. The rainfall at 1 Tc is centered at three hours. The rainfall at 2 Tc is started at three hours + 1/2Tc. The rainfall at 3 Tc is started at three hours - 1 1/2Tc, and so on.

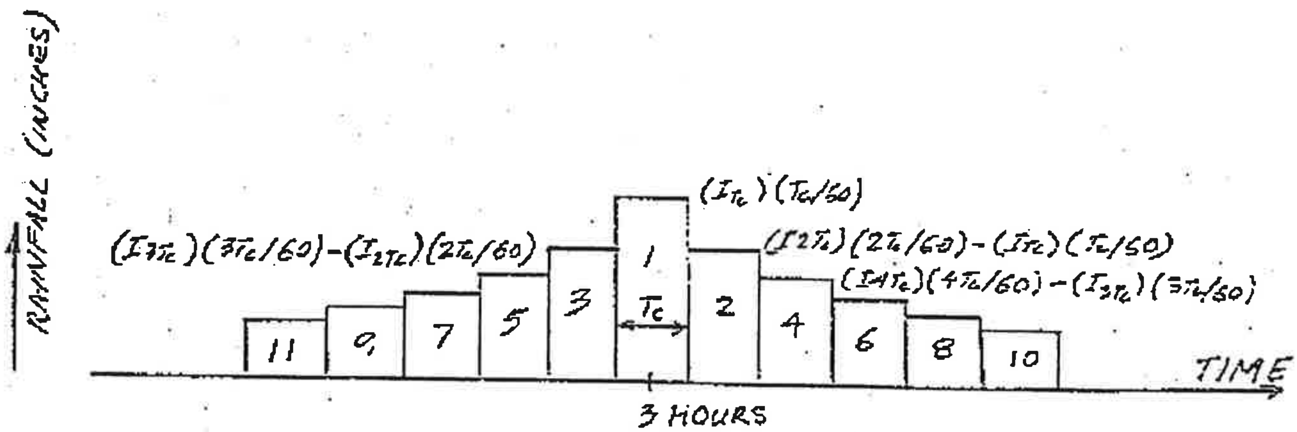

FIGURE 1.

In order to determine peak discharge of the hydrograph at any given multiple of the time of concentration, the following equation is used (Figure 2):

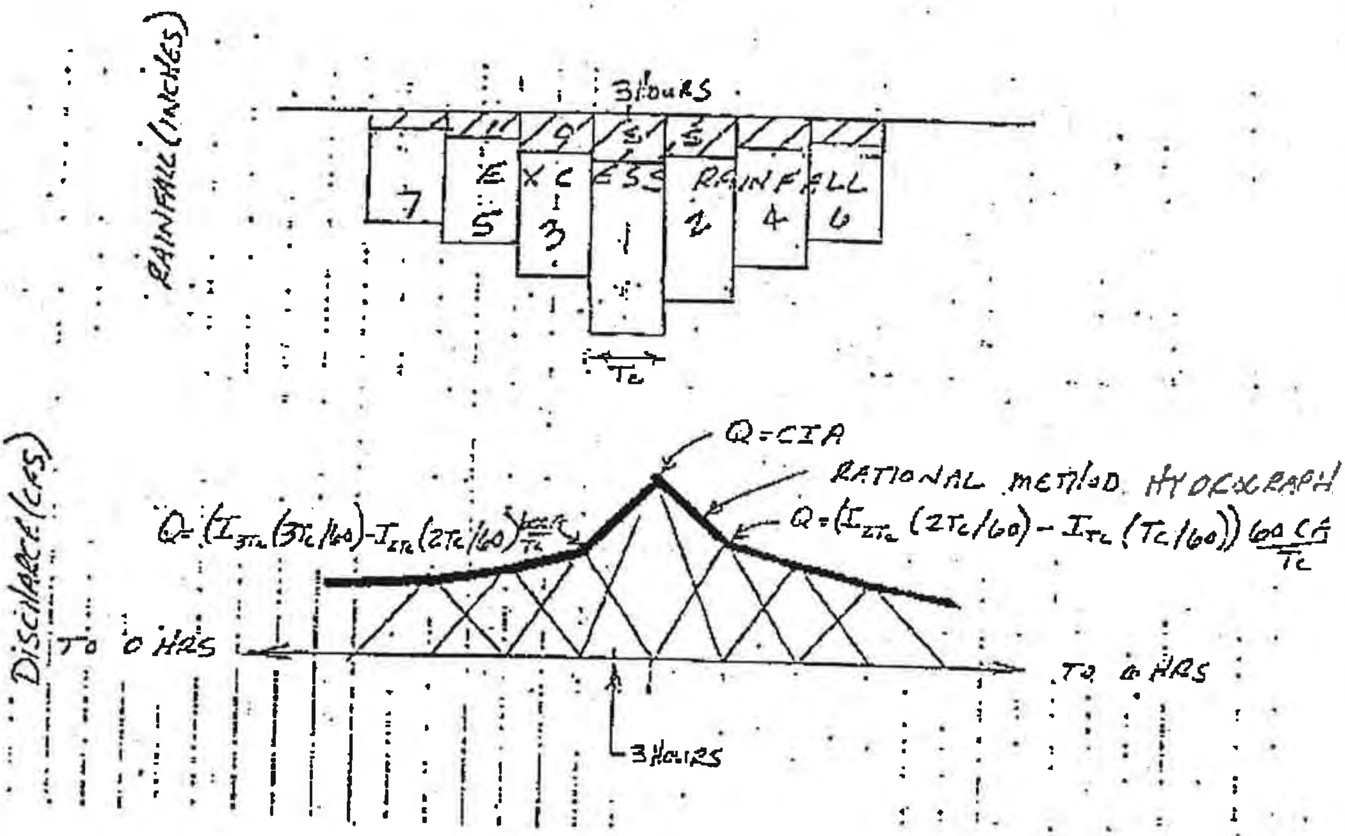

FIGURE 2

Equation 3

$Q_n = CIA$

$Q_n = ((I_{Tcn}) (Tcn /60) - (I_{Tc(n-1)}) (Tc(n-1)/60)) 60\ CA/Tc$

Q_n = Peak Discharge at Tcn (cubic feet per second)

n = Number of Hydrograph Ordinates

Tcn = Time of Concentration at Ordinate n (minutes)

I_{Tcn} = Rainfall Intensity at Time of Concentration Tcn (inches/hour)

C = Rational Method Runoff Coefficient

A = Area of the watershed (acres)

To develop the hydrograph for the six-hour design storm, a series of triangular hydrographs with ordinates at multiples of the given time of concentration are created and added to create the hydrograph. This hydrograph has its peak at three hours plus ½ of the time concentration. The total volume under the hydrograph is equal to the following equation:

Equation 4

$VOL = CP_6A$

VOL = Volume of runoff (acre-inches)

P_6 = 6-hour precipitation (inches)

C = Rational Method Runoff Coefficient

A = Area of the watershed (acres)

This six-hour storm hydrograph was then used in the HEC-1 Flood Hydrograph Program to perform routing calculations for the detention basin. In order to model a basin in the HEC-1 program, the storage volume of the basin and outflow characteristics of the discharge pipe are input at incremental elevations.

For the tentaive map, only volume calculations were performed for the 100-year storm event. Upon final design, outlet work calculations shall be performed and will be included with the revised report. Therefore, for the purposes of this report, an incremental elevation was used from 100 to 101, with the pre-project condition flow rate. Several iterations were performed to determine the correct volume that yielded a ponded water surface elevation of 101.

```
*****************************************
*                                       *
*   FLOOD HYDROGRAPH PACKAGE  (HEC-1)   *
*              JUN   1998               *
*             VERSION 4.1               *
*                                       *
* RUN DATE   17OCT03  TIME   11:37:16   *
*                                       *
*****************************************
```

```
*****************************************
*                                       *
*   U.S. ARMY CORPS OF ENGINEERS        *
*   HYDROLOGIC ENGINEERING CENTER       *
*         609 SECOND STREET             *
*      DAVIS, CALIFORNIA 95616          *
*          (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

$Q_{100,PRO}$ = 716.5 cfs
$Q_{100,EX}$ = 254.4 cfs
V_{DET} = 16.8 Ac-Ft ≅ 17.0 Ac-ft

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

*** FREE ***

          *DIAGRAM
          ID   VILLAGE 7    J-14483  08/22/03  FILE: V7B4DET.HC1
          ID   DETENTION FOR 100-YEAR STORM EVENT (BASED ON RATIONAL METHOD)
          ID   6-HR RAINFALL=2.5IN, TC=16.4MIN, R/O=.80
          ID   DETENTIN FOR EUC AND MCMILLIN OWNED PORTION
          IT       2 01JAN90     1200      200
          BA     .44
          IN      16 01JAN90     1140
          QI       0       0     33.4     35.5     37.9     40.8     44.3     48.8     54.6     62.6
          QI    74.5    94.5    138.8    716.5    179.8    111.3       83     67.9     58.3     51.5
          QI    46.4    42.5     39.3     36.6     34.4        0        0        0        0        0
          QI       0       0        0        0        0

          KK  DETAIN
          KM   Detain to Existing Flows
          RS       1    STOR       -1
          SV       0    16.8
          SQ       0   254.4
          SE     100     101
          ZZ
```

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

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

*** HEC1 ERROR 4 *** NO HYDROGRAPHS AVAILABLE TO ROUTE
                 V
                 V
  12        DETAIN

(***) RUNOFF ALSO COMPUTED AT THIS LOCATION

        1 ERRORS IN STREAM SYSTEM
```

```
*****************************************                    ***************************************
*                                       *                    *
*   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   17OCT03   TIME  11:37:16   *                    *         (916) 756-1104
*                                       *                    *
*****************************************                    ***************************************
```

```
                  VILLAGE 7    J-14483  08/22/03  FILE: V7B4DET.HC1
                  DETENTION FOR 100-YEAR STORM EVENT (BASED ON RATIONAL METHOD)
                  6-HR RAINFALL=2.5IN, TC=16.4MIN, R/O=.80
                  DETENTIN FOR EUC AND MCMILLIN OWNED PORTION

   IT       HYDROGRAPH TIME DATA
                 NMIN          2  MINUTES IN COMPUTATION INTERVAL
                IDATE    1JAN90  STARTING DATE
                ITIME      1200  STARTING TIME
                   NQ       200  NUMBER OF HYDROGRAPH ORDINATES
               NDDATE    1JAN90  ENDING DATE
               NDTIME      1838  ENDING TIME
                ICENT        19  CENTURY MARK

             COMPUTATION INTERVAL     .03 HOURS
                  TOTAL TIME BASE    6.63 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

 7 IN        TIME DATA FOR INPUT TIME SERIES
                JXMIN        16  TIME INTERVAL IN MINUTES
               JXDATE    1JAN90  STARTING DATE
               JXTIME      1140  STARTING TIME

          SUBBASIN RUNOFF DATA

 6 BA        SUBBASIN CHARACTERISTICS
                TAREA       .44  SUBBASIN AREA

                                         ***

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

                                 HYDROGRAPH AT STATION   DETAIN

*********************************************************************************************************
                          *                          *                          *
 DA MON HRMN  ORD    FLOW * DA MON HRMN  ORD    FLOW * DA MON HRMN  ORD    FLOW * DA MON HRMN  ORD    FLOW
                          *                          *                          *
```

1 JAN 1200	1	8.	*	1 JAN 1340	51	52.	*	1 JAN 1520	101	314.	*	1 JAN 1700	151	46
1 JAN 1202	2	13.	*	1 JAN 1342	52	52.	*	1 JAN 1522	102	247.	*	1 JAN 1702	152	46.
1 JAN 1204	3	17.	*	1 JAN 1344	53	53.	*	1 JAN 1524	103	180.	*	1 JAN 1704	153	45
1 JAN 1206	4	21.	*	1 JAN 1346	54	54.	*	1 JAN 1526	104	171.	*	1 JAN 1706	154	45
1 JAN 1208	5	25.	*	1 JAN 1348	55	55.	*	1 JAN 1528	105	163.	*	1 JAN 1708	155	44.
1 JAN 1210	6	29.	*	1 JAN 1350	56	56.	*	1 JAN 1530	106	154.	*	1 JAN 1710	156	44
1 JAN 1212	7	33.	*	1 JAN 1352	57	57.	*	1 JAN 1532	107	146.	*	1 JAN 1712	157	43
1 JAN 1214	8	34.	*	1 JAN 1354	58	58.	*	1 JAN 1534	108	137.	*	1 JAN 1714	158	43.
1 JAN 1216	9	34.	*	1 JAN 1356	59	59.	*	1 JAN 1536	109	128.	*	1 JAN 1716	159	43.
1 JAN 1218	10	34.	*	1 JAN 1358	60	60.	*	1 JAN 1538	110	120.	*	1 JAN 1718	160	42
1 JAN 1220	11	34.	*	1 JAN 1400	61	61.	*	1 JAN 1540	111	111.	*	1 JAN 1720	161	42
1 JAN 1222	12	35.	*	1 JAN 1402	62	62.	*	1 JAN 1542	112	108.	*	1 JAN 1722	162	41.
1 JAN 1224	13	35.	*	1 JAN 1404	63	63.	*	1 JAN 1544	113	104.	*	1 JAN 1724	163	41
1 JAN 1226	14	35.	*	1 JAN 1406	64	64.	*	1 JAN 1546	114	101.	*	1 JAN 1726	164	41
1 JAN 1228	15	36.	*	1 JAN 1408	65	66.	*	1 JAN 1548	115	97.	*	1 JAN 1728	165	40.
1 JAN 1230	16	36.	*	1 JAN 1410	66	67.	*	1 JAN 1550	116	94.	*	1 JAN 1730	166	40.
1 JAN 1232	17	36.	*	1 JAN 1412	67	69.	*	1 JAN 1552	117	90.	*	1 JAN 1732	167	39
1 JAN 1234	18	36.	*	1 JAN 1414	68	70.	*	1 JAN 1554	118	87.	*	1 JAN 1734	168	39
1 JAN 1236	19	37.	*	1 JAN 1416	69	72.	*	1 JAN 1556	119	83.	*	1 JAN 1736	169	39.
1 JAN 1238	20	37.	*	1 JAN 1418	70	73.	*	1 JAN 1558	120	81.	*	1 JAN 1738	170	38
1 JAN 1240	21	37.	*	1 JAN 1420	71	75.	*	1 JAN 1600	121	79.	*	1 JAN 1740	171	38
1 JAN 1242	22	38.	*	1 JAN 1422	72	77.	*	1 JAN 1602	122	77.	*	1 JAN 1742	172	38.
1 JAN 1244	23	38.	*	1 JAN 1424	73	80.	*	1 JAN 1604	123	75.	*	1 JAN 1744	173	37.
1 JAN 1246	24	38.	*	1 JAN 1426	74	82.	*	1 JAN 1606	124	74.	*	1 JAN 1746	174	37
1 JAN 1248	25	39.	*	1 JAN 1428	75	85.	*	1 JAN 1608	125	72.	*	1 JAN 1748	175	37
1 JAN 1250	26	39.	*	1 JAN 1430	76	87.	*	1 JAN 1610	126	70.	*	1 JAN 1750	176	36.
1 JAN 1252	27	39.	*	1 JAN 1432	77	90.	*	1 JAN 1612	127	68.	*	1 JAN 1752	177	36
1 JAN 1254	28	40.	*	1 JAN 1434	78	92.	*	1 JAN 1614	128	67.	*	1 JAN 1754	178	36
1 JAN 1256	29	40.	*	1 JAN 1436	79	95.	*	1 JAN 1616	129	66.	*	1 JAN 1756	179	36.
1 JAN 1258	30	40.	*	1 JAN 1438	80	100.	*	1 JAN 1618	130	64.	*	1 JAN 1758	180	35.
1 JAN 1300	31	41.	*	1 JAN 1440	81	106.	*	1 JAN 1620	131	63.	*	1 JAN 1800	181	35
1 JAN 1302	32	41.	*	1 JAN 1442	82	111.	*	1 JAN 1622	132	62.	*	1 JAN 1802	182	35
1 JAN 1304	33	42.	*	1 JAN 1444	83	117.	*	1 JAN 1624	133	61.	*	1 JAN 1804	183	34.
1 JAN 1306	34	42.	*	1 JAN 1446	84	122.	*	1 JAN 1626	134	60.	*	1 JAN 1806	184	30.
1 JAN 1308	35	43.	*	1 JAN 1448	85	128.	*	1 JAN 1628	135	58.	*	1 JAN 1808	185	26
1 JAN 1310	36	43.	*	1 JAN 1450	86	133.	*	1 JAN 1630	136	57.	*	1 JAN 1810	186	22.
1 JAN 1312	37	43.	*	1 JAN 1452	87	139.	*	1 JAN 1632	137	57.	*	1 JAN 1812	187	17.
1 JAN 1314	38	44.	*	1 JAN 1454	88	211.	*	1 JAN 1634	138	56.	*	1 JAN 1814	188	13
1 JAN 1316	39	44.	*	1 JAN 1456	89	283.	*	1 JAN 1636	139	55.	*	1 JAN 1816	189	9
1 JAN 1318	40	45.	*	1 JAN 1458	90	355.	*	1 JAN 1638	140	54.	*	1 JAN 1818	190	4.
1 JAN 1320	41	45.	*	1 JAN 1500	91	428.	*	1 JAN 1640	141	53.	*	1 JAN 1820	191	0.
1 JAN 1322	42	46.	*	1 JAN 1502	92	500.	*	1 JAN 1642	142	52.	*	1 JAN 1822	192	0
1 JAN 1324	43	47.	*	1 JAN 1504	93	572.	*	1 JAN 1644	143	52.	*	1 JAN 1824	193	0.
1 JAN 1326	44	47.	*	1 JAN 1506	94	644.	*	1 JAN 1646	144	51.	*	1 JAN 1826	194	0.
1 JAN 1328	45	48.	*	1 JAN 1508	95	717.	*	1 JAN 1648	145	50.	*	1 JAN 1828	195	0.
1 JAN 1330	46	48.	*	1 JAN 1510	96	649.	*	1 JAN 1650	146	50.	*	1 JAN 1830	196	0.
1 JAN 1332	47	49.	*	1 JAN 1512	97	582.	*	1 JAN 1652	147	49.	*	1 JAN 1832	197	0.
1 JAN 1334	48	50.	*	1 JAN 1514	98	515.	*	1 JAN 1654	148	48.	*	1 JAN 1834	198	0.
1 JAN 1336	49	50.	*	1 JAN 1516	99	448.	*	1 JAN 1656	149	48.	*	1 JAN 1836	199	0
1 JAN 1338	50	51.	*	1 JAN 1518	100	381.	*	1 JAN 1658	150	47.	*	1 JAN 1838	200	0.
			*				*				*			

****

PEAK FLOW	TIME		MAXIMUM AVERAGE FLOW			
(CFS)	(HR)		6-HR	24-HR	72-HR	6.63-HR
717.	3.13	(CFS)	94.	86.	86.	86.
		(INCHES)	.000	.000	.000	.000
		(AC-FT)	47.	47.	47.	47.

CUMULATIVE AREA = .00 SQ MI

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

          **************
          *            *
 12 KK    *   DETAIN   *
          *            *
          **************
                        Detain to Existing Flows

            HYDROGRAPH ROUTING DATA

 14 RS          STORAGE ROUTING
                     NSTPS           1  NUMBER OF SUBREACHES
                      ITYP        STOR  TYPE OF INITIAL CONDITION
                    RSVRIC       -1.00  INITIAL CONDITION
                         X         .00  WORKING R AND D COEFFICIENT

 15 SV            STORAGE          .0      16.8

 16 SQ          DISCHARGE          0.      254.

 17 SE          ELEVATION      100.00    101.00

                                                  ***

WARNING --- ROUTED OUTFLOW (    255.) IS GREATER THAN MAXIMUM OUTFLOW (    254.) IN STORAGE-OUTFLOW TABLE

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

                                          HYDROGRAPH AT STATION   DETAIN

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

DA	MON	HRMN	ORD	OUTFLOW	STORAGE	STAGE	*	DA	MON	HRMN	ORD	OUTFLOW	STORAGE	STAGE	*	DA	MON	HRMN	ORD	OUTFLOW	STORAGE	STAGE
1	JAN	1200	1	8.	.6	100.0	*	1	JAN	1414	68	50.	3.3	100.2	*	1	JAN	1628	135	129.	8.5	100.5
1	JAN	1202	2	8.	.6	100.0	*	1	JAN	1416	69	51.	3.3	100.2	*	1	JAN	1630	136	126.	8.3	100.5
1	JAN	1204	3	9.	.6	100.0	*	1	JAN	1418	70	51.	3.4	100.2	*	1	JAN	1632	137	123.	8.1	100.5
1	JAN	1206	4	9.	.6	100.0	*	1	JAN	1420	71	52.	3.5	100.2	*	1	JAN	1634	138	120.	7.9	100.5
1	JAN	1208	5	10.	.6	100.0	*	1	JAN	1422	72	53.	3.5	100.2	*	1	JAN	1636	139	118.	7.8	100.5
1	JAN	1210	6	10.	.7	100.0	*	1	JAN	1424	73	54.	3.6	100.2	*	1	JAN	1638	140	115.	7.6	100.5
1	JAN	1212	7	11.	.7	100.0	*	1	JAN	1426	74	55.	3.7	100.2	*	1	JAN	1640	141	112.	7.4	100.4
1	JAN	1214	8	12.	.8	100.0	*	1	JAN	1428	75	57.	3.7	100.2	*	1	JAN	1642	142	110.	7.3	100.4
1	JAN	1216	9	13.	.9	100.1	*	1	JAN	1430	76	58.	3.8	100.2	*	1	JAN	1644	143	108.	7.1	100.4
1	JAN	1218	10	14.	.9	100.1	*	1	JAN	1432	77	59.	3.9	100.2	*	1	JAN	1646	144	105.	7.0	100.4
1	JAN	1220	11	15.	1.0	100.1	*	1	JAN	1434	78	60.	4.0	100.2	*	1	JAN	1648	145	103.	6.8	100.4
1	JAN	1222	12	16.	1.0	100.1	*	1	JAN	1436	79	62.	4.1	100.2	*	1	JAN	1650	146	101.	6.7	100.4
1	JAN	1224	13	16.	1.1	100.1	*	1	JAN	1438	80	63.	4.2	100.2	*	1	JAN	1652	147	99.	6.5	100.4
1	JAN	1226	14	17.	1.1	100.1	*	1	JAN	1440	81	65.	4.3	100.3	*	1	JAN	1654	148	97.	6.4	100.4
1	JAN	1228	15	18.	1.2	100.1	*	1	JAN	1442	82	66.	4.4	100.3	*	1	JAN	1656	149	95.	6.3	100.4
1	JAN	1230	16	19.	1.2	100.1	*	1	JAN	1444	83	68.	4.5	100.3	*	1	JAN	1658	150	93.	6.1	100.4
1	JAN	1232	17	19.	1.3	100.1	*	1	JAN	1446	84	71.	4.7	100.3	*	1	JAN	1700	151	91.	6.0	100.4
1	JAN	1234	18	20.	1.3	100.1	*	1	JAN	1448	85	73.	4.8	100.3	*	1	JAN	1702	152	89.	5.9	100.4
1	JAN	1236	19	21.	1.4	100.1	*	1	JAN	1450	86	75.	5.0	100.3	*	1	JAN	1704	153	87.	5.8	100.3
1	JAN	1238	20	21.	1.4	100.1	*	1	JAN	1452	87	78.	5.1	100.3	*	1	JAN	1706	154	86.	5.7	100.3

1 JAN 1240	21	22.	1.5	100.1	*	1 JAN 1454	88	82.	5.4	100.3	*	1 JAN 1708	155	84.	5.5	[illegible]
1 JAN 1242	22	23.	1.5	100.1	*	1 JAN 1456	89	88.	5.8	100.3	*	1 JAN 1710	156	82.	5.4	100.3
1 JAN 1244	23	23.	1.5	100.1	*	1 JAN 1458	90	98.	6.5	100.4	*	1 JAN 1712	157	81.	5.3	[illegible]
1 JAN 1246	24	24.	1.6	100.1	*	1 JAN 1500	91	110.	7.2	100.4	*	1 JAN 1714	158	79.	5.2	[illegible]
1 JAN 1248	25	24.	1.6	100.1	*	1 JAN 1502	92	124.	8.2	100.5	*	1 JAN 1716	159	78.	5.1	100.3
1 JAN 1250	26	25.	1.7	100.1	*	1 JAN 1504	93	141.	9.3	100.6	*	1 JAN 1718	160	76.	5.0	[illegible]
1 JAN 1252	27	26.	1.7	100.1	*	1 JAN 1506	94	160.	10.6	100.6	*	1 JAN 1720	161	75.	4.9	[illegible]
1 JAN 1254	28	26.	1.7	100.1	*	1 JAN 1508	95	181.	12.0	100.7	*	1 JAN 1722	162	74.	4.9	[illegible]
1 JAN 1256	29	27.	1.8	100.1	*	1 JAN 1510	96	202.	13.3	100.8	*	1 JAN 1724	163	72.	4.8	100.3
1 JAN 1258	30	27.	1.8	100.1	*	1 JAN 1512	97	219.	14.5	100.9	*	1 JAN 1726	164	71.	4.7	[illegible]
1 JAN 1300	31	28.	1.8	100.1	*	1 JAN 1514	98	232.	15.3	100.9	*	1 JAN 1728	165	70.	4.6	[illegible]
1 JAN 1302	32	28.	1.9	100.1	*	1 JAN 1516	99	242.	16.0	101.0	*	1 JAN 1730	166	68.	4.5	100.3
1 JAN 1304	33	29.	1.9	100.1	*	1 JAN 1518	100	250.	16.5	101.0	*	1 JAN 1732	167	67.	4.4	[illegible]
1 JAN 1306	34	29.	1.9	100.1	*	1 JAN 1520	101	254.	16.7	101.0	*	1 JAN 1734	168	66.	4.4	[illegible]
1 JAN 1308	35	30.	2.0	100.1	*	1 JAN 1522	102	255.	16.8	101.0	*	1 JAN 1736	169	65.	4.3	100.3
1 JAN 1310	36	30.	2.0	100.1	*	1 JAN 1524	103	253.	16.7	101.0	*	1 JAN 1738	170	64.	4.2	100.3
1 JAN 1312	37	31.	2.0	100.1	*	1 JAN 1526	104	250.	16.5	101.0	*	1 JAN 1740	171	63.	4.2	[illegible]
1 JAN 1314	38	32.	2.1	100.1	*	1 JAN 1528	105	246.	16.3	101.0	*	1 JAN 1742	172	62.	4.1	[illegible]
1 JAN 1316	39	32.	2.1	100.1	*	1 JAN 1530	106	243.	16.0	101.0	*	1 JAN 1744	173	61.	4.0	100.2
1 JAN 1318	40	33.	2.1	100.1	*	1 JAN 1532	107	239.	15.8	100.9	*	1 JAN 1746	174	60.	4.0	[illegible]
1 JAN 1320	41	33.	2.2	100.1	*	1 JAN 1534	108	235.	15.5	100.9	*	1 JAN 1748	175	59.	3.9	[illegible]
1 JAN 1322	42	34.	2.2	100.1	*	1 JAN 1536	109	231.	15.2	100.9	*	1 JAN 1750	176	58.	3.8	[illegible]
1 JAN 1324	43	34.	2.3	100.1	*	1 JAN 1538	110	226.	15.0	100.9	*	1 JAN 1752	177	57.	3.8	100.2
1 JAN 1326	44	35.	2.3	100.1	*	1 JAN 1540	111	222.	14.7	100.9	*	1 JAN 1754	178	56.	3.7	[illegible]
1 JAN 1328	45	35.	2.3	100.1	*	1 JAN 1542	112	217.	14.4	100.9	*	1 JAN 1756	179	55.	3.7	[illegible]
1 JAN 1330	46	36.	2.4	100.1	*	1 JAN 1544	113	213.	14.1	100.8	*	1 JAN 1758	180	55.	3.6	100.2
1 JAN 1332	47	36.	2.4	100.1	*	1 JAN 1546	114	208.	13.8	100.8	*	1 JAN 1800	181	54.	3.6	[illegible]
1 JAN 1334	48	37.	2.4	100.1	*	1 JAN 1548	115	204.	13.5	100.8	*	1 JAN 1802	182	53.	3.5	[illegible]
1 JAN 1336	49	37.	2.5	100.1	*	1 JAN 1550	116	199.	13.2	100.8	*	1 JAN 1804	183	52.	3.5	[illegible]
1 JAN 1338	50	38.	2.5	100.1	*	1 JAN 1552	117	195.	12.9	100.8	*	1 JAN 1806	184	51.	3.4	100.2
1 JAN 1340	51	38.	2.5	100.2	*	1 JAN 1554	118	191.	12.6	100.7	*	1 JAN 1808	185	50.	3.3	[illegible]
1 JAN 1342	52	39.	2.6	100.2	*	1 JAN 1556	119	186.	12.3	100.7	*	1 JAN 1810	186	49.	3.3	[illegible]
1 JAN 1344	53	39.	2.6	100.2	*	1 JAN 1558	120	182.	12.0	100.7	*	1 JAN 1812	187	48.	3.2	100.2
1 JAN 1346	54	40.	2.6	100.2	*	1 JAN 1600	121	178.	11.7	100.7	*	1 JAN 1814	188	47.	3.1	[illegible]
1 JAN 1348	55	41.	2.7	100.2	*	1 JAN 1602	122	174.	11.5	100.7	*	1 JAN 1816	189	45.	3.0	[illegible]
1 JAN 1350	56	41.	2.7	100.2	*	1 JAN 1604	123	170.	11.2	100.7	*	1 JAN 1818	190	44.	2.9	[illegible]
1 JAN 1352	57	42.	2.8	100.2	*	1 JAN 1606	124	166.	11.0	100.7	*	1 JAN 1820	191	42.	2.8	100.2
1 JAN 1354	58	42.	2.8	100.2	*	1 JAN 1608	125	162.	10.7	100.6	*	1 JAN 1822	192	40.	2.7	[illegible]
1 JAN 1356	59	43.	2.8	100.2	*	1 JAN 1610	126	158.	10.5	100.6	*	1 JAN 1824	193	39.	2.6	[illegible]
1 JAN 1358	60	44.	2.9	100.2	*	1 JAN 1612	127	155.	10.2	100.6	*	1 JAN 1826	194	37.	2.4	100.1
1 JAN 1400	61	44.	2.9	100.2	*	1 JAN 1614	128	151.	10.0	100.6	*	1 JAN 1828	195	36.	2.3	100.1
1 JAN 1402	62	45.	3.0	100.2	*	1 JAN 1616	129	148.	9.8	100.6	*	1 JAN 1830	196	34.	2.3	[illegible]
1 JAN 1404	63	46.	3.0	100.2	*	1 JAN 1618	130	144.	9.5	100.6	*	1 JAN 1832	197	33.	2.2	[illegible]
1 JAN 1406	64	47.	3.1	100.2	*	1 JAN 1620	131	141.	9.3	100.6	*	1 JAN 1834	198	31.	2.1	100.1
1 JAN 1408	65	47.	3.1	100.2	*	1 JAN 1622	132	138.	9.1	100.5	*	1 JAN 1836	199	30.	2.0	[illegible]
1 JAN 1410	66	48.	3.2	100.2	*	1 JAN 1624	133	135.	8.9	100.5	*	1 JAN 1838	200	29.	1.9	[illegible]
1 JAN 1412	67	49.	3.2	100.2	*	1 JAN 1626	134	132.	8.7	100.5	*					
					*						*					

****

PEAK FLOW	TIME		MAXIMUM AVERAGE FLOW			
(CFS)	(HR)		6-HR	24-HR	72-HR	6.63-HR
255.	3.37	(CFS)	91.	83.	83.	83.
		(INCHES)	.000	.000	.000	.000
		(AC-FT)	45.	46.	46.	46.

PEAK STORAGE	TIME	MAXIMUM AVERAGE STORAGE			
(AC-FT)	(HR)	6-HR	24-HR	72-HR	6.63-HR
17.	3.37	6.	5.	5.	5.

PEAK STAGE	TIME	MAXIMUM AVERAGE STAGE			
(FEET)	(HR)	6-HR	24-HR	72-HR	6.63-HR
101.00	3.37	100.36	100.33	100.33	100.33

CUMULATIVE AREA = .00 SQ MI

RUNOFF SUMMARY

FLOW IN CUBIC FEET PER SECOND

TIME IN HOURS, AREA IN SQUARE MILES

OPERATION	STATION	PEAK FLOW	TIME OF PEAK	AVERAGE FLOW FOR MAXIMUM PERIOD 6-HOUR	24-HOUR	72-HOUR	BASIN AREA	MAXIMUM STAGE	TIME OF MAX STAGE
HYDROGRAPH AT	DETAIN	717.	3.13	94.	86.	86.	.00		
ROUTED TO	DETAIN	255.	3.37	91.	83.	83.	.00	101.00	3.37

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

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

          RATIONAL METHOD HYDROLOGY COMPUTER PROGRAM PACKAGE
          Reference: SAN DIEGO COUNTY FLOOD CONTROL DISTRICT
                        1985,1981 HYDROLOGY MANUAL
       (c) Copyright 1982-2000 Advanced Engineering Software (aes)
          Ver. 1.5A Release Date: 01/01/2000  License ID 1261

                          Analysis prepared by:

                         Rick Engineering Company
                           5620 Friars Road
                          San Diego, CA  92110
                            (619) 291-0707

************************** DESCRIPTION OF STUDY **************************
* Otay Ranch Village 7    J-              08/14/03                        *
* 100-Year Storm Event                                                    *
* Basin 400  -  Existing Condition                                        *
**************************************************************************

  FILE NAME: 1HV7B4EX.DAT
  TIME/DATE OF STUDY: 08:31 08/18/2003
----------------------------------------------------------------------------
  USER SPECIFIED HYDROLOGY AND HYDRAULIC MODEL INFORMATION:
----------------------------------------------------------------------------
  1985 SAN DIEGO MANUAL CRITERIA

  USER SPECIFIED STORM EVENT(YEAR) = 100.00
  6-HOUR DURATION PRECIPITATION (INCHES) =   2.500
  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: ONLY PEAK CONFLUENCE VALUES CONSIDERED
  *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

  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    400.00 TO NODE    402.00 IS CODE =  21
----------------------------------------------------------------------------
  >>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<
============================================================================
  *USER SPECIFIED(SUBAREA):
  RURAL DEVELOPMENT RUNOFF COEFFICIENT = .5000
  S.C.S. CURVE NUMBER (AMC II) =    0
```

```
  NATURAL WATERSHED NOMOGRAPH TIME OF CONCENTRATION (APPENDIX X-A)
  WITH 10-MINUTES ADDED =  13.09(MINUTES)
  INITIAL SUBAREA FLOW-LENGTH =   500.00
  UPSTREAM ELEVATION =    662.40
  DOWNSTREAM ELEVATION =    640.00
  ELEVATION DIFFERENCE =     22.40
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  3.540
  SUBAREA RUNOFF(CFS) =      4.07
  TOTAL AREA(ACRES) =      2.30   TOTAL RUNOFF(CFS) =      4.07

****************************************************************************
  FLOW PROCESS FROM NODE    400.00 TO NODE    402.00 IS CODE =  22
----------------------------------------------------------------------------
  >>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<
============================================================================
  *USER SPECIFIED(SUBAREA):
  RURAL DEVELOPMENT RUNOFF COEFFICIENT = .5000
  S.C.S. CURVE NUMBER (AMC II) =   0
  USER SPECIFIED Tc(MIN.) =    9.090
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  4.480
  SUBAREA RUNOFF(CFS) =      5.15
  TOTAL AREA(ACRES) =      2.30   TOTAL RUNOFF(CFS) =      5.15

****************************************************************************
  FLOW PROCESS FROM NODE    402.00 TO NODE    404.00 IS CODE =  51
----------------------------------------------------------------------------
  >>>>>COMPUTE TRAPEZOIDAL CHANNEL FLOW<<<<<
  >>>>>TRAVELTIME THRU SUBAREA (NEW ELEMENT)<<<<<
============================================================================
  ELEVATION DATA: UPSTREAM(FEET) =    640.00  DOWNSTREAM(FEET) =    530.00
  CHANNEL LENGTH THRU SUBAREA(FEET) =  2780.00   CHANNEL SLOPE =  0.0396
  CHANNEL BASE(FEET) =     5.00   "Z" FACTOR =  10.000
  MANNING'S FACTOR = 0.045   MAXIMUM DEPTH(FEET) =   5.00
  CHANNEL FLOW THRU SUBAREA(CFS) =      5.15
  FLOW VELOCITY(FEET/SEC) =   2.30   FLOW DEPTH(FEET) =   0.29
  TRAVEL TIME(MIN.) =  20.14   Tc(MIN.) =  29.23
  LONGEST FLOWPATH FROM NODE    400.00 TO NODE    404.00 =  3280.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    402.00 TO NODE    404.00 IS CODE =  81
----------------------------------------------------------------------------
  >>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
============================================================================
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  2.109
  *USER SPECIFIED(SUBAREA):
  RURAL DEVELOPMENT RUNOFF COEFFICIENT = .5000
  S.C.S. CURVE NUMBER (AMC II) =   0
  SUBAREA AREA(ACRES) =    64.20   SUBAREA RUNOFF(CFS) =    67.70
  TOTAL AREA(ACRES) =    66.50   TOTAL RUNOFF(CFS) =    72.85
  TC(MIN) =  29.23

****************************************************************************
  FLOW PROCESS FROM NODE    402.00 TO NODE    404.00 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.) =  29.23
RAINFALL INTENSITY(INCH/HR) =   2.11
TOTAL STREAM AREA(ACRES) =   66.50
PEAK FLOW RATE(CFS) AT CONFLUENCE =     72.85

****************************************************************************
FLOW PROCESS FROM NODE    406.00 TO NODE    408.00 IS CODE =  21
----------------------------------------------------------------------------
>>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<
============================================================================
*USER SPECIFIED(SUBAREA):
RURAL DEVELOPMENT RUNOFF COEFFICIENT = .5000
S.C.S. CURVE NUMBER (AMC II) =    0
NATURAL WATERSHED NOMOGRAPH TIME OF CONCENTRATION (APPENDIX X-A)
WITH 10-MINUTES ADDED =  12.88(MINUTES)
INITIAL SUBAREA FLOW-LENGTH =   500.00
UPSTREAM ELEVATION =    667.00
DOWNSTREAM ELEVATION =    640.00
ELEVATION DIFFERENCE =     27.00
 100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  3.578
SUBAREA RUNOFF(CFS) =      4.83
TOTAL AREA(ACRES) =      2.70   TOTAL RUNOFF(CFS) =      4.83

****************************************************************************
FLOW PROCESS FROM NODE    406.00 TO NODE    408.00 IS CODE =  22
----------------------------------------------------------------------------
>>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<
============================================================================
*USER SPECIFIED(SUBAREA):
RURAL DEVELOPMENT RUNOFF COEFFICIENT = .5000
S.C.S. CURVE NUMBER (AMC II) =    0
USER SPECIFIED Tc(MIN.) =    8.880
 100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  4.548
SUBAREA RUNOFF(CFS) =      6.14
TOTAL AREA(ACRES) =      2.70   TOTAL RUNOFF(CFS) =      6.14

****************************************************************************
FLOW PROCESS FROM NODE    408.00 TO NODE    404.00 IS CODE =  51
----------------------------------------------------------------------------
>>>>>COMPUTE TRAPEZOIDAL CHANNEL FLOW<<<<<
>>>>>TRAVELTIME THRU SUBAREA (NEW ELEMENT)<<<<<
============================================================================
ELEVATION DATA: UPSTREAM(FEET) =    640.00  DOWNSTREAM(FEET) =    530.00
CHANNEL LENGTH THRU SUBAREA(FEET) =  2500.00   CHANNEL SLOPE =  0.0440
CHANNEL BASE(FEET) =     5.00    "Z" FACTOR =  10.000
MANNING'S FACTOR = 0.045   MAXIMUM DEPTH(FEET) =    5.00
CHANNEL FLOW THRU SUBAREA(CFS) =       6.14
FLOW VELOCITY(FEET/SEC) =    2.53   FLOW DEPTH(FEET) =    0.30
TRAVEL TIME(MIN.) =  16.46    Tc(MIN.) =  25.34
LONGEST FLOWPATH FROM NODE    406.00 TO NODE    404.00 =  3000.00 FEET.

****************************************************************************
FLOW PROCESS FROM NODE    408.00 TO NODE    404.00 IS CODE =  81
----------------------------------------------------------------------------
>>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
============================================================================
```

```
 100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  2.312
*USER SPECIFIED(SUBAREA):
RURAL DEVELOPMENT RUNOFF COEFFICIENT = .5000
S.C.S. CURVE NUMBER (AMC II) =    0
SUBAREA AREA(ACRES) =    64.70   SUBAREA RUNOFF(CFS) =    74.80
TOTAL AREA(ACRES) =     67.40   TOTAL RUNOFF(CFS) =    80.94
TC(MIN) =  25.34

****************************************************************************
FLOW PROCESS FROM NODE    408.00 TO NODE    404.00 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.) =  25.34
RAINFALL INTENSITY(INCH/HR) =   2.31
TOTAL STREAM AREA(ACRES) =   67.40
PEAK FLOW RATE(CFS) AT CONFLUENCE =      80.94

** CONFLUENCE DATA **
STREAM      RUNOFF       Tc       INTENSITY       AREA
NUMBER       (CFS)     (MIN.)    (INCH/HOUR)     (ACRE)
    1        72.85     29.23        2.109          66.50
    2        80.94     25.34        2.312          67.40

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       147.39     25.34       2.312
    2       146.68     29.23       2.109

COMPUTED CONFLUENCE ESTIMATES ARE AS FOLLOWS:
PEAK FLOW RATE(CFS) =     147.39   Tc(MIN.) =    25.34
TOTAL AREA(ACRES) =     133.90
LONGEST FLOWPATH FROM NODE    400.00 TO NODE    404.00 =   3280.00 FEET.

****************************************************************************
FLOW PROCESS FROM NODE    404.00 TO NODE    409.00 IS CODE =  51
----------------------------------------------------------------------------
>>>>>COMPUTE TRAPEZOIDAL CHANNEL FLOW<<<<<
>>>>>TRAVELTIME THRU SUBAREA (EXISTING ELEMENT)<<<<<
============================================================================
ELEVATION DATA: UPSTREAM(FEET) =    530.00  DOWNSTREAM(FEET) =    518.00
CHANNEL LENGTH THRU SUBAREA(FEET) =   300.00   CHANNEL SLOPE =  0.0400
CHANNEL BASE(FEET) =    5.00    "Z" FACTOR =  10.000
MANNING'S FACTOR = 0.045   MAXIMUM DEPTH(FEET) =   5.00
CHANNEL FLOW THRU SUBAREA(CFS) =     147.39
FLOW VELOCITY(FEET/SEC) =   5.65   FLOW DEPTH(FEET) =   1.38
TRAVEL TIME(MIN.) =   0.88   Tc(MIN.) =  26.23
LONGEST FLOWPATH FROM NODE    400.00 TO NODE    409.00 =   3580.00 FEET.

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

```
 FLOW PROCESS FROM NODE    404.00 TO NODE    409.00 IS CODE =  81
----------------------------------------------------------------------------
 >>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
============================================================================
  100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  2.262
 *USER SPECIFIED(SUBAREA):
 RURAL DEVELOPMENT RUNOFF COEFFICIENT = .5000
 S.C.S. CURVE NUMBER (AMC II) =   0
 SUBAREA AREA(ACRES) =    8.00   SUBAREA RUNOFF(CFS) =    9.05
 TOTAL AREA(ACRES) =   141.90   TOTAL RUNOFF(CFS) =  156.44
 TC(MIN) =  26.23

****************************************************************************
 FLOW PROCESS FROM NODE    409.00 TO NODE    410.00 IS CODE =  51
----------------------------------------------------------------------------
 >>>>>COMPUTE TRAPEZOIDAL CHANNEL FLOW<<<<<
 >>>>>TRAVELTIME THRU SUBAREA (EXISTING ELEMENT)<<<<<
============================================================================
 ELEVATION DATA: UPSTREAM(FEET) =    518.00  DOWNSTREAM(FEET) =    430.00
 CHANNEL LENGTH THRU SUBAREA(FEET) =  2300.00   CHANNEL SLOPE =  0.0383
 CHANNEL BASE(FEET) =    5.00   "Z" FACTOR =    5.000
 MANNING'S FACTOR = 0.045   MAXIMUM DEPTH(FEET) =    5.00
 CHANNEL FLOW THRU SUBAREA(CFS) =    156.44
 FLOW VELOCITY(FEET/SEC) =   6.64   FLOW DEPTH(FEET) =   1.73
 TRAVEL TIME(MIN.) =   5.77   Tc(MIN.) =  32.00
 LONGEST FLOWPATH FROM NODE    400.00 TO NODE    410.00 =  5880.00 FEET.

****************************************************************************
 FLOW PROCESS FROM NODE    409.00 TO NODE    410.00 IS CODE =  81
----------------------------------------------------------------------------
 >>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
============================================================================
  100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  1.989
 *USER SPECIFIED(SUBAREA):
 RURAL DEVELOPMENT RUNOFF COEFFICIENT = .5000
 S.C.S. CURVE NUMBER (AMC II) =   0
 SUBAREA AREA(ACRES) =   98.50   SUBAREA RUNOFF(CFS) =   97.98
 TOTAL AREA(ACRES) =   240.40   TOTAL RUNOFF(CFS) =  254.42
 TC(MIN) =  32.00

****************************************************************************
 FLOW PROCESS FROM NODE    410.00 TO NODE    412.00 IS CODE =  51
----------------------------------------------------------------------------
 >>>>>COMPUTE TRAPEZOIDAL CHANNEL FLOW<<<<<
 >>>>>TRAVELTIME THRU SUBAREA (EXISTING ELEMENT)<<<<<
============================================================================
 ELEVATION DATA: UPSTREAM(FEET) =    430.00  DOWNSTREAM(FEET) =    377.00
 CHANNEL LENGTH THRU SUBAREA(FEET) =  2180.00   CHANNEL SLOPE =  0.0243
 CHANNEL BASE(FEET) =    5.00   "Z" FACTOR =    5.000
 MANNING'S FACTOR = 0.045   MAXIMUM DEPTH(FEET) =    5.00
 CHANNEL FLOW THRU SUBAREA(CFS) =    254.42
 FLOW VELOCITY(FEET/SEC) =   6.35   FLOW DEPTH(FEET) =   2.38
 TRAVEL TIME(MIN.) =   5.73   Tc(MIN.) =  37.72
 LONGEST FLOWPATH FROM NODE    400.00 TO NODE    412.00 =  8060.00 FEET.

****************************************************************************
 FLOW PROCESS FROM NODE    410.00 TO NODE    412.00 IS CODE =  81
```

```
----------------------------------------------------------------------------
  >>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
============================================================================
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  1.789
  *USER SPECIFIED(SUBAREA):
  RURAL DEVELOPMENT RUNOFF COEFFICIENT = .5000
  S.C.S. CURVE NUMBER (AMC II) =   0
  SUBAREA AREA(ACRES) =  102.80   SUBAREA RUNOFF(CFS) =   91.95
  TOTAL AREA(ACRES) =   343.20   TOTAL RUNOFF(CFS) =  346.37
  TC(MIN) =  37.72

****************************************************************************
  FLOW PROCESS FROM NODE    412.00 TO NODE    414.00 IS CODE =  51
----------------------------------------------------------------------------
  >>>>>COMPUTE TRAPEZOIDAL CHANNEL FLOW<<<<<
  >>>>>TRAVELTIME THRU SUBAREA (EXISTING ELEMENT)<<<<<
============================================================================
  ELEVATION DATA: UPSTREAM(FEET) =    377.00  DOWNSTREAM(FEET) =    348.00
  CHANNEL LENGTH THRU SUBAREA(FEET) =    940.00   CHANNEL SLOPE =  0.0309
  CHANNEL BASE(FEET) =    5.00   "Z" FACTOR =   3.300
  MANNING'S FACTOR = 0.045   MAXIMUM DEPTH(FEET) =   5.00
  CHANNEL FLOW THRU SUBAREA(CFS) =    346.37
  FLOW VELOCITY(FEET/SEC) =   8.22   FLOW DEPTH(FEET) =   2.89
  TRAVEL TIME(MIN.) =   1.91   Tc(MIN.) =  39.63
  LONGEST FLOWPATH FROM NODE    400.00 TO NODE    414.00 =  9000.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    412.00 TO NODE    414.00 IS CODE =  81
----------------------------------------------------------------------------
  >>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
============================================================================
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  1.733
  *USER SPECIFIED(SUBAREA):
  RURAL DEVELOPMENT RUNOFF COEFFICIENT = .5000
  S.C.S. CURVE NUMBER (AMC II) =   0
  SUBAREA AREA(ACRES) =   40.80   SUBAREA RUNOFF(CFS) =   35.35
  TOTAL AREA(ACRES) =   384.00   TOTAL RUNOFF(CFS) =  381.72
  TC(MIN) =  39.63
============================================================================
  END OF STUDY SUMMARY:
  TOTAL AREA(ACRES)      =    384.00  TC(MIN.) =     39.63
  PEAK FLOW RATE(CFS)    =    381.72
============================================================================
============================================================================
  END OF RATIONAL METHOD ANALYSIS

1
```

```
****************************************************************************
                                                                            
         RATIONAL METHOD HYDROLOGY COMPUTER PROGRAM PACKAGE
         Reference: SAN DIEGO COUNTY FLOOD CONTROL DISTRICT
                      1985,1981 HYDROLOGY MANUAL
      (c) Copyright 1982-2000 Advanced Engineering Software (aes)
          Ver. 1.5A Release Date: 01/01/2000  License ID 1261

                        Analysis prepared by:

                       Rick Engineering Company
                          5620 Friars Road
                        San Diego, CA  92110
                           (619) 291-0707

 ************************** DESCRIPTION OF STUDY **************************
 * Otay Ranch Village 7  J-14883   09/02/03                               *
 * 100-Year Storm Event - Proposed Condition                              *
 * Basin 600b (Shown as basin 5 on Workmap - only includes 27.8 ac)       *
 **************************************************************************

  FILE NAME: 1HV7B6BP.DAT
  TIME/DATE OF STUDY: 10:06 09/02/2003
----------------------------------------------------------------------------
  USER SPECIFIED HYDROLOGY AND HYDRAULIC MODEL INFORMATION:
----------------------------------------------------------------------------
  1985 SAN DIEGO MANUAL CRITERIA

  USER SPECIFIED STORM EVENT(YEAR) = 100.00
  6-HOUR DURATION PRECIPITATION (INCHES) =   2.500
  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: ONLY PEAK CONFLUENCE VALUES CONSIDERED
  *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

  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    600.00 TO NODE    602.00 IS CODE =  21
----------------------------------------------------------------------------
  >>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<
============================================================================
  *USER SPECIFIED(SUBAREA):
  SINGLE FAMILY DEVELOPMENT RUNOFF COEFFICIENT = .6500
  S.C.S. CURVE NUMBER (AMC II) =   0
```

```
 INITIAL SUBAREA FLOW-LENGTH =   330.00
 UPSTREAM ELEVATION =     579.00
 DOWNSTREAM ELEVATION =     575.70
 ELEVATION DIFFERENCE =       3.30
 URBAN SUBAREA OVERLAND TIME OF FLOW(MINUTES) =   14.714
  100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  3.283
 SUBAREA RUNOFF(CFS) =       5.98
 TOTAL AREA(ACRES) =       2.80   TOTAL RUNOFF(CFS) =        5.98

****************************************************************************
 FLOW PROCESS FROM NODE    602.00 TO NODE    604.00 IS CODE =  31
----------------------------------------------------------------------------
 >>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
 >>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<
============================================================================
 ELEVATION DATA: UPSTREAM(FEET) =   565.00  DOWNSTREAM(FEET) =   535.00
 FLOW LENGTH(FEET) =  2300.00   MANNING'S N =  0.013
 ESTIMATED PIPE DIAMETER(INCH) INCREASED TO 18.000
 DEPTH OF FLOW IN  18.0 INCH PIPE IS   9.3 INCHES
 PIPE-FLOW VELOCITY(FEET/SEC.) =   6.52
 ESTIMATED PIPE DIAMETER(INCH) =  18.00    NUMBER OF PIPES =   1
 PIPE-FLOW(CFS) =       5.98
 PIPE TRAVEL TIME(MIN.) =   5.88    Tc(MIN.) =  20.59
 LONGEST FLOWPATH FROM NODE    600.00 TO NODE    604.00 =  2630.00 FEET.

****************************************************************************
 FLOW PROCESS FROM NODE    602.00 TO NODE    604.00 IS CODE =  81
----------------------------------------------------------------------------
 >>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
============================================================================
  100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  2.643
 *USER SPECIFIED(SUBAREA):
 SINGLE FAMILY DEVELOPMENT RUNOFF COEFFICIENT = .6500
 S.C.S. CURVE NUMBER (AMC II) =   0
 SUBAREA AREA(ACRES) =   25.00   SUBAREA RUNOFF(CFS) =   42.96
 TOTAL AREA(ACRES) =    27.80   TOTAL RUNOFF(CFS) =   48.93
 TC(MIN) =  20.59

****************************************************************************
 FLOW PROCESS FROM NODE    604.00 TO NODE    432.00 IS CODE =  31
----------------------------------------------------------------------------
 >>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
 >>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<
============================================================================
 ELEVATION DATA: UPSTREAM(FEET) =   535.00  DOWNSTREAM(FEET) =   500.00
 FLOW LENGTH(FEET) =  1700.00   MANNING'S N =  0.013
 DEPTH OF FLOW IN  30.0 INCH PIPE IS  21.8 INCHES
 PIPE-FLOW VELOCITY(FEET/SEC.) =  12.83
 ESTIMATED PIPE DIAMETER(INCH) =  30.00    NUMBER OF PIPES =   1
 PIPE-FLOW(CFS) =      48.93
 PIPE TRAVEL TIME(MIN.) =   2.21    Tc(MIN.) =  22.80
 LONGEST FLOWPATH FROM NODE    600.00 TO NODE    432.00 =  4330.00 FEET.

****************************************************************************
 FLOW PROCESS FROM NODE    604.00 TO NODE    432.00 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.) =  22.80
 RAINFALL INTENSITY(INCH/HR) =   2.48
 TOTAL STREAM AREA(ACRES) =   27.80
 PEAK FLOW RATE(CFS) AT CONFLUENCE =     48.93

****************************************************************************
 FLOW PROCESS FROM NODE    432.00 TO NODE    432.00 IS CODE =   7
----------------------------------------------------------------------------
 >>>>>USER SPECIFIED HYDROLOGY INFORMATION AT NODE<<<<<
============================================================================
 USER-SPECIFIED VALUES ARE AS FOLLOWS:
 TC(MIN) =  18.47   RAIN INTENSITY(INCH/HOUR) =  2.84
 TOTAL AREA(ACRES) =   270.40   TOTAL RUNOFF(CFS) =    683.15

****************************************************************************
 FLOW PROCESS FROM NODE    432.00 TO NODE    432.00 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.) =  18.47
 RAINFALL INTENSITY(INCH/HR) =   2.84
 TOTAL STREAM AREA(ACRES) =  270.40
 PEAK FLOW RATE(CFS) AT CONFLUENCE =    683.15

 ** CONFLUENCE DATA **
 STREAM      RUNOFF       Tc       INTENSITY      AREA
 NUMBER       (CFS)     (MIN.)   (INCH/HOUR)    (ACRE)
    1         48.93     22.80        2.475         27.80
    2        683.15     18.47        2.836        270.40

 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        725.87     18.47       2.836
    2        645.29     22.80       2.475

 COMPUTED CONFLUENCE ESTIMATES ARE AS FOLLOWS:
 PEAK FLOW RATE(CFS) =     725.87   Tc(MIN.) =    18.47
 TOTAL AREA(ACRES) =     298.20
 LONGEST FLOWPATH FROM NODE    600.00 TO NODE    432.00 =  4330.00 FEET.
============================================================================
 END OF STUDY SUMMARY:
 TOTAL AREA(ACRES)        =    298.20  TC(MIN.) =      18.47
 PEAK FLOW RATE(CFS)      =    725.87
============================================================================
============================================================================
 END OF RATIONAL METHOD ANALYSIS
```

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

          RATIONAL METHOD HYDROLOGY COMPUTER PROGRAM PACKAGE
          Reference: SAN DIEGO COUNTY FLOOD CONTROL DISTRICT
                     1985,1981 HYDROLOGY MANUAL
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           Ver. 1.5A Release Date: 01/01/2000  License ID 1261

                       Analysis prepared by:

                      Rick Engineering Company
                         5620 Friars Road
                        San Diego, CA  92110
                          (619) 291-0707

 ************************** DESCRIPTION OF STUDY **************************
 * Otay Ranch Village 7  J-14883   09/02/03                               *
 * 100-Year Storm Event                                                   *
 * Basin 600 (Shown as Basin 5 on P&D Workmap) - Proposed Condition       *
 **************************************************************************

  FILE NAME: 1HV7B6PR.DAT
  TIME/DATE OF STUDY: 09:25 09/02/2003
----------------------------------------------------------------------------
  USER SPECIFIED HYDROLOGY AND HYDRAULIC MODEL INFORMATION:
----------------------------------------------------------------------------
  1985 SAN DIEGO MANUAL CRITERIA

  USER SPECIFIED STORM EVENT(YEAR) = 100.00
  6-HOUR DURATION PRECIPITATION (INCHES) =   2.500
  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: ONLY PEAK CONFLUENCE VALUES CONSIDERED
  *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

  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    600.00 TO NODE    602.00 IS CODE =  21
----------------------------------------------------------------------------
  >>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<
============================================================================
  *USER SPECIFIED(SUBAREA):
  SINGLE FAMILY DEVELOPMENT RUNOFF COEFFICIENT = .6500
  S.C.S. CURVE NUMBER (AMC II) =   0
```

```
INITIAL SUBAREA FLOW-LENGTH =   330.00
UPSTREAM ELEVATION =      579.00
DOWNSTREAM ELEVATION =      575.70
ELEVATION DIFFERENCE =        3.30
URBAN SUBAREA OVERLAND TIME OF FLOW(MINUTES) =   14.714
 100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  3.283
SUBAREA RUNOFF(CFS) =        5.98
TOTAL AREA(ACRES) =       2.80   TOTAL RUNOFF(CFS) =       5.98

****************************************************************************
  FLOW PROCESS FROM NODE    602.00 TO NODE    604.00 IS CODE =  31
----------------------------------------------------------------------------
  >>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
  >>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<
============================================================================
  ELEVATION DATA: UPSTREAM(FEET) =    565.00  DOWNSTREAM(FEET) =    535.00
  FLOW LENGTH(FEET) =  2300.00   MANNING'S N =  0.013
  ESTIMATED PIPE DIAMETER(INCH) INCREASED TO 18.000
  DEPTH OF FLOW IN  18.0 INCH PIPE IS   9.3 INCHES
  PIPE-FLOW VELOCITY(FEET/SEC.) =   6.52
  ESTIMATED PIPE DIAMETER(INCH) =  18.00    NUMBER OF PIPES =   1
  PIPE-FLOW(CFS) =       5.98
  PIPE TRAVEL TIME(MIN.) =    5.88    Tc(MIN.) =  20.59
  LONGEST FLOWPATH FROM NODE    600.00 TO NODE    604.00 =  2630.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    602.00 TO NODE    604.00 IS CODE =  81
----------------------------------------------------------------------------
  >>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
============================================================================
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  2.643
  *USER SPECIFIED(SUBAREA):
  SINGLE FAMILY DEVELOPMENT RUNOFF COEFFICIENT = .6500
  S.C.S. CURVE NUMBER (AMC II) =   0
  SUBAREA AREA(ACRES) =    25.00   SUBAREA RUNOFF(CFS) =   42.96
  TOTAL AREA(ACRES) =     27.80   TOTAL RUNOFF(CFS) =   48.93
  TC(MIN) =   20.59

****************************************************************************
  FLOW PROCESS FROM NODE    604.00 TO NODE    606.00 IS CODE =  31
----------------------------------------------------------------------------
  >>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
  >>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<
============================================================================
  ELEVATION DATA: UPSTREAM(FEET) =    535.00  DOWNSTREAM(FEET) =    450.00
  FLOW LENGTH(FEET) =  1400.00   MANNING'S N =  0.013
  DEPTH OF FLOW IN  24.0 INCH PIPE IS  18.2 INCHES
  PIPE-FLOW VELOCITY(FEET/SEC.) =  19.11
  ESTIMATED PIPE DIAMETER(INCH) =  24.00    NUMBER OF PIPES =   1
  PIPE-FLOW(CFS) =      48.93
  PIPE TRAVEL TIME(MIN.) =    1.22    Tc(MIN.) =  21.81
  LONGEST FLOWPATH FROM NODE    600.00 TO NODE    606.00 =  4030.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    604.00 TO NODE    606.00 IS CODE =  81
----------------------------------------------------------------------------
  >>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
```

```
==========================================================================
  100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  2.547
 *USER SPECIFIED(SUBAREA):
 SINGLE FAMILY DEVELOPMENT RUNOFF COEFFICIENT = .6500
 S.C.S. CURVE NUMBER (AMC II) =   0
 SUBAREA AREA(ACRES) =   21.20   SUBAREA RUNOFF(CFS) =   35.10
 TOTAL AREA(ACRES) =    49.00   TOTAL RUNOFF(CFS) =   84.03
 TC(MIN) =  21.81

**************************************************************************
 FLOW PROCESS FROM NODE    606.00 TO NODE    442.00 IS CODE =  31
--------------------------------------------------------------------------
 >>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
 >>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<
==========================================================================
 ELEVATION DATA: UPSTREAM(FEET) =    450.00  DOWNSTREAM(FEET) =   400.00
 FLOW LENGTH(FEET) =  1700.00   MANNING'S N =  0.013
 DEPTH OF FLOW IN  33.0 INCH PIPE IS  26.3 INCHES
 PIPE-FLOW VELOCITY(FEET/SEC.) =  16.53
 ESTIMATED PIPE DIAMETER(INCH) =  33.00    NUMBER OF PIPES =   1
 PIPE-FLOW(CFS) =      84.03
 PIPE TRAVEL TIME(MIN.) =   1.71    Tc(MIN.) =  23.53
 LONGEST FLOWPATH FROM NODE    600.00 TO NODE    442.00 =  5730.00 FEET.

**************************************************************************
 FLOW PROCESS FROM NODE    606.00 TO NODE    442.00 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.) =  23.53
 RAINFALL INTENSITY(INCH/HR) =   2.43
 TOTAL STREAM AREA(ACRES) =   49.00
 PEAK FLOW RATE(CFS) AT CONFLUENCE =      84.03

**************************************************************************
 FLOW PROCESS FROM NODE    442.00 TO NODE    442.00 IS CODE =   7
--------------------------------------------------------------------------
 >>>>>USER SPECIFIED HYDROLOGY INFORMATION AT NODE<<<<<
==========================================================================
 USER-SPECIFIED VALUES ARE AS FOLLOWS:
 TC(MIN) =  21.65   RAIN INTENSITY(INCH/HOUR) =  2.56
 TOTAL AREA(ACRES) =   396.80   TOTAL RUNOFF(CFS) =    884.27

**************************************************************************
 FLOW PROCESS FROM NODE    442.00 TO NODE    442.00 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.) =  21.65
 RAINFALL INTENSITY(INCH/HR) =   2.56
 TOTAL STREAM AREA(ACRES) =  396.80
 PEAK FLOW RATE(CFS) AT CONFLUENCE =    884.27
```

```
** CONFLUENCE DATA **
STREAM     RUNOFF      Tc      INTENSITY      AREA
NUMBER      (CFS)    (MIN.)   (INCH/HOUR)    (ACRE)
   1        84.03    23.53       2.426        49.00
   2       884.27    21.65       2.559       396.80

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       963.91    21.65      2.559
   2       922.12    23.53      2.426

COMPUTED CONFLUENCE ESTIMATES ARE AS FOLLOWS:
PEAK FLOW RATE(CFS) =     963.91   Tc(MIN.) =   21.65
TOTAL AREA(ACRES) =    445.80
LONGEST FLOWPATH FROM NODE    600.00 TO NODE    442.00 =  5730.00 FEET.
============================================================================
END OF STUDY SUMMARY:
TOTAL AREA(ACRES)      =    445.80  TC(MIN.) =     21.65
PEAK FLOW RATE(CFS)    =    963.91
============================================================================
============================================================================
END OF RATIONAL METHOD ANALYSIS

1
```

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

          RATIONAL METHOD HYDROLOGY COMPUTER PROGRAM PACKAGE
          Reference: SAN DIEGO COUNTY FLOOD CONTROL DISTRICT
                       1985,1981 HYDROLOGY MANUAL
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           Ver. 1.5A Release Date: 01/01/2000  License ID 1261

                         Analysis prepared by:

                        Rick Engineering Company
                            5620 Friars Road
                          San Diego, CA  92110
                             (619) 291-0707

 ************************** DESCRIPTION OF STUDY **************************
 * Otay Ranch Village 7   J-14483                                          *
 * 100-Year storm Event - Proposed Condition                               *
 * Basin 400 (EUC and McMillin)                                            *
 **************************************************************************

  FILE NAME: 1HV7B4BP.DAT
  TIME/DATE OF STUDY: 09:28 10/17/2003
----------------------------------------------------------------------------
  USER SPECIFIED HYDROLOGY AND HYDRAULIC MODEL INFORMATION:
----------------------------------------------------------------------------
  1985 SAN DIEGO MANUAL CRITERIA

  USER SPECIFIED STORM EVENT(YEAR) = 100.00
  6-HOUR DURATION PRECIPITATION (INCHES) =   2.500
  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: ONLY PEAK CONFLUENCE VALUES CONSIDERED
  *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

  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 4 ~~5~~21.00 TO NODE 4 ~~5~~21.00 IS CODE = 7

```
----------------------------------------------------------------------------
  >>>>>USER SPECIFIED HYDROLOGY INFORMATION AT NODE<<<<<
============================================================================
  USER-SPECIFIED VALUES ARE AS FOLLOWS:
  TC(MIN) =  14.68    RAIN INTENSITY(INCH/HOUR) =  3.29
  TOTAL AREA(ACRES) =    172.80    TOTAL RUNOFF(CFS) =     469.56
```

```
****************************************************************************
  FLOW PROCESS FROM NODE    521.00 TO NODE    521.40 IS CODE =  31
----------------------------------------------------------------------------
  >>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
  >>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<
============================================================================
  ELEVATION DATA: UPSTREAM(FEET) =    550.00  DOWNSTREAM(FEET) =   525.00
  FLOW LENGTH(FEET) =    800.00   MANNING'S N =  0.013
  DEPTH OF FLOW IN  63.0 INCH PIPE IS  48.7 INCHES
  PIPE-FLOW VELOCITY(FEET/SEC.) =  26.14
  ESTIMATED PIPE DIAMETER(INCH) =  63.00     NUMBER OF PIPES =   1
  PIPE-FLOW(CFS) =    469.56
  PIPE TRAVEL TIME(MIN.) =    0.51    Tc(MIN.) =  15.19
  LONGEST FLOWPATH FROM NODE      0.00 TO NODE    521.40 =    800.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    421.00 TO NODE    421.40 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.) =  15.19
  RAINFALL INTENSITY(INCH/HR) =   3.22
  TOTAL STREAM AREA(ACRES) =  172.80
  PEAK FLOW RATE(CFS) AT CONFLUENCE =     469.56

****************************************************************************
  FLOW PROCESS FROM NODE    421.10 TO NODE    421.20 IS CODE =  21
----------------------------------------------------------------------------
  >>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<
============================================================================
  *USER SPECIFIED(SUBAREA):
  INDUSTRIAL DEVELOPMENT RUNOFF COEFFICIENT = [redacted]
  S.C.S. CURVE NUMBER (AMC II) =    0
  INITIAL SUBAREA FLOW-LENGTH =    208.00
  UPSTREAM ELEVATION =     591.00
  DOWNSTREAM ELEVATION =     589.00
  ELEVATION DIFFERENCE =       2.00
  URBAN SUBAREA OVERLAND TIME OF FLOW(MINUTES) =     5.260
  TIME OF CONCENTRATION ASSUMED AS 6-MINUTES
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  5.856
  SUBAREA RUNOFF(CFS) =       2.11
  TOTAL AREA(ACRES) =        0.40   TOTAL RUNOFF(CFS) =        2.11

****************************************************************************
  FLOW PROCESS FROM NODE    421.20 TO NODE    421.00 IS CODE =  61
----------------------------------------------------------------------------
  >>>>>COMPUTE STREET FLOW TRAVEL TIME THRU SUBAREA<<<<<
  >>>>>(STANDARD CURB SECTION USED)<<<<<
============================================================================
  UPSTREAM ELEVATION(FEET) =   589.00  DOWNSTREAM ELEVATION(FEET) =   583.00
  STREET LENGTH(FEET) =     750.00   CURB HEIGHT(INCHES) =  6.0
  STREET HALFWIDTH(FEET) = 90.00

  DISTANCE FROM CROWN TO CROSSFALL GRADEBREAK(FEET) =   1.00
```

```
 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) =       4.51
   STREETFLOW MODEL RESULTS USING ESTIMATED FLOW:
   STREET FLOW DEPTH(FEET) =  0.41
   HALFSTREET FLOOD WIDTH(FEET) =   14.36
   AVERAGE FLOW VELOCITY(FEET/SEC.) =    2.07
   PRODUCT OF DEPTH&VELOCITY(FT*FT/SEC.) =     0.85
 STREET FLOW TRAVEL TIME(MIN.) =   6.05    Tc(MIN.) =  12.05
  100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  3.735
 *USER SPECIFIED(SUBAREA):
 INDUSTRIAL DEVELOPMENT RUNOFF COEFFICIENT =[redacted]
 S.C.S. CURVE NUMBER (AMC II) =    0
 SUBAREA AREA(ACRES) =     1.40      SUBAREA RUNOFF(CFS) =    4.71
 TOTAL AREA(ACRES) =     1.80        PEAK FLOW RATE(CFS) =       6.81

 END OF SUBAREA STREET FLOW HYDRAULICS:
 DEPTH(FEET) = 0.46   HALFSTREET FLOOD WIDTH(FEET) =  16.92
 FLOW VELOCITY(FEET/SEC.) =  2.28   DEPTH*VELOCITY(FT*FT/SEC.) =   1.06
 LONGEST FLOWPATH FROM NODE    421.10 TO NODE    421.00 =   958.00 FEET.

****************************************************************************
 FLOW PROCESS FROM NODE    421.00 TO NODE    421.00 IS CODE =  81
----------------------------------------------------------------------------
 >>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
============================================================================
  100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  3.735
 *USER SPECIFIED(SUBAREA):
 INDUSTRIAL DEVELOPMENT RUNOFF COEFFICIENT = [redacted]
 S.C.S. CURVE NUMBER (AMC II) =    0
 SUBAREA AREA(ACRES) =     1.40  SUBAREA RUNOFF(CFS) =    4.71
 TOTAL AREA(ACRES) =      3.20   TOTAL RUNOFF(CFS) =   11.52
 TC(MIN) =  12.05

****************************************************************************
 FLOW PROCESS FROM NODE    421.00 TO NODE    421.40 IS CODE =  61
----------------------------------------------------------------------------
 >>>>>COMPUTE STREET FLOW TRAVEL TIME THRU SUBAREA<<<<<
 >>>>>(STANDARD CURB SECTION USED)<<<<<
============================================================================
 UPSTREAM ELEVATION(FEET) =  555.00  DOWNSTREAM ELEVATION(FEET) =  535.00
 STREET LENGTH(FEET) =   810.00   CURB HEIGHT(INCHES) =  6.0
 STREET HALFWIDTH(FEET) = 40.00

 DISTANCE FROM CROWN TO CROSSFALL GRADEBREAK(FEET) =    1.00
 INSIDE STREET CROSSFALL(DECIMAL) =  0.020
 OUTSIDE STREET CROSSFALL(DECIMAL)  =  0.020

 SPECIFIED NUMBER OF HALFSTREETS CARRYING RUNOFF =  2
 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) = 15.20
STREETFLOW MODEL RESULTS USING ESTIMATED FLOW:
STREET FLOW DEPTH(FEET) = 0.41
HALFSTREET FLOOD WIDTH(FEET) = 14.13
AVERAGE FLOW VELOCITY(FEET/SEC.) = 3.59
PRODUCT OF DEPTH&VELOCITY(FT*FT/SEC.) = 1.47
STREET FLOW TRAVEL TIME(MIN.) = 3.76 Tc(MIN.) = 15.81
100 YEAR RAINFALL INTENSITY(INCH/HOUR) = 3.135
*USER SPECIFIED(SUBAREA):
INDUSTRIAL DEVELOPMENT RUNOFF COEFFICIENT [redacted]
S.C.S. CURVE NUMBER (AMC II) = 0
SUBAREA AREA(ACRES) = 2.60 SUBAREA RUNOFF(CFS) = 7.34
TOTAL AREA(ACRES) = 5.80 PEAK FLOW RATE(CFS) = 18.86

END OF SUBAREA STREET FLOW HYDRAULICS:
DEPTH(FEET) = 0.44 HALFSTREET FLOOD WIDTH(FEET) = 15.45
FLOW VELOCITY(FEET/SEC.) = 3.76 DEPTH*VELOCITY(FT*FT/SEC.) = 1.64
LONGEST FLOWPATH FROM NODE 421.10 TO NODE 421.40 = 1768.00 FEET.

****

FLOW PROCESS FROM NODE 421.00 TO NODE 421.40 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.) = 15.81
RAINFALL INTENSITY(INCH/HR) = 3.14
TOTAL STREAM AREA(ACRES) = 5.80
PEAK FLOW RATE(CFS) AT CONFLUENCE = 18.86

** CONFLUENCE DATA **

STREAM NUMBER	RUNOFF (CFS)	Tc (MIN.)	INTENSITY (INCH/HOUR)	AREA (ACRE)
1	469.56	15.19	3.217	172.80
2	18.86	15.81	3.135	5.80

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

** PEAK FLOW RATE TABLE **

STREAM NUMBER	RUNOFF (CFS)	Tc (MIN.)	INTENSITY (INCH/HOUR)
1	487.94	15.19	3.217
2	476.50	15.81	3.135

COMPUTED CONFLUENCE ESTIMATES ARE AS FOLLOWS:
PEAK FLOW RATE(CFS) = 487.94 Tc(MIN.) = 15.19
TOTAL AREA(ACRES) = 178.60
LONGEST FLOWPATH FROM NODE 421.10 TO NODE 421.40 = 1768.00 FEET.

****

FLOW PROCESS FROM NODE 421.40 TO NODE 421.40 IS CODE = 10

```
 >>>>>MAIN-STREAM MEMORY COPIED ONTO MEMORY BANK # 1 <<<<<
==============================================================================

******************************************************************************
 FLOW PROCESS FROM NODE    422.00 TO NODE    423.00 IS CODE =  21
------------------------------------------------------------------------------
 >>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<
==============================================================================
 *USER SPECIFIED(SUBAREA):
 SINGLE FAMILY DEVELOPMENT RUNOFF COEFFICIENT = [redacted]
 S.C.S. CURVE NUMBER (AMC II) =    0
 INITIAL SUBAREA FLOW-LENGTH =   100.00
 UPSTREAM ELEVATION =    582.00
 DOWNSTREAM ELEVATION =    581.00
 ELEVATION DIFFERENCE =      1.00
 URBAN SUBAREA OVERLAND TIME OF FLOW(MINUTES) =    8.100
  100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  4.825
 SUBAREA RUNOFF(CFS) =      0.63
 TOTAL AREA(ACRES) =       0.20  TOTAL RUNOFF(CFS) =       0.63

******************************************************************************
 FLOW PROCESS FROM NODE    423.00 TO NODE    424.00 IS CODE =  61
------------------------------------------------------------------------------
 >>>>>COMPUTE STREET FLOW TRAVEL TIME THRU SUBAREA<<<<<
 >>>>>(STANDARD CURB SECTION USED)<<<<<
==============================================================================
 UPSTREAM ELEVATION(FEET) =  579.00  DOWNSTREAM ELEVATION(FEET) =  573.00
 STREET LENGTH(FEET) =   430.00   CURB HEIGHT(INCHES) =  6.0
 STREET HALFWIDTH(FEET) = 30.00

 DISTANCE FROM CROWN TO CROSSFALL GRADEBREAK(FEET) =   1.00
 INSIDE STREET CROSSFALL(DECIMAL) =  0.020
 OUTSIDE STREET CROSSFALL(DECIMAL)  =  0.020

 SPECIFIED NUMBER OF HALFSTREETS CARRYING RUNOFF =  2
 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) =       4.69
   STREETFLOW MODEL RESULTS USING ESTIMATED FLOW:
   STREET FLOW DEPTH(FEET) =  0.32
   HALFSTREET FLOOD WIDTH(FEET) =    9.80
   AVERAGE FLOW VELOCITY(FEET/SEC.) =    2.17
   PRODUCT OF DEPTH&VELOCITY(FT*FT/SEC.) =    0.70
 STREET FLOW TRAVEL TIME(MIN.) =   3.30   Tc(MIN.) =  11.40
  100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  3.871
 *USER SPECIFIED(SUBAREA):
 SINGLE FAMILY DEVELOPMENT RUNOFF COEFFICIENT = [redacted]
 S.C.S. CURVE NUMBER (AMC II) =    0
 SUBAREA AREA(ACRES) =    3.20     SUBAREA RUNOFF(CFS) =     8.05
 TOTAL AREA(ACRES) =     3.40      PEAK FLOW RATE(CFS) =        8.68

 END OF SUBAREA STREET FLOW HYDRAULICS:
 DEPTH(FEET) = 0.38   HALFSTREET FLOOD WIDTH(FEET) =  12.65
 FLOW VELOCITY(FEET/SEC.) =  2.53   DEPTH*VELOCITY(FT*FT/SEC.) =   0.96
 LONGEST FLOWPATH FROM NODE    422.00 TO NODE    424.00 =   530.00 FEET.
```

```
****************************************************************************
  FLOW PROCESS FROM NODE    424.00 TO NODE    424.00 IS CODE =  81
----------------------------------------------------------------------------
  >>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
============================================================================
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  3.871
  *USER SPECIFIED(SUBAREA):
  SINGLE FAMILY DEVELOPMENT RUNOFF COEFFICIENT = [redacted]
  S.C.S. CURVE NUMBER (AMC II) =   0
  SUBAREA AREA(ACRES) =    0.90   SUBAREA RUNOFF(CFS) =    2.26
  TOTAL AREA(ACRES) =     4.30   TOTAL RUNOFF(CFS) =   10.94
  TC(MIN) =  11.40

****************************************************************************
  FLOW PROCESS FROM NODE    424.00 TO NODE    426.00 IS CODE =  61
----------------------------------------------------------------------------
  >>>>>COMPUTE STREET FLOW TRAVEL TIME THRU SUBAREA<<<<<
  >>>>>(STANDARD CURB SECTION USED)<<<<<
============================================================================
  UPSTREAM ELEVATION(FEET) =  573.00  DOWNSTREAM ELEVATION(FEET) =  569.00
  STREET LENGTH(FEET) =   260.00   CURB HEIGHT(INCHES) =  6.0
  STREET HALFWIDTH(FEET) = 30.00

  DISTANCE FROM CROWN TO CROSSFALL GRADEBREAK(FEET) =   1.00
  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) =      12.35
    STREETFLOW MODEL RESULTS USING ESTIMATED FLOW:
    STREET FLOW DEPTH(FEET) =  0.50
    HALFSTREET FLOOD WIDTH(FEET) =   18.96
    AVERAGE FLOW VELOCITY(FEET/SEC.) =    3.37
    PRODUCT OF DEPTH&VELOCITY(FT*FT/SEC.) =     1.70
  STREET FLOW TRAVEL TIME(MIN.) =   1.28   Tc(MIN.) =  12.68
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  3.614
  *USER SPECIFIED(SUBAREA):
  RURAL DEVELOPMENT RUNOFF COEFFICIENT = [redacted]
  S.C.S. CURVE NUMBER (AMC II) =   0
  SUBAREA AREA(ACRES) =    1.20      SUBAREA RUNOFF(CFS) =    2.82
  TOTAL AREA(ACRES) =    5.50        PEAK FLOW RATE(CFS) =      13.76

  END OF SUBAREA STREET FLOW HYDRAULICS:
  DEPTH(FEET) = 0.52   HALFSTREET FLOOD WIDTH(FEET) =  20.57
  FLOW VELOCITY(FEET/SEC.) =  3.46   DEPTH*VELOCITY(FT*FT/SEC.) =   1.79
  LONGEST FLOWPATH FROM NODE    422.00 TO NODE    426.00 =   790.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    426.00 TO NODE    426.00 IS CODE =  81
----------------------------------------------------------------------------
  >>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
============================================================================
```

```
 100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  3.614
*USER SPECIFIED(SUBAREA):
SINGLE FAMILY DEVELOPMENT RUNOFF COEFFICIENT = [redacted]
S.C.S. CURVE NUMBER (AMC II) =    0
SUBAREA AREA(ACRES) =     1.20   SUBAREA RUNOFF(CFS) =     2.82
TOTAL AREA(ACRES) =      6.70   TOTAL RUNOFF(CFS) =    16.58
TC(MIN) =  12.68

****************************************************************************
 FLOW PROCESS FROM NODE    426.00 TO NODE    427.00 IS CODE =  31
----------------------------------------------------------------------------
 >>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
 >>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<
============================================================================
 ELEVATION DATA: UPSTREAM(FEET) =   559.00  DOWNSTREAM(FEET) =   549.00
 FLOW LENGTH(FEET) =    445.00   MANNING'S N =  0.013
 DEPTH OF FLOW IN  21.0 INCH PIPE IS  13.4 INCHES
 PIPE-FLOW VELOCITY(FEET/SEC.) =  10.25
 ESTIMATED PIPE DIAMETER(INCH) =  21.00    NUMBER OF PIPES =   1
 PIPE-FLOW(CFS) =     16.58
 PIPE TRAVEL TIME(MIN.) =   0.72    Tc(MIN.) =  13.41
 LONGEST FLOWPATH FROM NODE    422.00 TO NODE    427.00 =  1235.00 FEET.

****************************************************************************
 FLOW PROCESS FROM NODE    427.00 TO NODE    427.00 IS CODE =  81
----------------------------------------------------------------------------
 >>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
============================================================================
  100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  3.487
 *USER SPECIFIED(SUBAREA):
 SINGLE FAMILY DEVELOPMENT RUNOFF COEFFICIENT = [redacted]
 S.C.S. CURVE NUMBER (AMC II) =    0
 SUBAREA AREA(ACRES) =     2.90   SUBAREA RUNOFF(CFS) =     6.57
 TOTAL AREA(ACRES) =      9.60   TOTAL RUNOFF(CFS) =    23.15
 TC(MIN) =  13.41

****************************************************************************
 FLOW PROCESS FROM NODE    427.00 TO NODE    427.00 IS CODE =  81
----------------------------------------------------------------------------
 >>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
============================================================================
  100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  3.487
 *USER SPECIFIED(SUBAREA):
 SINGLE FAMILY DEVELOPMENT RUNOFF COEFFICIENT = [redacted]
 S.C.S. CURVE NUMBER (AMC II) =    0
 SUBAREA AREA(ACRES) =     2.00   SUBAREA RUNOFF(CFS) =     4.53
 TOTAL AREA(ACRES) =     11.60   TOTAL RUNOFF(CFS) =    27.69
 TC(MIN) =  13.41

****************************************************************************
 FLOW PROCESS FROM NODE    427.00 TO NODE    427.00 IS CODE =  81
----------------------------------------------------------------------------
 >>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
============================================================================
  100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  3.487
 *USER SPECIFIED(SUBAREA):
 SINGLE FAMILY DEVELOPMENT RUNOFF COEFFICIENT = [redacted]
```

```
  S.C.S. CURVE NUMBER (AMC II) =    0
  SUBAREA AREA(ACRES) =     1.90   SUBAREA RUNOFF(CFS) =     4.31
  TOTAL AREA(ACRES) =     13.50    TOTAL RUNOFF(CFS) =    31.99
  TC(MIN) =  13.41

****************************************************************************
  FLOW PROCESS FROM NODE    427.00 TO NODE    428.00 IS CODE =  31
----------------------------------------------------------------------------
  >>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
  >>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<
============================================================================
  ELEVATION DATA: UPSTREAM(FEET) =    549.00  DOWNSTREAM(FEET) =    539.00
  FLOW LENGTH(FEET) =   250.00   MANNING'S N =  0.013
  DEPTH OF FLOW IN  24.0 INCH PIPE IS  15.4 INCHES
  PIPE-FLOW VELOCITY(FEET/SEC.) =  14.99
  ESTIMATED PIPE DIAMETER(INCH) =  24.00    NUMBER OF PIPES =   1
  PIPE-FLOW(CFS) =     31.99
  PIPE TRAVEL TIME(MIN.) =   0.28    Tc(MIN.) =  13.68
  LONGEST FLOWPATH FROM NODE    422.00 TO NODE    428.00 =  1485.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    428.00 TO NODE    428.00 IS CODE =  81
----------------------------------------------------------------------------
  >>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
============================================================================
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  3.441
  *USER SPECIFIED(SUBAREA):
  SINGLE FAMILY DEVELOPMENT RUNOFF COEFFICIENT [redacted]
  S.C.S. CURVE NUMBER (AMC II) =    0
  SUBAREA AREA(ACRES) =     0.30   SUBAREA RUNOFF(CFS) =     0.93
  TOTAL AREA(ACRES) =     13.80    TOTAL RUNOFF(CFS) =    32.92
  TC(MIN) =  13.68

****************************************************************************
  FLOW PROCESS FROM NODE    428.00 TO NODE    428.00 IS CODE =  81
----------------------------------------------------------------------------
  >>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
============================================================================
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  3.441
  *USER SPECIFIED(SUBAREA):
  SINGLE FAMILY DEVELOPMENT RUNOFF COEFFICIENT [redacted]
  S.C.S. CURVE NUMBER (AMC II) =    0
  SUBAREA AREA(ACRES) =     2.00   SUBAREA RUNOFF(CFS) =     4.47
  TOTAL AREA(ACRES) =     15.80    TOTAL RUNOFF(CFS) =    37.40
  TC(MIN) =  13.68

****************************************************************************
  FLOW PROCESS FROM NODE    428.00 TO NODE    428.00 IS CODE =  81
----------------------------------------------------------------------------
  >>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
============================================================================
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  3.441
  *USER SPECIFIED(SUBAREA):
  SINGLE FAMILY DEVELOPMENT RUNOFF COEFFICIENT = [redacted]
  S.C.S. CURVE NUMBER (AMC II) =    0
  SUBAREA AREA(ACRES) =     1.20   SUBAREA RUNOFF(CFS) =     2.68
  TOTAL AREA(ACRES) =     17.00    TOTAL RUNOFF(CFS) =    40.08
```

```
 TC(MIN) =  13.68

****************************************************************************
 FLOW PROCESS FROM NODE    428.00 TO NODE    429.00 IS CODE =  31
----------------------------------------------------------------------------
 >>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
 >>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<
============================================================================
 ELEVATION DATA: UPSTREAM(FEET) =   539.00  DOWNSTREAM(FEET) =   530.00
 FLOW LENGTH(FEET) =   300.00   MANNING'S N =  0.013
 DEPTH OF FLOW IN  27.0 INCH PIPE IS  18.0 INCHES
 PIPE-FLOW VELOCITY(FEET/SEC.) =  14.19
 ESTIMATED PIPE DIAMETER(INCH) =  27.00    NUMBER OF PIPES =   1
 PIPE-FLOW(CFS) =      40.08
 PIPE TRAVEL TIME(MIN.) =   0.35    Tc(MIN.) =  14.04
 LONGEST FLOWPATH FROM NODE    422.00 TO NODE    429.00 =  1785.00 FEET.

****************************************************************************
 FLOW PROCESS FROM NODE    429.00 TO NODE    429.00 IS CODE =  81
----------------------------------------------------------------------------
 >>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
============================================================================
  100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  3.385
 *USER SPECIFIED(SUBAREA):
 SINGLE FAMILY DEVELOPMENT RUNOFF COEFFICIENT = [redacted]
 S.C.S. CURVE NUMBER (AMC II) =   0
 SUBAREA AREA(ACRES) =     0.40   SUBAREA RUNOFF(CFS) =     1.22
 TOTAL AREA(ACRES) =     17.40   TOTAL RUNOFF(CFS) =    41.30
 TC(MIN) =  14.04

****************************************************************************
 FLOW PROCESS FROM NODE    429.00 TO NODE    429.00 IS CODE =  81
----------------------------------------------------------------------------
 >>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
============================================================================
  100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  3.385
 *USER SPECIFIED(SUBAREA):
 SINGLE FAMILY DEVELOPMENT RUNOFF COEFFICIENT = [redacted]
 S.C.S. CURVE NUMBER (AMC II) =   0
 SUBAREA AREA(ACRES) =     1.70   SUBAREA RUNOFF(CFS) =     3.74
 TOTAL AREA(ACRES) =     19.10   TOTAL RUNOFF(CFS) =    45.04
 TC(MIN) =  14.04

****************************************************************************
 FLOW PROCESS FROM NODE    429.00 TO NODE    429.00 IS CODE =  81
----------------------------------------------------------------------------
 >>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
============================================================================
  100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  3.385
 *USER SPECIFIED(SUBAREA):
 SINGLE FAMILY DEVELOPMENT RUNOFF COEFFICIENT = [redacted]
 S.C.S. CURVE NUMBER (AMC II) =   0
 SUBAREA AREA(ACRES) =     1.20   SUBAREA RUNOFF(CFS) =     3.05
 TOTAL AREA(ACRES) =     20.30   TOTAL RUNOFF(CFS) =    48.09
 TC(MIN) =  14.04

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

```
FLOW PROCESS FROM NODE    429.00 TO NODE    429.00 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.) =  14.04
RAINFALL INTENSITY(INCH/HR) =   3.38
TOTAL STREAM AREA(ACRES) =   20.30
PEAK FLOW RATE(CFS) AT CONFLUENCE =     48.09

****************************************************************************
FLOW PROCESS FROM NODE    430.00 TO NODE    431.00 IS CODE =  21
----------------------------------------------------------------------------
>>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<
============================================================================
*USER SPECIFIED(SUBAREA):
SINGLE FAMILY DEVELOPMENT RUNOFF COEFFICIENT =[redacted]
S.C.S. CURVE NUMBER (AMC II) =   0
INITIAL SUBAREA FLOW-LENGTH =   100.00
UPSTREAM ELEVATION =    577.50
DOWNSTREAM ELEVATION =    576.50
ELEVATION DIFFERENCE =      1.00
URBAN SUBAREA OVERLAND TIME OF FLOW(MINUTES) =    8.100
 100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  4.825
SUBAREA RUNOFF(CFS) =        0.31
TOTAL AREA(ACRES) =       0.10   TOTAL RUNOFF(CFS) =       0.31

****************************************************************************
FLOW PROCESS FROM NODE    431.00 TO NODE    432.00 IS CODE =  61
----------------------------------------------------------------------------
>>>>>COMPUTE STREET FLOW TRAVEL TIME THRU SUBAREA<<<<<
>>>>>(STANDARD CURB SECTION USED)<<<<<
============================================================================
UPSTREAM ELEVATION(FEET) =  576.00  DOWNSTREAM ELEVATION(FEET) =  574.00
STREET LENGTH(FEET) =   175.00   CURB HEIGHT(INCHES) =  6.0
STREET HALFWIDTH(FEET) = 30.00

DISTANCE FROM CROWN TO CROSSFALL GRADEBREAK(FEET) =   1.00
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) =       1.01
  STREETFLOW MODEL RESULTS USING ESTIMATED FLOW:
  STREET FLOW DEPTH(FEET) =  0.27
  HALFSTREET FLOOD WIDTH(FEET) =    6.95
  AVERAGE FLOW VELOCITY(FEET/SEC.) =    1.67
  PRODUCT OF DEPTH&VELOCITY(FT*FT/SEC.) =    0.44
STREET FLOW TRAVEL TIME(MIN.) =    1.74   Tc(MIN.) =   9.84
 100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  4.256
*USER SPECIFIED(SUBAREA):
SINGLE FAMILY DEVELOPMENT RUNOFF COEFFICIENT =[redacted]
```

```
S.C.S. CURVE NUMBER (AMC II) =    0
SUBAREA AREA(ACRES) =     0.50        SUBAREA RUNOFF(CFS) =     1.38
TOTAL AREA(ACRES) =     0.60          PEAK FLOW RATE(CFS) =        1.70

END OF SUBAREA STREET FLOW HYDRAULICS:
DEPTH(FEET) = 0.30   HALFSTREET FLOOD WIDTH(FEET) =    8.89
FLOW VELOCITY(FEET/SEC.) =  1.87   DEPTH*VELOCITY(FT*FT/SEC.) =   0.57
LONGEST FLOWPATH FROM NODE    430.00 TO NODE    432.00 =   275.00 FEET.

****************************************************************************
FLOW PROCESS FROM NODE    432.00 TO NODE    432.00 IS CODE =  81
----------------------------------------------------------------------------
>>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
============================================================================
 100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  4.256
*USER SPECIFIED(SUBAREA):
SINGLE FAMILY DEVELOPMENT RUNOFF COEFFICIENT = [redacted]
S.C.S. CURVE NUMBER (AMC II) =    0
SUBAREA AREA(ACRES) =     0.60   SUBAREA RUNOFF(CFS) =     1.66
TOTAL AREA(ACRES) =     1.20   TOTAL RUNOFF(CFS) =     3.36
TC(MIN) =    9.84

****************************************************************************
FLOW PROCESS FROM NODE    432.00 TO NODE    433.00 IS CODE =  61
----------------------------------------------------------------------------
>>>>>COMPUTE STREET FLOW TRAVEL TIME THRU SUBAREA<<<<<
>>>>>(STANDARD CURB SECTION USED)<<<<<
============================================================================
UPSTREAM ELEVATION(FEET) =  574.00  DOWNSTREAM ELEVATION(FEET) =  566.00
STREET LENGTH(FEET) =   240.00   CURB HEIGHT(INCHES) =  6.0
STREET HALFWIDTH(FEET) = 30.00

DISTANCE FROM CROWN TO CROSSFALL GRADEBREAK(FEET) =    1.00
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) =        3.74
  STREETFLOW MODEL RESULTS USING ESTIMATED FLOW:
  STREET FLOW DEPTH(FEET) =  0.32
  HALFSTREET FLOOD WIDTH(FEET) =    9.91
  AVERAGE FLOW VELOCITY(FEET/SEC.) =    3.40
  PRODUCT OF DEPTH&VELOCITY(FT*FT/SEC.) =    1.10
STREET FLOW TRAVEL TIME(MIN.) =   1.18   Tc(MIN.) =  11.02
 100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  3.957
*USER SPECIFIED(SUBAREA):
SINGLE FAMILY DEVELOPMENT RUNOFF COEFFICIENT = [redacted]
S.C.S. CURVE NUMBER (AMC II) =    0
SUBAREA AREA(ACRES) =     0.30        SUBAREA RUNOFF(CFS) =     0.77
TOTAL AREA(ACRES) =     1.50          PEAK FLOW RATE(CFS) =        4.13

END OF SUBAREA STREET FLOW HYDRAULICS:
DEPTH(FEET) = 0.33   HALFSTREET FLOOD WIDTH(FEET) =  10.34
```

```
  FLOW VELOCITY(FEET/SEC.) =  3.48   DEPTH*VELOCITY(FT*FT/SEC.) =   1.16
  LONGEST FLOWPATH FROM NODE     430.00 TO NODE     433.00 =    515.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    433.00 TO NODE    433.00 IS CODE =  81
----------------------------------------------------------------------------
  >>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
============================================================================
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  3.957
  *USER SPECIFIED(SUBAREA):
  SINGLE FAMILY DEVELOPMENT RUNOFF COEFFICIENT = [redacted]
  S.C.S. CURVE NUMBER (AMC II) =    0
  SUBAREA AREA(ACRES) =    1.00   SUBAREA RUNOFF(CFS) =    2.57
  TOTAL AREA(ACRES) =      2.50   TOTAL RUNOFF(CFS) =     6.70
  TC(MIN) =  11.02

****************************************************************************
  FLOW PROCESS FROM NODE    433.00 TO NODE    434.00 IS CODE =  61
----------------------------------------------------------------------------
  >>>>>COMPUTE STREET FLOW TRAVEL TIME THRU SUBAREA<<<<<
  >>>>>(STANDARD CURB SECTION USED)<<<<<
============================================================================
  UPSTREAM ELEVATION(FEET) =  566.00  DOWNSTREAM ELEVATION(FEET) =  565.00
  STREET LENGTH(FEET) =    125.00   CURB HEIGHT(INCHES) =  6.0
  STREET HALFWIDTH(FEET) = 30.00

  DISTANCE FROM CROWN TO CROSSFALL GRADEBREAK(FEET) =   1.00
  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) =        7.07
    STREETFLOW MODEL RESULTS USING ESTIMATED FLOW:
    STREET FLOW DEPTH(FEET) =  0.47
    HALFSTREET FLOOD WIDTH(FEET) =   17.21
    AVERAGE FLOW VELOCITY(FEET/SEC.) =    2.29
    PRODUCT OF DEPTH&VELOCITY(FT*FT/SEC.) =    1.08
  STREET FLOW TRAVEL TIME(MIN.) =    0.91   Tc(MIN.) =  11.92
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  3.760
  *USER SPECIFIED(SUBAREA):
  SINGLE FAMILY DEVELOPMENT RUNOFF COEFFICIENT [redacted]
  S.C.S. CURVE NUMBER (AMC II) =    0
  SUBAREA AREA(ACRES) =    0.30       SUBAREA RUNOFF(CFS) =    0.73
  TOTAL AREA(ACRES) =    2.80         PEAK FLOW RATE(CFS) =       7.43

  END OF SUBAREA STREET FLOW HYDRAULICS:
  DEPTH(FEET) = 0.48   HALFSTREET FLOOD WIDTH(FEET) =  17.53
  FLOW VELOCITY(FEET/SEC.) =  2.33   DEPTH*VELOCITY(FT*FT/SEC.) =   1.11
  LONGEST FLOWPATH FROM NODE     430.00 TO NODE     434.00 =    640.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    434.00 TO NODE    434.00 IS CODE =  81
----------------------------------------------------------------------------
```

```
>>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
==============================================================================
 100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  3.760
 *USER SPECIFIED(SUBAREA):
 SINGLE FAMILY DEVELOPMENT RUNOFF COEFFICIENT =[redacted]
 S.C.S. CURVE NUMBER (AMC II) =   0
 SUBAREA AREA(ACRES) =     0.60   SUBAREA RUNOFF(CFS) =     1.47
 TOTAL AREA(ACRES) =     3.40   TOTAL RUNOFF(CFS) =     8.90
 TC(MIN) =  11.92

******************************************************************************
 FLOW PROCESS FROM NODE    434.00 TO NODE    434.00 IS CODE =  81
------------------------------------------------------------------------------
 >>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
==============================================================================
 100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  3.760
 *USER SPECIFIED(SUBAREA):
 RURAL DEVELOPMENT RUNOFF COEFFICIENT =[redacted]
 S.C.S. CURVE NUMBER (AMC II) =   0
 SUBAREA AREA(ACRES) =     1.00   SUBAREA RUNOFF(CFS) =     2.07
 TOTAL AREA(ACRES) =     4.40   TOTAL RUNOFF(CFS) =    10.97
 TC(MIN) =  11.92

******************************************************************************
 FLOW PROCESS FROM NODE    434.00 TO NODE    435.00 IS CODE =  31
------------------------------------------------------------------------------
 >>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
 >>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<
==============================================================================
 ELEVATION DATA: UPSTREAM(FEET) =   555.00  DOWNSTREAM(FEET) =   550.00
 FLOW LENGTH(FEET) =   226.00   MANNING'S N = 0.013
 DEPTH OF FLOW IN  18.0 INCH PIPE IS  11.5 INCHES
 PIPE-FLOW VELOCITY(FEET/SEC.) =    9.19
 ESTIMATED PIPE DIAMETER(INCH) =  18.00    NUMBER OF PIPES =   1
 PIPE-FLOW(CFS) =     10.97
 PIPE TRAVEL TIME(MIN.) =   0.41    Tc(MIN.) =  12.33
 LONGEST FLOWPATH FROM NODE    430.00 TO NODE    435.00 =   866.00 FEET.

******************************************************************************
 FLOW PROCESS FROM NODE    435.00 TO NODE    435.00 IS CODE =  81
------------------------------------------------------------------------------
 >>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
==============================================================================
 100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  3.679
 *USER SPECIFIED(SUBAREA):
 SINGLE FAMILY DEVELOPMENT RUNOFF COEFFICIENT = [redacted]
 S.C.S. CURVE NUMBER (AMC II) =   0
 SUBAREA AREA(ACRES) =     0.40   SUBAREA RUNOFF(CFS) =     1.32
 TOTAL AREA(ACRES) =     4.80   TOTAL RUNOFF(CFS) =    12.29
 TC(MIN) =  12.33

******************************************************************************
 FLOW PROCESS FROM NODE    435.00 TO NODE    435.00 IS CODE =  81
------------------------------------------------------------------------------
 >>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
==============================================================================
 100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  3.679
```

```
*USER SPECIFIED(SUBAREA):
SINGLE FAMILY DEVELOPMENT RUNOFF COEFFICIENT = [redacted]
S.C.S. CURVE NUMBER (AMC II) =  0
SUBAREA AREA(ACRES) =     1.30   SUBAREA RUNOFF(CFS) =     3.11
TOTAL AREA(ACRES) =      6.10    TOTAL RUNOFF(CFS) =    15.40
TC(MIN) =  12.33

****************************************************************************
FLOW PROCESS FROM NODE    435.00 TO NODE    435.00 IS CODE =  81
----------------------------------------------------------------------------
>>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
============================================================================
 100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  3.679
*USER SPECIFIED(SUBAREA):
SINGLE FAMILY DEVELOPMENT RUNOFF COEFFICIENT = [redacted]
S.C.S. CURVE NUMBER (AMC II) =  0
SUBAREA AREA(ACRES) =     1.30   SUBAREA RUNOFF(CFS) =     3.11
TOTAL AREA(ACRES) =      7.40    TOTAL RUNOFF(CFS) =    18.51
TC(MIN) =  12.33

****************************************************************************
FLOW PROCESS FROM NODE    435.00 TO NODE    436.00 IS CODE =  31
----------------------------------------------------------------------------
>>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
>>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<
============================================================================
ELEVATION DATA: UPSTREAM(FEET) =    550.00  DOWNSTREAM(FEET) =   542.00
FLOW LENGTH(FEET) =    258.00    MANNING'S N =  0.013
DEPTH OF FLOW IN  21.0 INCH PIPE IS  12.9 INCHES
PIPE-FLOW VELOCITY(FEET/SEC.) =  11.91
ESTIMATED PIPE DIAMETER(INCH) =  21.00     NUMBER OF PIPES =   1
PIPE-FLOW(CFS) =       18.51
PIPE TRAVEL TIME(MIN.) =    0.36    Tc(MIN.) =  12.70
LONGEST FLOWPATH FROM NODE    430.00 TO NODE    436.00 =  1124.00 FEET.

****************************************************************************
FLOW PROCESS FROM NODE    436.00 TO NODE    436.00 IS CODE =  81
----------------------------------------------------------------------------
>>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
============================================================================
 100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  3.611
*USER SPECIFIED(SUBAREA):
SINGLE FAMILY DEVELOPMENT RUNOFF COEFFICIENT = [redacted]
S.C.S. CURVE NUMBER (AMC II) =  0
SUBAREA AREA(ACRES) =     2.70   SUBAREA RUNOFF(CFS) =     6.34
TOTAL AREA(ACRES) =     10.10    TOTAL RUNOFF(CFS) =    24.85
TC(MIN) =  12.70

****************************************************************************
FLOW PROCESS FROM NODE    436.00 TO NODE    436.00 IS CODE =  81
----------------------------------------------------------------------------
>>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
============================================================================
 100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  3.611
*USER SPECIFIED(SUBAREA):
SINGLE FAMILY DEVELOPMENT RUNOFF COEFFICIENT = [redacted]
S.C.S. CURVE NUMBER (AMC II) =  0
```

```
SUBAREA AREA(ACRES) =     0.60    SUBAREA RUNOFF(CFS) =     1.41
TOTAL AREA(ACRES) =    10.70    TOTAL RUNOFF(CFS) =    26.26
TC(MIN) =  12.70

****************************************************************************
FLOW PROCESS FROM NODE    436.00 TO NODE    436.00 IS CODE =  81
----------------------------------------------------------------------------
>>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
============================================================================
 100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  3.611
*USER SPECIFIED(SUBAREA):
INDUSTRIAL DEVELOPMENT RUNOFF COEFFICIENT = [redacted]
S.C.S. CURVE NUMBER (AMC II) =   0
SUBAREA AREA(ACRES) =     0.40    SUBAREA RUNOFF(CFS) =     1.30
TOTAL AREA(ACRES) =    11.10    TOTAL RUNOFF(CFS) =    27.56
TC(MIN) =  12.70

****************************************************************************
FLOW PROCESS FROM NODE    436.00 TO NODE    438.00 IS CODE =  31
----------------------------------------------------------------------------
>>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
>>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<
============================================================================
ELEVATION DATA: UPSTREAM(FEET) =    542.00  DOWNSTREAM(FEET) =    534.00
FLOW LENGTH(FEET) =    298.00   MANNING'S N =  0.013
DEPTH OF FLOW IN  24.0 INCH PIPE IS  16.0 INCHES
PIPE-FLOW VELOCITY(FEET/SEC.) =  12.40
ESTIMATED PIPE DIAMETER(INCH) =  24.00    NUMBER OF PIPES =   1
PIPE-FLOW(CFS) =      27.56
PIPE TRAVEL TIME(MIN.) =   0.40    Tc(MIN.) =  13.10
LONGEST FLOWPATH FROM NODE    430.00 TO NODE    438.00 =  1422.00 FEET.

****************************************************************************
FLOW PROCESS FROM NODE    438.00 TO NODE    438.00 IS CODE =  81
----------------------------------------------------------------------------
>>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
============================================================================
 100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  3.540
*USER SPECIFIED(SUBAREA):
INDUSTRIAL DEVELOPMENT RUNOFF COEFFICIENT = [redacted]
S.C.S. CURVE NUMBER (AMC II) =   0
SUBAREA AREA(ACRES) =     0.50    SUBAREA RUNOFF(CFS) =     1.59
TOTAL AREA(ACRES) =    11.60    TOTAL RUNOFF(CFS) =    29.15
TC(MIN) =  13.10

****************************************************************************
FLOW PROCESS FROM NODE    438.00 TO NODE    438.00 IS CODE =  81
----------------------------------------------------------------------------
>>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
============================================================================
 100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  3.540
*USER SPECIFIED(SUBAREA):
SINGLE FAMILY DEVELOPMENT RUNOFF COEFFICIENT = [redacted]
S.C.S. CURVE NUMBER (AMC II) =   0
SUBAREA AREA(ACRES) =     1.40    SUBAREA RUNOFF(CFS) =     3.22
TOTAL AREA(ACRES) =    13.00    TOTAL RUNOFF(CFS) =    32.37
TC(MIN) =  13.10
```

```
****************************************************************************
  FLOW PROCESS FROM NODE     438.00 TO NODE     438.00 IS CODE =  81
----------------------------------------------------------------------------
  >>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
============================================================================
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  3.540
  *USER SPECIFIED(SUBAREA):
  SINGLE FAMILY DEVELOPMENT RUNOFF COEFFICIENT = [redacted]
  S.C.S. CURVE NUMBER (AMC II) =   0
  SUBAREA AREA(ACRES) =     1.20    SUBAREA RUNOFF(CFS) =     3.19
  TOTAL AREA(ACRES) =     14.20    TOTAL RUNOFF(CFS) =    35.56
  TC(MIN) =  13.10

****************************************************************************
  FLOW PROCESS FROM NODE     438.00 TO NODE     429.00 IS CODE =  31
----------------------------------------------------------------------------
  >>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
  >>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<
============================================================================
  ELEVATION DATA: UPSTREAM(FEET) =    534.00  DOWNSTREAM(FEET) =    530.00
  FLOW LENGTH(FEET) =    380.00   MANNING'S N =  0.013
  DEPTH OF FLOW IN  30.0 INCH PIPE IS  22.0 INCHES
  PIPE-FLOW VELOCITY(FEET/SEC.) =    9.20
  ESTIMATED PIPE DIAMETER(INCH) =  30.00    NUMBER OF PIPES =   1
  PIPE-FLOW(CFS) =      35.56
  PIPE TRAVEL TIME(MIN.) =    0.69    Tc(MIN.) =   13.78
  LONGEST FLOWPATH FROM NODE     430.00 TO NODE     429.00 =   1802.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE     438.00 TO NODE     429.00 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.) =  13.78
  RAINFALL INTENSITY(INCH/HR) =   3.42
  TOTAL STREAM AREA(ACRES) =   14.20
  PEAK FLOW RATE(CFS) AT CONFLUENCE =     35.56

  ** CONFLUENCE DATA **
  STREAM      RUNOFF      Tc       INTENSITY       AREA
  NUMBER       (CFS)     (MIN.)   (INCH/HOUR)     (ACRE)
      1        48.09     14.04        3.385          20.30
      2        35.56     13.78        3.425          14.20

  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        83.08     13.78        3.425
      2        83.23     14.04        3.385
```

```
 COMPUTED CONFLUENCE ESTIMATES ARE AS FOLLOWS:
 PEAK FLOW RATE(CFS) =      83.23   Tc(MIN.) =    14.04
 TOTAL AREA(ACRES) =     34.50
 LONGEST FLOWPATH FROM NODE    430.00 TO NODE    429.00 =  1802.00 FEET.

****************************************************************************
 FLOW PROCESS FROM NODE    429.00 TO NODE    421.40 IS CODE =  31
----------------------------------------------------------------------------
 >>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
 >>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<
============================================================================
 ELEVATION DATA: UPSTREAM(FEET) =    530.00  DOWNSTREAM(FEET) =    525.00
 FLOW LENGTH(FEET) =    175.00   MANNING'S N =  0.013
 DEPTH OF FLOW IN  33.0 INCH PIPE IS  26.5 INCHES
 PIPE-FLOW VELOCITY(FEET/SEC.) =  16.29
 ESTIMATED PIPE DIAMETER(INCH) =  33.00    NUMBER OF PIPES =   1
 PIPE-FLOW(CFS) =      83.23
 PIPE TRAVEL TIME(MIN.) =    0.18    Tc(MIN.) =  14.21
 LONGEST FLOWPATH FROM NODE    430.00 TO NODE    421.40 =  1977.00 FEET.

****************************************************************************
 FLOW PROCESS FROM NODE    421.40 TO NODE    421.40 IS CODE =  81
----------------------------------------------------------------------------
 >>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
============================================================================
  100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  3.357
 *USER SPECIFIED(SUBAREA):
 SINGLE FAMILY DEVELOPMENT RUNOFF COEFFICIENT = [redacted]
 S.C.S. CURVE NUMBER (AMC II) =   0
 SUBAREA AREA(ACRES) =    0.60    SUBAREA RUNOFF(CFS) =    1.31
 TOTAL AREA(ACRES) =    35.10   TOTAL RUNOFF(CFS) =    84.54
 TC(MIN) =  14.21

****************************************************************************
 FLOW PROCESS FROM NODE    421.40 TO NODE    421.40 IS CODE =  81
----------------------------------------------------------------------------
 >>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
============================================================================
  100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  3.357
 *USER SPECIFIED(SUBAREA):
 SINGLE FAMILY DEVELOPMENT RUNOFF COEFFICIENT = [redacted]
 S.C.S. CURVE NUMBER (AMC II) =   0
 SUBAREA AREA(ACRES) =    0.40    SUBAREA RUNOFF(CFS) =    1.01
 TOTAL AREA(ACRES) =    35.50   TOTAL RUNOFF(CFS) =    85.55
 TC(MIN) =  14.21

****************************************************************************
 FLOW PROCESS FROM NODE    429.00 TO NODE    421.40 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.) =  14.21
 RAINFALL INTENSITY(INCH/HR) =   3.36
 TOTAL STREAM AREA(ACRES) =   35.50
 PEAK FLOW RATE(CFS) AT CONFLUENCE =     85.55
```

```
****************************************************************************
  FLOW PROCESS FROM NODE     439.00 TO NODE     440.00 IS CODE =  21
----------------------------------------------------------------------------
  >>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<
============================================================================
  *USER SPECIFIED(SUBAREA):
  RURAL DEVELOPMENT RUNOFF COEFFICIENT = [redacted]
  S.C.S. CURVE NUMBER (AMC II) =    0
  INITIAL SUBAREA FLOW-LENGTH =   180.00
  UPSTREAM ELEVATION =     577.00
  DOWNSTREAM ELEVATION =     574.00
  ELEVATION DIFFERENCE =       3.00
  URBAN SUBAREA OVERLAND TIME OF FLOW(MINUTES) =   11.203
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  3.915
  SUBAREA RUNOFF(CFS) =        1.29
  TOTAL AREA(ACRES) =        0.60   TOTAL RUNOFF(CFS) =        1.29

****************************************************************************
  FLOW PROCESS FROM NODE     440.00 TO NODE     440.00 IS CODE =  51
----------------------------------------------------------------------------
  >>>>>COMPUTE TRAPEZOIDAL CHANNEL FLOW<<<<<
  >>>>>TRAVELTIME THRU SUBAREA (EXISTING ELEMENT)<<<<<
============================================================================
  ELEVATION DATA: UPSTREAM(FEET) =    574.00  DOWNSTREAM(FEET) =    561.00
  CHANNEL LENGTH THRU SUBAREA(FEET) =    670.00   CHANNEL SLOPE =  0.0194
  CHANNEL BASE(FEET) =     5.00   "Z" FACTOR =   1.500
  MANNING'S FACTOR = 0.015   MAXIMUM DEPTH(FEET) =   2.00
  CHANNEL FLOW THRU SUBAREA(CFS) =        1.29
  FLOW VELOCITY(FEET/SEC) =    2.66   FLOW DEPTH(FEET) =    0.09
  TRAVEL TIME(MIN.) =    4.20   Tc(MIN.) =  15.40
  LONGEST FLOWPATH FROM NODE     439.00 TO NODE     440.00 =    850.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE     440.00 TO NODE     441.00 IS CODE =  81
----------------------------------------------------------------------------
  >>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
============================================================================
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  3.188
  *USER SPECIFIED(SUBAREA):
  RURAL DEVELOPMENT RUNOFF COEFFICIENT = [redacted]
  S.C.S. CURVE NUMBER (AMC II) =    0
  SUBAREA AREA(ACRES) =      7.40   SUBAREA RUNOFF(CFS) =    12.98
  TOTAL AREA(ACRES) =      8.00   TOTAL RUNOFF(CFS) =    14.27
  TC(MIN) =  15.40

****************************************************************************
  FLOW PROCESS FROM NODE     441.00 TO NODE     421.40 IS CODE =  31
----------------------------------------------------------------------------
  >>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
  >>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<
============================================================================
  ELEVATION DATA: UPSTREAM(FEET) =    551.00  DOWNSTREAM(FEET) =    525.00
  FLOW LENGTH(FEET) =    750.00   MANNING'S N =  0.013
  DEPTH OF FLOW IN  18.0 INCH PIPE IS  11.8 INCHES
  PIPE-FLOW VELOCITY(FEET/SEC.) =  11.59
  ESTIMATED PIPE DIAMETER(INCH) =  18.00    NUMBER OF PIPES =   1
```

```
PIPE-FLOW(CFS) =      14.27
PIPE TRAVEL TIME(MIN.) =    1.08    Tc(MIN.) =  16.48
LONGEST FLOWPATH FROM NODE     439.00 TO NODE     421.40 =   1600.00 FEET.

****************************************************************************
FLOW PROCESS FROM NODE     421.40 TO NODE     421.40 IS CODE =  81
----------------------------------------------------------------------------
>>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
============================================================================
 100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  3.052
*USER SPECIFIED(SUBAREA):
INDUSTRIAL DEVELOPMENT RUNOFF COEFFICIENT = [redacted]
S.C.S. CURVE NUMBER (AMC II) =   0
SUBAREA AREA(ACRES) =     1.70   SUBAREA RUNOFF(CFS) =     4.67
TOTAL AREA(ACRES) =     9.70   TOTAL RUNOFF(CFS) =   18.94
TC(MIN) =  16.48

****************************************************************************
FLOW PROCESS FROM NODE     421.40 TO NODE     421.40 IS CODE =  81
----------------------------------------------------------------------------
>>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
============================================================================
 100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  3.052
*USER SPECIFIED(SUBAREA):
COMMERCIAL DEVELOPMENT RUNOFF COEFFICIENT = [redacted]
S.C.S. CURVE NUMBER (AMC II) =   0
SUBAREA AREA(ACRES) =    11.30   SUBAREA RUNOFF(CFS) =    29.32
TOTAL AREA(ACRES) =    21.00   TOTAL RUNOFF(CFS) =   48.26
TC(MIN) =  16.48

****************************************************************************
FLOW PROCESS FROM NODE     421.40 TO NODE     421.40 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.) =  16.48
RAINFALL INTENSITY(INCH/HR) =   3.05
TOTAL STREAM AREA(ACRES) =   21.00
PEAK FLOW RATE(CFS) AT CONFLUENCE =     48.26

** CONFLUENCE DATA **
STREAM     RUNOFF      Tc      INTENSITY      AREA
NUMBER      (CFS)    (MIN.)   (INCH/HOUR)    (ACRE)
   1        85.55    14.21       3.357        35.50
   2        48.26    16.48       3.052        21.00

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       126.03    16.48      3.052
   2       129.42    56.50      1.379
```

```
COMPUTED CONFLUENCE ESTIMATES ARE AS FOLLOWS:
PEAK FLOW RATE(CFS) =     129.42   Tc(MIN.) =    14.21
TOTAL AREA(ACRES) =     56.50
LONGEST FLOWPATH FROM NODE     430.00 TO NODE     421.40 =  1977.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    421.40 TO NODE    421.40 IS CODE =  11
----------------------------------------------------------------------------
  >>>>>CONFLUENCE MEMORY BANK # 1 WITH THE MAIN-STREAM MEMORY<<<<<
============================================================================

  ** MAIN STREAM CONFLUENCE DATA **
  STREAM     RUNOFF      Tc       INTENSITY      AREA
  NUMBER      (CFS)    (MIN.)    (INCH/HOUR)    (ACRE)
     1       129.42    14.21        3.357        56.50
  LONGEST FLOWPATH FROM NODE     430.00 TO NODE     421.40 =  1977.00 FEET.

  ** MEMORY BANK #  1 CONFLUENCE DATA **
  STREAM     RUNOFF      Tc       INTENSITY      AREA
  NUMBER      (CFS)    (MIN.)    (INCH/HOUR)    (ACRE)
     1       487.94    15.19        3.217       178.60
  LONGEST FLOWPATH FROM NODE     421.10 TO NODE     421.40 =  1768.00 FEET.

  ** PEAK FLOW RATE TABLE **
  STREAM    RUNOFF        Tc        INTENSITY
  NUMBER     (CFS)      (MIN.)    (INCH/HOUR)
     1      596.90      14.21        3.357
     2      611.93      15.19        3.217

  COMPUTED CONFLUENCE ESTIMATES ARE AS FOLLOWS:
  PEAK FLOW RATE(CFS) =     611.93   Tc(MIN.) =    15.19
  TOTAL AREA(ACRES) =    235.10

****************************************************************************
  FLOW PROCESS FROM NODE    421.40 TO NODE    421.40 IS CODE =  12
----------------------------------------------------------------------------
  >>>>>CLEAR MEMORY BANK # 1 <<<<<
============================================================================

****************************************************************************
  FLOW PROCESS FROM NODE    421.40 TO NODE    442.00 IS CODE =  51
----------------------------------------------------------------------------
  >>>>>COMPUTE TRAPEZOIDAL CHANNEL FLOW<<<<<
  >>>>>TRAVELTIME THRU SUBAREA (EXISTING ELEMENT)<<<<<
============================================================================
  ELEVATION DATA: UPSTREAM(FEET) =     525.00  DOWNSTREAM(FEET) =     516.00
  CHANNEL LENGTH THRU SUBAREA(FEET) =     460.00   CHANNEL SLOPE =  0.0196
  CHANNEL BASE(FEET) =     5.00   "Z" FACTOR =    3.000
  MANNING'S FACTOR = 0.015   MAXIMUM DEPTH(FEET) =    4.00
  CHANNEL FLOW THRU SUBAREA(CFS) =     611.93
  FLOW VELOCITY(FEET/SEC) =  18.51   FLOW DEPTH(FEET) =    2.59
  TRAVEL TIME(MIN.) =    0.41   Tc(MIN.) =   15.60
  LONGEST FLOWPATH FROM NODE     430.00 TO NODE     442.00 =  2437.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    421.40 TO NODE    442.00 IS CODE =  81
```

```
-------------------------------------------------------------------------------
 >>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
===============================================================================
  100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  3.161
 *USER SPECIFIED(SUBAREA):
 RURAL DEVELOPMENT RUNOFF COEFFICIENT = [redacted]
 S.C.S. CURVE NUMBER (AMC II) =   0
 SUBAREA AREA(ACRES) =     2.20   SUBAREA RUNOFF(CFS) =    3.83
 TOTAL AREA(ACRES) =   237.30   TOTAL RUNOFF(CFS) =  615.75
 TC(MIN) =  15.60

*******************************************************************************
 FLOW PROCESS FROM NODE    421.40 TO NODE    442.00 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.) =  15.60
 RAINFALL INTENSITY(INCH/HR) =   3.16
 TOTAL STREAM AREA(ACRES) =  237.30
 PEAK FLOW RATE(CFS) AT CONFLUENCE =    615.75

*******************************************************************************
 FLOW PROCESS FROM NODE    443.00 TO NODE    444.00 IS CODE =  21
-------------------------------------------------------------------------------
 >>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<
===============================================================================
 *USER SPECIFIED(SUBAREA):
 SINGLE FAMILY DEVELOPMENT RUNOFF COEFFICIENT = [redacted]
 S.C.S. CURVE NUMBER (AMC II) =   0
 INITIAL SUBAREA FLOW-LENGTH =   100.00
 UPSTREAM ELEVATION =     558.50
 DOWNSTREAM ELEVATION =     557.50
 ELEVATION DIFFERENCE =       1.00
 URBAN SUBAREA OVERLAND TIME OF FLOW(MINUTES) =    8.100
  100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  4.825
 SUBAREA RUNOFF(CFS) =       0.31
 TOTAL AREA(ACRES) =       0.10   TOTAL RUNOFF(CFS) =       0.31

*******************************************************************************
 FLOW PROCESS FROM NODE    444.00 TO NODE    446.00 IS CODE =  61
-------------------------------------------------------------------------------
 >>>>>COMPUTE STREET FLOW TRAVEL TIME THRU SUBAREA<<<<<
 >>>>>(STANDARD CURB SECTION USED)<<<<<
===============================================================================
 UPSTREAM ELEVATION(FEET) =  557.00  DOWNSTREAM ELEVATION(FEET) =  551.50
 STREET LENGTH(FEET) =    551.00   CURB HEIGHT(INCHES) =  6.0
 STREET HALFWIDTH(FEET) = 30.00

 DISTANCE FROM CROWN TO CROSSFALL GRADEBREAK(FEET) =    1.00
 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) =       1.70
  STREETFLOW MODEL RESULTS USING ESTIMATED FLOW:
  STREET FLOW DEPTH(FEET) =  0.31
  HALFSTREET FLOOD WIDTH(FEET) =    9.15
  AVERAGE FLOW VELOCITY(FEET/SEC.) =    1.78
  PRODUCT OF DEPTH&VELOCITY(FT*FT/SEC.) =    0.55
STREET FLOW TRAVEL TIME(MIN.) =    5.16   Tc(MIN.) =  13.26
 100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  3.512
*USER SPECIFIED(SUBAREA):
SINGLE FAMILY DEVELOPMENT RUNOFF COEFFICIENT = [redacted]
S.C.S. CURVE NUMBER (AMC II) =   0
SUBAREA AREA(ACRES) =     1.20     SUBAREA RUNOFF(CFS) =    2.74
TOTAL AREA(ACRES) =     1.30       PEAK FLOW RATE(CFS) =       3.05

END OF SUBAREA STREET FLOW HYDRAULICS:
DEPTH(FEET) = 0.36   HALFSTREET FLOOD WIDTH(FEET) =  11.73
FLOW VELOCITY(FEET/SEC.) =  2.04   DEPTH*VELOCITY(FT*FT/SEC.) =   0.74
LONGEST FLOWPATH FROM NODE    443.00 TO NODE    446.00 =   651.00 FEET.

****************************************************************************
 FLOW PROCESS FROM NODE    446.00 TO NODE    446.00 IS CODE =  81
----------------------------------------------------------------------------
 >>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
============================================================================
  100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  3.512
 *USER SPECIFIED(SUBAREA):
 SINGLE FAMILY DEVELOPMENT RUNOFF COEFFICIENT = [redacted]
 S.C.S. CURVE NUMBER (AMC II) =  0
 SUBAREA AREA(ACRES) =     0.40   SUBAREA RUNOFF(CFS) =    0.91
 TOTAL AREA(ACRES) =      1.70   TOTAL RUNOFF(CFS) =    3.97
 TC(MIN) =  13.26

****************************************************************************
 FLOW PROCESS FROM NODE    446.00 TO NODE    447.00 IS CODE =  61
----------------------------------------------------------------------------
 >>>>>COMPUTE STREET FLOW TRAVEL TIME THRU SUBAREA<<<<<
 >>>>>(STANDARD CURB SECTION USED)<<<<<
============================================================================
 UPSTREAM ELEVATION(FEET) =  551.50  DOWNSTREAM ELEVATION(FEET) =  546.00
 STREET LENGTH(FEET) =    290.00   CURB HEIGHT(INCHES) =  6.0
 STREET HALFWIDTH(FEET) = 30.00

 DISTANCE FROM CROWN TO CROSSFALL GRADEBREAK(FEET) =   1.00
 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) =       4.81
   STREETFLOW MODEL RESULTS USING ESTIMATED FLOW:
   STREET FLOW DEPTH(FEET) =  0.37
   HALFSTREET FLOOD WIDTH(FEET) =   12.38
```

```
  AVERAGE FLOW VELOCITY(FEET/SEC.) =    2.92
  PRODUCT OF DEPTH&VELOCITY(FT*FT/SEC.) =    1.09
 STREET FLOW TRAVEL TIME(MIN.) =    1.66   Tc(MIN.) =  14.91
 100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  3.255
 *USER SPECIFIED(SUBAREA):
 SINGLE FAMILY DEVELOPMENT RUNOFF COEFFICIENT =[redacted]
 S.C.S. CURVE NUMBER (AMC II) =  0
 SUBAREA AREA(ACRES) =     0.80     SUBAREA RUNOFF(CFS) =    1.69
 TOTAL AREA(ACRES) =    2.50        PEAK FLOW RATE(CFS) =       5.66

 END OF SUBAREA STREET FLOW HYDRAULICS:
 DEPTH(FEET) = 0.39   HALFSTREET FLOOD WIDTH(FEET) =  13.24
 FLOW VELOCITY(FEET/SEC.) =  3.03   DEPTH*VELOCITY(FT*FT/SEC.) =   1.18
 LONGEST FLOWPATH FROM NODE    443.00 TO NODE    447.00 =   941.00 FEET.

****************************************************************************
 FLOW PROCESS FROM NODE    447.00 TO NODE    447.00 IS CODE =  81
----------------------------------------------------------------------------
 >>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
============================================================================
 100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  3.255
 *USER SPECIFIED(SUBAREA):
 SINGLE FAMILY DEVELOPMENT RUNOFF COEFFICIENT = [redacted]
 S.C.S. CURVE NUMBER (AMC II) =  0
 SUBAREA AREA(ACRES) =     2.00   SUBAREA RUNOFF(CFS) =    4.23
 TOTAL AREA(ACRES) =     4.50   TOTAL RUNOFF(CFS) =    9.89
 TC(MIN) =  14.91

****************************************************************************
 FLOW PROCESS FROM NODE    447.00 TO NODE    447.00 IS CODE =  81
----------------------------------------------------------------------------
 >>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
============================================================================
 100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  3.255
 *USER SPECIFIED(SUBAREA):
 SINGLE FAMILY DEVELOPMENT RUNOFF COEFFICIENT =[redacted]
 S.C.S. CURVE NUMBER (AMC II) =  0
 SUBAREA AREA(ACRES) =     1.70   SUBAREA RUNOFF(CFS) =    4.15
 TOTAL AREA(ACRES) =     6.20   TOTAL RUNOFF(CFS) =   14.04
 TC(MIN) =  14.91

****************************************************************************
 FLOW PROCESS FROM NODE    447.00 TO NODE    448.00 IS CODE =  31
----------------------------------------------------------------------------
 >>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
 >>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<
============================================================================
 ELEVATION DATA: UPSTREAM(FEET) =    536.00  DOWNSTREAM(FEET) =    528.00
 FLOW LENGTH(FEET) =    333.00   MANNING'S N =  0.013
 DEPTH OF FLOW IN  18.0 INCH PIPE IS  13.5 INCHES
 PIPE-FLOW VELOCITY(FEET/SEC.) =   9.91
 ESTIMATED PIPE DIAMETER(INCH) =  18.00    NUMBER OF PIPES =   1
 PIPE-FLOW(CFS) =      14.04
 PIPE TRAVEL TIME(MIN.) =   0.56    Tc(MIN.) =  15.47
 LONGEST FLOWPATH FROM NODE    443.00 TO NODE    448.00 =  1274.00 FEET.

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

```
 FLOW PROCESS FROM NODE    448.00 TO NODE    448.00 IS CODE =  81
----------------------------------------------------------------------------
 >>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
============================================================================
  100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  3.179
 *USER SPECIFIED(SUBAREA):
 SINGLE FAMILY DEVELOPMENT RUNOFF COEFFICIENT = [redacted]
 S.C.S. CURVE NUMBER (AMC II) =  0
 SUBAREA AREA(ACRES) =     1.80   SUBAREA RUNOFF(CFS) =     4.29
 TOTAL AREA(ACRES) =     8.00   TOTAL RUNOFF(CFS) =    18.33
 TC(MIN) =  15.47

****************************************************************************
 FLOW PROCESS FROM NODE    448.00 TO NODE    448.00 IS CODE =  81
----------------------------------------------------------------------------
 >>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
============================================================================
  100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  3.179
 *USER SPECIFIED(SUBAREA):
 SINGLE FAMILY DEVELOPMENT RUNOFF COEFFICIENT [redacted]
 S.C.S. CURVE NUMBER (AMC II) =  0
 SUBAREA AREA(ACRES) =     2.90   SUBAREA RUNOFF(CFS) =     6.91
 TOTAL AREA(ACRES) =    10.90   TOTAL RUNOFF(CFS) =    25.24
 TC(MIN) =  15.47

****************************************************************************
 FLOW PROCESS FROM NODE    448.00 TO NODE    449.00 IS CODE =  31
----------------------------------------------------------------------------
 >>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
 >>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<
============================================================================
 ELEVATION DATA: UPSTREAM(FEET) =   528.00  DOWNSTREAM(FEET) =   522.00
 FLOW LENGTH(FEET) =   630.00   MANNING'S N =  0.013
 DEPTH OF FLOW IN  27.0 INCH PIPE IS  19.6 INCHES
 PIPE-FLOW VELOCITY(FEET/SEC.) =   8.15
 ESTIMATED PIPE DIAMETER(INCH) =  27.00    NUMBER OF PIPES =   1
 PIPE-FLOW(CFS) =     25.24
 PIPE TRAVEL TIME(MIN.) =   1.29    Tc(MIN.) =  16.76
 LONGEST FLOWPATH FROM NODE    443.00 TO NODE    449.00 =  1904.00 FEET.

****************************************************************************
 FLOW PROCESS FROM NODE    449.00 TO NODE    449.00 IS CODE =  81
----------------------------------------------------------------------------
 >>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
============================================================================
  100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  3.019
 *USER SPECIFIED(SUBAREA):
 SINGLE FAMILY DEVELOPMENT RUNOFF COEFFICIENT = [redacted]
 S.C.S. CURVE NUMBER (AMC II) =  0
 SUBAREA AREA(ACRES) =     2.10   SUBAREA RUNOFF(CFS) =     4.75
 TOTAL AREA(ACRES) =    13.00   TOTAL RUNOFF(CFS) =    30.00
 TC(MIN) =  16.76

****************************************************************************
 FLOW PROCESS FROM NODE    449.00 TO NODE    449.00 IS CODE =  81
----------------------------------------------------------------------------
 >>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
```

```
================================================================================
  100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  3.019
 *USER SPECIFIED(SUBAREA):
 SINGLE FAMILY DEVELOPMENT RUNOFF COEFFICIENT = [redacted]
 S.C.S. CURVE NUMBER (AMC II) =  0
 SUBAREA AREA(ACRES) =     0.90   SUBAREA RUNOFF(CFS) =    2.04
 TOTAL AREA(ACRES) =     13.90   TOTAL RUNOFF(CFS) =    32.04
 TC(MIN) =  16.76

********************************************************************************
 FLOW PROCESS FROM NODE    449.00 TO NODE    449.00 IS CODE =  81
--------------------------------------------------------------------------------
 >>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
================================================================================
  100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  3.019
 *USER SPECIFIED(SUBAREA):
 RURAL DEVELOPMENT RUNOFF COEFFICIENT = [redacted]
 S.C.S. CURVE NUMBER (AMC II) =  0
 SUBAREA AREA(ACRES) =     1.10   SUBAREA RUNOFF(CFS) =    1.83
 TOTAL AREA(ACRES) =     15.00   TOTAL RUNOFF(CFS) =    33.86
 TC(MIN) =  16.76

********************************************************************************
 FLOW PROCESS FROM NODE    449.00 TO NODE    442.00 IS CODE =  31
--------------------------------------------------------------------------------
 >>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
 >>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<
================================================================================
 ELEVATION DATA: UPSTREAM(FEET) =   522.00  DOWNSTREAM(FEET) =   516.00
 FLOW LENGTH(FEET) =   240.00   MANNING'S N =  0.013
 DEPTH OF FLOW IN  24.0 INCH PIPE IS  19.6 INCHES
 PIPE-FLOW VELOCITY(FEET/SEC.) =  12.33
 ESTIMATED PIPE DIAMETER(INCH) =  24.00    NUMBER OF PIPES =   1
 PIPE-FLOW(CFS) =     33.86
 PIPE TRAVEL TIME(MIN.) =   0.32    Tc(MIN.) =  17.09
 LONGEST FLOWPATH FROM NODE    443.00 TO NODE    442.00 =  2144.00 FEET.

********************************************************************************
 FLOW PROCESS FROM NODE    449.00 TO NODE    442.00 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.) =  17.09
 RAINFALL INTENSITY(INCH/HR) =   2.98
 TOTAL STREAM AREA(ACRES) =   15.00
 PEAK FLOW RATE(CFS) AT CONFLUENCE =     33.86

 ** CONFLUENCE DATA **
 STREAM    RUNOFF      Tc      INTENSITY      AREA
 NUMBER     (CFS)    (MIN.)   (INCH/HOUR)    (ACRE)
    1     615.75    15.60       3.161       237.30
    2      33.86    17.09       2.982        15.00

 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       647.69     15.60        3.161
    2       614.61     17.09        2.982

COMPUTED CONFLUENCE ESTIMATES ARE AS FOLLOWS:
PEAK FLOW RATE(CFS) =     647.69   Tc(MIN.) =    15.60
TOTAL AREA(ACRES) =     252.30
LONGEST FLOWPATH FROM NODE      430.00 TO NODE      442.00 =   2437.00 FEET.

****************************************************************************
FLOW PROCESS FROM NODE     442.00 TO NODE     450.00 IS CODE =  51
----------------------------------------------------------------------------
>>>>>COMPUTE TRAPEZOIDAL CHANNEL FLOW<<<<<
>>>>>TRAVELTIME THRU SUBAREA (EXISTING ELEMENT)<<<<<
============================================================================
ELEVATION DATA: UPSTREAM(FEET) =     516.00  DOWNSTREAM(FEET) =     490.00
CHANNEL LENGTH THRU SUBAREA(FEET) =  1000.00   CHANNEL SLOPE =  0.0260
CHANNEL BASE(FEET) =    10.00    "Z" FACTOR =    3.000
MANNING'S FACTOR = 0.015   MAXIMUM DEPTH(FEET) =    4.00
CHANNEL FLOW THRU SUBAREA(CFS) =     647.69
FLOW VELOCITY(FEET/SEC) =  20.11   FLOW DEPTH(FEET) =    2.01
TRAVEL TIME(MIN.) =    0.83    Tc(MIN.) =  16.43
LONGEST FLOWPATH FROM NODE     430.00 TO NODE     450.00 =  3437.00 FEET.

****************************************************************************
FLOW PROCESS FROM NODE     442.00 TO NODE     450.00 IS CODE =  81
----------------------------------------------------------------------------
>>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
============================================================================
 100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  3.058
*USER SPECIFIED(SUBAREA):
RURAL DEVELOPMENT RUNOFF COEFFICIENT = [redacted]
S.C.S. CURVE NUMBER (AMC II) =    0
SUBAREA AREA(ACRES) =      5.60   SUBAREA RUNOFF(CFS) =      9.42
TOTAL AREA(ACRES) =    257.90   TOTAL RUNOFF(CFS) =   657.11
TC(MIN) =  16.43

****************************************************************************
FLOW PROCESS FROM NODE     442.00 TO NODE     450.00 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.) =  16.43
RAINFALL INTENSITY(INCH/HR) =   3.06
TOTAL STREAM AREA(ACRES) =  257.90
PEAK FLOW RATE(CFS) AT CONFLUENCE =     657.11

****************************************************************************
FLOW PROCESS FROM NODE     451.00 TO NODE     452.00 IS CODE =  21
----------------------------------------------------------------------------
>>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<
```

```
==============================================================================
 *USER SPECIFIED(SUBAREA):
 INDUSTRIAL DEVELOPMENT RUNOFF COEFFICIENT = [redacted]
 S.C.S. CURVE NUMBER (AMC II) =    0
 INITIAL SUBAREA FLOW-LENGTH =   240.00
 UPSTREAM ELEVATION =     561.00
 DOWNSTREAM ELEVATION =     556.00
 ELEVATION DIFFERENCE =       5.00
 URBAN SUBAREA OVERLAND TIME OF FLOW(MINUTES) =    4.367
 TIME OF CONCENTRATION ASSUMED AS 6-MINUTES
  100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  5.856
 SUBAREA RUNOFF(CFS) =       2.64
 TOTAL AREA(ACRES) =       0.50  TOTAL RUNOFF(CFS) =       2.64

******************************************************************************
 FLOW PROCESS FROM NODE    452.00 TO NODE    453.00 IS CODE =  61
------------------------------------------------------------------------------
 >>>>>COMPUTE STREET FLOW TRAVEL TIME THRU SUBAREA<<<<<
 >>>>>(STANDARD CURB SECTION USED)<<<<<
==============================================================================
 UPSTREAM ELEVATION(FEET) =  556.00  DOWNSTREAM ELEVATION(FEET) =  438.00
 STREET LENGTH(FEET) =    950.00   CURB HEIGHT(INCHES) =  6.0
 STREET HALFWIDTH(FEET) = 30.00

 DISTANCE FROM CROWN TO CROSSFALL GRADEBREAK(FEET) =   1.00
 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) =       6.89
   STREETFLOW MODEL RESULTS USING ESTIMATED FLOW:
   STREET FLOW DEPTH(FEET) =  0.32
   HALFSTREET FLOOD WIDTH(FEET) =    9.69
   AVERAGE FLOW VELOCITY(FEET/SEC.) =    6.52
   PRODUCT OF DEPTH&VELOCITY(FT*FT/SEC.) =    2.09
 STREET FLOW TRAVEL TIME(MIN.) =   2.43   Tc(MIN.) =   8.43
  100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  4.703
 *USER SPECIFIED(SUBAREA):
 INDUSTRIAL DEVELOPMENT RUNOFF COEFFICIENT = [redacted]
 S.C.S. CURVE NUMBER (AMC II) =   0
 SUBAREA AREA(ACRES) =     2.00     SUBAREA RUNOFF(CFS) =     8.47
 TOTAL AREA(ACRES) =     2.50       PEAK FLOW RATE(CFS) =      11.10

 END OF SUBAREA STREET FLOW HYDRAULICS:
 DEPTH(FEET) = 0.36   HALFSTREET FLOOD WIDTH(FEET) =  11.89
 FLOW VELOCITY(FEET/SEC.) =  7.24   DEPTH*VELOCITY(FT*FT/SEC.) =   2.64
 LONGEST FLOWPATH FROM NODE    451.00 TO NODE    453.00 =  1190.00 FEET.

******************************************************************************
 FLOW PROCESS FROM NODE    453.00 TO NODE    453.00 IS CODE =  81
------------------------------------------------------------------------------
 >>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
==============================================================================
```

```
 100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  4.703
 *USER SPECIFIED(SUBAREA):
 COMMERCIAL DEVELOPMENT RUNOFF COEFFICIENT = [redacted]
 S.C.S. CURVE NUMBER (AMC II) =  0
 SUBAREA AREA(ACRES) =     1.00   SUBAREA RUNOFF(CFS) =     4.00
 TOTAL AREA(ACRES) =     3.50   TOTAL RUNOFF(CFS) =   15.10
 TC(MIN) =    8.43

****************************************************************************
 FLOW PROCESS FROM NODE    453.00 TO NODE    454.00 IS CODE =  31
----------------------------------------------------------------------------
 >>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
 >>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<
============================================================================
 ELEVATION DATA: UPSTREAM(FEET) =   528.00  DOWNSTREAM(FEET) =   505.00
 FLOW LENGTH(FEET) =   616.00   MANNING'S N =  0.013
 DEPTH OF FLOW IN  18.0 INCH PIPE IS  12.0 INCHES
 PIPE-FLOW VELOCITY(FEET/SEC.) =  12.08
 ESTIMATED PIPE DIAMETER(INCH) =  18.00    NUMBER OF PIPES =   1
 PIPE-FLOW(CFS) =     15.10
 PIPE TRAVEL TIME(MIN.) =   0.85    Tc(MIN.) =   9.28
 LONGEST FLOWPATH FROM NODE    451.00 TO NODE    454.00 =  1806.00 FEET.

****************************************************************************
 FLOW PROCESS FROM NODE    454.00 TO NODE    454.00 IS CODE =  81
----------------------------------------------------------------------------
 >>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
============================================================================
 100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  4.421
 *USER SPECIFIED(SUBAREA):
 INDUSTRIAL DEVELOPMENT RUNOFF COEFFICIENT = [redacted]
 S.C.S. CURVE NUMBER (AMC II) =  0
 SUBAREA AREA(ACRES) =     1.60   SUBAREA RUNOFF(CFS) =    6.37
 TOTAL AREA(ACRES) =     5.10   TOTAL RUNOFF(CFS) =   21.46
 TC(MIN) =    9.28

****************************************************************************
 FLOW PROCESS FROM NODE    454.00 TO NODE    454.00 IS CODE =  81
----------------------------------------------------------------------------
 >>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
============================================================================
 100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  4.421
 *USER SPECIFIED(SUBAREA):
 SINGLE FAMILY DEVELOPMENT RUNOFF COEFFICIENT = [redacted]
 S.C.S. CURVE NUMBER (AMC II) =  0
 SUBAREA AREA(ACRES) =    15.90   SUBAREA RUNOFF(CFS) =   52.72
 TOTAL AREA(ACRES) =    21.00   TOTAL RUNOFF(CFS) =   74.18
 TC(MIN) =    9.28

****************************************************************************
 FLOW PROCESS FROM NODE    454.00 TO NODE    450.00 IS CODE =  31
----------------------------------------------------------------------------
 >>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
 >>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<
============================================================================
 ELEVATION DATA: UPSTREAM(FEET) =   505.00  DOWNSTREAM(FEET) =   490.00
 FLOW LENGTH(FEET) =   235.00   MANNING'S N =  0.013
```

```
 DEPTH OF FLOW IN  27.0 INCH PIPE IS  22.1 INCHES
 PIPE-FLOW VELOCITY(FEET/SEC.) =  21.31
 ESTIMATED PIPE DIAMETER(INCH) =  27.00    NUMBER OF PIPES =   1
 PIPE-FLOW(CFS) =      74.18
 PIPE TRAVEL TIME(MIN.) =    0.18    Tc(MIN.) =   9.46
 LONGEST FLOWPATH FROM NODE    451.00 TO NODE    450.00 =  2041.00 FEET.

****************************************************************************
 FLOW PROCESS FROM NODE    450.00 TO NODE    450.00 IS CODE =  81
----------------------------------------------------------------------------
 >>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
============================================================================
  100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  4.365
 *USER SPECIFIED(SUBAREA):
 INDUSTRIAL DEVELOPMENT RUNOFF COEFFICIENT =[redacted]
 S.C.S. CURVE NUMBER (AMC II) =  0
 SUBAREA AREA(ACRES) =     0.40   SUBAREA RUNOFF(CFS) =     1.57
 TOTAL AREA(ACRES) =     21.40   TOTAL RUNOFF(CFS) =    75.75
 TC(MIN) =    9.46

****************************************************************************
 FLOW PROCESS FROM NODE    450.00 TO NODE    450.00 IS CODE =  81
----------------------------------------------------------------------------
 >>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
============================================================================
  100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  4.365
 *USER SPECIFIED(SUBAREA):
 INDUSTRIAL DEVELOPMENT RUNOFF COEFFICIENT = [redacted]
 S.C.S. CURVE NUMBER (AMC II) =  0
 SUBAREA AREA(ACRES) =     2.30   SUBAREA RUNOFF(CFS) =     9.04
 TOTAL AREA(ACRES) =     23.70   TOTAL RUNOFF(CFS) =    84.79
 TC(MIN) =    9.46

****************************************************************************
 FLOW PROCESS FROM NODE    454.00 TO NODE    450.00 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.) =    9.46
 RAINFALL INTENSITY(INCH/HR) =    4.36
 TOTAL STREAM AREA(ACRES) =    23.70
 PEAK FLOW RATE(CFS) AT CONFLUENCE =      84.79

 ** CONFLUENCE DATA **
 STREAM      RUNOFF       Tc       INTENSITY       AREA
 NUMBER       (CFS)     (MIN.)    (INCH/HOUR)     (ACRE)
     1       657.11     16.43        3.058        257.90
     2        84.79      9.46        4.365         23.70

 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	545.07	9.46	4.365
2	716.50	281.60	0.489

COMPUTED CONFLUENCE ESTIMATES ARE AS FOLLOWS:
PEAK FLOW RATE(CFS) = 716.50 Tc(MIN.) = 16.43
TOTAL AREA(ACRES) = 281.60
LONGEST FLOWPATH FROM NODE 430.00 TO NODE 450.00 = 3437.00 FEET.

==

END OF STUDY SUMMARY:
TOTAL AREA(ACRES) = 281.60 TC(MIN.) = 16.43
PEAK FLOW RATE(CFS) = 716.50

==

==

END OF RATIONAL METHOD ANALYSIS

1

[redacted] (~~472.8~~) + [redacted] (0.40 + 1.40 + 1.40 + 2.60 + 0.30 + 0.40 +
0.40 + 0.40 + 0.50 + 1.70 + 0.50 + 2.00 + 1.60 + 0.40 + 2.30)
+ 1.20 + 1.70 + 3.40 + 1.3 +
+ [redacted] (13.50 + 2.0 + 1.20 + 1.70 + 3.40 + 1.3 + 2.7 + 0.6 + 1.40 + 0.60 + 4.50) + 0[redacted] (1.20 +
1.20 + 0.40 + 1.70 + 1.80 + 2.90 + 2.10 + 0.90 + 15.90) +
[redacted] (1.0 + 0.60 + 7.40 + 2.20 + 1.10 + 5.60)
+ [redacted] (11.30 + 1.0)

281.6

R/O = 0.80

```
****************************************************************************
          RATIONAL METHOD HYDROLOGY COMPUTER PROGRAM PACKAGE
          Reference: SAN DIEGO COUNTY FLOOD CONTROL DISTRICT
                       1985,1981 HYDROLOGY MANUAL
       (c) Copyright 1982-2000 Advanced Engineering Software (aes)
           Ver. 1.5A Release Date: 01/01/2000  License ID 1261

                         Analysis prepared by:

                        Rick Engineering Company
                           5620 Friars Road
                          San Diego, CA  92110
                            (619) 291-0707

 ************************** DESCRIPTION OF STUDY **************************
 * Otay Ranch Village 7   J-14483                                          *
 * 100-Year storm Event                                                    *
 * Basin 400 (EUC) - Proposed Condition                                    *
 **************************************************************************

  FILE NAME: 1HV7B4P2.DAT
  TIME/DATE OF STUDY: 08:52 10/17/2003
----------------------------------------------------------------------------
  USER SPECIFIED HYDROLOGY AND HYDRAULIC MODEL INFORMATION:
----------------------------------------------------------------------------
  1985 SAN DIEGO MANUAL CRITERIA

  USER SPECIFIED STORM EVENT(YEAR) = 100.00
  6-HOUR DURATION PRECIPITATION (INCHES) =   2.500
  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: ONLY PEAK CONFLUENCE VALUES CONSIDERED
  *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

  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    400.00 TO NODE    402.00 IS CODE =  21
----------------------------------------------------------------------------
  >>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<
============================================================================
  COMMERCIAL DEVELOPMENT RUNOFF COEFFICIENT = .8500
  SOIL CLASSIFICATION IS "D"
  S.C.S. CURVE NUMBER (AMC II) =  92
```

```
 INITIAL SUBAREA FLOW-LENGTH =   500.00
 UPSTREAM ELEVATION =     650.00
 DOWNSTREAM ELEVATION =     645.00
 ELEVATION DIFFERENCE =       5.00
 URBAN SUBAREA OVERLAND TIME OF FLOW(MINUTES) =   10.062
  100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  4.195
 SUBAREA RUNOFF(CFS) =     14.98
 TOTAL AREA(ACRES) =      4.20   TOTAL RUNOFF(CFS) =     14.98

****************************************************************************
 FLOW PROCESS FROM NODE    402.00 TO NODE    404.00 IS CODE =  31
----------------------------------------------------------------------------
 >>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
 >>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<
============================================================================
 ELEVATION DATA: UPSTREAM(FEET) =   635.00  DOWNSTREAM(FEET) =   580.00
 FLOW LENGTH(FEET) =  2100.00   MANNING'S N =  0.013
 DEPTH OF FLOW IN  18.0 INCH PIPE IS  13.7 INCHES
 PIPE-FLOW VELOCITY(FEET/SEC.) =  10.36
 ESTIMATED PIPE DIAMETER(INCH) =  18.00    NUMBER OF PIPES =   1
 PIPE-FLOW(CFS) =     14.98
 PIPE TRAVEL TIME(MIN.) =   3.38    Tc(MIN.) =  13.44
 LONGEST FLOWPATH FROM NODE    400.00 TO NODE    404.00 =  2600.00 FEET.

****************************************************************************
 FLOW PROCESS FROM NODE    402.00 TO NODE    404.00 IS CODE =  81
----------------------------------------------------------------------------
 >>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
============================================================================
  100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  3.481
 COMMERCIAL DEVELOPMENT RUNOFF COEFFICIENT = .8500
 SOIL CLASSIFICATION IS "D"
 S.C.S. CURVE NUMBER (AMC II) =  92
 SUBAREA AREA(ACRES) =    32.20   SUBAREA RUNOFF(CFS) =    95.27
 TOTAL AREA(ACRES) =     36.40   TOTAL RUNOFF(CFS) =   110.25
 TC(MIN) =  13.44

****************************************************************************
 FLOW PROCESS FROM NODE    402.00 TO NODE    404.00 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.) =  13.44
 RAINFALL INTENSITY(INCH/HR) =   3.48
 TOTAL STREAM AREA(ACRES) =   36.40
 PEAK FLOW RATE(CFS) AT CONFLUENCE =    110.25

****************************************************************************
 FLOW PROCESS FROM NODE    406.00 TO NODE    408.00 IS CODE =  21
----------------------------------------------------------------------------
 >>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<
============================================================================
 COMMERCIAL DEVELOPMENT RUNOFF COEFFICIENT = .8500
 SOIL CLASSIFICATION IS "D"
 S.C.S. CURVE NUMBER (AMC II) =  92
```

```
INITIAL SUBAREA FLOW-LENGTH =   500.00
UPSTREAM ELEVATION =    600.00
DOWNSTREAM ELEVATION =     595.00
ELEVATION DIFFERENCE =       5.00
URBAN SUBAREA OVERLAND TIME OF FLOW(MINUTES) =   10.062
 100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  4.195
SUBAREA RUNOFF(CFS) =     20.33
TOTAL AREA(ACRES) =      5.70   TOTAL RUNOFF(CFS) =     20.33

****************************************************************************
 FLOW PROCESS FROM NODE    408.00 TO NODE    404.00 IS CODE =  31
----------------------------------------------------------------------------
 >>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
 >>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<
============================================================================
 ELEVATION DATA: UPSTREAM(FEET) =    585.00  DOWNSTREAM(FEET) =    580.00
 FLOW LENGTH(FEET) =  1060.00   MANNING'S N =  0.013
 DEPTH OF FLOW IN  30.0 INCH PIPE IS  19.6 INCHES
 PIPE-FLOW VELOCITY(FEET/SEC.) =   5.99
 ESTIMATED PIPE DIAMETER(INCH) =  30.00    NUMBER OF PIPES =   1
 PIPE-FLOW(CFS) =     20.33
 PIPE TRAVEL TIME(MIN.) =    2.95    Tc(MIN.) =  13.01
 LONGEST FLOWPATH FROM NODE    406.00 TO NODE    404.00 =  1560.00 FEET.

****************************************************************************
 FLOW PROCESS FROM NODE    408.00 TO NODE    404.00 IS CODE =  81
----------------------------------------------------------------------------
 >>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
============================================================================
  100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  3.555
 COMMERCIAL DEVELOPMENT RUNOFF COEFFICIENT = .8500
 SOIL CLASSIFICATION IS "D"
 S.C.S. CURVE NUMBER (AMC II) =  92
 SUBAREA AREA(ACRES) =   28.00   SUBAREA RUNOFF(CFS) =    84.60
 TOTAL AREA(ACRES) =    33.70   TOTAL RUNOFF(CFS) =  104.93
 TC(MIN) =  13.01

****************************************************************************
 FLOW PROCESS FROM NODE    408.00 TO NODE    404.00 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.) =  13.01
 RAINFALL INTENSITY(INCH/HR) =   3.55
 TOTAL STREAM AREA(ACRES) =   33.70
 PEAK FLOW RATE(CFS) AT CONFLUENCE =    104.93

 ** CONFLUENCE DATA **
 STREAM      RUNOFF      Tc      INTENSITY      AREA
 NUMBER       (CFS)    (MIN.)   (INCH/HOUR)    (ACRE)
     1       110.25    13.44        3.481        36.40
     2       104.93    13.01        3.555        33.70

 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       212.90     13.01        3.555
    2       213.01     13.44        3.481

COMPUTED CONFLUENCE ESTIMATES ARE AS FOLLOWS:
PEAK FLOW RATE(CFS) =     213.01   Tc(MIN.) =    13.44
TOTAL AREA(ACRES) =     70.10
LONGEST FLOWPATH FROM NODE     400.00 TO NODE     404.00 =  2600.00 FEET.

****************************************************************************
FLOW PROCESS FROM NODE    404.00 TO NODE    410.00 IS CODE =  31
----------------------------------------------------------------------------
>>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
>>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<
============================================================================
ELEVATION DATA: UPSTREAM(FEET) =    580.00  DOWNSTREAM(FEET) =    555.00
FLOW LENGTH(FEET) =  1800.00   MANNING'S N =  0.013
DEPTH OF FLOW IN  54.0 INCH PIPE IS  42.8 INCHES
PIPE-FLOW VELOCITY(FEET/SEC.) =  15.75
ESTIMATED PIPE DIAMETER(INCH) =  54.00    NUMBER OF PIPES =   1
PIPE-FLOW(CFS) =    213.01
PIPE TRAVEL TIME(MIN.) =    1.90    Tc(MIN.) =  15.34
LONGEST FLOWPATH FROM NODE     400.00 TO NODE     410.00 =  4400.00 FEET.

****************************************************************************
FLOW PROCESS FROM NODE    404.00 TO NODE    410.00 IS CODE =  10
----------------------------------------------------------------------------
>>>>>MAIN-STREAM MEMORY COPIED ONTO MEMORY BANK # 1 <<<<<
============================================================================

****************************************************************************
FLOW PROCESS FROM NODE    412.00 TO NODE    414.00 IS CODE =  21
----------------------------------------------------------------------------
>>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<
============================================================================
COMMERCIAL DEVELOPMENT RUNOFF COEFFICIENT = .8500
SOIL CLASSIFICATION IS "D"
S.C.S. CURVE NUMBER (AMC II) =  92
INITIAL SUBAREA FLOW-LENGTH =   400.00
UPSTREAM ELEVATION =    585.00
DOWNSTREAM ELEVATION =    581.00
ELEVATION DIFFERENCE =      4.00
URBAN SUBAREA OVERLAND TIME OF FLOW(MINUTES) =     9.000
 100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  4.508
SUBAREA RUNOFF(CFS) =     15.33
TOTAL AREA(ACRES) =       4.00   TOTAL RUNOFF(CFS) =      15.33

****************************************************************************
FLOW PROCESS FROM NODE    414.00 TO NODE    416.00 IS CODE =  31
----------------------------------------------------------------------------
>>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
>>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<
============================================================================
```

```
  ELEVATION DATA: UPSTREAM(FEET) =    571.00  DOWNSTREAM(FEET) =    555.00
  FLOW LENGTH(FEET) =  2900.00   MANNING'S N =  0.013
  DEPTH OF FLOW IN  24.0 INCH PIPE IS  18.9 INCHES
  PIPE-FLOW VELOCITY(FEET/SEC.) =   5.78
  ESTIMATED PIPE DIAMETER(INCH) =  24.00    NUMBER OF PIPES =   1
  PIPE-FLOW(CFS) =     15.33
  PIPE TRAVEL TIME(MIN.) =   8.36    Tc(MIN.) =  17.36
  LONGEST FLOWPATH FROM NODE    412.00 TO NODE    416.00 =  3300.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    414.00 TO NODE    416.00 IS CODE =  81
----------------------------------------------------------------------------
  >>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
============================================================================
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  2.951
  COMMERCIAL DEVELOPMENT RUNOFF COEFFICIENT = .8500
  SOIL CLASSIFICATION IS "D"
  S.C.S. CURVE NUMBER (AMC II) =  92
  SUBAREA AREA(ACRES) =   49.30   SUBAREA RUNOFF(CFS) =  123.65
  TOTAL AREA(ACRES) =    53.30   TOTAL RUNOFF(CFS) =  138.98
  TC(MIN) =  17.36

****************************************************************************
  FLOW PROCESS FROM NODE    414.00 TO NODE    416.00 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.) =  17.36
  RAINFALL INTENSITY(INCH/HR) =   2.95
  TOTAL STREAM AREA(ACRES) =    53.30
  PEAK FLOW RATE(CFS) AT CONFLUENCE =    138.98

****************************************************************************
  FLOW PROCESS FROM NODE    418.00 TO NODE    420.00 IS CODE =  21
----------------------------------------------------------------------------
  >>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<
============================================================================
  COMMERCIAL DEVELOPMENT RUNOFF COEFFICIENT = .8500
  SOIL CLASSIFICATION IS "D"
  S.C.S. CURVE NUMBER (AMC II) =  92
  INITIAL SUBAREA FLOW-LENGTH =   300.00
  UPSTREAM ELEVATION =    585.00
  DOWNSTREAM ELEVATION =    582.00
  ELEVATION DIFFERENCE =      3.00
  URBAN SUBAREA OVERLAND TIME OF FLOW(MINUTES) =    7.794
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  4.947
  SUBAREA RUNOFF(CFS) =     11.77
  TOTAL AREA(ACRES) =      2.80   TOTAL RUNOFF(CFS) =     11.77

****************************************************************************
  FLOW PROCESS FROM NODE    420.00 TO NODE    416.00 IS CODE =  31
----------------------------------------------------------------------------
  >>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
  >>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<
============================================================================
```

```
ELEVATION DATA: UPSTREAM(FEET) =   582.00  DOWNSTREAM(FEET) =   555.00
FLOW LENGTH(FEET) =  2600.00  MANNING'S N =  0.013
DEPTH OF FLOW IN  21.0 INCH PIPE IS  13.8 INCHES
PIPE-FLOW VELOCITY(FEET/SEC.) =   7.02
ESTIMATED PIPE DIAMETER(INCH) =  21.00    NUMBER OF PIPES =   1
PIPE-FLOW(CFS) =     11.77
PIPE TRAVEL TIME(MIN.) =   6.17    Tc(MIN.) =  13.96
LONGEST FLOWPATH FROM NODE     418.00 TO NODE     416.00 =  2900.00 FEET.

****************************************************************************
FLOW PROCESS FROM NODE    420.00 TO NODE    416.00 IS CODE =  81
----------------------------------------------------------------------------
>>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
============================================================================
 100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  3.396
COMMERCIAL DEVELOPMENT RUNOFF COEFFICIENT = .8500
SOIL CLASSIFICATION IS "D"
S.C.S. CURVE NUMBER (AMC II) =  92
SUBAREA AREA(ACRES) =    46.60   SUBAREA RUNOFF(CFS) =  134.52
TOTAL AREA(ACRES) =     49.40   TOTAL RUNOFF(CFS) =   146.30
TC(MIN) =  13.96

****************************************************************************
FLOW PROCESS FROM NODE    420.00 TO NODE    416.00 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.) =  13.96
RAINFALL INTENSITY(INCH/HR) =   3.40
TOTAL STREAM AREA(ACRES) =   49.40
PEAK FLOW RATE(CFS) AT CONFLUENCE =    146.30

** CONFLUENCE DATA **
STREAM     RUNOFF      Tc      INTENSITY      AREA
NUMBER      (CFS)    (MIN.)   (INCH/HOUR)    (ACRE)
    1      138.98    17.36       2.951        53.30
    2      146.30    13.96       3.396        49.40

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      267.05    13.96      3.396
    2      266.09    17.36      2.951

COMPUTED CONFLUENCE ESTIMATES ARE AS FOLLOWS:
PEAK FLOW RATE(CFS) =     267.05   Tc(MIN.) =    13.96
TOTAL AREA(ACRES) =     102.70
LONGEST FLOWPATH FROM NODE     412.00 TO NODE     416.00 =  3300.00 FEET.

****************************************************************************
FLOW PROCESS FROM NODE    416.00 TO NODE    410.00 IS CODE =  31
```

```
----------------------------------------------------------------------------
  >>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
  >>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<
============================================================================
  ELEVATION DATA: UPSTREAM(FEET) =    555.00  DOWNSTREAM(FEET) =    553.00
  FLOW LENGTH(FEET) =    200.00   MANNING'S N =  0.013
  DEPTH OF FLOW IN  63.0 INCH PIPE IS  49.0 INCHES
  PIPE-FLOW VELOCITY(FEET/SEC.) =  14.79
  ESTIMATED PIPE DIAMETER(INCH) =  63.00     NUMBER OF PIPES =   1
  PIPE-FLOW(CFS) =    267.05
  PIPE TRAVEL TIME(MIN.) =   0.23     Tc(MIN.) =   14.19
  LONGEST FLOWPATH FROM NODE     412.00 TO NODE     410.00 =   3500.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE     416.00 TO NODE     410.00 IS CODE =  11
----------------------------------------------------------------------------
  >>>>>CONFLUENCE MEMORY BANK # 1 WITH THE MAIN-STREAM MEMORY<<<<<
============================================================================

  ** MAIN STREAM CONFLUENCE DATA **
  STREAM      RUNOFF      Tc       INTENSITY     AREA
  NUMBER       (CFS)    (MIN.)    (INCH/HOUR)   (ACRE)
     1        267.05    14.19        3.361      102.70
  LONGEST FLOWPATH FROM NODE     412.00 TO NODE     410.00 =   3500.00 FEET.

  ** MEMORY BANK #  1 CONFLUENCE DATA **
  STREAM      RUNOFF      Tc       INTENSITY     AREA
  NUMBER       (CFS)    (MIN.)    (INCH/HOUR)   (ACRE)
     1        213.01    15.34        3.196       70.10
  LONGEST FLOWPATH FROM NODE     400.00 TO NODE     410.00 =   4400.00 FEET.

  ** PEAK FLOW RATE TABLE **
  STREAM     RUNOFF         Tc       INTENSITY
  NUMBER      (CFS)       (MIN.)    (INCH/HOUR)
      1      469.56       14.19         3.361
      2      466.90       15.34         3.196

  COMPUTED CONFLUENCE ESTIMATES ARE AS FOLLOWS:
  PEAK FLOW RATE(CFS) =     469.56   Tc(MIN.) =    14.19
  TOTAL AREA(ACRES) =     172.80

****************************************************************************
  FLOW PROCESS FROM NODE     416.00 TO NODE     410.00 IS CODE =  12
----------------------------------------------------------------------------
  >>>>>CLEAR MEMORY BANK # 1 <<<<<
============================================================================

****************************************************************************
  FLOW PROCESS FROM NODE     410.00 TO NODE     421.00 IS CODE =  31
----------------------------------------------------------------------------
  >>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
  >>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<
============================================================================
  ELEVATION DATA: UPSTREAM(FEET) =    555.00  DOWNSTREAM(FEET) =    550.00
  FLOW LENGTH(FEET) =    500.00   MANNING'S N =  0.013
  DEPTH OF FLOW IN  78.0 INCH PIPE IS  60.3 INCHES
  PIPE-FLOW VELOCITY(FEET/SEC.) =  17.05
```

```
 ESTIMATED PIPE DIAMETER(INCH) =  78.00    NUMBER OF PIPES =   1
 PIPE-FLOW(CFS) =     469.56
 PIPE TRAVEL TIME(MIN.) =   0.49    Tc(MIN.) =  14.68
 LONGEST FLOWPATH FROM NODE    400.00 TO NODE    421.00 =  4900.00 FEET.
============================================================================
 END OF STUDY SUMMARY:
 TOTAL AREA(ACRES)       =    172.80  TC(MIN.) =      14.68
 PEAK FLOW RATE(CFS)     =    469.56
============================================================================
============================================================================
 END OF RATIONAL METHOD ANALYSIS

1
```

Otay Ranch Village 7 Pre- and Post-Project Condition Hydrology Work Map

Prepared By:
Rick Engineering Company – Water Resources Division

DCB:JW:nd/Report/14483.005
9-19-03
Revised: 10-20-03
Revised: 02-27-04
Revised: 03-26-04
Revised: 05-24-04

OVERSIZED EXHIBIT
"OTAY RANCH VILLAGE 7 PRE- AND POST-PROJECT CONDITION HYDROLOGY WORK MAP"

This exhibit is on file at the City of Chula Vista, Planning Department located at 276 Fourth Avenue, Chula Vista, CA 91910

Inlet Filter Inserts

Location of Post-Construction BMP*	Street (acres)	Lots (acres)	Slopes/Open Area (acres)	Composite Runoff Coefficient**	Total Area (acres)
1	0.5		0.43	0.7	0.93
2	0.7		0.4	0.8	1.06
3	0.3			1.0	0.27
4	0.6	0.7	0.1	0.8	1.39
5	0.4	0.5		0.8	0.9
6	0.5	1.8		0.7	2.32
7	0.5	1.4		0.7	1.95
8	0.2	0.8		0.7	0.97
9	0.2	0.8		0.7	0.98
10	0.3	0.9		0.7	1.25
11	0.3	1.0		0.7	1.33
12	0.3	0.8		0.7	1
13	0.5	1.2		0.7	1.69
14	0.2	0.4		0.8	0.66
15	0.2	0.6		0.7	0.82
16	0.1			1.0	0.11
17	0.1			1.0	0.11
18	0.3	0.9		0.7	1.13
19	0.4	0.6		0.8	0.94
20	0.7	0.9	0.5	0.7	2.01
21	0.3	0.8		0.7	1.11
22	0.8	2.8		0.7	3.65
23	0.8	2.0		0.7	2.76
24	0.3	0.2		0.8	0.45
25	0.4	0.5		0.8	0.89
26	0.3	0.9		0.7	1.21
27	2.9	2.4	0.5	0.8	5.82
28	0.6	0.7		0.8	1.29
29	0.2			1.0	0.15
30	0.3			1.0	0.33
31	0.5			1.0	0.53
32	0.9		0.2	0.9	1.13
33	0.9			1.0	0.88
34	0.9		0.1	0.9	0.91
35	1.0		0.1	0.9	1.09
36	0.7		0.6	0.7	1.27
37	0.5		0.1	0.9	0.64
38	0.6		0.1	0.9	0.69
39	0.8		0.3	0.8	1.14
40	0.7		0.1	0.9	0.78
41	0.9			1.0	0.85
42	0.8		0.3	0.8	1.12
43	1.0		0.5	0.8	1.49
44	0.9		0.8	0.7	1.73
45	0.1		0.1	0.8	0.15
46	0.1			1.0	0.13
47	0.9		0.4	0.8	1.25
48	0.9		0.3	0.8	1.15
49	0.6		0.1	0.9	0.76

* Refer to Water Quality Technical Report Exhibit for Otay Ranch Village 7, located in Map Pocket 1

** Calculations for Composite Runoff Coefficient are found in Appendix B: Inlet Filter Inserts

RICK ENGINEERING COMPANY
5620 Friars Road • San Diego, California 92110-2596 • (619) 291-3588 • www.rickengineering.com

3/29/04
14483

1. Total area: 0.93 acres
slope: 0.43 acres
road: 0.50 acres

2. Total area: 1.06 acres
slope: 0.39 acres
road: 0.67 acres

3. Total area: 0.27 ac
Road: 0.27 ac

4. Total area: 1.33 ac
Lots: 0.71 ac
Road: 0.62 ac
slope: 0.06 ac

5. Total area: 0.90 ac
Lots: 0.47 ac
Roads: 0.43 ac

6. Total area: 2.32 ac
Lots: 1.78 ac
Roads: 0.54 ac

7. Total area: 1.95 ac
Lots: 1.42 ac
Roads: 0.53 ac

8. Total area: 0.97 ac
Lots: 0.75 ac
Road: 0.22 ac

9. Total area: 0.98 ac
Lots: 0.76 ac
Road: 0.22 ac

10. Total area: 1.25 ac
Lots: 0.93 ac
Road: 0.32 ac

11. Total area: 1.33 ac
Lots: 1.03 ac
Road: 0.30 ac

12. Total area: 1.0 ac
Lots: 0.75 ac
Roads: 0.25 ac

13. Total area: 1.69 ac
Lots: 1.22 ac
Roads: 0.47 ac

14. Total area: 0.66 ac
Lots: 0.43 ac
Road: 0.23 ac

15. Total area: 0.82 ac
Lots: 0.61 ac
Road: 0.21 ac

16. Total area: 0.11 ac
Roads: 0.11 ac

17. Total area: 0.11 ac
Road: 0.11 ac

18. Total area: 1.13 ac
Lots: 0.87 ac
Road: 0.26 ac

RICK ENGINEERING COMPANY
5620 Friars Road • San Diego, California 92110-2596 • (619) 291-3588 • www.rickengineering.com

3/29/04
14483
2

19. Total area: 0.94ac
Lots: 0.58ac
Road: 0.36ac

20. Total area: 2.01 ac
Lots: 0.86ac
Road: 0.66ac
Park: 0.49ac

21. Total area: 1.11ac
Lots: 0.81ac
Road: 0.30ac

22. Total area: 3.65ac
Lots: 2.82ac
Road: 0.83ac

23. Total area: 2.76ac
Lots: 1.96 ac
Road: 0.8 ac

24. Total area: 0.45ac
Lots: 0.18ac
Road: 0.27ac

25. Total area: 0.89ac
Lots: 0.51ac
Roads: 0.38ac

26. Total area: 1.21ac
Lots: 0.94ac
Roads: 0.27ac

27. Total area: 4.20ac
Lots: 2.47ac
Road: 2.91ac

28. Total area: 1.29ac
Lots: 0.66ac
Road: 0.63ac

29. Total area: 0.15ac
Road: 0.15ac

30. Total area: 0.33ac
Road: 0.33ac

31. Total area: 0.53ac
Road: 0.53ac

32. Total area: 1.15ac
Slope: 0.20ac
Road: 0.93ac

33. Total area: 0.88ac
Road: 0.88ac

34. Total area: 0.91ac
Slope: 0.05ac
Road: 0.86ac

35. Total area: 1.09ac
Slope: 0.13ac
Road: 0.96ac

36. Total area: 1.27ac
Slope: 0.55ac
Road: 0.72ac

37. Total area: 0.64ac
Slope: 0.11ac
Road: 0.53ac

In-line Treatment Facility

Prepared By:
Rick Engineering Company – Water Resources Division

DCB:JW:nd/Report/14483.005
9-19-03
Revised: 10-20-03
Revised: 02-27-04
Revised: 03-26-04
Revised: 05-24-04

OTAY RANCH VILLAGE 7
J-14483
05/24/04

CALCULATIONS FOR IN-LINE TREATMENT FACILITY:

$Q_T = CIA$

Q_T = TREATMENT FLOW RATE (cubic feet / second)
C = COMPOSITE COEFFICIENT
I = INTENSITY (AS DEFINED BY SUSMP = 0.2 inches / hour)
A = DRAINAGE AREA

IN-LINE TREATMENT FACILITY #1

$Q_T = (0.8)(0.2)(41.3) = 6.6$ cfs

IN-LINE TREATMENT FACILITY #2

$Q_T = (0.95)(0.2)(9.0) = 1.7$ cfs

IN-LINE TREATMENT FACILITY #3

$Q_T = (0.8)(0.2)(16.9) = 2.7$ cfs

NOTE: FOR UNIT #1 & #3 A CONSERVATIVE RUNOFF COEFFICIENT WAS USED OF 0.8. THIS WILL MODEL A HIGH (CONSERVATIVE) DENSITY RESIDENTIAL LAND USE.

FOR UNIT #2 A CONSERVATIVE RUN OFF COEFFICIENT WAS USED OF 0.95. THIS WILL MODEL A INDUSTRIAL (CONSERVATIVE) LAND USE.

Temporary Desilting Basin (High School)

Prepared By:
Rick Engineering Company – Water Resources Division

DCB:JW:nd/Report/14483.005
9-19-03
Revised: 10-20-03
Revised: 02-27-04
Revised: 03-26-04
Revised: 05-24-04

TEMPORARY DESILTING BASIN

Methodology and Criteria

A temporary desilting basin will be constructed for the proposed high school site that will remain in a mass graded condition for an indefinite period of time. The location of the temporary desilting basins is shown on the exhit titled, *Wter Quality Technical Report Exhibit for Village 7*, located in Map Pocket 2. The basin will be constructed to meet the requirements outlined in the State Water Resources Control Board (SWRCB) Order No. 99-08-DWQ National Pollutant Discharge Elimination System (NPDES) General Permit No. CAS000002 Waste Discharge Requirements (WDRs) for Storm Water Runoff Associated with Construction Activity (General Construction Permit), Section A: Storm Water Pollution Prevention Plan, Item 8, Sediment Control, Option 2. The following equation from the General Construction Permit was used to determine the capacity (volume) required for the basins from the bottom of the basin to the principal outlet:

$$\text{Volume} = 3{,}600 \text{ cubic feet} / A$$

Where A is the drainage area measured in acres.

In addition to the criteria for capacity, the General Construction Permit specifies that the length of the basin shall be greater than twice the width of the basin. The depth must not be less than three feet nor greater than five feet (for safety reasons and for maximum efficiency). The 100-year 6-hour peak runoff from the drainage area in a mass graded condition is determined using the rational method.

The temporary desilting basin will utilize a 48-inch corrugated metal pipe (CMP) risers for the principal outlet. Because the type of outflow through a riser (weir flow or orifice flow) and the weir coefficient for weir flow vary depending on the amount of head (water depth) over the riser crest elevation, a spreadsheet was utilized to calculate weir flow and orifice flow at incremental depths above the riser crest. Weir coefficients were obtained from Figure 9-57, *Relationship of Circular Crest Coefficient C_o to H_o/R_s for different Approach Depths (aerated nappe)* [where H_0 is head and R_s is the radius of the riser], and from Design of Small Dams (United States Department of the Interior Bureau of Reclamation, 1987). The riser crest elevation shown in the spreadsheet is 100 feet. The actual head above the riser crest elevation required to convey the 100-year 6-hour peak runoff from the drainage area in a mass graded condition is determined by finding the water surface elevation, "E," from the first column of the spreadsheet at which the controlling discharge, "Qout," from the tenth column of the spreadsheet is equal to or greater than the 100-year 6-hour peak runoff from the drainage area in a mass graded condition, and subtracting 100 feet.

The total depth of the basin is determined by adding the riser height, the head above the riser crest elevation required to convey the 100-year 6-hour peak runoff from the drainage area in a mass graded condition, and two feet of freeboard. The proposed basin will also have a 48-inch outlet riser with two feet of freeboard. A dual 48-inch riser with two feet of head will also be installed to act as an emergency outlet/spillway.

Job Name:	**Otay Ranch V7**
Job Number:	**14483**
Date:	**02/27/04**
Revised:	

BASIN DIMENSIONS:

Spreadsheet Instructions: **Enter project specific information into yellow boxes.**
Input A, H, Z, and adjust Cell B33 so that Cell C41 is "more than 2.00 feet"...

Location:

Sediment Basin(s), as measured from the bottom of the basin to the principal outlet, shall have at least a capacity equivalent to 3,600 ft^3 per acre draining into the site.

The length of the basin shall be more than twice the width of the basin.

The depth must not be less than three feet nor greater than five feet (for safety reasons and for maximum efficiency).

The side slopes shall be 2 to 1 (2:1) or greater.

Input:

Drainage Area to Basin, A:	52.7 acres
Required Volume: (3600 ft^3 per acre)	189720 ft^3
assume riser height, H =	3.0 ft
assume side slopes, Z =	3 to 1

Calculations:

		Bottom of Basin (@ 0 feet)	ROUND UP... Bottom of Basin (@ 0 feet)	THEREFOR... Length and Width (@ Riser Height, H)
length to width ratio at H/2...	2.35			
surface area needed at H/2=	63240 ft^2			
Length at H/2=	385.5 ft	376.5 ft	377.0 ft	395.0 ft
Width at H/2=	164.0 ft	155.0 ft	156.0 ft	174.0 ft

Results:

Length to Width ratio at Riser Crest, H =	**2.27 ft**	okay
Volume provided at Riser Crest, H =	**191313 ft**	okay

Filename: LOT X - Design of Desilting Basins.xls

STAND PIPE:

Enter information in yellow boxes. All other information is self-generated.

Job Name:	**Otay Ranch V7**
Job Number:	**14483**
Date:	**02/27/04**
Revised:	

Location:

	inches	feet
Diameter of Riser Stand Pipe, D (in.):	60	5
Crest Elevation, Z (ft):	100	
Step Increments for Elevation (ft):	0.25	

Q_{100} (cfs): 109.5

150% x Q_{100} = 164.25 cfs

Water Surface Elevation, E	Weir Ho/Rs	Weir C	Weir L	Weir H	$Q_{weir} = CL(H)^{1.5}$	Orifice (0.6) C	Orifice A	Orifice H	$Q_{orifice} = CA(2gH)^{0.5}$	(Pass 150% of Q_{100}) Controlling Discharge, Qout	Type
100.0	0.00	4.05	15.71	0.0	0.00	0.6	19.63	0.0	0.00	0.00	weir flow
100.3	0.10	3.99	15.71	0.3	7.84	0.6	19.63	0.3	47.27	7.84	weir flow
100.5	0.20	3.90	15.71	0.5	21.67	0.6	19.63	0.5	66.85	21.67	weir flow
100.8	0.30	3.76	15.71	0.8	38.38	0.6	19.63	0.8	81.88	38.38	weir flow
101.0	0.40	3.57	15.71	1.0	56.06	0.6	19.63	1.0	94.54	56.06	weir flow
101.3	0.50	3.33	15.71	1.3	73.12	0.6	19.63	1.3	105.70	73.12	weir flow
101.5	0.60	3.06	15.71	1.5	88.38	0.6	19.63	1.5	115.79	88.38	weir flow
101.8	0.70	2.78	15.71	1.8	101.18	0.6	19.63	1.8	125.07	101.18	weir flow
102.0	0.80	2.51	15.71	2.0	111.39	0.6	19.63	2.0	133.70	111.39	weir flow
102.3	0.90	2.25	15.71	2.3	119.40	0.6	19.63	2.3	141.81	119.40	weir flow
102.5	1.00	2.03	15.71	2.5	125.92	0.6	19.63	2.5	149.48	125.92	weir flow
102.8	1.10	1.84	15.71	2.8	131.82	0.6	19.63	2.8	156.78	131.82	weir flow
103.0	1.20	1.69	15.71	3.0	137.83	0.6	19.63	3.0	163.75	137.83	weir flow
103.3	1.30	1.57	15.71	3.3	144.35	0.6	19.63	3.3	170.44	144.35	weir flow
103.5	1.40	1.47	15.71	3.5	151.31	0.6	19.63	3.5	176.87	151.31	weir flow
103.8	1.50	1.39	15.71	3.8	158.10	0.6	19.63	3.8	183.08	158.10	weir flow
104.0	1.60	1.30	15.71	4.0	163.78	0.6	19.63	4.0	189.08	163.78	weir flow
104.3	1.70	1.22	15.71	4.3	167.58	0.6	19.63	4.3	194.90	167.58	weir flow
104.5	1.80	1.13	15.71	4.5	169.70	0.6	19.63	4.5	200.55	169.70	weir flow
104.8	1.90	1.06	15.71	4.8	172.59	0.6	19.63	4.8	206.05	172.59	weir flow

*C From Design of Small Dams, based on Ho/Rs.
For Ho/Rs greater than 2.0, Co = 1.0.

NOTES:

Provide 1.0-foot freeboard above $H_{req'd}$ to pass Q_{100}

Total depth in Desilt Basin shall equal: Riser Height + $H_{req'd}$ + 1.0'

APPENDIX C

Storm Water Costs and Details

BioClean

BIO CLEAN
ENVIRONMENTAL SERVICES, INC.

STORMWATER FILTRATION SYSTEMS
FILTER PRICE LIST

Effective June 2003

Grate Inlet Skimmer Box

Flange Dimensions	Price Per Unit
12" x 12" up to 18" x 18"	$ 745.00
19" x 19" up to 24" x 24"	$ 795.00
25" x 25" up to 28" x 36"	$ 945.00
29" x 37" up to 36" x 48"	$1,064.00
37" x 49" up to 48" x 54"	$1,183.00

For larger sizes – please call for quote

Curb Inlet Easy Maintenance Units

Product Description	Size	Price Per Unit
Curb Inlet Units w/Shelves & Basket	2' up to 4'	$ 845.00
Curb Inlet Units w/Shelves & Basket	5' up to 7'	$ 875.00
Curb Inlet Units w/Shelves & Basket	8' up to 10'	$ 975.00
Curb Inlet Units w/Shelves & Basket	11' up to 12'	$1,075.00
Curb Inlet Units w/Shelves & Basket	13' up to 14'	$1,225.00
Curb Inlet Units w/Shelves & Basket	15' up to 16'	$1,375.00

All units above include Standard Rectangle Baskets. An additional $100.00 will be added to any unit that is upgraded to the round baskets and collars, regardless of size. Prices DO NOT include installation.

Curb Inlet Baskets with no Shelves

	Price Per Foot
Curb Inlet Baskets	$145.00 (includes installation)

NOTE: SHIPPING CHARGES AND INSTALLATION ARE EXTRA

BIO CLEAN ENVIRONMENTAL SERVICES, INC
P O BOX 869, OCEANSIDE CA 92049
(760) 433-7640 FAX (760) 433-3176

BIO CLEAN
ENVIRONMENTAL SERVICES, INC

STORMWATER FILTRATION SYSTEMS
FILTER INSTALLATION PRICE LIST

Effective June 2003

Grate Inlet Skimmer Box

	Up to 10 Units	10 or More Units
Up to 24" x 24"	$ 65.00 ea.	$ 55.00 ea.
25" x 25" up to 28" x 36"	$ 65.00 ea.	$ 55.00 ea.
29" x 37" up to 36" x 48"	$ 65.00 ea.	$ 55.00 ea.
37" x 49" up to 48" x 54"	$ 75.00 ea.	$ 65.00 ea.
Over 48" x 54"	Per Quote	Per Quote

Curb Inlet Basket

Unit Size 2' up to 7': Installation includes installing a fiberglass shelf mounted to the concrete vault. Installation of the back basket, located under manhole. Unit caulked in place.

$125.00 each

Unit Sizes 7' up to 10': Installation includes installing a fiberglass shelf mounted to the concrete vault. Installation of the back basket, located under manhole. Unit caulked in place.

$155.00 each

Unit Sizes 10' up to 16': Installation includes installing a fiberglass shelf mounted to the concrete vault. Installation of the back basket, located under manhole. Unit caulked in place.

$200.00 each

An additional $50.00 will be added to any unit that is upgraded to the round baskets and collars, regardless of size.

Additional Pricing for Grate Curb Inlet Unit Custom Work:

Wing Extensions up to 8' long	$100.00 each Wing
Deflector Shields up to 8' long	$150.00 each Shield

BIO CLEAN ENVIRONMENTAL SERVICES, INC
P O BOX 869, OCEANSIDE CA 92049
(760) 433-7640 FAX (760) 433-3176

BIO CLEAN
ENVIRONMENTAL SERVICES, INC.

STORMWATER FILTRATION SYSTEMS
FILTER MAINTENANCE PRICE LIST

Effective June 2003

Grate Inlet Skimmer Box

Up to 28" x 36"	$69.00 per Unit
29" x 37" up to 48" x 54"	$79.00 per Unit

Curb Inlet Basket

30" to 84"	Shelf & Basket	$69.00 per Unit
84" to 144"	Shelf & Basket	$79.00 per Unit
144" to 180"	Shelf & Basket	$89.00 per Unit

Bio Clean Environmental Services, Inc. recommends cleaning and maintenance of stormwater filters on a quarterly basis (4 x per year).

A MINIMUM CHARGE OF $150.00 IS REQUIRED ON ALL SERVICES

Service and Maintenance Includes:

- Disposal of debris captured by filtration device.
- Evaluation of Hydrocarbon booms. Booms will be changed out at a minimum of at least twice per year.
- Hydrocarbon booms to be disposed of in accordance with local and state requirements
- Transportation of debris, sediments and organics to approved facility and in accordance with local and state requirements.
- Report on collected debris, type of debris and condition of filters will be provided to landowner, city or municipality.

The Bio Clean Environmental Services Maintenance Program incorporates a tracking program used to identify each inlet unit and to preserve its history.

Bio Clean Environmental Services reserves the right to *not* service filter systems that have been misused, vandalized, illegally dumped into or not used for intended purposes. Service does not provide for cleaning of the vault in which the filter is located.

Please see Bio Clean Service Agreement for specific details.

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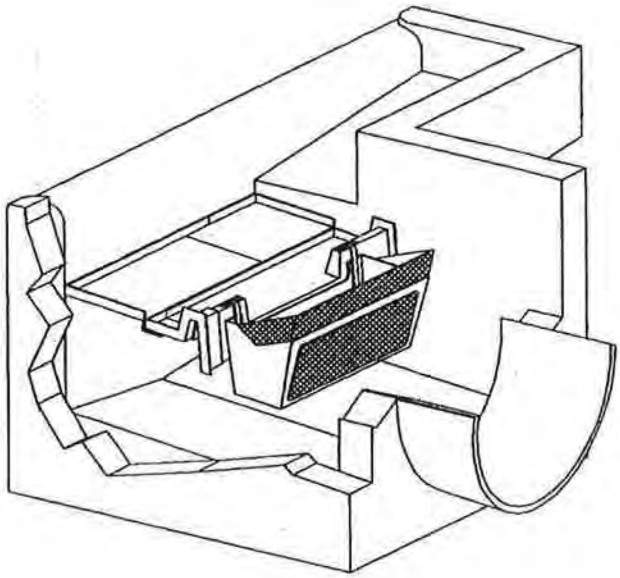
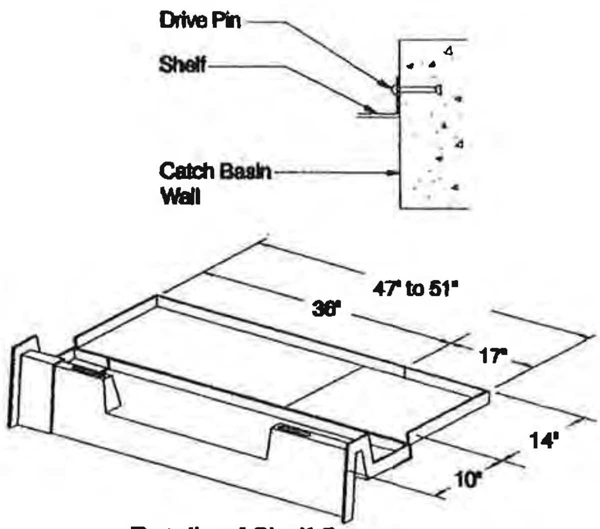
gkent@biocleanenvironmental.net
www.biocleanenvironmental.net

THE CALIFORNIA CURB SHELF BASKET WATER CLEANSING SYSTEM



The California Curb Shelf Basket
Shelf Water Cleaning System

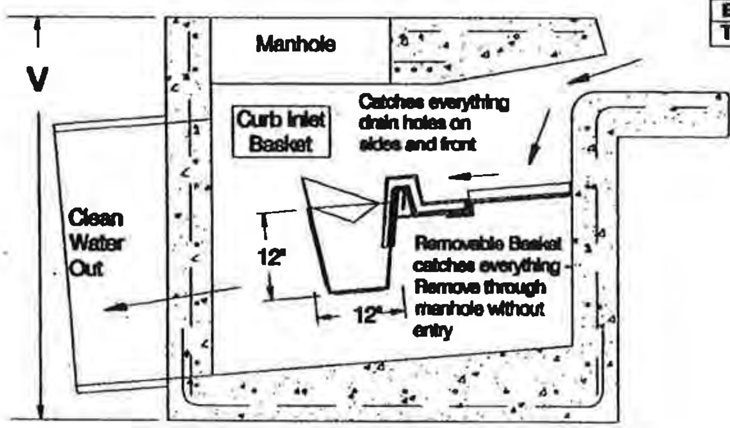
Figure 1



Details of Shelf System
(Dimensions will vary)

Figure 2

San Diego regional standard Curb Inlet - Type B



Patented

Figure 3

FLOW RATES per 3 ft BASKET				
$Q = SO * c_d * A\sqrt{2 * g * h}$			$c_d = \frac{\text{Coefficient of}}{\text{Discharge}} = .67$	
	SO	A (ft²)	h (ft)	Q (ft³/s)
Top Front	.62	65.1	7.9	1.6
Bottom Front	.56	179.4	12.40	3.6
Bottom	.68	165.9	18.0	5.1
TOTAL				10.6

NOTES:

1. Shelf system provides for entire coverage of inlet opening so to divert all flow to basket.
2. Shelf system manufactured from marine grade fiberglass, gel coated for UV protection.
3. Shelf system attched to catch basin with non corrosive hardware.
4. Filtration Basket structure manufactured of marine grade fiberglass, gel coated for UV protection.
5. Filtration Basket fine screen and coarse containment screen manufactured from stainless steel.
6. Filtration Basket holds boom of absorbent media to capture hydrocarbons. Boom is easily replaced without removing mounting hardware.
7. Filtration Basket location is directly under manhole access for easy maintenance.

In-Line Treatment Facilities

Prepared By:
Rick Engineering Company – Water Resources Division

DCB:JW:nd/Report/14483.005
9-19-03
Revised: 10-20-03
Revised: 02-27-04
Revised: 03-26-04
Revised: 05-24-04

APPROXIMATE CDS COSTS

	MODEL	Q treatment (CFS)	AREA (ACRES)	EQUIPMENT COSTS*	INSTALLATION COSTS**	TOTAL
D ≤ 12"	PM U 20_15	0.7	4	$4,600	$2,400	$7,000
D ≤ 18"	PMSU 20_15_4	0.7	4	$6,900	$2,400	$9,300
	PMSU 20_15	0.7	4	$8,200	$3,400	$11,600
	PMSU 20_20	1.1	6	$10,500	$3,800	$14,300
	PMSU 20_25	1.6	9	$14,700	$3,800	$18,500
	PMSU 30_20	2.0	11	$19,200	$3,800	$23,000
	PMSU 30_30	3.0	17	$24,500	$4,500	$29,000
	PSW 30_30	3.0	17	$19,700	$9,300	$29,000
	PMSU 40_30	4.5	25	$28,000	$6,000	$34,000
	PMSU 40_40	6.0	33	$32,700	$7,000	$39,700 -
	PSWC 40_30	4.5	25	$23,900	$13,000	$36,900
	PSWC 40_40	6.0	33	$28,000	$13,000	$41,000 -
	PSW 50_42	9.0	50	$35,500	$18,000	$53,500
	PSW 50_50	11	61	$36,200	$20,000	$56,200
	PSWC 56_53	14	78	$42,000	$26,000	$68,000
	PSWC 56_68	19	106	$51,600	$33,000	$84,600
	PSW 70_70	26	144	$64,900	$40,000	$104,900
	PSW 100_60	38	211	$115,300	$60,000	$175,300
	PSW 100_80	50	278	$121,600	$70,000	$191,600
	PSW 100_100	64	356	$127,900	$80,000	$207,900

A = Q/CI (I = 0.2 IN/HR, C = 0.9) (Based on Regional Water Quality Control Board Requirements (SUSMP)).

* Equipment costs include delivery to jobsite (FOB). Prices are based on finished grade to invert depths of 5 ft. Prices are subject to change.

** Installation costs may vary depending on site conditions. Cost includes the cast-in-place weir box (if required).

APPROXIMATE CDS MAINTENANCE COSTS

MODEL	Q_{TREAT} (CFS)	AREA (ACRES)	SUMP VOLUME (CU YDS)	SUMP MATERIAL WT. (TONS)	DISPOSAL COSTS $40/TON	VACTOR TIME (HRS)	VACTOR CHARGE $175/HR	TOTAL COST
PMIU 20_15	0.7	4	1	1	$35	4	$700	$735
PMSU 20_15_4	0.7	4	1	1	$35	4	$700	$735
PMSU 20_15	0.7	4	1	1	$50	4	$700	$750
PMSU 20_20	1.1	6	1	1	$50	4	$700	$750
PMSU 20_25	1.6	8	1	1	$50	4	$700	$750
PMSU 30_20	2.0	10	2	2	$100	4	$700	$800
PMSU 30_30	3.0	20	2	2	$100	4	$700	$800
PSW 30_30	3.0	20	1	2	$65	4	$700	$765
PMSU 40_30	4.5	25	6	6	$250	4	$700	$950
PMSU 40_40	6.0	30	6	6	$250	4	$700	$950
PSWC 40_30	4.5	80	6	6	$250	4	$700	$950
PSWC 40_40	6.0	80	6	6	$250	4	$700	$950
PSW 50_42	9.0	50	2	2	$85	4	$700	$785
PSW 50_50	11	60	2	2	$85	4	$700	$785
PSWC 56_53	14	110	6	6	$250	5	$875	$1,125
PSWC 56_68	19	110	6	6	$250	5	$875	$1,125
PSW 70_70	26	150	4	4	$180	6	$1,050	$1,230
PSW 100_60	30	210	14	16	$635	7	$1,225	$1,860
PSW 100_80	50	280	14	16	$635	7	$1,225	$1,860
PSW 100_100	64	350	14	16	$635	7	$1,225	$1,860

Assumptions:

1. Sump material weighs 85 lbs per cubic foot
2. Cleanout intervals are based on the amount and types of floatables and sediment captured by each CDS unit. CDS installations in Southern California typically require 1-2 cleanouts per year.
3. This estimate does not include hazardous waste disposal fees, if required.
4. Liquids may be decanted back into the CDS unit or discharged into the sanitay sewer without fees.
5. Minimum vactor charge of 4 hours applies. Actual maintenance time is less. Therefore, cleaning multiple CDS units in one day is recommended to reduce cost.

```
  SPECIFIED NUMBER OF HALFSTREETS CARRYING RUNOFF =  1
  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) =      16.37
    STREETFLOW MODEL RESULTS USING ESTIMATED FLOW:
    STREET FLOW DEPTH(FEET) =  0.56
    HALFSTREET FLOOD WIDTH(FEET) =   22.30
    AVERAGE FLOW VELOCITY(FEET/SEC.) =    3.53
    PRODUCT OF DEPTH&VELOCITY(FT*FT/SEC.) =     1.98
  STREET FLOW TRAVEL TIME(MIN.) =   4.43    Tc(MIN.) =    9.38
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  4.742
  *USER SPECIFIED(SUBAREA):
  USER-SPECIFIED RUNOFF COEFFICIENT = .6500
  S.C.S. CURVE NUMBER (AMC II) =   0
  AREA-AVERAGE RUNOFF COEFFICIENT =  0.650
  SUBAREA AREA(ACRES) =      9.47      SUBAREA RUNOFF(CFS) =   29.19
  TOTAL AREA(ACRES) =        9.77        PEAK FLOW RATE(CFS) =       30.11

  END OF SUBAREA STREET FLOW HYDRAULICS:
  DEPTH(FEET) = 0.67   HALFSTREET FLOOD WIDTH(FEET) =  28.40
  FLOW VELOCITY(FEET/SEC.) =  4.09   DEPTH*VELOCITY(FT*FT/SEC.) =   2.74
  *NOTE: INITIAL SUBAREA NOMOGRAPH WITH SUBAREA PARAMETERS,
        AND L =  938.3 FT WITH ELEVATION-DROP =  10.0 FT, IS   37.2 CFS,
        WHICH EXCEEDS THE TOP-OF-CURB STREET CAPACITY AT NODE    110.00
  LONGEST FLOWPATH FROM NODE       1.00 TO NODE     110.00 =  1008.33 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    110.00 TO NODE    113.00 IS CODE =  31
----------------------------------------------------------------------------
  >>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
  >>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<
============================================================================
  ELEVATION DATA: UPSTREAM(FEET) =   485.00  DOWNSTREAM(FEET) =   482.40
  FLOW LENGTH(FEET) =   329.22   MANNING'S N =  0.013
  DEPTH OF FLOW IN  33.0 INCH PIPE IS  24.6 INCHES
  PIPE-FLOW VELOCITY(FEET/SEC.) =   6.34
  ESTIMATED PIPE DIAMETER(INCH) =  33.00    NUMBER OF PIPES =   1
  PIPE-FLOW(CFS) =       30.11
  PIPE TRAVEL TIME(MIN.) =   0.87    Tc(MIN.) =  10.24
  LONGEST FLOWPATH FROM NODE       1.00 TO NODE     113.00 =  1337.55 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    113.00 TO NODE    113.00 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.) =  10.24
  RAINFALL INTENSITY(INCH/HR) =   4.48
  TOTAL STREAM AREA(ACRES) =      9.77
  PEAK FLOW RATE(CFS) AT CONFLUENCE =      30.11

****************************************************************************
  FLOW PROCESS FROM NODE      3.00 TO NODE      4.00 IS CODE =  21
----------------------------------------------------------------------------
  >>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<
============================================================================
  *USER SPECIFIED(SUBAREA):
  USER-SPECIFIED RUNOFF COEFFICIENT = .6500
  S.C.S. CURVE NUMBER (AMC II) =   0
  INITIAL SUBAREA FLOW-LENGTH(FEET) =     70.00
  UPSTREAM ELEVATION(FEET) =     533.30
  DOWNSTREAM ELEVATION(FEET) =     531.50
  ELEVATION DIFFERENCE(FEET) =       1.80
  SUBAREA OVERLAND TIME OF FLOW(MIN.) =    4.947
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  7.114
  NOTE: RAINFALL INTENSITY IS BASED ON Tc = 5-MINUTE.
  SUBAREA RUNOFF(CFS) =       0.55
  TOTAL AREA(ACRES) =       0.12   TOTAL RUNOFF(CFS) =       0.55

****************************************************************************
  FLOW PROCESS FROM NODE      4.00 TO NODE    140.00 IS CODE =  62
----------------------------------------------------------------------------
  >>>>>COMPUTE STREET FLOW TRAVEL TIME THRU SUBAREA<<<<<
  >>>>>(STREET TABLE SECTION #  1 USED)<<<<<
============================================================================
  UPSTREAM ELEVATION(FEET) =  531.50  DOWNSTREAM ELEVATION(FEET) =  493.60
```

```
 STREET LENGTH(FEET) =   767.26   CURB HEIGHT(INCHES) =  8.0
 STREET HALFWIDTH(FEET) = 30.00

 DISTANCE FROM CROWN TO CROSSFALL GRADEBREAK(FEET) =  20.00
 INSIDE STREET CROSSFALL(DECIMAL) =  0.018
 OUTSIDE STREET CROSSFALL(DECIMAL)  =  0.018

 SPECIFIED NUMBER OF HALFSTREETS CARRYING RUNOFF =  1
 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) =       5.32
   STREETFLOW MODEL RESULTS USING ESTIMATED FLOW:
   STREET FLOW DEPTH(FEET) =  0.34
   HALFSTREET FLOOD WIDTH(FEET) =   10.04
   AVERAGE FLOW VELOCITY(FEET/SEC.) =    4.86
   PRODUCT OF DEPTH&VELOCITY(FT*FT/SEC.) =    1.66
 STREET FLOW TRAVEL TIME(MIN.) =    2.63   Tc(MIN.) =    7.58
  100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  5.441
 *USER SPECIFIED(SUBAREA):
 USER-SPECIFIED RUNOFF COEFFICIENT = .6500
 S.C.S. CURVE NUMBER (AMC II) =    0
 AREA-AVERAGE RUNOFF COEFFICIENT =  0.650
 SUBAREA AREA(ACRES) =      2.66      SUBAREA RUNOFF(CFS) =     9.41
 TOTAL AREA(ACRES) =        2.78        PEAK FLOW RATE(CFS) =        9.83

 END OF SUBAREA STREET FLOW HYDRAULICS:
 DEPTH(FEET) = 0.40   HALFSTREET FLOOD WIDTH(FEET) =  13.24
 FLOW VELOCITY(FEET/SEC.) =  5.59   DEPTH*VELOCITY(FT*FT/SEC.) =   2.23
 LONGEST FLOWPATH FROM NODE       3.00 TO NODE     140.00 =   837.26 FEET.

****************************************************************************
 FLOW PROCESS FROM NODE    140.00 TO NODE    113.00 IS CODE =  31
----------------------------------------------------------------------------
 >>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
 >>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<
============================================================================
 ELEVATION DATA: UPSTREAM(FEET) =   483.60  DOWNSTREAM(FEET) =   482.40
 FLOW LENGTH(FEET) =   196.91   MANNING'S N =  0.013
 DEPTH OF FLOW IN  24.0 INCH PIPE IS  16.0 INCHES
 PIPE-FLOW VELOCITY(FEET/SEC.) =    4.40
 ESTIMATED PIPE DIAMETER(INCH) =  24.00    NUMBER OF PIPES =   1
 PIPE-FLOW(CFS) =        9.83
 PIPE TRAVEL TIME(MIN.) =    0.75    Tc(MIN.) =    8.32
 LONGEST FLOWPATH FROM NODE       3.00 TO NODE     113.00 =  1034.17 FEET.

****************************************************************************
 FLOW PROCESS FROM NODE    113.00 TO NODE    113.00 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.) =    8.32
 RAINFALL INTENSITY(INCH/HR) =    5.12
 TOTAL STREAM AREA(ACRES) =      2.78
 PEAK FLOW RATE(CFS) AT CONFLUENCE =       9.83

 ** CONFLUENCE DATA **
 STREAM     RUNOFF       Tc       INTENSITY       AREA
 NUMBER      (CFS)     (MIN.)    (INCH/HOUR)     (ACRE)
    1        30.11     10.24        4.480          9.77
    2         9.83      8.32        5.122          2.78

 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        34.30      8.32        5.122
    2        38.71     10.24        4.480

 COMPUTED CONFLUENCE ESTIMATES ARE AS FOLLOWS:
 PEAK FLOW RATE(CFS) =      38.71   Tc(MIN.) =     10.24
 TOTAL AREA(ACRES) =       12.55
 LONGEST FLOWPATH FROM NODE       1.00 TO NODE     113.00 =  1337.55 FEET.

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

```
 FLOW PROCESS FROM NODE    113.00 TO NODE    114.00 IS CODE =  31
----------------------------------------------------------------------------
 >>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
 >>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<
============================================================================
 ELEVATION DATA: UPSTREAM(FEET) =   482.40  DOWNSTREAM(FEET) =   480.00
 FLOW LENGTH(FEET) =   235.92   MANNING'S N =  0.013
 DEPTH OF FLOW IN  36.0 INCH PIPE IS  24.7 INCHES
 PIPE-FLOW VELOCITY(FEET/SEC.) =   7.50
 ESTIMATED PIPE DIAMETER(INCH) =  36.00    NUMBER OF PIPES =   1
 PIPE-FLOW(CFS) =     38.71
 PIPE TRAVEL TIME(MIN.) =   0.52    Tc(MIN.) =  10.77
 LONGEST FLOWPATH FROM NODE      1.00 TO NODE    114.00 =  1573.47 FEET.

****************************************************************************
 FLOW PROCESS FROM NODE    114.00 TO NODE    114.00 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.) =  10.77
 RAINFALL INTENSITY(INCH/HR) =   4.34
 TOTAL STREAM AREA(ACRES) =    12.55
 PEAK FLOW RATE(CFS) AT CONFLUENCE =     38.71

****************************************************************************
 FLOW PROCESS FROM NODE      5.00 TO NODE      6.00 IS CODE =  21
----------------------------------------------------------------------------
 >>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<
============================================================================
 *USER SPECIFIED(SUBAREA):
 USER-SPECIFIED RUNOFF COEFFICIENT = .6500
 S.C.S. CURVE NUMBER (AMC II) =   0
 INITIAL SUBAREA FLOW-LENGTH(FEET) =     70.00
 UPSTREAM ELEVATION(FEET) =    501.10
 DOWNSTREAM ELEVATION(FEET) =    499.00
 ELEVATION DIFFERENCE(FEET) =      2.10
 SUBAREA OVERLAND TIME OF FLOW(MIN.) =     4.699
  100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  7.114
 NOTE: RAINFALL INTENSITY IS BASED ON Tc = 5-MINUTE.
 SUBAREA RUNOFF(CFS) =      0.60
 TOTAL AREA(ACRES) =      0.13   TOTAL RUNOFF(CFS) =      0.60

****************************************************************************
 FLOW PROCESS FROM NODE      6.00 TO NODE    115.00 IS CODE =  62
----------------------------------------------------------------------------
 >>>>>COMPUTE STREET FLOW TRAVEL TIME THRU SUBAREA<<<<<
 >>>>>(STREET TABLE SECTION #  1 USED)<<<<<
============================================================================
 UPSTREAM ELEVATION(FEET) =  499.00  DOWNSTREAM ELEVATION(FEET) =  491.00
 STREET LENGTH(FEET) =   851.01   CURB HEIGHT(INCHES) =  8.0
 STREET HALFWIDTH(FEET) = 30.00

 DISTANCE FROM CROWN TO CROSSFALL GRADEBREAK(FEET) =  20.00
 INSIDE STREET CROSSFALL(DECIMAL) =  0.018
 OUTSIDE STREET CROSSFALL(DECIMAL)  =  0.018

 SPECIFIED NUMBER OF HALFSTREETS CARRYING RUNOFF =  1
 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) =       9.48
   STREETFLOW MODEL RESULTS USING ESTIMATED FLOW:
   STREET FLOW DEPTH(FEET) =  0.49
   HALFSTREET FLOOD WIDTH(FEET) =   18.40
   AVERAGE FLOW VELOCITY(FEET/SEC.) =    2.95
   PRODUCT OF DEPTH&VELOCITY(FT*FT/SEC.) =     1.45
 STREET FLOW TRAVEL TIME(MIN.) =   4.81   Tc(MIN.) =    9.51
  100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  4.698
 *USER SPECIFIED(SUBAREA):
 USER-SPECIFIED RUNOFF COEFFICIENT = .6500
 S.C.S. CURVE NUMBER (AMC II) =   0
 AREA-AVERAGE RUNOFF COEFFICIENT =  0.650
 SUBAREA AREA(ACRES) =     5.65      SUBAREA RUNOFF(CFS) =   17.25
 TOTAL AREA(ACRES) =      5.78        PEAK FLOW RATE(CFS) =      17.65

 END OF SUBAREA STREET FLOW HYDRAULICS:
 DEPTH(FEET) = 0.58   HALFSTREET FLOOD WIDTH(FEET) =  23.55
 FLOW VELOCITY(FEET/SEC.) =  3.43   DEPTH*VELOCITY(FT*FT/SEC.) =   2.00
```

```
 LONGEST FLOWPATH FROM NODE      5.00 TO NODE    115.00 =   921.01 FEET.

****************************************************************************
 FLOW PROCESS FROM NODE    115.00 TO NODE    114.00 IS CODE =  31
----------------------------------------------------------------------------
 >>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
 >>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<
============================================================================
 ELEVATION DATA: UPSTREAM(FEET) =   481.00  DOWNSTREAM(FEET) =   480.00
 FLOW LENGTH(FEET) =    85.77  MANNING'S N =  0.013
 DEPTH OF FLOW IN  27.0 INCH PIPE IS  17.4 INCHES
 PIPE-FLOW VELOCITY(FEET/SEC.) =   6.52
 ESTIMATED PIPE DIAMETER(INCH) =  27.00    NUMBER OF PIPES =   1
 PIPE-FLOW(CFS) =      17.65
 PIPE TRAVEL TIME(MIN.) =   0.22    Tc(MIN.) =    9.73
 LONGEST FLOWPATH FROM NODE      5.00 TO NODE    114.00 =  1006.78 FEET.

****************************************************************************
 FLOW PROCESS FROM NODE    114.00 TO NODE    114.00 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.) =    9.73
 RAINFALL INTENSITY(INCH/HR) =    4.63
 TOTAL STREAM AREA(ACRES) =      5.78
 PEAK FLOW RATE(CFS) AT CONFLUENCE =      17.65

 ** CONFLUENCE DATA **
 STREAM     RUNOFF      Tc      INTENSITY      AREA
 NUMBER      (CFS)     (MIN.)   (INCH/HOUR)    (ACRE)
     1       38.71     10.77       4.338         12.55
     2       17.65      9.73       4.630          5.78

 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       53.92      9.73      4.630
     2       55.25     10.77      4.338

 COMPUTED CONFLUENCE ESTIMATES ARE AS FOLLOWS:
 PEAK FLOW RATE(CFS) =      55.25   Tc(MIN.) =    10.77
 TOTAL AREA(ACRES) =      18.33
 LONGEST FLOWPATH FROM NODE      1.00 TO NODE    114.00 =  1573.47 FEET.

****************************************************************************
 FLOW PROCESS FROM NODE    114.00 TO NODE    118.00 IS CODE =  31
----------------------------------------------------------------------------
 >>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
 >>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<
============================================================================
 ELEVATION DATA: UPSTREAM(FEET) =   480.00  DOWNSTREAM(FEET) =   477.00
 FLOW LENGTH(FEET) =   248.55  MANNING'S N =  0.013
 DEPTH OF FLOW IN  39.0 INCH PIPE IS  27.9 INCHES
 PIPE-FLOW VELOCITY(FEET/SEC.) =   8.70
 ESTIMATED PIPE DIAMETER(INCH) =  39.00    NUMBER OF PIPES =   1
 PIPE-FLOW(CFS) =      55.25
 PIPE TRAVEL TIME(MIN.) =   0.48    Tc(MIN.) =  11.24
 LONGEST FLOWPATH FROM NODE      1.00 TO NODE    118.00 =  1822.02 FEET.

****************************************************************************
 FLOW PROCESS FROM NODE    118.00 TO NODE    118.00 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.) =  11.24
 RAINFALL INTENSITY(INCH/HR) =    4.22
 TOTAL STREAM AREA(ACRES) =     18.33
 PEAK FLOW RATE(CFS) AT CONFLUENCE =      55.25

****************************************************************************
 FLOW PROCESS FROM NODE      7.00 TO NODE      8.00 IS CODE =  21
----------------------------------------------------------------------------
 >>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<
```

```
==============================================================================
  *USER SPECIFIED(SUBAREA):
  USER-SPECIFIED RUNOFF COEFFICIENT = .6500
  S.C.S. CURVE NUMBER (AMC II) =   0
  INITIAL SUBAREA FLOW-LENGTH(FEET) =     70.00
  UPSTREAM ELEVATION(FEET) =    498.80
  DOWNSTREAM ELEVATION(FEET) =    497.00
  ELEVATION DIFFERENCE(FEET) =      1.80
  SUBAREA OVERLAND TIME OF FLOW(MIN.) =    4.947
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  7.114
  NOTE: RAINFALL INTENSITY IS BASED ON Tc = 5-MINUTE.
  SUBAREA RUNOFF(CFS) =      0.55
  TOTAL AREA(ACRES) =      0.12   TOTAL RUNOFF(CFS) =      0.55

****************************************************************************
  FLOW PROCESS FROM NODE      8.00 TO NODE    119.00 IS CODE =  62
----------------------------------------------------------------------------
  >>>>>COMPUTE STREET FLOW TRAVEL TIME THRU SUBAREA<<<<<
  >>>>>(STREET TABLE SECTION #  1 USED)<<<<<
============================================================================
  UPSTREAM ELEVATION(FEET) =  497.00  DOWNSTREAM ELEVATION(FEET) =  488.50
  STREET LENGTH(FEET) =   305.91   CURB HEIGHT(INCHES) =  8.0
  STREET HALFWIDTH(FEET) = 30.00

  DISTANCE FROM CROWN TO CROSSFALL GRADEBREAK(FEET) =  20.00
  INSIDE STREET CROSSFALL(DECIMAL) =  0.018
  OUTSIDE STREET CROSSFALL(DECIMAL)  =  0.018

  SPECIFIED NUMBER OF HALFSTREETS CARRYING RUNOFF =  1
  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) =      11.14
    STREETFLOW MODEL RESULTS USING ESTIMATED FLOW:
    STREET FLOW DEPTH(FEET) =  0.44
    HALFSTREET FLOOD WIDTH(FEET) =   15.74
    AVERAGE FLOW VELOCITY(FEET/SEC.) =    4.63
    PRODUCT OF DEPTH&VELOCITY(FT*FT/SEC.) =     2.05
  STREET FLOW TRAVEL TIME(MIN.) =   1.10   Tc(MIN.) =   6.05
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  6.292
  *USER SPECIFIED(SUBAREA):
  USER-SPECIFIED RUNOFF COEFFICIENT = .6500
  S.C.S. CURVE NUMBER (AMC II) =   0
  AREA-AVERAGE RUNOFF COEFFICIENT =  0.650
  SUBAREA AREA(ACRES) =    5.16      SUBAREA RUNOFF(CFS) =    21.10
  TOTAL AREA(ACRES) =      5.28       PEAK FLOW RATE(CFS) =      21.59

  END OF SUBAREA STREET FLOW HYDRAULICS:
  DEPTH(FEET) = 0.53   HALFSTREET FLOOD WIDTH(FEET) =  20.59
  FLOW VELOCITY(FEET/SEC.) =  5.43   DEPTH*VELOCITY(FT*FT/SEC.) =   2.88
  LONGEST FLOWPATH FROM NODE      7.00 TO NODE    119.00 =   375.91 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    119.00 TO NODE    118.00 IS CODE =  31
----------------------------------------------------------------------------
  >>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
  >>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<
============================================================================
  ELEVATION DATA: UPSTREAM(FEET) =   478.50  DOWNSTREAM(FEET) =   477.00
  FLOW LENGTH(FEET) =    83.43   MANNING'S N =  0.013
  DEPTH OF FLOW IN  27.0 INCH PIPE IS  17.2 INCHES
  PIPE-FLOW VELOCITY(FEET/SEC.) =   8.07
  ESTIMATED PIPE DIAMETER(INCH) =  27.00    NUMBER OF PIPES =   1
  PIPE-FLOW(CFS) =     21.59
  PIPE TRAVEL TIME(MIN.) =   0.17    Tc(MIN.) =   6.22
  LONGEST FLOWPATH FROM NODE      7.00 TO NODE    118.00 =   459.34 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    118.00 TO NODE    118.00 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.) =   6.22
  RAINFALL INTENSITY(INCH/HR) =   6.18
  TOTAL STREAM AREA(ACRES) =     5.28
  PEAK FLOW RATE(CFS) AT CONFLUENCE =     21.59
```

```
** CONFLUENCE DATA **
STREAM      RUNOFF      Tc        INTENSITY       AREA
NUMBER       (CFS)     (MIN.)    (INCH/HOUR)     (ACRE)
    1        55.25     11.24        4.218          18.33
    2        21.59      6.22        6.179           5.28

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        59.31      6.22        6.179
    2        69.99     11.24        4.218

COMPUTED CONFLUENCE ESTIMATES ARE AS FOLLOWS:
PEAK FLOW RATE(CFS) =     69.99   Tc(MIN.) =    11.24
TOTAL AREA(ACRES) =      23.61
LONGEST FLOWPATH FROM NODE       1.00 TO NODE     118.00 =  1822.02 FEET.

****************************************************************************
 FLOW PROCESS FROM NODE     118.00 TO NODE     125.00 IS CODE =  31
----------------------------------------------------------------------------
 >>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
 >>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<
============================================================================
 ELEVATION DATA: UPSTREAM(FEET) =    477.00  DOWNSTREAM(FEET) =    473.00
 FLOW LENGTH(FEET) =    248.12   MANNING'S N =  0.013
 DEPTH OF FLOW IN  39.0 INCH PIPE IS  30.2 INCHES
 PIPE-FLOW VELOCITY(FEET/SEC.) =  10.16
 ESTIMATED PIPE DIAMETER(INCH) =  39.00    NUMBER OF PIPES =   1
 PIPE-FLOW(CFS) =     69.99
 PIPE TRAVEL TIME(MIN.) =   0.41    Tc(MIN.) =  11.65
 LONGEST FLOWPATH FROM NODE       1.00 TO NODE     125.00 =  2070.14 FEET.

****************************************************************************
 FLOW PROCESS FROM NODE     125.00 TO NODE     125.00 IS CODE =  10
----------------------------------------------------------------------------
 >>>>>MAIN-STREAM MEMORY COPIED ONTO MEMORY BANK # 1 <<<<<
============================================================================

****************************************************************************
 FLOW PROCESS FROM NODE       9.00 TO NODE      10.00 IS CODE =  21
----------------------------------------------------------------------------
 >>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<
============================================================================
 *USER SPECIFIED(SUBAREA):
 USER-SPECIFIED RUNOFF COEFFICIENT = .6500
 S.C.S. CURVE NUMBER (AMC II) =   0
 INITIAL SUBAREA FLOW-LENGTH(FEET) =     70.00
 UPSTREAM ELEVATION(FEET) =    498.80
 DOWNSTREAM ELEVATION(FEET) =    497.00
 ELEVATION DIFFERENCE(FEET) =      1.80
 SUBAREA OVERLAND TIME OF FLOW(MIN.) =    4.947
  100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  7.114
 NOTE: RAINFALL INTENSITY IS BASED ON Tc = 5-MINUTE.
 SUBAREA RUNOFF(CFS) =      0.88
 TOTAL AREA(ACRES) =      0.19   TOTAL RUNOFF(CFS) =      0.88

****************************************************************************
 FLOW PROCESS FROM NODE      10.00 TO NODE     123.00 IS CODE =  62
----------------------------------------------------------------------------
 >>>>>COMPUTE STREET FLOW TRAVEL TIME THRU SUBAREA<<<<<
 >>>>>(STREET TABLE SECTION #  1 USED)<<<<<
============================================================================
 UPSTREAM ELEVATION(FEET) =  497.00  DOWNSTREAM ELEVATION(FEET) =  485.00
 STREET LENGTH(FEET) =  1117.08   CURB HEIGHT(INCHES) =  8.0
 STREET HALFWIDTH(FEET) = 30.00

 DISTANCE FROM CROWN TO CROSSFALL GRADEBREAK(FEET) =  20.00
 INSIDE STREET CROSSFALL(DECIMAL) =  0.018
 OUTSIDE STREET CROSSFALL(DECIMAL)  =  0.018

 SPECIFIED NUMBER OF HALFSTREETS CARRYING RUNOFF =  1
 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) =      13.36
   STREETFLOW MODEL RESULTS USING ESTIMATED FLOW:
   STREET FLOW DEPTH(FEET) =  0.53
```

```
   HALFSTREET FLOOD WIDTH(FEET) =   20.59
   AVERAGE FLOW VELOCITY(FEET/SEC.) =     3.36
   PRODUCT OF DEPTH&VELOCITY(FT*FT/SEC.) =     1.78
 STREET FLOW TRAVEL TIME(MIN.) =    5.54   Tc(MIN.) =  10.49
  100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  4.411
 *USER SPECIFIED(SUBAREA):
 USER-SPECIFIED RUNOFF COEFFICIENT = .6500
 S.C.S. CURVE NUMBER (AMC II) =   0
 AREA-AVERAGE RUNOFF COEFFICIENT =  0.650
 SUBAREA AREA(ACRES) =     8.41      SUBAREA RUNOFF(CFS) =   24.11
 TOTAL AREA(ACRES) =       8.60          PEAK FLOW RATE(CFS) =      24.66

 END OF SUBAREA STREET FLOW HYDRAULICS:
 DEPTH(FEET) = 0.63   HALFSTREET FLOOD WIDTH(FEET) =  26.13
 FLOW VELOCITY(FEET/SEC.) =  3.92   DEPTH*VELOCITY(FT*FT/SEC.) =   2.47
 *NOTE: INITIAL SUBAREA NOMOGRAPH WITH SUBAREA PARAMETERS,
        AND L = 1117.1 FT WITH ELEVATION-DROP =  12.0 FT, IS   33.1 CFS,
        WHICH EXCEEDS THE TOP-OF-CURB STREET CAPACITY AT NODE    123.00
 LONGEST FLOWPATH FROM NODE       9.00 TO NODE    123.00 =  1187.08 FEET.

****************************************************************************
 FLOW PROCESS FROM NODE    123.00 TO NODE    124.00 IS CODE =  31
----------------------------------------------------------------------------
 >>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
 >>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<
============================================================================
 ELEVATION DATA: UPSTREAM(FEET) =   474.00  DOWNSTREAM(FEET) =   473.50
 FLOW LENGTH(FEET) =     13.00    MANNING'S N =  0.013
 DEPTH OF FLOW IN  24.0 INCH PIPE IS  16.0 INCHES
 PIPE-FLOW VELOCITY(FEET/SEC.) =  11.06
 ESTIMATED PIPE DIAMETER(INCH) =  24.00    NUMBER OF PIPES =   1
 PIPE-FLOW(CFS) =     24.66
 PIPE TRAVEL TIME(MIN.) =   0.02    Tc(MIN.) =  10.51
 LONGEST FLOWPATH FROM NODE       9.00 TO NODE    124.00 =  1200.08 FEET.

****************************************************************************
 FLOW PROCESS FROM NODE    124.00 TO NODE    124.00 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.) =   10.51
 RAINFALL INTENSITY(INCH/HR) =    4.41
 TOTAL STREAM AREA(ACRES) =      8.60
 PEAK FLOW RATE(CFS) AT CONFLUENCE =      24.66

****************************************************************************
 FLOW PROCESS FROM NODE     11.00 TO NODE     12.00 IS CODE =  21
----------------------------------------------------------------------------
 >>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<
============================================================================
 *USER SPECIFIED(SUBAREA):
 USER-SPECIFIED RUNOFF COEFFICIENT = .6500
 S.C.S. CURVE NUMBER (AMC II) =   0
 INITIAL SUBAREA FLOW-LENGTH(FEET) =    100.00
 UPSTREAM ELEVATION(FEET) =     530.00
 DOWNSTREAM ELEVATION(FEET) =     507.40
 ELEVATION DIFFERENCE(FEET) =      22.60
 SUBAREA OVERLAND TIME OF FLOW(MIN.) =     3.760
 WARNING: THE MAXIMUM OVERLAND FLOW SLOPE, 10.%, IS USED IN Tc CALCULATION!
  100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  7.114
 NOTE: RAINFALL INTENSITY IS BASED ON Tc = 5-MINUTE.
 SUBAREA RUNOFF(CFS) =      2.17
 TOTAL AREA(ACRES) =       0.47   TOTAL RUNOFF(CFS) =       2.17

****************************************************************************
 FLOW PROCESS FROM NODE     12.00 TO NODE    127.00 IS CODE =  62
----------------------------------------------------------------------------
 >>>>>COMPUTE STREET FLOW TRAVEL TIME THRU SUBAREA<<<<<
 >>>>>(STREET TABLE SECTION #  1 USED)<<<<<
============================================================================
 UPSTREAM ELEVATION(FEET) =  507.40  DOWNSTREAM ELEVATION(FEET) =  483.00
 STREET LENGTH(FEET) =  1152.23   CURB HEIGHT(INCHES) =  8.0
 STREET HALFWIDTH(FEET) = 30.00

 DISTANCE FROM CROWN TO CROSSFALL GRADEBREAK(FEET) =  20.00
 INSIDE STREET CROSSFALL(DECIMAL) =  0.018
 OUTSIDE STREET CROSSFALL(DECIMAL)  =  0.018

 SPECIFIED NUMBER OF HALFSTREETS CARRYING RUNOFF =  1
```

```
 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) =      21.01
   STREETFLOW MODEL RESULTS USING ESTIMATED FLOW:
   STREET FLOW DEPTH(FEET) =  0.55
   HALFSTREET FLOOD WIDTH(FEET) =   21.52
   AVERAGE FLOW VELOCITY(FEET/SEC.) =    4.85
   PRODUCT OF DEPTH&VELOCITY(FT*FT/SEC.) =    2.65
 STREET FLOW TRAVEL TIME(MIN.) =   3.96   Tc(MIN.) =   7.72
  100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  5.376
 *USER SPECIFIED(SUBAREA):
 USER-SPECIFIED RUNOFF COEFFICIENT = .6500
 S.C.S. CURVE NUMBER (AMC II) =   0
 AREA-AVERAGE RUNOFF COEFFICIENT =  0.650
 SUBAREA AREA(ACRES) =   10.58      SUBAREA RUNOFF(CFS) =   36.97
 TOTAL AREA(ACRES) =     11.05        PEAK FLOW RATE(CFS) =      38.62

 END OF SUBAREA STREET FLOW HYDRAULICS:
 DEPTH(FEET) = 0.65   HALFSTREET FLOOD WIDTH(FEET) =  27.30
 FLOW VELOCITY(FEET/SEC.) =  5.64   DEPTH*VELOCITY(FT*FT/SEC.) =   3.66
 *NOTE: INITIAL SUBAREA NOMOGRAPH WITH SUBAREA PARAMETERS,
       AND L = 1152.2 FT WITH ELEVATION-DROP =  24.4 FT, IS   45.0 CFS,
       WHICH EXCEEDS THE TOP-OF-CURB STREET CAPACITY AT NODE    127.00
 LONGEST FLOWPATH FROM NODE     11.00 TO NODE    127.00 =  1252.23 FEET.

****************************************************************************
 FLOW PROCESS FROM NODE    127.00 TO NODE    124.00 IS CODE =  31
----------------------------------------------------------------------------
 >>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
 >>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<
============================================================================
 ELEVATION DATA: UPSTREAM(FEET) =   475.00  DOWNSTREAM(FEET) =   473.50
 FLOW LENGTH(FEET) =   319.85    MANNING'S N =  0.013
 DEPTH OF FLOW IN  39.0 INCH PIPE IS  30.8 INCHES
 PIPE-FLOW VELOCITY(FEET/SEC.) =    5.49
 ESTIMATED PIPE DIAMETER(INCH) =  39.00    NUMBER OF PIPES =   1
 PIPE-FLOW(CFS) =      38.62
 PIPE TRAVEL TIME(MIN.) =   0.97    Tc(MIN.) =    8.69
 LONGEST FLOWPATH FROM NODE     11.00 TO NODE    124.00 =  1572.08 FEET.

****************************************************************************
 FLOW PROCESS FROM NODE    124.00 TO NODE    124.00 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.) =    8.69
 RAINFALL INTENSITY(INCH/HR) =   4.98
 TOTAL STREAM AREA(ACRES) =     11.05
 PEAK FLOW RATE(CFS) AT CONFLUENCE =     38.62

 ** CONFLUENCE DATA **
 STREAM     RUNOFF      Tc      INTENSITY      AREA
 NUMBER      (CFS)    (MIN.)   (INCH/HOUR)    (ACRE)
     1       24.66    10.51       4.406          8.60
     2       38.62     8.69       4.981         11.05

 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       59.00     8.69      4.981
     2       58.82    10.51      4.406

 COMPUTED CONFLUENCE ESTIMATES ARE AS FOLLOWS:
 PEAK FLOW RATE(CFS) =      59.00   Tc(MIN.) =     8.69
 TOTAL AREA(ACRES) =     19.65
 LONGEST FLOWPATH FROM NODE     11.00 TO NODE    124.00 =  1572.08 FEET.

****************************************************************************
 FLOW PROCESS FROM NODE    124.00 TO NODE    125.00 IS CODE =  31
----------------------------------------------------------------------------
 >>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
 >>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<
============================================================================
```

```
ELEVATION DATA: UPSTREAM(FEET) =    473.50  DOWNSTREAM(FEET) =    473.00
FLOW LENGTH(FEET) =     30.89    MANNING'S N =  0.013
DEPTH OF FLOW IN  36.0 INCH PIPE IS  29.0 INCHES
PIPE-FLOW VELOCITY(FEET/SEC.) =    9.68
ESTIMATED PIPE DIAMETER(INCH) =  36.00    NUMBER OF PIPES =    1
PIPE-FLOW(CFS) =       59.00
PIPE TRAVEL TIME(MIN.) =    0.05     Tc(MIN.) =    8.74
LONGEST FLOWPATH FROM NODE       11.00 TO NODE     125.00 =   1602.97 FEET.

****************************************************************************
FLOW PROCESS FROM NODE     125.00 TO NODE     125.00 IS CODE =  11
----------------------------------------------------------------------------
>>>>>CONFLUENCE MEMORY BANK # 1 WITH THE MAIN-STREAM MEMORY<<<<<
============================================================================

** MAIN STREAM CONFLUENCE DATA **
STREAM     RUNOFF      Tc        INTENSITY      AREA
NUMBER      (CFS)    (MIN.)    (INCH/HOUR)    (ACRE)
   1        59.00     8.74        4.961        19.65
LONGEST FLOWPATH FROM NODE       11.00 TO NODE     125.00 =   1602.97 FEET.

** MEMORY BANK #  1 CONFLUENCE DATA **
STREAM     RUNOFF      Tc        INTENSITY      AREA
NUMBER      (CFS)    (MIN.)    (INCH/HOUR)    (ACRE)
   1        69.99    11.65        4.122        23.61
LONGEST FLOWPATH FROM NODE        1.00 TO NODE     125.00 =   2070.14 FEET.

** PEAK FLOW RATE TABLE **
STREAM    RUNOFF        Tc       INTENSITY
NUMBER     (CFS)      (MIN.)    (INCH/HOUR)
    1      111.53       8.74         4.961
    2      119.02      11.65         4.122

COMPUTED CONFLUENCE ESTIMATES ARE AS FOLLOWS:
PEAK FLOW RATE(CFS) =      119.02    Tc(MIN.) =    11.65
TOTAL AREA(ACRES) =       43.26

****************************************************************************
FLOW PROCESS FROM NODE     125.00 TO NODE     125.00 IS CODE =  12
----------------------------------------------------------------------------
>>>>>CLEAR MEMORY BANK # 1 <<<<<
============================================================================

****************************************************************************
FLOW PROCESS FROM NODE     125.00 TO NODE     126.00 IS CODE =  31
----------------------------------------------------------------------------
>>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
>>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<
============================================================================
ELEVATION DATA: UPSTREAM(FEET) =    473.00  DOWNSTREAM(FEET) =    446.10
FLOW LENGTH(FEET) =    431.74    MANNING'S N =  0.013
DEPTH OF FLOW IN  39.0 INCH PIPE IS  26.8 INCHES
PIPE-FLOW VELOCITY(FEET/SEC.) =  19.59
ESTIMATED PIPE DIAMETER(INCH) =  39.00    NUMBER OF PIPES =    1
PIPE-FLOW(CFS) =      119.02
PIPE TRAVEL TIME(MIN.) =    0.37     Tc(MIN.) =  12.02
LONGEST FLOWPATH FROM NODE        1.00 TO NODE     126.00 =   2501.88 FEET.

****************************************************************************
FLOW PROCESS FROM NODE     126.00 TO NODE     126.00 IS CODE =   1
----------------------------------------------------------------------------
>>>>>DESIGNATE INDEPENDENT STREAM FOR CONFLUENCE<<<<<
============================================================================
TOTAL NUMBER OF STREAMS =  3
CONFLUENCE VALUES USED FOR INDEPENDENT STREAM  1 ARE:
TIME OF CONCENTRATION(MIN.) =  12.02
RAINFALL INTENSITY(INCH/HR) =   4.04
TOTAL STREAM AREA(ACRES) =     43.26
PEAK FLOW RATE(CFS) AT CONFLUENCE =      119.02

****************************************************************************
FLOW PROCESS FROM NODE      13.00 TO NODE      14.00 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) =      70.00
UPSTREAM ELEVATION(FEET) =     481.10
DOWNSTREAM ELEVATION(FEET) =     480.50
```

```
  ELEVATION DIFFERENCE(FEET) =        0.60
  SUBAREA OVERLAND TIME OF FLOW(MIN.) =     3.581
  WARNING: INITIAL SUBAREA FLOW PATH LENGTH IS GREATER THAN
           THE MAXIMUM OVERLAND FLOW LENGTH =     57.14
           (Reference: Table 3-1B of Hydrology Manual)
           THE MAXIMUM OVERLAND FLOW LENGTH IS USED IN Tc CALCULATION!
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  7.114
  NOTE: RAINFALL INTENSITY IS BASED ON Tc = 5-MINUTE.
  SUBAREA RUNOFF(CFS) =      2.54
  TOTAL AREA(ACRES) =      0.42   TOTAL RUNOFF(CFS) =      2.54

****************************************************************************
  FLOW PROCESS FROM NODE      14.00 TO NODE     128.00 IS CODE =  62
----------------------------------------------------------------------------
  >>>>>COMPUTE STREET FLOW TRAVEL TIME THRU SUBAREA<<<<<
  >>>>>(STREET TABLE SECTION #  2 USED)<<<<<
============================================================================
  UPSTREAM ELEVATION(FEET) =  480.50  DOWNSTREAM ELEVATION(FEET) =  456.10
  STREET LENGTH(FEET) =   438.05    CURB HEIGHT(INCHES) =  6.0
  STREET HALFWIDTH(FEET) = 60.00

  DISTANCE FROM CROWN TO CROSSFALL GRADEBREAK(FEET) =  30.00
  INSIDE STREET CROSSFALL(DECIMAL) =  0.020
  OUTSIDE STREET CROSSFALL(DECIMAL)  =  0.020

  SPECIFIED NUMBER OF HALFSTREETS CARRYING RUNOFF =  1
  Manning's FRICTION FACTOR for Streetflow Section(curb-to-curb) =   0.0160

    **TRAVEL TIME COMPUTED USING ESTIMATED FLOW(CFS) =        6.59
    STREETFLOW MODEL RESULTS USING ESTIMATED FLOW:
    STREET FLOW DEPTH(FEET) =  0.34
    HALFSTREET FLOOD WIDTH(FEET) =   10.78
    AVERAGE FLOW VELOCITY(FEET/SEC.) =     5.15
    PRODUCT OF DEPTH&VELOCITY(FT*FT/SEC.) =     1.76
  STREET FLOW TRAVEL TIME(MIN.) =   1.42   Tc(MIN.) =    5.00
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  7.114
  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) =     1.34      SUBAREA RUNOFF(CFS) =     8.10
  TOTAL AREA(ACRES) =      1.76         PEAK FLOW RATE(CFS) =      10.64

  END OF SUBAREA STREET FLOW HYDRAULICS:
  DEPTH(FEET) = 0.39   HALFSTREET FLOOD WIDTH(FEET) =  13.13
  FLOW VELOCITY(FEET/SEC.) =  5.78   DEPTH*VELOCITY(FT*FT/SEC.) =   2.25
  LONGEST FLOWPATH FROM NODE      13.00 TO NODE     128.00 =   508.05 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE     128.00 TO NODE     126.00 IS CODE =  31
----------------------------------------------------------------------------
  >>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
  >>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<
============================================================================
  ELEVATION DATA: UPSTREAM(FEET) =    446.10  DOWNSTREAM(FEET) =    445.50
  FLOW LENGTH(FEET) =     68.52    MANNING'S N =  0.013
  DEPTH OF FLOW IN  24.0 INCH PIPE IS  15.0 INCHES
  PIPE-FLOW VELOCITY(FEET/SEC.) =    5.17
  ESTIMATED PIPE DIAMETER(INCH) =  24.00    NUMBER OF PIPES =   1
  PIPE-FLOW(CFS) =      10.64
  PIPE TRAVEL TIME(MIN.) =   0.22    Tc(MIN.) =    5.22
  LONGEST FLOWPATH FROM NODE      13.00 TO NODE     126.00 =   576.57 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE     126.00 TO NODE     126.00 IS CODE =   1
----------------------------------------------------------------------------
  >>>>>DESIGNATE INDEPENDENT STREAM FOR CONFLUENCE<<<<<
============================================================================
  TOTAL NUMBER OF STREAMS =  3
  CONFLUENCE VALUES USED FOR INDEPENDENT STREAM  2 ARE:
  TIME OF CONCENTRATION(MIN.) =    5.22
  RAINFALL INTENSITY(INCH/HR) =    6.92
  TOTAL STREAM AREA(ACRES) =      1.76
  PEAK FLOW RATE(CFS) AT CONFLUENCE =     10.64

****************************************************************************
  FLOW PROCESS FROM NODE      15.00 TO NODE      16.00 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) =     70.00
UPSTREAM ELEVATION(FEET) =    481.10
DOWNSTREAM ELEVATION(FEET) =    480.50
ELEVATION DIFFERENCE(FEET) =      0.60
SUBAREA OVERLAND TIME OF FLOW(MIN.) =    3.581
WARNING: INITIAL SUBAREA FLOW PATH LENGTH IS GREATER THAN
         THE MAXIMUM OVERLAND FLOW LENGTH =    57.14
         (Reference: Table 3-1B of Hydrology Manual)
         THE MAXIMUM OVERLAND FLOW LENGTH IS USED IN Tc CALCULATION!
 100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  7.114
NOTE: RAINFALL INTENSITY IS BASED ON Tc = 5-MINUTE.
SUBAREA RUNOFF(CFS) =       2.06
TOTAL AREA(ACRES) =       0.34   TOTAL RUNOFF(CFS) =       2.06

****************************************************************************
  FLOW PROCESS FROM NODE     16.00 TO NODE    127.00 IS CODE =  62
----------------------------------------------------------------------------
  >>>>>COMPUTE STREET FLOW TRAVEL TIME THRU SUBAREA<<<<<
  >>>>>(STREET TABLE SECTION #  2 USED)<<<<<
============================================================================
  UPSTREAM ELEVATION(FEET) =  480.50  DOWNSTREAM ELEVATION(FEET) =  456.10
  STREET LENGTH(FEET) =   433.11   CURB HEIGHT(INCHES) =  6.0
  STREET HALFWIDTH(FEET) = 60.00

  DISTANCE FROM CROWN TO CROSSFALL GRADEBREAK(FEET) =  30.00
  INSIDE STREET CROSSFALL(DECIMAL) =  0.020
  OUTSIDE STREET CROSSFALL(DECIMAL)  =  0.020

  SPECIFIED NUMBER OF HALFSTREETS CARRYING RUNOFF =  1
  Manning's FRICTION FACTOR for Streetflow Section(curb-to-curb) =   0.0160

    **TRAVEL TIME COMPUTED USING ESTIMATED FLOW(CFS) =       6.68
    STREETFLOW MODEL RESULTS USING ESTIMATED FLOW:
    STREET FLOW DEPTH(FEET) =  0.34
    HALFSTREET FLOOD WIDTH(FEET) =   10.78
    AVERAGE FLOW VELOCITY(FEET/SEC.) =    5.22
    PRODUCT OF DEPTH&VELOCITY(FT*FT/SEC.) =    1.78
  STREET FLOW TRAVEL TIME(MIN.) =   1.38   Tc(MIN.) =   4.96
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  7.114
  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) =      1.53      SUBAREA RUNOFF(CFS) =     9.25
  TOTAL AREA(ACRES) =       1.87         PEAK FLOW RATE(CFS) =      11.31

  END OF SUBAREA STREET FLOW HYDRAULICS:
  DEPTH(FEET) = 0.39   HALFSTREET FLOOD WIDTH(FEET) =  13.41
  FLOW VELOCITY(FEET/SEC.) =  5.90   DEPTH*VELOCITY(FT*FT/SEC.) =   2.33
  LONGEST FLOWPATH FROM NODE      15.00 TO NODE     127.00 =   503.11 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    127.00 TO NODE    126.00 IS CODE =  31
----------------------------------------------------------------------------
  >>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
  >>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<
============================================================================
  ELEVATION DATA: UPSTREAM(FEET) =   446.10  DOWNSTREAM(FEET) =   445.40
  FLOW LENGTH(FEET) =    47.36   MANNING'S N =  0.013
  DEPTH OF FLOW IN  21.0 INCH PIPE IS  14.6 INCHES
  PIPE-FLOW VELOCITY(FEET/SEC.) =   6.33
  ESTIMATED PIPE DIAMETER(INCH) =  21.00    NUMBER OF PIPES =   1
  PIPE-FLOW(CFS) =      11.31
  PIPE TRAVEL TIME(MIN.) =   0.12    Tc(MIN.) =    5.09
  LONGEST FLOWPATH FROM NODE      15.00 TO NODE     126.00 =   550.47 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    126.00 TO NODE    126.00 IS CODE =   1
----------------------------------------------------------------------------
  >>>>>DESIGNATE INDEPENDENT STREAM FOR CONFLUENCE<<<<<
  >>>>>AND COMPUTE VARIOUS CONFLUENCED STREAM VALUES<<<<<
============================================================================
  TOTAL NUMBER OF STREAMS =  3
  CONFLUENCE VALUES USED FOR INDEPENDENT STREAM  3 ARE:
  TIME OF CONCENTRATION(MIN.) =    5.09
  RAINFALL INTENSITY(INCH/HR) =    7.03
  TOTAL STREAM AREA(ACRES) =      1.87
```

```
 PEAK FLOW RATE(CFS) AT CONFLUENCE =     11.31

 ** CONFLUENCE DATA **
 STREAM     RUNOFF      Tc       INTENSITY     AREA
 NUMBER      (CFS)     (MIN.)   (INCH/HOUR)   (ACRE)
    1       119.02    12.02        4.041        43.26
    2        10.64     5.22        6.918         1.76
    3        11.31     5.09        7.033         1.87

 RAINFALL INTENSITY AND TIME OF CONCENTRATION RATIO
 CONFLUENCE FORMULA USED FOR  3 STREAMS.

 ** PEAK FLOW RATE TABLE **
 STREAM     RUNOFF      Tc       INTENSITY
 NUMBER      (CFS)     (MIN.)   (INCH/HOUR)
    1        72.09     5.09        7.033
    2        73.47     5.22        6.918
    3       131.73    12.02        4.041

 COMPUTED CONFLUENCE ESTIMATES ARE AS FOLLOWS:
 PEAK FLOW RATE(CFS) =     131.73   Tc(MIN.) =    12.02
 TOTAL AREA(ACRES) =     46.89
 LONGEST FLOWPATH FROM NODE       1.00 TO NODE     126.00 =  2501.88 FEET.

****************************************************************************
 FLOW PROCESS FROM NODE     126.00 TO NODE     130.00 IS CODE =  31
----------------------------------------------------------------------------
 >>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
 >>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<
============================================================================
 ELEVATION DATA: UPSTREAM(FEET) =   445.50  DOWNSTREAM(FEET) =   398.00
 FLOW LENGTH(FEET) =  1061.58   MANNING'S N =  0.013
 DEPTH OF FLOW IN  42.0 INCH PIPE IS  30.4 INCHES
 PIPE-FLOW VELOCITY(FEET/SEC.) =  17.63
 ESTIMATED PIPE DIAMETER(INCH) =  42.00    NUMBER OF PIPES =   1
 PIPE-FLOW(CFS) =     131.73
 PIPE TRAVEL TIME(MIN.) =    1.00    Tc(MIN.) =   13.02
 LONGEST FLOWPATH FROM NODE       1.00 TO NODE     130.00 =  3563.46 FEET.

****************************************************************************
 FLOW PROCESS FROM NODE     130.00 TO NODE     130.00 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.) =  13.02
 RAINFALL INTENSITY(INCH/HR) =   3.84
 TOTAL STREAM AREA(ACRES) =     46.89
 PEAK FLOW RATE(CFS) AT CONFLUENCE =    131.73

****************************************************************************
 FLOW PROCESS FROM NODE      17.00 TO NODE      18.00 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) =     465.00
 DOWNSTREAM ELEVATION(FEET) =     445.00
 ELEVATION DIFFERENCE(FEET) =      20.00
 SUBAREA OVERLAND TIME OF FLOW(MIN.) =    2.089
 WARNING: THE MAXIMUM OVERLAND FLOW SLOPE, 10.%, IS USED IN Tc CALCULATION!
  100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  7.114
 NOTE: RAINFALL INTENSITY IS BASED ON Tc = 5-MINUTE.
 SUBAREA RUNOFF(CFS) =       2.96
 TOTAL AREA(ACRES) =       0.49   TOTAL RUNOFF(CFS) =       2.96

****************************************************************************
 FLOW PROCESS FROM NODE      18.00 TO NODE     129.00 IS CODE =  62
----------------------------------------------------------------------------
 >>>>>COMPUTE STREET FLOW TRAVEL TIME THRU SUBAREA<<<<<
 >>>>>(STREET TABLE SECTION #  2 USED)<<<<<
============================================================================
 UPSTREAM ELEVATION(FEET) =  445.00  DOWNSTREAM ELEVATION(FEET) =  399.00
 STREET LENGTH(FEET) =    875.78   CURB HEIGHT(INCHES) =  6.0
 STREET HALFWIDTH(FEET) = 60.00

 DISTANCE FROM CROWN TO CROSSFALL GRADEBREAK(FEET) =  30.00
```

```
 INSIDE STREET CROSSFALL(DECIMAL) =  0.020
 OUTSIDE STREET CROSSFALL(DECIMAL)  =  0.020

 SPECIFIED NUMBER OF HALFSTREETS CARRYING RUNOFF =  1
 Manning's FRICTION FACTOR for Streetflow Section(curb-to-curb) =   0.0160

   **TRAVEL TIME COMPUTED USING ESTIMATED FLOW(CFS) =      15.12
   STREETFLOW MODEL RESULTS USING ESTIMATED FLOW:
   STREET FLOW DEPTH(FEET) =  0.43
   HALFSTREET FLOOD WIDTH(FEET) =  15.30
   AVERAGE FLOW VELOCITY(FEET/SEC.) =    6.15
   PRODUCT OF DEPTH&VELOCITY(FT*FT/SEC.) =    2.66
 STREET FLOW TRAVEL TIME(MIN.) =   2.37   Tc(MIN.) =   4.46
  100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  7.114
 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) =      4.02      SUBAREA RUNOFF(CFS) =   24.31
 TOTAL AREA(ACRES) =        4.51        PEAK FLOW RATE(CFS) =       27.27

 END OF SUBAREA STREET FLOW HYDRAULICS:
 DEPTH(FEET) = 0.51   HALFSTREET FLOOD WIDTH(FEET) =  19.30
 FLOW VELOCITY(FEET/SEC.) =  7.10   DEPTH*VELOCITY(FT*FT/SEC.) =   3.64
 LONGEST FLOWPATH FROM NODE      17.00 TO NODE     129.00 =   975.78 FEET.

****************************************************************************
 FLOW PROCESS FROM NODE    129.00 TO NODE    130.00 IS CODE =  31
----------------------------------------------------------------------------
 >>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
 >>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<
============================================================================
 ELEVATION DATA: UPSTREAM(FEET) =   399.00  DOWNSTREAM(FEET) =   398.00
 FLOW LENGTH(FEET) =     48.00   MANNING'S N =  0.013
 DEPTH OF FLOW IN  27.0 INCH PIPE IS  19.3 INCHES
 PIPE-FLOW VELOCITY(FEET/SEC.) =   8.94
 ESTIMATED PIPE DIAMETER(INCH) =  27.00    NUMBER OF PIPES =   1
 PIPE-FLOW(CFS) =      27.27
 PIPE TRAVEL TIME(MIN.) =    0.09    Tc(MIN.) =    4.55
 LONGEST FLOWPATH FROM NODE      17.00 TO NODE     130.00 =  1023.78 FEET.

****************************************************************************
 FLOW PROCESS FROM NODE    130.00 TO NODE    130.00 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.) =    4.55
 RAINFALL INTENSITY(INCH/HR) =    7.11
 TOTAL STREAM AREA(ACRES) =      4.51
 PEAK FLOW RATE(CFS) AT CONFLUENCE =      27.27

 ** CONFLUENCE DATA **
 STREAM      RUNOFF       Tc       INTENSITY       AREA
 NUMBER       (CFS)     (MIN.)    (INCH/HOUR)     (ACRE)
     1       131.73     13.02         3.837          46.89
     2        27.27      4.55         7.114           4.51

 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        98.33     4.55        7.114
     2       146.44    13.02        3.837

 COMPUTED CONFLUENCE ESTIMATES ARE AS FOLLOWS:
 PEAK FLOW RATE(CFS) =     146.44   Tc(MIN.) =    13.02
 TOTAL AREA(ACRES) =      51.40
 LONGEST FLOWPATH FROM NODE       1.00 TO NODE     130.00 =  3563.46 FEET.

****************************************************************************
 FLOW PROCESS FROM NODE    130.00 TO NODE    131.00 IS CODE =  31
----------------------------------------------------------------------------
 >>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
 >>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<
============================================================================
```

```
 ELEVATION DATA: UPSTREAM(FEET) =   398.00  DOWNSTREAM(FEET) =   394.80
 FLOW LENGTH(FEET) =   193.81   MANNING'S N =  0.013
 DEPTH OF FLOW IN  51.0 INCH PIPE IS  39.9 INCHES
 PIPE-FLOW VELOCITY(FEET/SEC.) =  12.31
 ESTIMATED PIPE DIAMETER(INCH) =  51.00    NUMBER OF PIPES =   1
 PIPE-FLOW(CFS) =    146.44
 PIPE TRAVEL TIME(MIN.) =   0.26    Tc(MIN.) =  13.28
 LONGEST FLOWPATH FROM NODE      1.00 TO NODE    131.00 =  3757.27 FEET.

****************************************************************************
 FLOW PROCESS FROM NODE    131.00 TO NODE    131.00 IS CODE =  10
----------------------------------------------------------------------------
 >>>>>MAIN-STREAM MEMORY COPIED ONTO MEMORY BANK # 2 <<<<<
============================================================================

****************************************************************************
 FLOW PROCESS FROM NODE     27.00 TO NODE     28.00 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) =     70.00
 UPSTREAM ELEVATION(FEET) =     460.90
 DOWNSTREAM ELEVATION(FEET) =     460.00
 ELEVATION DIFFERENCE(FEET) =       0.90
 SUBAREA OVERLAND TIME OF FLOW(MIN.) =     3.281
 WARNING: INITIAL SUBAREA FLOW PATH LENGTH IS GREATER THAN
          THE MAXIMUM OVERLAND FLOW LENGTH =     62.86
          (Reference: Table 3-1B of Hydrology Manual)
          THE MAXIMUM OVERLAND FLOW LENGTH IS USED IN Tc CALCULATION!
  100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  7.114
 NOTE: RAINFALL INTENSITY IS BASED ON Tc = 5-MINUTE.
 SUBAREA RUNOFF(CFS) =      2.12
 TOTAL AREA(ACRES) =      0.35   TOTAL RUNOFF(CFS) =      2.12

****************************************************************************
 FLOW PROCESS FROM NODE     28.00 TO NODE    136.00 IS CODE =  62
----------------------------------------------------------------------------
 >>>>>COMPUTE STREET FLOW TRAVEL TIME THRU SUBAREA<<<<<
 >>>>>(STREET TABLE SECTION #  2 USED)<<<<<
============================================================================
 UPSTREAM ELEVATION(FEET) =  460.00  DOWNSTREAM ELEVATION(FEET) =  432.00
 STREET LENGTH(FEET) =   820.38   CURB HEIGHT(INCHES) =  6.0
 STREET HALFWIDTH(FEET) = 60.00

 DISTANCE FROM CROWN TO CROSSFALL GRADEBREAK(FEET) =  30.00
 INSIDE STREET CROSSFALL(DECIMAL) =  0.020
 OUTSIDE STREET CROSSFALL(DECIMAL)  =  0.020

 SPECIFIED NUMBER OF HALFSTREETS CARRYING RUNOFF =  1
 Manning's FRICTION FACTOR for Streetflow Section(curb-to-curb) =   0.0160

   **TRAVEL TIME COMPUTED USING ESTIMATED FLOW(CFS) =      13.83
   STREETFLOW MODEL RESULTS USING ESTIMATED FLOW:
   STREET FLOW DEPTH(FEET) =  0.45
   HALFSTREET FLOOD WIDTH(FEET) =    16.10
   AVERAGE FLOW VELOCITY(FEET/SEC.) =    5.11
   PRODUCT OF DEPTH&VELOCITY(FT*FT/SEC.) =    2.29
 STREET FLOW TRAVEL TIME(MIN.) =   2.68   Tc(MIN.) =    5.96
  100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  6.353
 *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) =     4.31      SUBAREA RUNOFF(CFS) =   23.27
 TOTAL AREA(ACRES) =      4.66        PEAK FLOW RATE(CFS) =      25.16

 END OF SUBAREA STREET FLOW HYDRAULICS:
 DEPTH(FEET) = 0.53   HALFSTREET FLOOD WIDTH(FEET) =  20.32
 FLOW VELOCITY(FEET/SEC.) =  5.92   DEPTH*VELOCITY(FT*FT/SEC.) =   3.16
 *NOTE: INITIAL SUBAREA NOMOGRAPH WITH SUBAREA PARAMETERS,
        AND L =  820.4 FT WITH ELEVATION-DROP =  28.0 FT, IS   26.1 CFS,
        WHICH EXCEEDS THE TOP-OF-CURB STREET CAPACITY AT NODE    136.00
 LONGEST FLOWPATH FROM NODE     27.00 TO NODE    136.00 =   890.38 FEET.

****************************************************************************
 FLOW PROCESS FROM NODE    136.00 TO NODE    134.00 IS CODE =  31
----------------------------------------------------------------------------
 >>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
```

```
>>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<
==========================================================================
 ELEVATION DATA: UPSTREAM(FEET) =   422.00  DOWNSTREAM(FEET) =   410.00
 FLOW LENGTH(FEET) =   309.71   MANNING'S N =  0.013
 DEPTH OF FLOW IN  24.0 INCH PIPE IS  16.2 INCHES
 PIPE-FLOW VELOCITY(FEET/SEC.) =  11.13
 ESTIMATED PIPE DIAMETER(INCH) =  24.00    NUMBER OF PIPES =   1
 PIPE-FLOW(CFS) =     25.16
 PIPE TRAVEL TIME(MIN.) =   0.46    Tc(MIN.) =    6.42
 LONGEST FLOWPATH FROM NODE     27.00 TO NODE    134.00 =  1200.09 FEET.

****************************************************************************
 FLOW PROCESS FROM NODE    134.00 TO NODE    134.00 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.) =   6.42
 RAINFALL INTENSITY(INCH/HR) =   6.05
 TOTAL STREAM AREA(ACRES) =     4.66
 PEAK FLOW RATE(CFS) AT CONFLUENCE =     25.16

****************************************************************************
 FLOW PROCESS FROM NODE     25.00 TO NODE     26.00 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) =    70.00
 UPSTREAM ELEVATION(FEET) =    460.90
 DOWNSTREAM ELEVATION(FEET) =    460.00
 ELEVATION DIFFERENCE(FEET) =      0.90
 SUBAREA OVERLAND TIME OF FLOW(MIN.) =    3.281
 WARNING: INITIAL SUBAREA FLOW PATH LENGTH IS GREATER THAN
          THE MAXIMUM OVERLAND FLOW LENGTH =    62.86
          (Reference: Table 3-1B of Hydrology Manual)
          THE MAXIMUM OVERLAND FLOW LENGTH IS USED IN Tc CALCULATION!
  100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  7.114
 NOTE: RAINFALL INTENSITY IS BASED ON Tc = 5-MINUTE.
 SUBAREA RUNOFF(CFS) =     26.61
 TOTAL AREA(ACRES) =      4.40   TOTAL RUNOFF(CFS) =     26.61

****************************************************************************
 FLOW PROCESS FROM NODE     26.00 TO NODE    135.00 IS CODE =  62
----------------------------------------------------------------------------
 >>>>>COMPUTE STREET FLOW TRAVEL TIME THRU SUBAREA<<<<<
 >>>>>(STREET TABLE SECTION #  2 USED)<<<<<
============================================================================
 UPSTREAM ELEVATION(FEET) =  460.00  DOWNSTREAM ELEVATION(FEET) =  422.00
 STREET LENGTH(FEET) =  1051.15   CURB HEIGHT(INCHES) =  6.0
 STREET HALFWIDTH(FEET) = 60.00

 DISTANCE FROM CROWN TO CROSSFALL GRADEBREAK(FEET) =  30.00
 INSIDE STREET CROSSFALL(DECIMAL) =  0.020
 OUTSIDE STREET CROSSFALL(DECIMAL)  =  0.020

 SPECIFIED NUMBER OF HALFSTREETS CARRYING RUNOFF =  1
 Manning's FRICTION FACTOR for Streetflow Section(curb-to-curb) =   0.0160

   **TRAVEL TIME COMPUTED USING ESTIMATED FLOW(CFS) =      38.62
   STREETFLOW MODEL RESULTS USING ESTIMATED FLOW:
        NOTE: STREET FLOW EXCEEDS TOP OF CURB.
        THE FOLLOWING STREET FLOW RESULTS ARE BASED ON THE ASSUMPTION
        THAT NEGLIBLE FLOW OCCURS OUTSIDE OF THE STREET CHANNEL.
        THAT IS, ALL FLOW ALONG THE PARKWAY, ETC., IS NEGLECTED.
   STREET FLOW DEPTH(FEET) =  0.60
   HALFSTREET FLOOD WIDTH(FEET) =   23.69
   AVERAGE FLOW VELOCITY(FEET/SEC.) =    6.74
   PRODUCT OF DEPTH&VELOCITY(FT*FT/SEC.) =    4.04
 STREET FLOW TRAVEL TIME(MIN.) =   2.60   Tc(MIN.) =    5.88
  100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  6.406
 *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) =    4.40      SUBAREA RUNOFF(CFS) =   23.96
 TOTAL AREA(ACRES) =      8.80        PEAK FLOW RATE(CFS) =      47.92
```

```
  END OF SUBAREA STREET FLOW HYDRAULICS:
  DEPTH(FEET) = 0.64   HALFSTREET FLOOD WIDTH(FEET) =  25.69
  FLOW VELOCITY(FEET/SEC.) =  7.13   DEPTH*VELOCITY(FT*FT/SEC.) =   4.56
  *NOTE: INITIAL SUBAREA NOMOGRAPH WITH SUBAREA PARAMETERS,
         AND L = 1051.2 FT WITH ELEVATION-DROP =  38.0 FT, IS   26.6 CFS,
         WHICH EXCEEDS THE TOP-OF-CURB STREET CAPACITY AT NODE    135.00
  LONGEST FLOWPATH FROM NODE     25.00 TO NODE    135.00 =  1121.15 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    135.00 TO NODE    134.00 IS CODE =  31
----------------------------------------------------------------------------
  >>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
  >>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<
============================================================================
  ELEVATION DATA: UPSTREAM(FEET) =   423.00  DOWNSTREAM(FEET) =   410.00
  FLOW LENGTH(FEET) =    50.46   MANNING'S N =  0.013
  DEPTH OF FLOW IN  21.0 INCH PIPE IS  14.8 INCHES
  PIPE-FLOW VELOCITY(FEET/SEC.) =  26.50
  ESTIMATED PIPE DIAMETER(INCH) =  21.00    NUMBER OF PIPES =   1
  PIPE-FLOW(CFS) =      47.92
  PIPE TRAVEL TIME(MIN.) =   0.03    Tc(MIN.) =    5.91
  LONGEST FLOWPATH FROM NODE     25.00 TO NODE    134.00 =  1171.61 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    134.00 TO NODE    134.00 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.) =   5.91
  RAINFALL INTENSITY(INCH/HR) =   6.38
  TOTAL STREAM AREA(ACRES) =     8.80
  PEAK FLOW RATE(CFS) AT CONFLUENCE =     47.92

  ** CONFLUENCE DATA **
  STREAM     RUNOFF      Tc       INTENSITY      AREA
  NUMBER      (CFS)     (MIN.)   (INCH/HOUR)    (ACRE)
     1        25.16     6.42        6.053          4.66
     2        47.92     5.91        6.384          8.80

  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        71.09     5.91       6.384
     2        70.60     6.42       6.053

  COMPUTED CONFLUENCE ESTIMATES ARE AS FOLLOWS:
  PEAK FLOW RATE(CFS) =      71.09   Tc(MIN.) =    5.91
  TOTAL AREA(ACRES) =      13.46
  LONGEST FLOWPATH FROM NODE     27.00 TO NODE    134.00 =  1200.09 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    134.00 TO NODE    131.00 IS CODE =  31
----------------------------------------------------------------------------
  >>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
  >>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<
============================================================================
  ELEVATION DATA: UPSTREAM(FEET) =   410.00  DOWNSTREAM(FEET) =   394.80
  FLOW LENGTH(FEET) =   625.51   MANNING'S N =  0.013
  DEPTH OF FLOW IN  36.0 INCH PIPE IS  28.5 INCHES
  PIPE-FLOW VELOCITY(FEET/SEC.) =  11.85
  ESTIMATED PIPE DIAMETER(INCH) =  36.00    NUMBER OF PIPES =   1
  PIPE-FLOW(CFS) =      71.09
  PIPE TRAVEL TIME(MIN.) =   0.88    Tc(MIN.) =    6.79
  LONGEST FLOWPATH FROM NODE     27.00 TO NODE    131.00 =  1825.60 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    131.00 TO NODE    131.00 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.) =   6.79
  RAINFALL INTENSITY(INCH/HR) =   5.84
  TOTAL STREAM AREA(ACRES) =     13.46
```

```
  PEAK FLOW RATE(CFS) AT CONFLUENCE =     71.09

****************************************************************************
  FLOW PROCESS FROM NODE      21.00 TO NODE      22.00 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) =    440.00
  DOWNSTREAM ELEVATION(FEET) =    420.00
  ELEVATION DIFFERENCE(FEET) =     20.00
  SUBAREA OVERLAND TIME OF FLOW(MIN.) =    2.089
  WARNING: THE MAXIMUM OVERLAND FLOW SLOPE, 10.%, IS USED IN Tc CALCULATION!
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  7.114
  NOTE: RAINFALL INTENSITY IS BASED ON Tc = 5-MINUTE.
  SUBAREA RUNOFF(CFS) =      1.87
  TOTAL AREA(ACRES) =      0.31   TOTAL RUNOFF(CFS) =      1.87

****************************************************************************
  FLOW PROCESS FROM NODE      22.00 TO NODE     132.00 IS CODE =  62
----------------------------------------------------------------------------
  >>>>>COMPUTE STREET FLOW TRAVEL TIME THRU SUBAREA<<<<<
  >>>>>(STREET TABLE SECTION #  2 USED)<<<<<
============================================================================
  UPSTREAM ELEVATION(FEET) =  420.00  DOWNSTREAM ELEVATION(FEET) =  395.30
  STREET LENGTH(FEET) =   516.45   CURB HEIGHT(INCHES) =  6.0
  STREET HALFWIDTH(FEET) = 60.00

  DISTANCE FROM CROWN TO CROSSFALL GRADEBREAK(FEET) =  30.00
  INSIDE STREET CROSSFALL(DECIMAL) =  0.020
  OUTSIDE STREET CROSSFALL(DECIMAL)  =  0.020

  SPECIFIED NUMBER OF HALFSTREETS CARRYING RUNOFF =  1
  Manning's FRICTION FACTOR for Streetflow Section(curb-to-curb) =   0.0160

    **TRAVEL TIME COMPUTED USING ESTIMATED FLOW(CFS) =       5.68
    STREETFLOW MODEL RESULTS USING ESTIMATED FLOW:
    STREET FLOW DEPTH(FEET) =  0.34
    HALFSTREET FLOOD WIDTH(FEET) =   10.44
    AVERAGE FLOW VELOCITY(FEET/SEC.) =    4.70
    PRODUCT OF DEPTH&VELOCITY(FT*FT/SEC.) =    1.58
  STREET FLOW TRAVEL TIME(MIN.) =   1.83   Tc(MIN.) =   3.92
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  7.114
  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) =    1.26     SUBAREA RUNOFF(CFS) =    7.62
  TOTAL AREA(ACRES) =      1.57        PEAK FLOW RATE(CFS) =       9.49

  END OF SUBAREA STREET FLOW HYDRAULICS:
  DEPTH(FEET) = 0.38   HALFSTREET FLOOD WIDTH(FEET) =  12.90
  FLOW VELOCITY(FEET/SEC.) =  5.33   DEPTH*VELOCITY(FT*FT/SEC.) =   2.05
  LONGEST FLOWPATH FROM NODE      21.00 TO NODE     132.00 =   616.45 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE     132.00 TO NODE     131.00 IS CODE =  31
----------------------------------------------------------------------------
  >>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
  >>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<
============================================================================
  ELEVATION DATA: UPSTREAM(FEET) =    395.30  DOWNSTREAM(FEET) =   394.80
  FLOW LENGTH(FEET) =     27.10   MANNING'S N =  0.013
  DEPTH OF FLOW IN  18.0 INCH PIPE IS  13.9 INCHES
  PIPE-FLOW VELOCITY(FEET/SEC.) =   6.49
  ESTIMATED PIPE DIAMETER(INCH) =  18.00    NUMBER OF PIPES =   1
  PIPE-FLOW(CFS) =       9.49
  PIPE TRAVEL TIME(MIN.) =   0.07    Tc(MIN.) =   3.99
  LONGEST FLOWPATH FROM NODE      21.00 TO NODE     131.00 =   643.55 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE     131.00 TO NODE     131.00 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.) =    3.99
 RAINFALL INTENSITY(INCH/HR) =    7.11
 TOTAL STREAM AREA(ACRES) =      1.57
 PEAK FLOW RATE(CFS) AT CONFLUENCE =       9.49

 ** CONFLUENCE DATA **
 STREAM      RUNOFF       Tc       INTENSITY       AREA
 NUMBER       (CFS)     (MIN.)    (INCH/HOUR)     (ACRE)
     1        71.09      6.79        5.838          13.46
     2         9.49      3.99        7.114           1.57

 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        67.83      3.99        7.114
     2        78.88      6.79        5.838

 COMPUTED CONFLUENCE ESTIMATES ARE AS FOLLOWS:
 PEAK FLOW RATE(CFS) =      78.88   Tc(MIN.) =      6.79
 TOTAL AREA(ACRES) =      15.03
 LONGEST FLOWPATH FROM NODE      27.00 TO NODE     131.00 =  1825.60 FEET.

****************************************************************************
 FLOW PROCESS FROM NODE     131.00 TO NODE     131.00 IS CODE =  11
----------------------------------------------------------------------------
 >>>>>CONFLUENCE MEMORY BANK # 2 WITH THE MAIN-STREAM MEMORY<<<<<
============================================================================

 ** MAIN STREAM CONFLUENCE DATA **
 STREAM      RUNOFF      Tc       INTENSITY      AREA
 NUMBER       (CFS)    (MIN.)    (INCH/HOUR)    (ACRE)
    1        78.88      6.79       5.838         15.03
 LONGEST FLOWPATH FROM NODE      27.00 TO NODE     131.00 =  1825.60 FEET.

 ** MEMORY BANK #  2 CONFLUENCE DATA **
 STREAM      RUNOFF      Tc       INTENSITY      AREA
 NUMBER       (CFS)    (MIN.)    (INCH/HOUR)    (ACRE)
    1       146.44    13.28       3.788         51.40
 LONGEST FLOWPATH FROM NODE       1.00 TO NODE     131.00 =  3757.27 FEET.

 ** PEAK FLOW RATE TABLE **
 STREAM     RUNOFF        Tc       INTENSITY
 NUMBER      (CFS)      (MIN.)    (INCH/HOUR)
     1      153.77       6.79        5.838
     2      197.63      13.28        3.788

 COMPUTED CONFLUENCE ESTIMATES ARE AS FOLLOWS:
 PEAK FLOW RATE(CFS) =     197.63   Tc(MIN.) =    13.28
 TOTAL AREA(ACRES) =      66.43

****************************************************************************
 FLOW PROCESS FROM NODE     131.00 TO NODE     131.00 IS CODE =  12
----------------------------------------------------------------------------
 >>>>>CLEAR MEMORY BANK # 2 <<<<<
============================================================================

****************************************************************************
 FLOW PROCESS FROM NODE     131.00 TO NODE     133.00 IS CODE =  31
----------------------------------------------------------------------------
 >>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
 >>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<
============================================================================
 ELEVATION DATA: UPSTREAM(FEET) =   394.80  DOWNSTREAM(FEET) =   393.00
 FLOW LENGTH(FEET) =     87.10   MANNING'S N =  0.013
 DEPTH OF FLOW IN  54.0 INCH PIPE IS  43.7 INCHES
 PIPE-FLOW VELOCITY(FEET/SEC.) =  14.33
 ESTIMATED PIPE DIAMETER(INCH) =  54.00    NUMBER OF PIPES =   1
 PIPE-FLOW(CFS) =    197.63
 PIPE TRAVEL TIME(MIN.) =   0.10    Tc(MIN.) =  13.38
 LONGEST FLOWPATH FROM NODE       1.00 TO NODE     133.00 =  3844.37 FEET.

****************************************************************************
 FLOW PROCESS FROM NODE     133.00 TO NODE     133.00 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.) =  13.38
 RAINFALL INTENSITY(INCH/HR) =   3.77
 TOTAL STREAM AREA(ACRES) =    66.43
 PEAK FLOW RATE(CFS) AT CONFLUENCE =    197.63

****************************************************************************
 FLOW PROCESS FROM NODE     19.00 TO NODE     20.00 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) =    488.00
 DOWNSTREAM ELEVATION(FEET) =    450.00
 ELEVATION DIFFERENCE(FEET) =     38.00
 SUBAREA OVERLAND TIME OF FLOW(MIN.) =    2.089
 WARNING: THE MAXIMUM OVERLAND FLOW SLOPE, 10.%, IS USED IN Tc CALCULATION!
  100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  7.114
 NOTE: RAINFALL INTENSITY IS BASED ON Tc = 5-MINUTE.
 SUBAREA RUNOFF(CFS) =      2.60
 TOTAL AREA(ACRES) =      0.43   TOTAL RUNOFF(CFS) =      2.60

****************************************************************************
 FLOW PROCESS FROM NODE     20.00 TO NODE    133.00 IS CODE =  62
----------------------------------------------------------------------------
 >>>>>COMPUTE STREET FLOW TRAVEL TIME THRU SUBAREA<<<<<
 >>>>>(STREET TABLE SECTION #  2 USED)<<<<<
============================================================================
 UPSTREAM ELEVATION(FEET) =  450.00  DOWNSTREAM ELEVATION(FEET) =  404.80
 STREET LENGTH(FEET) =  1124.94   CURB HEIGHT(INCHES) =  6.0
 STREET HALFWIDTH(FEET) = 60.00

 DISTANCE FROM CROWN TO CROSSFALL GRADEBREAK(FEET) =  30.00
 INSIDE STREET CROSSFALL(DECIMAL) =  0.020
 OUTSIDE STREET CROSSFALL(DECIMAL)  =  0.020

 SPECIFIED NUMBER OF HALFSTREETS CARRYING RUNOFF =  1
 Manning's FRICTION FACTOR for Streetflow Section(curb-to-curb) =   0.0160

   **TRAVEL TIME COMPUTED USING ESTIMATED FLOW(CFS) =      31.41
   STREETFLOW MODEL RESULTS USING ESTIMATED FLOW:
        NOTE: STREET FLOW EXCEEDS TOP OF CURB.
        THE FOLLOWING STREET FLOW RESULTS ARE BASED ON THE ASSUMPTION
        THAT NEGLIBLE FLOW OCCURS OUTSIDE OF THE STREET CHANNEL.
        THAT IS, ALL FLOW ALONG THE PARKWAY, ETC., IS NEGLECTED.
   STREET FLOW DEPTH(FEET) =  0.55
   HALFSTREET FLOOD WIDTH(FEET) =   21.41
   AVERAGE FLOW VELOCITY(FEET/SEC.) =    6.68
   PRODUCT OF DEPTH&VELOCITY(FT*FT/SEC.) =    3.70
 STREET FLOW TRAVEL TIME(MIN.) =   2.81   Tc(MIN.) =   4.90
  100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  7.114
 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) =     9.53      SUBAREA RUNOFF(CFS) =   57.63
 TOTAL AREA(ACRES) =      9.96        PEAK FLOW RATE(CFS) =      60.23

 END OF SUBAREA STREET FLOW HYDRAULICS:
 DEPTH(FEET) = 0.68   HALFSTREET FLOOD WIDTH(FEET) =  27.47
 FLOW VELOCITY(FEET/SEC.) =  7.86   DEPTH*VELOCITY(FT*FT/SEC.) =   5.31
 *NOTE: INITIAL SUBAREA NOMOGRAPH WITH SUBAREA PARAMETERS,
       AND L = 1124.9 FT WITH ELEVATION-DROP =  45.2 FT, IS   57.6 CFS,
       WHICH EXCEEDS THE TOP-OF-CURB STREET CAPACITY AT NODE    133.00
 LONGEST FLOWPATH FROM NODE     19.00 TO NODE    133.00 =  1224.94 FEET.

****************************************************************************
 FLOW PROCESS FROM NODE    133.00 TO NODE    133.00 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.) =   4.90
 RAINFALL INTENSITY(INCH/HR) =   7.11
 TOTAL STREAM AREA(ACRES) =     9.96
```

```
 PEAK FLOW RATE(CFS) AT CONFLUENCE =     60.23

 ** CONFLUENCE DATA **
 STREAM      RUNOFF      Tc       INTENSITY      AREA
 NUMBER       (CFS)    (MIN.)   (INCH/HOUR)    (ACRE)
     1       197.63    13.38        3.770         66.43
     2        60.23     4.90        7.114          9.96

 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       132.51     4.90       7.114
     2       229.54    13.38       3.770

 COMPUTED CONFLUENCE ESTIMATES ARE AS FOLLOWS:
 PEAK FLOW RATE(CFS) =     229.54   Tc(MIN.) =    13.38
 TOTAL AREA(ACRES) =      76.39
 LONGEST FLOWPATH FROM NODE       1.00 TO NODE     133.00 =  3844.37 FEET.

****************************************************************************
 FLOW PROCESS FROM NODE    133.00 TO NODE    300.00 IS CODE =  31
----------------------------------------------------------------------------
 >>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
 >>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<
============================================================================
 ELEVATION DATA: UPSTREAM(FEET) =    393.00  DOWNSTREAM(FEET) =    390.00
 FLOW LENGTH(FEET) =     78.66    MANNING'S N =  0.013
 DEPTH OF FLOW IN  51.0 INCH PIPE IS  41.1 INCHES
 PIPE-FLOW VELOCITY(FEET/SEC.) =  18.74
 ESTIMATED PIPE DIAMETER(INCH) =  51.00     NUMBER OF PIPES =   1
 PIPE-FLOW(CFS) =     229.54
 PIPE TRAVEL TIME(MIN.) =    0.07     Tc(MIN.) =  13.45
 LONGEST FLOWPATH FROM NODE       1.00 TO NODE     300.00 =  3923.03 FEET.

****************************************************************************
 FLOW PROCESS FROM NODE    300.00 TO NODE    300.00 IS CODE =  10
----------------------------------------------------------------------------
 >>>>>MAIN-STREAM MEMORY COPIED ONTO MEMORY BANK # 3 <<<<<
============================================================================

****************************************************************************
 FLOW PROCESS FROM NODE     35.00 TO NODE     36.00 IS CODE =  21
----------------------------------------------------------------------------
 >>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<
============================================================================
 *USER SPECIFIED(SUBAREA):
 USER-SPECIFIED RUNOFF COEFFICIENT = .6500
 S.C.S. CURVE NUMBER (AMC II) =   0
 INITIAL SUBAREA FLOW-LENGTH(FEET) =     70.00
 UPSTREAM ELEVATION(FEET) =     532.80
 DOWNSTREAM ELEVATION(FEET) =     531.00
 ELEVATION DIFFERENCE(FEET) =       1.80
 SUBAREA OVERLAND TIME OF FLOW(MIN.) =     4.947
  100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  7.114
 NOTE: RAINFALL INTENSITY IS BASED ON Tc = 5-MINUTE.
 SUBAREA RUNOFF(CFS) =       0.46
 TOTAL AREA(ACRES) =       0.10   TOTAL RUNOFF(CFS) =       0.46

****************************************************************************
 FLOW PROCESS FROM NODE     36.00 TO NODE    134.00 IS CODE =  62
----------------------------------------------------------------------------
 >>>>>COMPUTE STREET FLOW TRAVEL TIME THRU SUBAREA<<<<<
 >>>>>(STREET TABLE SECTION #  1 USED)<<<<<
============================================================================
 UPSTREAM ELEVATION(FEET) =  531.00  DOWNSTREAM ELEVATION(FEET) =  505.50
 STREET LENGTH(FEET) =  1081.77   CURB HEIGHT(INCHES) =  8.0
 STREET HALFWIDTH(FEET) = 30.00

 DISTANCE FROM CROWN TO CROSSFALL GRADEBREAK(FEET) =  20.00
 INSIDE STREET CROSSFALL(DECIMAL) =  0.018
 OUTSIDE STREET CROSSFALL(DECIMAL)  =  0.018

 SPECIFIED NUMBER OF HALFSTREETS CARRYING RUNOFF =  1
 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) =       9.97
```

```
   STREETFLOW MODEL RESULTS USING ESTIMATED FLOW:
   STREET FLOW DEPTH(FEET) =  0.44
   HALFSTREET FLOOD WIDTH(FEET) =   15.59
   AVERAGE FLOW VELOCITY(FEET/SEC.) =    4.22
   PRODUCT OF DEPTH&VELOCITY(FT*FT/SEC.) =    1.86
 STREET FLOW TRAVEL TIME(MIN.) =   4.27   Tc(MIN.) =   9.22
  100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  4.794
 *USER SPECIFIED(SUBAREA):
 USER-SPECIFIED RUNOFF COEFFICIENT = .6500
 S.C.S. CURVE NUMBER (AMC II) =   0
 AREA-AVERAGE RUNOFF COEFFICIENT =  0.650
 SUBAREA AREA(ACRES) =      5.95      SUBAREA RUNOFF(CFS) =   18.54
 TOTAL AREA(ACRES) =        6.05        PEAK FLOW RATE(CFS) =      18.85

 END OF SUBAREA STREET FLOW HYDRAULICS:
 DEPTH(FEET) = 0.52   HALFSTREET FLOOD WIDTH(FEET) =  20.20
 FLOW VELOCITY(FEET/SEC.) =  4.91   DEPTH*VELOCITY(FT*FT/SEC.) =   2.57
 LONGEST FLOWPATH FROM NODE      35.00 TO NODE     134.00 =  1151.77 FEET.

****************************************************************************
 FLOW PROCESS FROM NODE    134.00 TO NODE    138.00 IS CODE =  31
----------------------------------------------------------------------------
 >>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
 >>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<
============================================================================
 ELEVATION DATA: UPSTREAM(FEET) =   495.50  DOWNSTREAM(FEET) =   487.00
 FLOW LENGTH(FEET) =  1111.41   MANNING'S N =  0.013
 DEPTH OF FLOW IN  27.0 INCH PIPE IS  21.8 INCHES
 PIPE-FLOW VELOCITY(FEET/SEC.) =   5.49
 ESTIMATED PIPE DIAMETER(INCH) =  27.00    NUMBER OF PIPES =   1
 PIPE-FLOW(CFS) =      18.85
 PIPE TRAVEL TIME(MIN.) =   3.37    Tc(MIN.) =  12.59
 LONGEST FLOWPATH FROM NODE      35.00 TO NODE     138.00 =  2263.18 FEET.

****************************************************************************
 FLOW PROCESS FROM NODE    138.00 TO NODE    138.00 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.) =  12.59
 RAINFALL INTENSITY(INCH/HR) =   3.92
 TOTAL STREAM AREA(ACRES) =     6.05
 PEAK FLOW RATE(CFS) AT CONFLUENCE =     18.85

****************************************************************************
 FLOW PROCESS FROM NODE     37.00 TO NODE     38.00 IS CODE =  21
----------------------------------------------------------------------------
 >>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<
============================================================================
 *USER SPECIFIED(SUBAREA):
 USER-SPECIFIED RUNOFF COEFFICIENT = .6500
 S.C.S. CURVE NUMBER (AMC II) =   0
 INITIAL SUBAREA FLOW-LENGTH(FEET) =     70.00
 UPSTREAM ELEVATION(FEET) =     507.30
 DOWNSTREAM ELEVATION(FEET) =     506.00
 ELEVATION DIFFERENCE(FEET) =       1.30
 SUBAREA OVERLAND TIME OF FLOW(MIN.) =    5.514
  100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  6.679
 SUBAREA RUNOFF(CFS) =       0.48
 TOTAL AREA(ACRES) =       0.11   TOTAL RUNOFF(CFS) =       0.48

****************************************************************************
 FLOW PROCESS FROM NODE     38.00 TO NODE    137.00 IS CODE =  62
----------------------------------------------------------------------------
 >>>>>COMPUTE STREET FLOW TRAVEL TIME THRU SUBAREA<<<<<
 >>>>>(STREET TABLE SECTION #  1 USED)<<<<<
============================================================================
 UPSTREAM ELEVATION(FEET) =  506.00  DOWNSTREAM ELEVATION(FEET) =  496.00
 STREET LENGTH(FEET) =   904.59   CURB HEIGHT(INCHES) =  8.0
 STREET HALFWIDTH(FEET) = 30.00

 DISTANCE FROM CROWN TO CROSSFALL GRADEBREAK(FEET) =  20.00
 INSIDE STREET CROSSFALL(DECIMAL) =  0.018
 OUTSIDE STREET CROSSFALL(DECIMAL)  =  0.018

 SPECIFIED NUMBER OF HALFSTREETS CARRYING RUNOFF =  1
 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) =       4.84
  STREETFLOW MODEL RESULTS USING ESTIMATED FLOW:
  STREET FLOW DEPTH(FEET) =  0.40
  HALFSTREET FLOOD WIDTH(FEET) =   13.48
  AVERAGE FLOW VELOCITY(FEET/SEC.) =    2.66
  PRODUCT OF DEPTH&VELOCITY(FT*FT/SEC.) =    1.07
STREET FLOW TRAVEL TIME(MIN.) =   5.66   Tc(MIN.) =  11.17
 100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  4.236
*USER SPECIFIED(SUBAREA):
USER-SPECIFIED RUNOFF COEFFICIENT = .6500
S.C.S. CURVE NUMBER (AMC II) =   0
AREA-AVERAGE RUNOFF COEFFICIENT =  0.650
SUBAREA AREA(ACRES) =     3.07      SUBAREA RUNOFF(CFS) =    8.45
TOTAL AREA(ACRES) =       3.18        PEAK FLOW RATE(CFS) =       8.75

END OF SUBAREA STREET FLOW HYDRAULICS:
DEPTH(FEET) = 0.47   HALFSTREET FLOOD WIDTH(FEET) =  17.23
FLOW VELOCITY(FEET/SEC.) =  3.08   DEPTH*VELOCITY(FT*FT/SEC.) =   1.45
LONGEST FLOWPATH FROM NODE      37.00 TO NODE    137.00 =   974.59 FEET.

****************************************************************************
FLOW PROCESS FROM NODE    137.00 TO NODE    138.00 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) =   150.55   MANNING'S N =  0.013
DEPTH OF FLOW IN  21.0 INCH PIPE IS  12.7 INCHES
PIPE-FLOW VELOCITY(FEET/SEC.) =   5.77
ESTIMATED PIPE DIAMETER(INCH) =  21.00    NUMBER OF PIPES =   1
PIPE-FLOW(CFS) =       8.75
PIPE TRAVEL TIME(MIN.) =   0.43    Tc(MIN.) =  11.61
LONGEST FLOWPATH FROM NODE      37.00 TO NODE    138.00 =  1125.14 FEET.

****************************************************************************
FLOW PROCESS FROM NODE    138.00 TO NODE    138.00 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.) =  11.61
RAINFALL INTENSITY(INCH/HR) =   4.13
TOTAL STREAM AREA(ACRES) =     3.18
PEAK FLOW RATE(CFS) AT CONFLUENCE =      8.75

** CONFLUENCE DATA **
STREAM     RUNOFF      Tc       INTENSITY       AREA
NUMBER      (CFS)    (MIN.)   (INCH/HOUR)     (ACRE)
    1       18.85    12.59        3.920          6.05
    2        8.75    11.61        4.132          3.18

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       26.13    11.61       4.132
    2       27.16    12.59       3.920

COMPUTED CONFLUENCE ESTIMATES ARE AS FOLLOWS:
PEAK FLOW RATE(CFS) =      27.16    Tc(MIN.) =   12.59
TOTAL AREA(ACRES) =       9.23
LONGEST FLOWPATH FROM NODE      35.00 TO NODE    138.00 =  2263.18 FEET.

****************************************************************************
FLOW PROCESS FROM NODE    138.00 TO NODE    300.00 IS CODE =  31
----------------------------------------------------------------------------
>>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
>>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<
============================================================================
ELEVATION DATA: UPSTREAM(FEET) =   487.00  DOWNSTREAM(FEET) =   390.00
FLOW LENGTH(FEET) =   256.12   MANNING'S N =  0.013
ESTIMATED PIPE DIAMETER(INCH) INCREASED TO 18.000
DEPTH OF FLOW IN  18.0 INCH PIPE IS  10.0 INCHES
PIPE-FLOW VELOCITY(FEET/SEC.) =  26.97
ESTIMATED PIPE DIAMETER(INCH) =  18.00    NUMBER OF PIPES =   1
```

```
  PIPE-FLOW(CFS) =      27.16
  PIPE TRAVEL TIME(MIN.) =   0.16    Tc(MIN.) =  12.75
  LONGEST FLOWPATH FROM NODE     35.00 TO NODE    300.00 =  2519.30 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    300.00 TO NODE    300.00 IS CODE =   1
----------------------------------------------------------------------------
  >>>>>DESIGNATE INDEPENDENT STREAM FOR CONFLUENCE<<<<<
============================================================================
  TOTAL NUMBER OF STREAMS =  3
  CONFLUENCE VALUES USED FOR INDEPENDENT STREAM  1 ARE:
  TIME OF CONCENTRATION(MIN.) =  12.75
  RAINFALL INTENSITY(INCH/HR) =   3.89
  TOTAL STREAM AREA(ACRES) =      9.23
  PEAK FLOW RATE(CFS) AT CONFLUENCE =      27.16

****************************************************************************
  FLOW PROCESS FROM NODE     29.00 TO NODE     30.00 IS CODE =  21
----------------------------------------------------------------------------
  >>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<
============================================================================
  *USER SPECIFIED(SUBAREA):
  USER-SPECIFIED RUNOFF COEFFICIENT = .5500
  S.C.S. CURVE NUMBER (AMC II) =   0
  INITIAL SUBAREA FLOW-LENGTH(FEET) =    100.00
  UPSTREAM ELEVATION(FEET) =     552.00
  DOWNSTREAM ELEVATION(FEET) =     510.00
  ELEVATION DIFFERENCE(FEET) =      42.00
  SUBAREA OVERLAND TIME OF FLOW(MIN.) =     4.596
  WARNING: THE MAXIMUM OVERLAND FLOW SLOPE, 10.%, IS USED IN Tc CALCULATION!
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  7.114
  NOTE: RAINFALL INTENSITY IS BASED ON Tc = 5-MINUTE.
  SUBAREA RUNOFF(CFS) =       3.83
  TOTAL AREA(ACRES) =       0.98   TOTAL RUNOFF(CFS) =       3.83

****************************************************************************
  FLOW PROCESS FROM NODE     30.00 TO NODE    139.00 IS CODE =  52
----------------------------------------------------------------------------
  >>>>>COMPUTE NATURAL VALLEY CHANNEL FLOW<<<<<
  >>>>>TRAVELTIME THRU SUBAREA<<<<<
============================================================================
  ELEVATION DATA: UPSTREAM(FEET) =     510.00  DOWNSTREAM(FEET) =     448.00
  CHANNEL LENGTH THRU SUBAREA(FEET) =    608.20    CHANNEL SLOPE =  0.1019
  NOTE: CHANNEL SLOPE OF .1 WAS ASSUMED IN VELOCITY ESTIMATION
  CHANNEL FLOW THRU SUBAREA(CFS) =        3.83
  FLOW VELOCITY(FEET/SEC) =    6.27 (PER LACFCD/RCFC&WCD HYDROLOGY MANUAL)
  TRAVEL TIME(MIN.) =    1.62    Tc(MIN.) =    6.21
  LONGEST FLOWPATH FROM NODE     29.00 TO NODE    139.00 =    708.20 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE     30.00 TO NODE    139.00 IS CODE =  81
----------------------------------------------------------------------------
  >>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
============================================================================
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  6.185
  *USER SPECIFIED(SUBAREA):
  USER-SPECIFIED RUNOFF COEFFICIENT = .5500
  S.C.S. CURVE NUMBER (AMC II) =   0
  AREA-AVERAGE RUNOFF COEFFICIENT = 0.5500
  SUBAREA AREA(ACRES) =      5.65   SUBAREA RUNOFF(CFS) =   19.22
  TOTAL AREA(ACRES) =        6.63   TOTAL RUNOFF(CFS) =    22.55
  TC(MIN.) =    6.21

****************************************************************************
  FLOW PROCESS FROM NODE    139.00 TO NODE    140.00 IS CODE =  31
----------------------------------------------------------------------------
  >>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
  >>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<
============================================================================
  ELEVATION DATA: UPSTREAM(FEET) =    448.00  DOWNSTREAM(FEET) =    411.00
  FLOW LENGTH(FEET) =    398.83   MANNING'S N =  0.013
  DEPTH OF FLOW IN  18.0 INCH PIPE IS  14.7 INCHES
  PIPE-FLOW VELOCITY(FEET/SEC.) =  14.60
  ESTIMATED PIPE DIAMETER(INCH) =  18.00    NUMBER OF PIPES =   1
  PIPE-FLOW(CFS) =      22.55
  PIPE TRAVEL TIME(MIN.) =   0.46    Tc(MIN.) =    6.67
  LONGEST FLOWPATH FROM NODE     29.00 TO NODE    140.00 =  1107.03 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    140.00 TO NODE    300.00 IS CODE =  31
----------------------------------------------------------------------------
```

```
 >>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
 >>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<
==============================================================================
 ELEVATION DATA: UPSTREAM(FEET) =   411.00  DOWNSTREAM(FEET) =   390.00
 FLOW LENGTH(FEET) =    92.29   MANNING'S N = 0.013
 DEPTH OF FLOW IN  18.0 INCH PIPE IS  10.4 INCHES
 PIPE-FLOW VELOCITY(FEET/SEC.) =  21.25
 ESTIMATED PIPE DIAMETER(INCH) =  18.00    NUMBER OF PIPES =   1
 PIPE-FLOW(CFS) =      22.55
 PIPE TRAVEL TIME(MIN.) =   0.07    Tc(MIN.) =    6.74
 LONGEST FLOWPATH FROM NODE      29.00 TO NODE     300.00 =  1199.32 FEET.

******************************************************************************
 FLOW PROCESS FROM NODE    300.00 TO NODE    300.00 IS CODE =   1
------------------------------------------------------------------------------
 >>>>>DESIGNATE INDEPENDENT STREAM FOR CONFLUENCE<<<<<
==============================================================================
 TOTAL NUMBER OF STREAMS =  3
 CONFLUENCE VALUES USED FOR INDEPENDENT STREAM  2 ARE:
 TIME OF CONCENTRATION(MIN.) =    6.74
 RAINFALL INTENSITY(INCH/HR) =    5.87
 TOTAL STREAM AREA(ACRES) =     6.63
 PEAK FLOW RATE(CFS) AT CONFLUENCE =      22.55

******************************************************************************
 FLOW PROCESS FROM NODE     33.00 TO NODE     34.00 IS CODE =  21
------------------------------------------------------------------------------
 >>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<
==============================================================================
 *USER SPECIFIED(SUBAREA):
 USER-SPECIFIED RUNOFF COEFFICIENT = .5500
 S.C.S. CURVE NUMBER (AMC II) =   0
 INITIAL SUBAREA FLOW-LENGTH(FEET) =   100.00
 UPSTREAM ELEVATION(FEET) =    497.00
 DOWNSTREAM ELEVATION(FEET) =    460.00
 ELEVATION DIFFERENCE(FEET) =     37.00
 SUBAREA OVERLAND TIME OF FLOW(MIN.) =     4.596
 WARNING: THE MAXIMUM OVERLAND FLOW SLOPE, 10.%, IS USED IN Tc CALCULATION!
  100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  7.114
 NOTE: RAINFALL INTENSITY IS BASED ON Tc = 5-MINUTE.
 SUBAREA RUNOFF(CFS) =      1.02
 TOTAL AREA(ACRES) =      0.26   TOTAL RUNOFF(CFS) =      1.02

******************************************************************************
 FLOW PROCESS FROM NODE     34.00 TO NODE    300.00 IS CODE =  52
------------------------------------------------------------------------------
 >>>>>COMPUTE NATURAL VALLEY CHANNEL FLOW<<<<<
 >>>>>TRAVELTIME THRU SUBAREA<<<<<
==============================================================================
 ELEVATION DATA: UPSTREAM(FEET) =    460.00  DOWNSTREAM(FEET) =    390.00
 CHANNEL LENGTH THRU SUBAREA(FEET) =    310.15   CHANNEL SLOPE =  0.2257
 NOTE: CHANNEL SLOPE OF .1 WAS ASSUMED IN VELOCITY ESTIMATION
 CHANNEL FLOW THRU SUBAREA(CFS) =       1.02
 FLOW VELOCITY(FEET/SEC) =   4.76  (PER LACFCD/RCFC&WCD HYDROLOGY MANUAL)
 TRAVEL TIME(MIN.) =    1.09    Tc(MIN.) =    5.68
 LONGEST FLOWPATH FROM NODE      33.00 TO NODE     300.00 =    410.15 FEET.

******************************************************************************
 FLOW PROCESS FROM NODE     34.00 TO NODE    300.00 IS CODE =  81
------------------------------------------------------------------------------
 >>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
==============================================================================
  100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  6.551
 *USER SPECIFIED(SUBAREA):
 USER-SPECIFIED RUNOFF COEFFICIENT = .5500
 S.C.S. CURVE NUMBER (AMC II) =   0
 AREA-AVERAGE RUNOFF COEFFICIENT = 0.5500
 SUBAREA AREA(ACRES) =     6.31   SUBAREA RUNOFF(CFS) =    22.73
 TOTAL AREA(ACRES) =       6.57   TOTAL RUNOFF(CFS) =    23.67
 TC(MIN.) =    5.68

******************************************************************************
 FLOW PROCESS FROM NODE    300.00 TO NODE    300.00 IS CODE =   1
------------------------------------------------------------------------------
 >>>>>DESIGNATE INDEPENDENT STREAM FOR CONFLUENCE<<<<<
 >>>>>AND COMPUTE VARIOUS CONFLUENCED STREAM VALUES<<<<<
==============================================================================
 TOTAL NUMBER OF STREAMS =  3
 CONFLUENCE VALUES USED FOR INDEPENDENT STREAM  3 ARE:
 TIME OF CONCENTRATION(MIN.) =    5.68
 RAINFALL INTENSITY(INCH/HR) =    6.55
```

```
 TOTAL STREAM AREA(ACRES) =      6.57
 PEAK FLOW RATE(CFS) AT CONFLUENCE =      23.67

 ** CONFLUENCE DATA **
 STREAM     RUNOFF      Tc       INTENSITY       AREA
 NUMBER      (CFS)    (MIN.)    (INCH/HOUR)     (ACRE)
     1       27.16    12.75        3.889           9.23
     2       22.55     6.74        5.868           6.63
     3       23.67     5.68        6.551           6.57

 RAINFALL INTENSITY AND TIME OF CONCENTRATION RATIO
 CONFLUENCE FORMULA USED FOR  3 STREAMS.

 ** PEAK FLOW RATE TABLE **
 STREAM     RUNOFF      Tc       INTENSITY
 NUMBER      (CFS)    (MIN.)    (INCH/HOUR)
     1       58.81     5.68       6.551
     2       61.76     6.74       5.868
     3       56.16    12.75       3.889

 COMPUTED CONFLUENCE ESTIMATES ARE AS FOLLOWS:
 PEAK FLOW RATE(CFS) =      61.76   Tc(MIN.) =     6.74
 TOTAL AREA(ACRES) =      22.43
 LONGEST FLOWPATH FROM NODE      35.00 TO NODE     300.00 =  2519.30 FEET.

****************************************************************************
 FLOW PROCESS FROM NODE     300.00 TO NODE     300.00 IS CODE =  11
----------------------------------------------------------------------------
 >>>>>CONFLUENCE MEMORY BANK # 3 WITH THE MAIN-STREAM MEMORY<<<<<
============================================================================

 ** MAIN STREAM CONFLUENCE DATA **
 STREAM     RUNOFF      Tc       INTENSITY       AREA
 NUMBER      (CFS)    (MIN.)    (INCH/HOUR)     (ACRE)
     1       61.76     6.74        5.868         22.43
 LONGEST FLOWPATH FROM NODE      35.00 TO NODE     300.00 =  2519.30 FEET.

 ** MEMORY BANK #  3 CONFLUENCE DATA **
 STREAM     RUNOFF      Tc       INTENSITY       AREA
 NUMBER      (CFS)    (MIN.)    (INCH/HOUR)     (ACRE)
     1      229.54    13.45        3.757         76.39
 LONGEST FLOWPATH FROM NODE       1.00 TO NODE     300.00 =  3923.03 FEET.

 ** PEAK FLOW RATE TABLE **
 STREAM    RUNOFF        Tc       INTENSITY
 NUMBER     (CFS)      (MIN.)    (INCH/HOUR)
     1     176.73       6.74         5.868
     2     269.08      13.45         3.757

 COMPUTED CONFLUENCE ESTIMATES ARE AS FOLLOWS:
 PEAK FLOW RATE(CFS) =     269.08   Tc(MIN.) =    13.45
 TOTAL AREA(ACRES) =      98.82

****************************************************************************
 FLOW PROCESS FROM NODE     300.00 TO NODE     300.00 IS CODE =  12
----------------------------------------------------------------------------
 >>>>>CLEAR MEMORY BANK # 3 <<<<<
============================================================================

****************************************************************************
 FLOW PROCESS FROM NODE      31.00 TO NODE      32.00 IS CODE =  21
----------------------------------------------------------------------------
 >>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<
============================================================================
 *USER SPECIFIED(SUBAREA):
 USER-SPECIFIED RUNOFF COEFFICIENT = .5500
 S.C.S. CURVE NUMBER (AMC II) =   0
 INITIAL SUBAREA FLOW-LENGTH(FEET) =    100.00
 UPSTREAM ELEVATION(FEET) =     570.00
 DOWNSTREAM ELEVATION(FEET) =      525.00
 ELEVATION DIFFERENCE(FEET) =      45.00
 SUBAREA OVERLAND TIME OF FLOW(MIN.) =     4.596
 WARNING: THE MAXIMUM OVERLAND FLOW SLOPE, 10.%, IS USED IN Tc CALCULATION!
  100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  7.114
 NOTE: RAINFALL INTENSITY IS BASED ON Tc = 5-MINUTE.
 SUBAREA RUNOFF(CFS) =      19.60
 TOTAL AREA(ACRES) =       5.01   TOTAL RUNOFF(CFS) =      19.60

****************************************************************************
 FLOW PROCESS FROM NODE      32.00 TO NODE     141.00 IS CODE =  52
----------------------------------------------------------------------------
```

```
>>>>>COMPUTE NATURAL VALLEY CHANNEL FLOW<<<<<
>>>>>TRAVELTIME THRU SUBAREA<<<<<
==============================================================================
ELEVATION DATA: UPSTREAM(FEET) =    525.00  DOWNSTREAM(FEET) =    420.00
CHANNEL LENGTH THRU SUBAREA(FEET) =   650.78   CHANNEL SLOPE =  0.1613
NOTE: CHANNEL SLOPE OF .1 WAS ASSUMED IN VELOCITY ESTIMATION
CHANNEL FLOW THRU SUBAREA(CFS) =     19.60
FLOW VELOCITY(FEET/SEC) =   9.42 (PER LACFCD/RCFC&WCD HYDROLOGY MANUAL)
TRAVEL TIME(MIN.) =   1.15   Tc(MIN.) =   5.75
LONGEST FLOWPATH FROM NODE     31.00 TO NODE    141.00 =   750.78 FEET.

******************************************************************************
FLOW PROCESS FROM NODE     32.00 TO NODE    141.00 IS CODE =  81
------------------------------------------------------------------------------
>>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
==============================================================================
 100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  6.503
*USER SPECIFIED(SUBAREA):
USER-SPECIFIED RUNOFF COEFFICIENT = .5500
S.C.S. CURVE NUMBER (AMC II) =   0
AREA-AVERAGE RUNOFF COEFFICIENT = 0.5500
SUBAREA AREA(ACRES) =   34.08   SUBAREA RUNOFF(CFS) =  121.90
TOTAL AREA(ACRES) =      39.09   TOTAL RUNOFF(CFS) =  139.82
TC(MIN.) =   5.75
==============================================================================
END OF STUDY SUMMARY:
TOTAL AREA(ACRES)      =      39.09  TC(MIN.) =       5.75
PEAK FLOW RATE(CFS)    =    139.82
==============================================================================
==============================================================================
END OF RATIONAL METHOD ANALYSIS
```

OVERSIZED EXHIBIT
"EXISTING CONDITIONS HYDROLOGY MAP"

This exhibit is on file at the City of Chula Vista, Planning Department located at 276 Fourth Avenue, Chula Vista, CA 91910

APPENDIX E-4

PRELIMINARY WATER QUALITY TECHNICAL REPORT

HUNSAKER & ASSOCIATES

SAN DIEGO, INC.

PLANNING
ENGINEERING
SURVEYING

IRVINE
LOS ANGELES
RIVERSIDE
SAN DIEGO

PRELIMINARY WATER QUALITY TECHNICAL REPORT for OTAY RANCH VILLAGE 7 NEIGHBORHOOD R-2 AND VILLAGE 4 PARK SITE

City of Chula Vista, California

Prepared for:
Otay Ranch Company
610 West Ash Street
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W.O. 0025-346

May 21, 2004

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Exp. 09/30/05
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STATE OF CALIFORNIA

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ATTACHMENTS

Developed Site Map

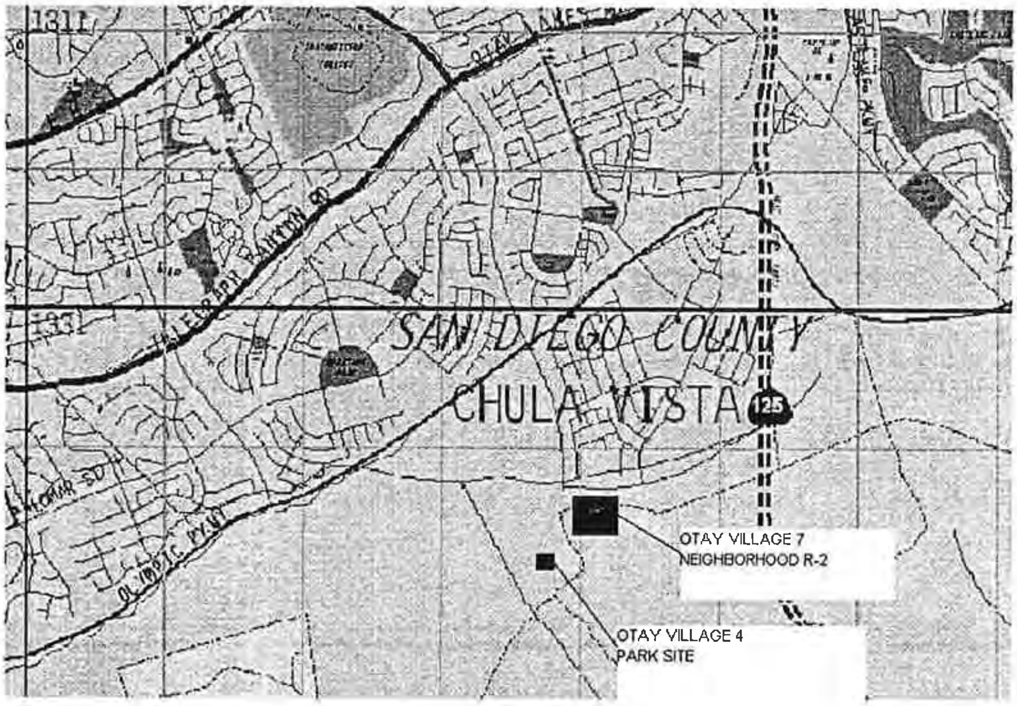
List of Tables and Figures

CHAPTER 1 - EXECUTIVE SUMMARY

CHAPTER 1 – EXECUTIVE SUMMARY

1.1 – Introduction

The Otay Ranch Village 7, Neighborhood R-2 and Village 4 Park Site developments are located in the City of Chula Vista south of Birch Road and west of the proposed alignment of State Route 125. Village 7 lies to the east of La Media Road and is bounded by McMillin's portion of the Otay 7 development to the east. The Village 4 park site is located southwest of the Village 7 Neighborhood R-2 site (west of La Media Road). The vicinity map below shows the approximate project site location.



Per the City of Chula Vista Storm Water Requirements Manual, the Otay Ranch Village 7 Neighborhood R-2 and Village 4 park site projects are classified as Priority Projects and are subject to the City's Permanent Storm Water BMP and Standard Urban Storm Water Management Plan (SUSMP) Requirements. The Village 4 Community Park Site is included in this report for the purpose of addressing NPDES compliance during grading operations as the park site shall be used as a "balancing" area. Construction of the park shall comply with the complete SUSMP process. Additional details regarding ultimate conditions water quality treatment for the Village 4 park site are included in a separate report entitled "Water Quality Technical Report for Otay Ranch Villages 2, 3, 4 and PA 18B" prepared by Hunsaker & Associated, dated May, 2004.

This Water Quality Technical Report (WQTR) has been prepared pursuant to requirements set forth in the City of Chula Vista's "Storm Water Requirements Manual." All calculations are consistent with criteria set forth by the Regional Water

Quality Control Board's Order No. 2001-01, and the City of Chula Vista Standard Urban Storm Water Mitigation Plan (SUSMP).

This WQTR recommends the location and sizing of site Best Management Practices (BMPs) which includes three flow-based Vortechs, or approved equivalent flow-based treatment units (see Developed Site Map in the Attachments section).

Furthermore, this report determines anticipated project pollutants, pollutants of concern in the receiving watershed, peak flow mitigation, recommended source control BMPs, and methodology used for the design of flow-based BMPs.

1.2 – Summary of Pre-Developed Conditions

The existing Otay Ranch Village 7 Neighborhood R-2 site and Village 4 park site currently contain no development and are characterized by rolling hills and heavily grazed land. Runoff from the existing site areas drain via incised canyon channels to Wolf Canyon.

Wolf Canyon Creek's main drainage course forms the southern boundary for the Otay Ranch Village 7 Neighborhood R-2 site and Otay Ranch Village 2 further downstream. The Village 4 park site is located south of Wolf Canyon Creek

Downstream of Otay Ranch Village 2, Wolf Canyon Creek flows in a southerly direction just east of the Village 3 property boundary. Wolf Canyon discharges runoff to the Otay River near the proposed location of the Otay Valley Road detention basin.

1.3 – Summary of Proposed Development

Development of the Otay Ranch Village 7 Neighborhood R-2 project site will include the construction of single-family homes as well as the associated streets, sidewalks, landscaping and utilities. Development of the Village 4 park site will consist of the construction of a neighborhood park.

Two flow-based BMPs will be located on the Village 7 Neighborhood R-2 project site as shown on the developed site map included in the attachment section of this report.

The "TM Drainage Study for Otay Ranch Village 7 Neighborhood R-2 and Village 4 Park Site", prepared by Hunsaker & Associates, details the design of the detention basin for Village 7 Neighborhood R-2. As shown in the referenced report, the detention facility mitigates the design peak flow increase from Village 7 below pre-development levels. 85^{th} percentile flow will be treated in one of the proposed Vortechs, or approved equivalent flow-based treatment units. Two offline treatment units will treat flow from the Village 7 Neighborhood R-2 site. One flow-based BMP will be located on the Village 4 park site to mitigate 85^{th} percentile runoff from this site.

The "Master Drainage Study for Wolf Canyon," prepared by Hunsaker & Associates, addresses pre and post developed peak flows produced from the overall development site as well as the McMillin development upstream and Village 2 downstream.

1.4 – Results and Recommendations

Two flow-based Vortechs systems, or approved equivalent systems will treat the 85th percentile runoff from Village 7, Neighborhood R-2 as well as runoff from a portion of La Media Road and the adjacent slopes (see BMP Location Map on the following page).

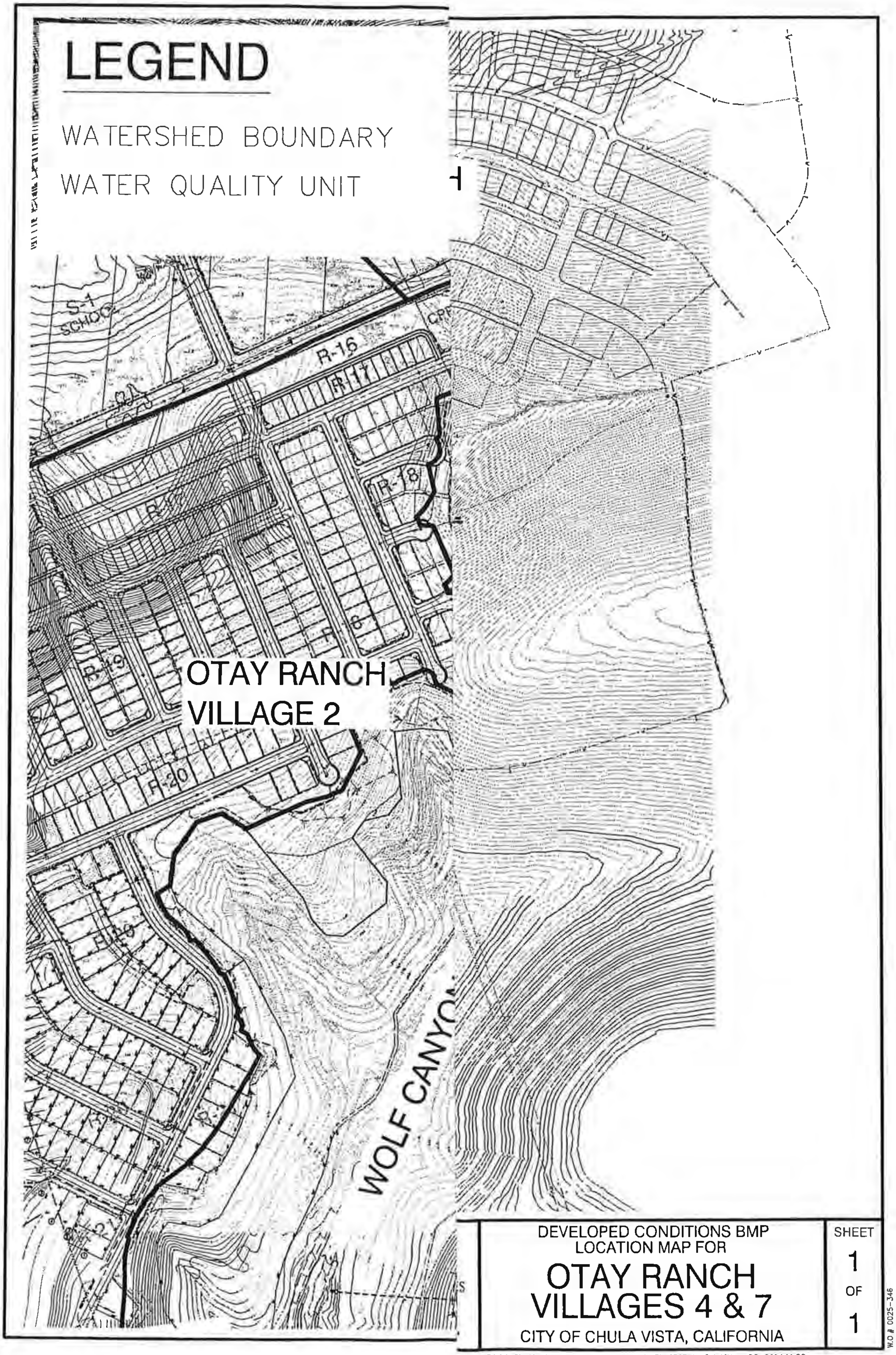
Using the 85th percentile rainfall of 0.60 inches, Rational Method calculations were utilized to predict the 85th percentile runoff flow to each outlet location within the post-developed project boundaries.

Vortechs, or approved equivalent systems are flow-based BMPs that function as hydrodynamic separators. Two units are proposed for the Village 7, Neighborhood R-2 site, one upstream of each outlet location to the proposed detention basin. The Vortechs storm water quality treatment unit proposed for the Village 4 park site is located at the southwest corner of the park footprint area (see BMP Location Map on the following page).

A Vortechs Model 7000, or approved equivalent treatment unit is proposed to treat the flow from the majority of the Village 7, Neighborhood R-2 site as well as flow from La Media prior to it's outlet into the detention basin. A Vortechs model 5000 will be used to treat the remainder of the flow from the Village 7 site. A Vortechs Model 4000, or approved equivalent treatment unit, is proposed to treat the flow from the Village 4 park site.

The proposed Vortechs, or approved equivalent units will be designed as offline, pre-cast treatment units. In addition to a site specific treatment unit, this design will include a diversion structure that will divert the entire 85th percentile runoff flow from the watershed into the treatment unit. Flows in excess of the design flow rate pass over the weir and proceed downstream. Additional information about the Vortechs system can be found in Chapter 7 of this report.

Chapter 1 – BMP Location Map



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CHAPTER 2 –STORM WATER CRITERIA

CHAPTER 2 – STORM WATER CRITERIA

2.1 – Regional Water Quality Control Board Criteria

All runoff conveyed in the proposed storm drain systems will be treated in compliance with Regional Water Quality Control Board regulations and NPDES criteria prior to discharging to natural watercourses. California Regional Water Quality Control Board Order No. 2001-01, dated February 21, 2001, sets waste discharge requirements for discharges of urban runoff from municipal storm separate drainage systems draining the watersheds of San Diego County.

Per the RWQCB Order, post-development runoff from a site shall not contain pollutant loads which cause or contribute to an exceedance of receiving water quality objectives or which have not been reduced to the maximum extent practicable. Post-construction Best Management Practices (BMPs), which refer to specific storm water management techniques that are applied to manage construction and post-construction site runoff and minimize erosion, include source control – aimed at reducing the amount of sediment and other pollutants – and treatment controls that keep soil and other pollutants onsite once they have been loosened by storm water erosion.

Post construction pollutants are a result of the urban development of the property and the effects of automobile use. Runoff from paved surfaces can contain both sediment (in the form of silt and sand) as well as a variety of pollutants transported by the sediment. Landscape activities by homeowners are an additional source of sediment.

All structural BMPs shall be located to infiltrate, filter, or treat the required runoff volume or flow (based on the 85th percentile rainfall) prior to its discharge to any receiving watercourse supporting beneficial uses.

2.2 – City of Chula Vista SUSMP Criteria

Per the City of Chula Vista "Development and Redevelopment Projects Storm Water Standards Requirements Manual", the Otay Ranch Village 7 project is classified as a Priority Project and subject to the City's Permanent Storm Water BMP Requirements. These requirements required the preparation of this Water Quality Technical Report.

The Standard Storm Water BMPs Requirements, which must be included along with Grading Plan applications, is included on the following page.

Chapter 2 - City of Chula Vista BMP Requirements (Form 5500)

CITY OF CHULA VISTA	ENGINEERING 276 Fourth Avenue, Chula Vista, CA 91910 619-691-5021 619-691-5171 FAX	**PROJECT PERMANENT STORM WATER BMPs (SUSMP) REQUIREMENTS**
FORM 5500		

Appendix A

Complete the following checklist to determine the project's per manent and construction best management practices requirements. This form must be completed and submitted with the permit application.

If one or more questions in the checklist are answered "Yes," the project is subject to the "Priority Project Permanent Storm Water BMPS (SUSMP)" requirements in Appendix B. If all answers are "No", please complete Form 5501 to determine if the project is subject to the "Standard Permanent Storm Water BMP" requirements.

Does the project meet the definition of one or mor e of the priority project categories? Also, refer to the definition in Appendix F for expanded definition of the Significant Redevelopment priority project

		Yes	No
1.	Detached residential development of 10 or more units	✓	
2.	Attached residential devel opment of 10 or more units	✓	
3.	Commercial development greater than 100,000 square feet		✓
4.	Automotive repair shop		✓
5.	Restaurant		✓
6.	Steep hillside development greater than 5,000 square feet		✓
7.	Project discharging to receiving waters within Envi ronmentally Sensitive Areas		✓
8.	Parking lots greater than or equal to 5,000 square feet or with at least 15 parking spaces, and potentially exposed to urban runoff		✓
9.	Streets, roads, highways, and freeways which create a new paved surface that is 5,00 0 square feet or greater	✓	

* Refer to the definitions in Appendix F for expanded definitions of the priority project categories.

Limited Exclusion: Trenching and resurfacing work associated with utility projects are not considered priority projects. Parking lots, buildings and other structures associated with utility projects are priority projects if one or more of the criteria is met.

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Chapter 2 - City of Chula Vista BMP Requirements (Form 5501)

CITY OF CHULA VISTA

ENGINEERING

276 Fourth Avenue, Chula Vista, CA. 91910

619-691-5021 619-691-5171 FAX

FORM 5501

PERMANENT STANDARD STORM WATER BMPs REQUIREMENTS

Appendix A

Section 1

Complete the following checklist to determine if the project is subject to "Permanent Standard Storm Water BMPs" requirements.

If one or more questions in the following checklist are answered "Yes", the project is subject to the applicable "Permanent Standard Storm Water BMPs" requirements identified in Section 2 of this Form 5501. If all answers are "No", the project is exempt from permanent storm water BMPs requirements.

	Does the project propose:	Yes	No	Applicable BMP (refer to Section 2 of this Form 5501)
1.	New impervious areas, such as rooftops, ro ads, parking lots, driveways, paths, and sidewalks?	✓		A.1, A.2, B1, C.1, C.2, C.8, C11
2.	New pervious landscape areas and irrigation systems?	✓		A.1, A.2, B.4, C.10
3.	Permanent structures within 100 feet of any natural water body?		✓	A.1, A.2, A.3
4.	Trash storage areas?		✓	B.3
5.	Liquid or solid material loading and unloading areas?		✓	B.2, C.3
6.	Vehicle or equipment fueling, washing, or maintenance areas?		✓	C.4, C.5, C6, C.7, C.9
7.	Require a General NPDES permit for Storm Water Discharges Associated with Industrial Activities (except Construction)? *		✓	Applicable BMPs
8.	Commercial or industrial waste handling or storage, excluding typical office or household waste?		✓	B2, B3, C.3, C.6
9.	Any grading or ground disturbance during construction?	✓		A.1, A.2, A.3, C10
10	Any new storm drains, or alteration to existing storm drains?	✓		A.3, B.1, C11

*To find out if the project is required to obtain an individual General NPDES Permit for Storm Water Discharges Associated with Industrial Activities, visit the State Water Resources Control Board web site at, www.swrcb.ca.gov/stormwtr/industrial.html. Applicable BMPs shall be selected from Section 2 of this Form 5501.

Development Projects subject to permanent standard BMP requirements shall complete and incorporate all necessary permanent BMPs into the project plans prior to submittal, regardless of project type. The City may approve proposed alternatives to the BMP requirements in this manual if they are determined by the City to be applicable and equally effective. Also, additional BMPs, analysis or information may be required by the City to enable staff to determine the adequacy of proposed BMPs, and will be requested through the project review process. Refer to Step 2 in the Manual: "Prepare & Submit Appropriate Plans," for guidance in the BMP design process.

Projects shall incorporate, where applicable, storm water BMPs into the project design, in the following progression:

- Site Design BMPs
- Source Control BMPs
- BMPs for Individual Project Categories

The series of BMPs listed below have organized sequentially to allow the applicant and design professional to incorporate the site design, source control BMPs, and where necessary, requirements applicable to individual project categories in this progression.

A. Site Design BMPs

1. Minimize Project's Impervious Footprint & Conserve Natural Areas

The following site design options shall be considered and, incorporated and implemented where determined applicable and feasible by the developer, and as approved by the City of Chula Vista, during the site planning and approval process, consistent with applicable General Plan policies and other development regulations.

a. Minimize impervious footprint. This can be achieved in various ways, including, but not limited to increasing building density (number of stories above or below ground) and developing land use regulations seeking to limit impervious surfaces. Decreasing the project's footprint can substantially reduce the project's impacts to water quality and hydrologic conditions.

b. Conserve natural areas where feasible. This can be achieved by concentrating or clustering development on the least environmentally sensitive portions of a site, while leaving the remaining land in a natural, undisturbed condition. The following list provides a guideline for determining the least sensitive portions of the site, in order of increasing sensitivity. Developers should also refer to the City's Multiple Species Conservation Plan or other biological regulations, as appropriate.

   - Areas devoid of vegetation, including previously graded areas and agricultural fields.
   - Areas of non-native vegetation, disturbed habitats and eucalyptus woodlands.
   - Areas of chamise or mixed chaparral, and non-native grasslands.
   - Areas containing coastal scrub communities.
   - All other upland communities.
   - Occupied habitat of sensitive species and all wetlands (as both are defined by the City of Chula Vista).
   - All areas necessary to maintain the viability of wildlife corridors. Within each of the previous categories, areas containing hillsides (as defined in Appendix E) should be considered more sensitive than the same category without hillsides.

c. Construct walkways, trails, patios, overflow parking lots and alleys and other low-traffic areas with permeable surfaces, such as pervious concrete, porous asphalt, unit pavers, and granular materials.

d. Construct streets, sidewalks and parking lot aisles to the minimum widths necessary, provided that public safety and a walkable environment for pedestrians are not compromised.

e. Maximize canopy interception and water conservation by preserving existing native trees and shrubs, and planting additional native or drought tolerant trees and large shrubs.

f. Minimize the use of impervious surfaces, such as decorative concrete, in the landscape design.
g. Use natural drainage systems to the maximum extent practicable.
h. Other site design options that are comparable, and equally effective, as approved by the City.

2. Minimize Directly Connected Impervious Areas (DCIAs)

Projects shall consider, and incorporate and implement the following design characteristics, where determined applicable and feasible by the City.

a. Where landscaping is proposed, drain rooftops into adjacent landscaping prior to discharging to the storm drain.
b. Where landscaping is proposed, drain impervious sidewalks, walkways, trails, and patios into adjacent landscaping.
c. Other design characteristics that are comparable and equally effective, as approved by the City.

3. Protect Slopes and Channels

Project plans shall include storm water BMPs to decrease the potential for erosion of slopes and/or channels, consistent with local codes and ordinances and with the approval of all agencies with jurisdiction over the project, e.g., the U.S. Army Corps of Engineers, the San Diego Regional Water Quality Control Board, and the California Department of Fish and Game. The following design principles shall be considered, and incorporated and implemented where determined applicable and feasible by the City of Chula Vista:

a. Convey runoff safely from the tops of slopes.
b. Vegetate slopes with native or drought tolerant vegetation.
c. Control and treat flows in landscaping and/or other controls prior to reaching existing natural drainage systems.
d. Stabilize permanent channel crossings.
e. Install energy dissipaters, such as riprap, at the outlets of new storm drains, culverts, conduits, or channels that enter unlined channels in accordance with applicable specifications to minimize erosion. Energy dissipaters shall be installed in such a way as to minimize impacts to receiving waters.
f. Other design principles that are comparable and equally effective, as approved by the City.

B. Source Control BMPs.

1. Provide Storm Drain System Stenciling and Signage

Storm drain stencils are highly visible source control messages, typically placed directly adjacent to storm drain inlets. The stencils contain a brief statement that prohibits the dumping of improper materials into the urban runoff conveyance system. Graphical icons, either illustrating anti-dumping symbols or images of receiving water fauna, are effective supplements to the anti-dumping message. Projects shall include the following requirements in the project design.

a. Provide stenciling, labeling, or stamping in fresh concrete of all storm drain inlets and catch basins within the project area with prohibitive language (such as: "NO DUMPING – I LIVE DOWNSTREAM") and graphical icons to discourage illegal dumping, according to City approved designs.
b. Post signs and prohibitive language and/or graphical icons, which prohibit illegal dumping at public access points along channels and creeks within the project area, according to City approved design.
c. Maintain legibility of stencils and signs.

2. Design Outdoor Material Storage Areas to Reduce Pollution Introduction

Improper storage of materials outdoors may increase the potential for toxic compounds, oil and grease, heavy metals, nutrients, suspended solids, and other pollutants to enter the urban runoff conveyance system. Where the project plans include outdoor areas for storage of hazardous materials that may contribute pollutants to the urban runoff conveyance system, the following storm water BMPs are required:

a. Hazardous materials with the potential to contaminate urban runoff shall either be: (1) placed in an enclosure such as, but not limited to, a cabinet, shed, or similar structure that prevents contact with runoff or spillage to the storm water conveyance system; or (2) protected by secondary containment structures such as berms, dikes, or curbs.

b. The storage area shall be paved and sufficiently impervious to contain leaks and spills.
c. The storage area shall have a roof or awning to minimize direct precipitation within the secondary containment area.
d. Other methods that are comparable and equally effective within the projects, as approved by the City.

3. Design Trash Storage Areas to Reduce Pollution Introduction

All trash container areas shall meet the following requirements (limited exclusion: detached residential homes):

a. Paved with an impervious surface e, designed not to allow run-on from adjoining areas, screened or walled to prevent off-site transport of trash; and
b. Provide attached lids on all trash containers, that exclude rain; or roof or awning to minimize direct precipitation.
c. Other design characteristics that are comparable and equally effective, as approved by the City.

4. Use Efficient Irrigation Systems & Landscape Design, and Employ Integrated Pest Management Principles

Priority projects shall design the timing and application methods of irrigation water to minimize the runoff of excess irrigation water into the storm water conveyance system. (Limited exclusion: detached residential homes.) The following methods to reduce excessive irrigation runoff shall be considered, and incorporated and implemented where determined applicable and feasible by the City:

a. Employing rain shutoff devices to prevent irrigation after precipitation;
b. Designing irrigation systems to each landscape area's specific water requirements;
c. Using flow reducers or shutoff valves triggered by a pressure drop to control water loss in the event of broken sprinkler heads or lines;
d. Employing other comparable, equally effective, methods to reduce irrigation water runoff.

Employ Integrated Pest Management Principles

Integrated Pest Management (IPM) is an ecosystem-based pollution prevention strategy that focuses on long-term prevention of pests or their damage through a combination of techniques such as biological control, habitat manipulation, modification of cultural practices, and use of resistant plant varieties. Pesticides are used only after monitoring indicates they are needed according to established guidelines. Pest control materials are selected and applied in a manner that minimizes risks to human health, beneficial and non-target organisms, and the environment. More information may be obtained at the UC Davies website (http://www.ipm.ucdavis.edu/WATER/U/index.html)

Eliminate and/or reduce the need for pesticide use in the project design by:

a. Plant pest-resistant or well-adapted plant varieties such as native plants; and
b. Discourage pests by modifying the site and landscaping design. Pollution prevention is the primary "first line of defense" because pollutants that are never used do not have to be controlled or treated (methods which are inherently less efficient).

Distribute IPM educational materials to future site residents/tenants. Minimally, educational materials must address the following topics:

a. Keeping pests out of buildings and landscaping using barriers, screens, and caulking;
b. Physical pest elimination techniques, such as, weeding, squashing, trapping, washing, or pruning out pests;
c. Relying on natural enemies to eat pests;
d. Proper use of pesticides as a last line of defense.

C. BMPs Applicable to Individual Project Categories

1. Private Roads

The design of private roadway drainage shall use at least one of the following (for further guidance, see *Start at the Source* [1999]):

a. Rural swale system: Street sheet flows to vegetated swale or gravel shoulder, curbs at street corners, culverts under driveways and street crossings;
b. Urban curb/swale system: street slopes to curb, periodic swale inlets drain to vegetated swale/biofilter;
c. Dual drainage system: First flush captured in street catch basins and discharged to adjacent vegetated swale or gravel shoulder, high flows connect directly to storm water conveyance system.
d. Other methods that are comparable and equally effective within the project, as approved by the City.

2. Residential Driveways & Guest Parking

The design of driveways and private residential parking areas shall use one at least of the following features.

a. Design driveways with shared access, flared (single lane at street) or wheelstrips (paving only under tires); or, drain into landscaping prior to discharging to the storm water conveyance system.
b. Uncovered temporary or guest parking on private residential lots may be: paved with a permeable surface; or, designed to drain into landscaping prior to discharging to the storm water conveyance system.
c. Other features which are comparable and equally effective, as approved by the City.

3. Dock Areas

Loading/unloading dock areas shall include the following:

a. Cover loading dock areas, or design drainage to preclude urban run-on and runoff.
b. Direct connections to storm drains from depressed loading docks (truck wells) are prohibited.
c. Other features which are comparable and equal effective, as approved by the City.

4. Maintenance Bays

Maintenance bays shall include the following:

a. Repair/Maintenance bays shall be indoors; or designed to preclude urban run-on and run-off; and
b. Design a repair/maintenance bay drainage system to capture all wash water, leaks, and spills. Connect drains to a sump for collection and disposal. Direct connection of the repair/maintenance bays to the storm drain system is prohibited. I required by the City, obtain an Industrial Waste Discharge Permit

OR

c. Other features which are comparable and equally effective, as approved by the City.

5. Vehicle Wash Areas

Projects that include areas for washing/steam cleaning of vehicles shall use the following:

a. Self-contained; or covered with a roof or overhang;
b. Equipped with a clarifier or other pretreatment facility;
c. Properly connected to a sanitary sewer, as approved by the City;
d. Other features which are comparable and equally effective, as approved by the City.

6. Outdoor Processing Areas

Outdoor process equipment operations, such as rock grinding or crushing, painting or coating, grinding or sanding, degreasing or parts cleaning, landfills, waste piles, and wastewater and solid waste treatment and disposal, and other operations determined to be a potential threat to water quality by the City of Chula Vista shall adhere to the following requirements.

a. Cover or enclose areas that would be the most significant source of pollutants; or, slope the area toward a dead-end sump; or, discharge to the sanitary sewer system following appropriate treatment in accordance with conditions established by the applicable sewer agency.
b. Grade or berm area to prevent run-on from surrounding areas.
c. Installation of storm drains in areas of equipment repair is prohibited.
d. Other features which are comparable or equally effective, as approved by the City.

7. Equipment Wash Areas

Outdoor equipment/accessory washing and steam cleaning activities at projects shall use the following:

a. Be self-contained; or covered with a roof or overhang;
b. Be equipped with a clarifier, grease trap or other pretreatment facility, as appropriate;
c. Be properly connected to a sanitary sewer after obtaining a permit from the City of San Diego Metropolitan Wastewater Department.
d. Other features which are comparable or equally effective, as approved by the City.

8 Parking Areas

To minimize the offsite transport of pollutants from parking areas, the following design concepts shall be considered, and incorporated and implemented where determined applicable and feasible by the City of Chula Vista:

a. Where landscaping is proposed in parking areas, incorporate landscape areas into the drainage design.
b. Overflow parking (parking stalls provided in excess of the City of Chula Vista's minimum parking requirements) may be constructed with permeable paving.
c. Other design concepts that are comparable and equally effective, as approved by the City.

9. Fueling Area

Fuel dispensing areas shall contain the following:

a. Overhanging roof structure or canopy. The cover's minimum dimensions must be equal to or greater than the area within the grade break. The cover must not drain onto the fuel dispensing area and the downspouts must be routed to prevent drainage across the fueling area. The fueling area shall drain to the project's treatment control BMP(s) prior to discharging to the storm water conveyance system.
b. Paved with Portland cement concrete (or equivalent smooth impervious surface). The use of asphalt concrete shall be prohibited.
c. Have an appropriate slope to prevent ponding, and must be separated from the rest of the site by a grade break that prevents run-on of urban runoff.
d. At a minimum, the concrete fuel dispensing area must extend 6.5 feet (2.0 meters) from the corner of each fuel dispenser, or the length at which the hose and nozzle assembly may be operated plus 1 foot (0.3 meter), whichever is less.
e. Other features which are comparable or equally effective, as approved by the City.

10 Hillside Landscaping

a. Hillside areas disturbed by project development shall be landscaped with deep-rooted, drought tolerant plant species selected for erosion control, satisfactory to the City of Chula Vista.
b. Other features which are comparable or equally effective, as approved by the City.

11 Design of Drainage Systems for Industrial/Commercial facilities

As required by the City in its sole discretion, Industrial/Commercial facilities with paved outdoor areas shall avoid sheet flow of runoff to the street gutter. Instead, all outdoor paved areas shall be directed to one or more storm drain sump(s) catch basin(s) before discharging to the public street gutter and/or public storm drainage systems. The sump(s) catch basin(s) shall be equipped with filters (inserts) or other Best Management Practices, satisfactory to the City of Chula Vista. Also, all private storm water facilities proposed shall be maintained by the property owner or approved private entity. The ongoing storm drainage systems maintenance records shall be kept on site indicating at the minimum, type of system, operator name, maintenance date, and maintenance activity type.

No maintenance agreement may be required. Maintenance of the proposed storm water facilities would be enforced by the City in accordance with the applicable City of Chula Vista ordinances, policies and regulations.

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CHAPTER 3 - IDENTIFICATION OF TYPICAL POLLUTANTS

CHAPTER 3 – IDENTIFICATION OF TYPICAL POLLUTANTS

3.1 – Anticipated Pollutants from Project Site

The following table details typical anticipated and potential pollutants generated by various land use types. The Otay Ranch Village 7development will consist of detached single-family residences. Thus, the *Detached Residential Development,* category has been highlighted to clearly illustrate which general pollutant categories are anticipated from the project area.

	General Pollutant Categories								
Priority Project Categories	**Sediments**	**Nutrients**	**Heavy Metals**	**Organic Compounds**	**Trash & Debris**	**Oxygen Demanding Substances**	**Oil & Grease**	**Bacteria & Viruses**	**Pesticides**
Detached Residential Development	X	X			X	X	X	X	X
Attached Residential Development	X	X			X	P(1)	P(2)	P	X
Commercial Development >100,000 ft^2	P(1)	P(1)		P(2)	X	P(5)	X	P(3)	P(5)
Automotive Repair Shops			X	X(4)(5)	X		X		
Restaurants					X	X	X	X	
Hillside Development >5,000 ft^2	X	X			X	X	X		X
Parking Lots	P(1)	P(1)	X		X	P(1)	X		P(1)
Streets, Highways & Freeways	X	P(1)	X	X(4)	X	P(5)	X		
Retail Gas Outlets			X	X(4)	X		X		

X = anticipated
P = potential
(1) A potential pollutant if landscaping exists on-site.
(2) A potential pollutant if the project includes uncovered parking areas.
(3) A potential pollutant if land use involves food or animal waste products.
(4) Including petroleum hydrocarbons.
(5) Including solvents.

*Source: "Water Quality Control Plan for San Diego Basin (9) RWQCB – San Diego Region"

3.2 – Sediment

Soils or other surface materials eroded and then transported or deposited by the action of wind, water, ice, or gravity. Sediments can increase turbidity, clog fish gills, reduce spawning habitat, smother bottom dwelling organisms, and suppress aquatic vegetative growth.

3.3 – Nutrients

Inorganic substances, such as nitrogen and phosphorous, that commonly exist in the form of mineral salts that are either dissolved or suspended in water. Primary sources of nutrients in urban runoff are fertilizers and eroded soils. Excessive discharge of nutrients to water bodies and streams can cause excessive aquatic algae and plant growth. Such excessive production, referred to as cultural eutrophication, may lead to excessive decay of organic matter in the water body, loss of oxygen in the water, release of toxins in sediment, and the eventual death of aquatic organisms.

3.4 Heavy Metals

Raw material components in non-metal products such as fuels, adhesives, paints and other coatings. Metals of concern include cadmium, chromium, copper, lead, mercury, and zinc. At high concentrations, metals can be toxic to aquatic life.

3.5 Organic Compounds

Carbon based commercially available or naturally occurring substances found in pesticides, solvents and hydrocarbons. Organic compounds can, at certain concentrations indirectly or directly constitute a hazard to life or health. When rinsing off objects toxic levels of solvents and cleaning compounds can be discharged to storm drains. Dirt, grease, and grime retained in the cleaning fluid or rinse water may also adsorb levels of organic compounds that are harmful or hazardous to aquatic life.

3.6 – Trash & Debris

Examples include paper, plastic, leaves, grass cuttings, and food waste, which may have a significant impact on the recreational value of a water body and aquatic habitat. Excess organic matter can create a high biochemical oxygen demand in a stream and thereby lower its water quality. In areas where stagnant water is present, the presence of excess organic matter can promote septic conditions resulting in the growth of undesirable organisms and the release of odorous and hazardous compounds such as hydrogen sulfide.

3.7 – Oxygen-Demanding Substances

Biodegradable organic material as well as chemicals that react with dissolved oxygen in water to form other compounds. Compounds such as ammonia and hydrogen sulfide are examples of oxygen-demanding compounds. The oxygen demand of a substance can lead to depletion of dissolved oxygen in a water body and possibly the development of septic conditions.

3.8 – Oil & Grease

Characterized as high high-molecular weight organic compounds. Primary sources of oil and grease are petroleum hydrocarbon products, motor products from leaking vehicles, oils, waxes, and high-molecular weight fatty acids. Elevated oil and grease content can decrease the aesthetic value of the water body, as well as the water quality.

3.9 – Bacteria and Viruses

Ubiquitous microorganisms that thrive under certain environmental conditions. Water, containing excessive bacteria and viruses can alter the aquatic habitat and create a harmful environment for humans and aquatic life. Decomposition of excess organic waste causes increased growth of undesirable organisms in water.

3.10 – Pesticides

Chemical compounds commonly used to control nuisance growth or prevalence of organisms. Excessive application of a pesticide may result in runoff containing toxic levels of its active component.

CHAPTER 4 – CONDITIONS OF CONCERN

CHAPTER 4 – CONDITIONS OF CONCERN

4.1 – Receiving Watershed Descriptions

As shown in the watershed map on the following page, the entire pre-developed and post developed Otay Ranch Village 7 project site drains to Wolf Canyon which eventually confluences with the Otay River. The Regional Water Quality Control Board has identified the Otay River as part of the Otay River Hydrologic Unit (basin number 10.20). The Otay River Hydrologic Unit drains to San Diego Bay at the Coronado Shoreline.

4.2 – Pollutants of Concern in Receiving Watersheds

The Otay River is not listed on the EPA's 303(d) List of endangered waterways however the San Diego Bay is included on the EPA 303(d) list (included in this Chapter). The Otay River drains to San Diego Bay near Imperial Beach. The San Diego Bay Shoreline at Tidelands Park in Coronado (significantly north of the Otay River outlet to San Diego Bay) is listed for bacterial indicators on the EPA's 303 (d) list. Per the "Water Quality Plan for the San Diego Basin", Table 2-2 (included at the end of this Chapter) the beneficial uses for the Otay River waterway includes agricultural supply, non-contact water recreation, warm freshwater habitat, and wildlife habitat. The Otay River is also listed for the potential beneficial use as industrial service supply, and contact water recreation. Table 2-2 also indicates that this waterway has been exempted by the Regional Board from the municipal use designation.

Table 3-2 from the "Water Quality Plan for the San Diego Basin" (included at the end of this Chapter) lists water quality objectives for a variety of potential pollutants required to sustain the beneficial uses of the Otay River hydrologic area.

4.3 – Peak Flow Attenuation

The "TM Drainage Study for Otay Ranch Village 7 Neighborhood R-2", prepared by Hunsaker & Associates, details the design of the detention facility for Village 7 and portions of La Media road. This report includes details of the HEC HMS analysis for the basin.

Table 1 on the following page summarizes contributing areas and 100-year flows for existing and developed conditions for the site's major discharge locations. As shown in the table, the proposed detention facility mitigates the 100-year peak flowrates well below the pre-developed peak flowrates from the study area at the proposed La Media crossing. The outflow of 109 cfs from the Village 7 detention basin is significantly lower than the pre-developed study area 100-year flow of 132 cfs at the same location. A schematic exhibit for the Village 7 basin appears at the end of this chapter.

TABLE 1

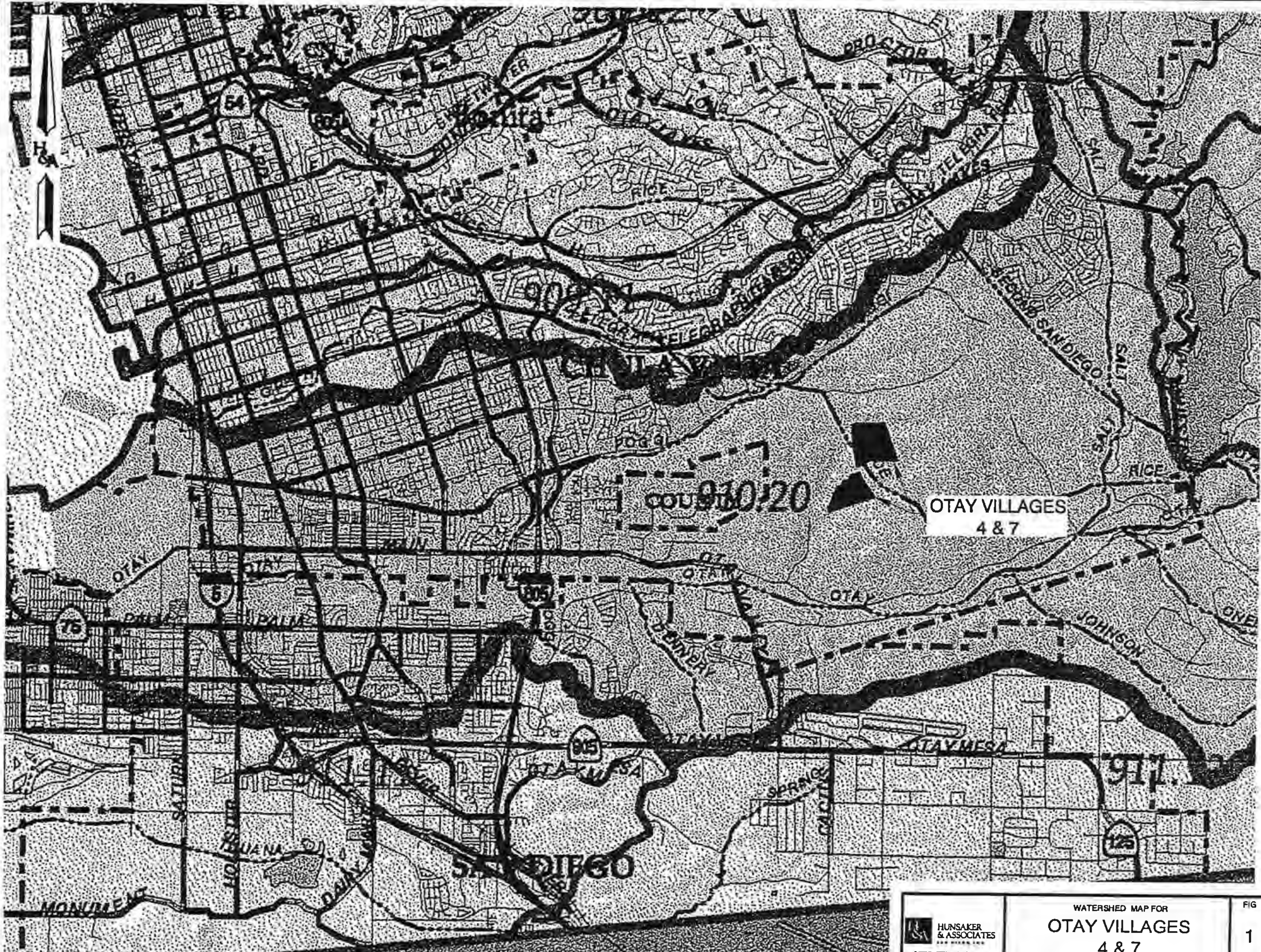
Location	Node Number	Existing Condition Drainage Area (acres)	Developed Condition Drainage Area (acres)	Difference (acres)	Existing Condition 100-Year Flow (cfs)	Developed Condition 100-Year Flow (cfs)	Difference (cfs)
Village 7 Site & La Media Flow	133	n/a	98.8	n/a	n/a	269.1	n/a
Natural Area South of Creek	30,32	n/a	39.1	n/a	n/a	139.8	n/a
Area South of Creek (Ultimate Conditions – Developed)	30,32	n/a	39.1	n/a	n/a	210.3*	n/a
La Media without detention	n/a	83.7	137.9	54.2	131.8	408.9	277.1
Detention Basin inflow	300	n/a	137.9	n/a	n/a	408.9	n/a
Detention Basin outflow	n/a	n/a	137.9	n/a	n/a	109.3	n/a
La Media with detention	n/a	83.7	137.9	54.2	131.8	84	-47.8

*Flow for Vortec property approximated based on ultimate conditions

The first 10 cfs of flow from the upstream McMillin development will be conveyed through the channel section between the McMillin development and La Media Road. Thus, all dry weather runoff will flow down the channel and sustain a steady water supply for the stream. Flows in excess of this 10 cfs will be conveyed through a storm drain pipe west of La Media Road. Due to the steep gradient of the channel reach in this area, riprap will be buried every 100 feet to prevent erosion.

Riprap downstream of La Media Road will be sized to mitigate culvert outlet velocities to non-erosive levels.

Chapter 4 – San Diego Region Hydrologic Divisions



WATERSHED MAP FOR

OTAY VILLAGES

4 & 7

CITY OF CHULA VISTA, CALIFORNIA

FIG 1

HUNSAKER & ASSOCIATES

Chapter 4 – Combined 1998 and Draft 2002 Section 303(d) Update

	San Diego Bay Shoreline Telegraph HSA (909.11&909.12)	at Chula Vista Marina	Bacterial Indicators[E]	0.4 miles	1998
	San Diego Bay Shoreline Coronado HA (910.10)	at Tidelands Park	Bacterial Indicators[E]	0.4 miles	2002
49 Tijuana HU (911.00) 911.11	Pacific Ocean Shoreline	from the border, extending north along the shore	Bacterial Indicators[E]	3.2 miles	1998
50 Tijuana HU (911.00) 911.41	Pine Valley Creek, Upper	lower portion	Enterococci	lower 2.9 miles	2002
51 San Ysidora HSA (911.11)	Tijuana River		Bacterial Indicators[E]	5.8 miles	1998
			Dissolved Oxygen, low		
			Eutrophic		
			Pesticides		
			Solids		
			Synthetic Organics		
			Trace Elements		
			Trash		
52 San Ysidora HSA (911.11)	Tijuana River Estuary		Bacterial Indicators[E]	150 acres	1998
			Eutrophic	1 acre	1998
			Lead		
			Nickel		
			Pesticides		
			Thallium		
			Trash		
			Oxygen (dissolved)	Entire Estuary	2002

[A] The 1998 List has been corrected as described in the text, pgs 17-18.

[B] The 1998 list, as adopted by the Regional Board, contained specific locations of impairment. These specific locations were omitted from the list as adopted by the USEPA. In 2002, it is recommended that these specific locations be included to better illustrate the location of impairment.

[C] In 1998, unless more information was available, the extent of impairment was assumed to be 0.1 miles for each shoreline impairment due to bacteria. The extents of impairment have been increased to 0.4 miles. Extents of impairment that were greater than 0.4 miles in 1998 were not changed. Rationale is described in Appendix B, pgs B69-

[D] This location was previously listed as "Pacific Ocean, Laguna Beach HSA"

[E] In 1998, Bacterial Indicators implies that impairment was due to either total coliform, fecal coliform, or both. In 2002, impairment may have also been caused by enterococci.

[F] The entire reach (7.2 miles) is listed for enterococci, *E. coli*, fecal coliform and toxicity. Additionally, Aliso Hills Channel, English Canyon Creek, Dairy Fork Creek, Sulphur Creek and Wood Canyon Creek are also listed for enterococci and *E. coli*. The lower 4 miles of Aliso Creek is listed for phosphorus.

[H] These locations and extents of impairment are approximated from interpretation of the 1996 Section 303(d) Report.

[J] This location was previously known as "San Diego Bay, at Downtown Piers."

[K] Area at the end of Switzer Creek, bound by piers on the north and south side of the outlet, extending to the edge of the piers.

Chapter 4 – Beneficial Uses of Inland Surface Waters

Table 2-2. BENEFICIAL USES OF INLAND SURFACE WATERS

Inland Surface Waters [1,2]	Hydrologic Unit Basin Number	MUN	AGR	IND	PROC	GWR	FRSH	POW	REC1	REC2	BIOL	WARM	COLD	WILD	RARE	SPWN
		BENEFICIAL USE														
Otay River Watershed - continued																
Wolf Canyon	10.20	+	●	O					O	●		●		●		
Dennery Canyon	10.20	+	●	O					O	●		●		●		
Pogi Canyon	10.20	+	●	O					O	●		●		●		
Tijuana River Watershed																
Tijuana River	11.11	+		O					O	●		●		●	●	
Moody Canyon	11.11	+		O					O	●		●		●		
Smugglers Gulch	11.11	+		O					O	●		●		●		
Goat Canyon	11.11	+		O					O	●		●		●		
Tijuana River Estuary	11.11	See Coastal Waters- Table 2-3														
Spring Canyon	11.12	+	●	O					O	●		●		●		
Dillon Canyon	11.12	+	●	O					O	●		●		●		
Finger Canyon	11.12	+	●	O					O	●		●		●		
Wruck Canyon	11.12	+	●	O					O	●		●		●		
unnamed intermittent streams	11.12	+	●	O					O	●		●		●		
unnamed intermittent streams	11.21	+							●	●		●		●		
Tijuana River	11.21	+							●	●		●		●		
Tecate Creek	11.23	+							●	●		●		●		

● Existing Beneficial Use

O Potential Beneficial Use

\+ Excepted From MUN (See Text)

[1] Waterbodies are listed multiple times if they cross hydrologic area or sub area boundaries.

[2] Beneficial use designations apply to all tributaries to the indicated waterbody, if not listed separately.

Chapter 4 – Water Quality Objectives

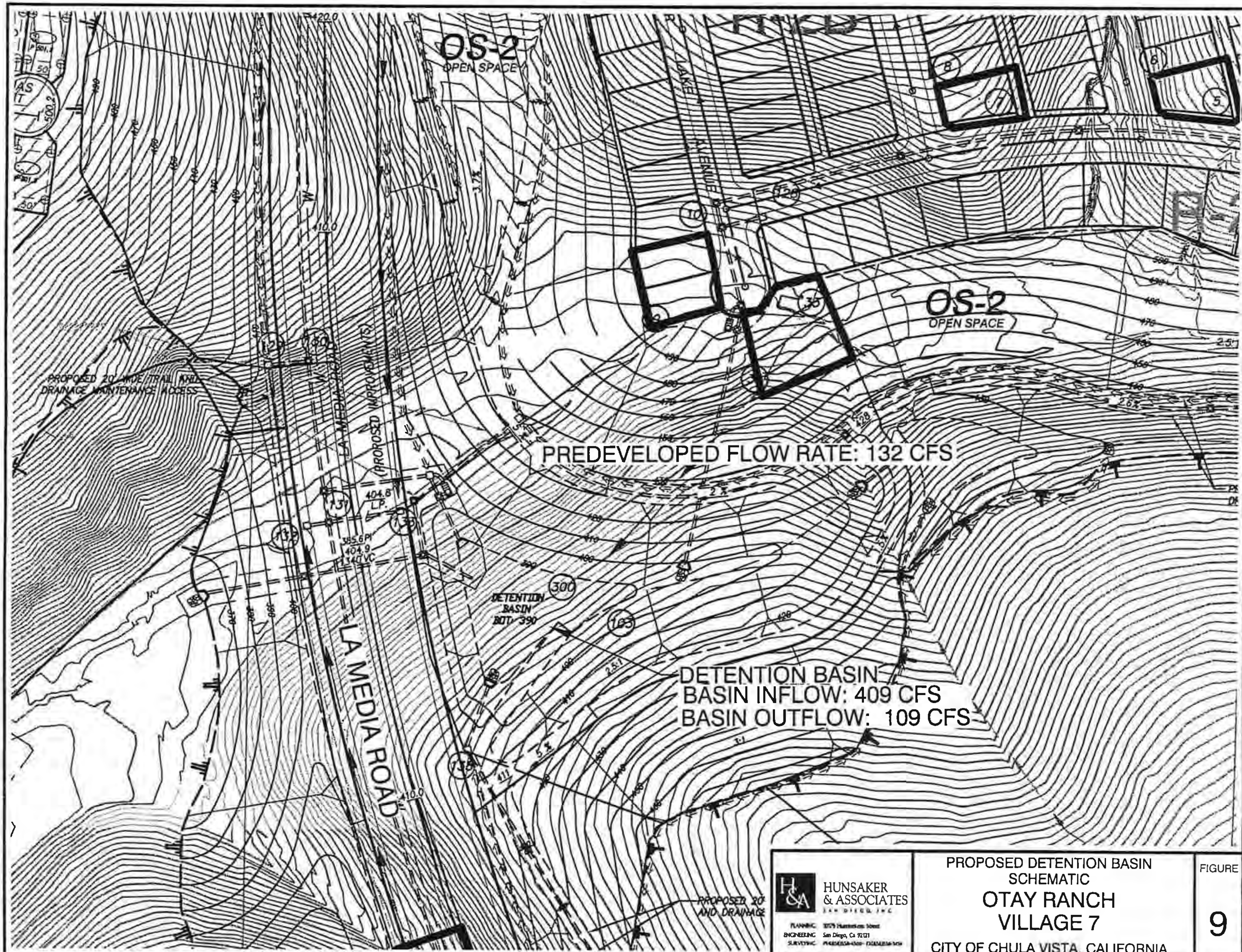
Table 3-2. WATER QUALITY OBJECTIVES

Concentrations not to be exceeded more than 10% of the time during any one one year period.

Inland Surface Waters		Hydrologic Unit Basin Number	Constituent (mg/L or as noted)												
			TDS	Cl	SO_4	%Na	N&P	Fe	Mn	MBAS	B	ODOR	Turb NTU	Color Units	F
OTAY HYDROLOGIC UNIT		910.00													
Coronado	HA	10.10	-	-	-	-	-	-	-	-	-	-	-	-	-
Otay Valley	HA	10.20	1000	400	500	60	a	0.3	0.05	0.5	0.75	none	20	20	1.0
Dulzura	HA	10.30	500	250	250	60	a	0.3	0.05	0.5	0.75	none	20	20	1.0
TIJUANA HYDROLOGIC UNIT		911.00													
Tijuana Valley	HA	11.10	-	-	-	-	-	-	-	-	-	-	-	-	-
San Ysidro	HSA	11.11	2100	-	-	-	a	-	-	-	-	none	20	20	-
Potrero	HA	11.20	500	250	250	60	a	0.3	0.05	0.5	1.0	none	20	20	1.0
Barrett Lake	HA	11.30	500	250	250	60	a	0.3	0.05	0.5	1.0	none	20	20	1.0
Monument	HA	11.40	500	250	250	60	a	0.3	0.05	0.5	1.0	none	20	20	1.0
Morena	HA	11.50	500	250	250	60	a	0.3	0.05	0.5	1.0	none	20	20	1.0
Cottonwood	HA	11.60	500	250	250	60	a	0.3	0.05	0.5	1.0	none	20	20	1.0
Cameron	HA	11.70	500	250	250	60	a	0.3	0.05	0.5	1.0	none	20	20	1.0
Campo	HA	11.80	500	250	250	60	a	0.3	0.05	0.5	1.0	none	20	20	1.0

HA - Hydrologic Area
HSA - Hydrologic Sub Area (Lower case letters indicate endnotes following the table.)

Chapter 4 – Detention Basin Schematic



HUNSAKER & ASSOCIATES SAN DIEGO, INC.

PROPOSED DETENTION BASIN SCHEMATIC

OTAY RANCH VILLAGE 7

CITY OF CHULA VISTA, CALIFORNIA

FIGURE 9

W.O.# 0025-346

R:\0188\&Hyd\02.04 Submittal\0188$H28-FIG 9-V7 WQ BASIN.dwg[kt]May-24-2004:08:08

CHAPTER 5 – FLOW-BASED BEST MANAGEMENT PRACTICES (BMPS)

CHAPTER 5 – FLOW-BASED BMPs

5.1 – Design Criteria

Flow-based BMPs shall be designed to mitigate the maximum flowrate of runoff produced from a rainfall intensity of 0.2 inch per hour. Such basins utilize either mechanical devices (such as vaults that produce vortex effects) or non-mechanical devices (based on weir hydraulics and specially designed filters) to promote settling and removal of pollutants from the runoff.

The Rational Method was the method used to determine the 85th percentile design runoff flow for the Otay Ranch Village 7 Neighborhood R-2 and Village 4 park site2.

As stated in the Introduction of Section II, Regional Water Quality Control Board regulations and NPDES criteria have established that flow-based BMPs shall be designed to mitigate a rainfall intensity of 0.2 inch per hour.

The Rational Method was used to calculate the 85th percentile runoff. The basic Rational Method runoff procedure is as follows:

Design Flow (Q) = C * I * A

Runoff Coefficient (C) – In accordance with the City of Chula Vista standards, the weighted runoff coefficient for all the areas tributary to the treatment unit was determined using the areas analyzed in the hydrology report. The runoff coefficient is based on the following characteristics of the watershed:

- Land Use – Single Family and Park Site
- Soil Type - Hydrologic soil group D was assumed for all areas. Group D soils have very slow infiltration rates when thoroughly wetted. Consisting chiefly of clay soils with a high swelling potential, soils with a high permanent water table, soils with clay pan or clay layer at or near the surface, and shallow soils over nearly impervious materials, Group D soils have a very slow rate of water transmission.

Rainfall Intensity (I) – Regional Water Quality Control Board regulations have established that flow-based BMPs shall be designed to mitigate a rainfall intensity of 0.2 inches per hour.

Watershed Area (A) – Project Area to Treatment Unit

The 85th percentile, 24-hour rainfall was derived from the isopluvial map provided by the County of San Diego (attached).

5.2 – Vortechs Treatment Units

The Vortechs Storm Water Treatment System is designed to efficiently remove grit, contaminated sediments, metals, hydrocarbons and floating contaminants from surface runoff. Combining swirl-concentrator and flow-control technologies to eliminate turbulence within the system, the Vortechs System ensures the effective capture of sediment and oils and prevents resuspension of trapped pollutants for flows up to 25 cfs.

Other features of the Vortechs Systems include the following:

- Large capacity system provides an 80 percent net annual Total Suspended Solids (TSS) removal rate
- Unit is installed below grade
- Low pump-out volume and one-point access reduce maintenance costs
- Design prevents oils and other floatables from escaping the system during cleanout
- Enhanced removal efficiencies of nutrients and heavy metals with offline configuration

The tangential inlet to the system creates a swirling motion that directs settleable solids into a pile towards the center of the grit chamber. Sediment is caught in the swirling flow path and settles back onto the pile after the storm event is over.

Floatable entrapment is achieved by sizing the low flow control to create a rise in the water level of the vault that is sufficient to just submerge the inlet pipe with the 85th percentile flow.

Chapter 5 – Pollutant Removal Efficiency Table (Flow-Based BMPs)

5.3 – Pollutant Removal Efficiency Table

Pollutant of Concern	BMP Categories	
	Hydrodynamic Separation Devices(2)	Vortechs™ Stormwater Treatment System
Sediment	M-H	H
Nutrients	L-M	L-M
Heavy Metals	L-M	L-M
Organic Compounds	L-M	L-M
Trash & Debris	M-H	H
Oxygen Demanding Substances	L	L
Bacteria	L	L
Oil & Grease	L-H	H
Pesticides	L	L

(1) The County will periodically assess the performance characteristics of these BMPs to update this table.
(2) Proprietary Structural BMPs. Not all serve the same function.
L (Low): Low removal efficiency (roughly 0-25%)
M (Medium): Medium removal efficiency (roughly 25-75%)
H (High): High removal efficiency (roughly 75-100%)
U: Unknown removal efficiency, applicant must provide evidence supporting use

Sources: *Guidance Specifying Management Measures for Sources of Nonpoint Pollution in Coastal Waters* (1993), *National Stormwater Best Management Practices Database* (2001), and *Guide for BMP Selection in Urban Developed Areas* (2001).

5.4 – Maintenance Requirements

Flow-based storm water treatment devices should be inspected periodically to assure their condition to treat anticipated runoff. Maintenance of the proposed Vortechnics units includes inspection and maintenance 1 to 4 times per year. Maintenance activities as well as the associated funding will be the responsibility of the Homeowners Association.

Maintenance of the Vortechs, or approved equivalent units involves the use of a "vactor truck", which clears the grit chamber of the treatment unit by vacuuming all the grit, oil and grease, and water from the sump. Typically a 3-man crew is required to perform the maintenance of the treatment unit. Properly maintained Vortechs, or approved equivalent Systems will only require evacuation of the grit chamber portion of the system. In some cases, it may be necessary to pump out all chambers. In the event of cleaning other chambers, it is imperative that the grit chamber be drained first.

Proper inspection includes a visual observation to ascertain whether the unit is functioning properly and measuring the amount of deposition in the unit. Floatables should be removed and sumps cleaned when the sump storage exceeds 85 percent of capacity specifically, or when the sediment depth has accumulated within 6 inches of the dry-weather water level. The rate at which the system collects pollutants will depend more heavily on site activities than the size of the unit.

5.5 – Schedule of Maintenance Activities

Target Maintenance Date – March 15th
Maintenance Activity – Annual inspection and cleanout. Clear grit chamber of each unit with vactor truck. Perform visual inspection. Remove floatables.

DrainPac Filter Units:

Target Maintenance Dates – June 15th, September 15th (Dry Season Inspections)
Maintenance Activity - Regular inspection to ensure that filter unit is functioning properly, has not become clogged, and does not need to be replaced;

Target Maintenance Dates – 15th of each month; October through April (Rainy Season Inspections)
Maintenance Activity - Regular inspection to ensure that filter unit is functioning properly, has not become clogged, and does not need to be replaced;

Target Maintenance Date – March 15th, June 15th, September 15th, December 15th
Maintenance Activity – Quarterly cleanouts; Cleanout filter, remove trash, debris and excess sediment.

Target Maintenance Dates – March 15th
Maintenance Activity – Annual filter replacement; Remove and replace filter. Dispose of used filter according to state and federal environmental protection guidelines. Place new filter in existing bracket below the storm drain entrance.

For proper maintenance to be performed, the storm water treatment facility must be accessible to both maintenance personnel and their equipment and materials.

5.6 – Annual Operations & Maintenance Costs

The following costs are intended only to provide a magnitude of the costs involved in maintaining BMPs. Specific unit costs shall be verified prior to the formation of the respective maintenance CFD.

Approximate annual maintenance costs for each of the proposed Vortechs units are outlined below. Costs assume a 3 man crew:

Maintenance for Model 7000 (Unit #1 at Node 133)

Periodic Inspection, Maintenance and Monitoring = $800
Annual Cleanout Cost = $1,800

Subtotal = $2,600
Contingency = $260

Total = $2,860

Maintenance for Model 5,000 (Unit #2 at Node 128)

Periodic Inspection, Maintenance and Monitoring = $800
Annual Cleanout Cost = $1,500

Subtotal = $2,300
Contingency = $230

Total = $2,530

Maintenance for Model 4,000 (at Village 4 Park Site)

Periodic Inspection, Maintenance and Monitoring = $800
Annual Cleanout Cost = $1,250

Subtotal = $2,050
Contingency = $205

Total = $2,255

CHAPTER 6 – SITE DESIGN/ SOURCE CONTROL BMPS

CHAPTER 6 – SITE DESIGN/SOURCE CONTROL BMPs

6.1 Site Design

Priority projects, such as the Otay Village 7 and Village 4 park site projects, shall be designed to minimize, to the maximum extent practicable, the introduction of pollutants and conditions of concern from site runoff to the storm water conveyance system. Site design components can significantly reduce the impact of a project on the environment. The following design techniques have been proposed to accomplish this goal.

- Implementing on-lot hydrologically functional landscape design and management practices; Additional detail regarding landscaping design is discussed in section 7.2.
- Minimizing project's impervious footprint. Methods of accomplishing this goal include constructing streets, sidewalks, and parking lots to the minimum widths necessary without compromising public safety. Another method for minimizing impervious area includes incorporating landscaped areas in the drainage system to encourage infiltration and reduce the amount of directly connected impervious areas.

6.2 – Landscaping

Manufactured slopes shall be landscaped with suitable ground cover or installed with an erosion control system. Homeowners should be educated as to the proper routine maintenance to landscaped areas including trimming, pruning, weeding, mowing, replacement or substitution of vegetation in ornamental and required landscapes.

Per the RWQCB Order, the following landscaping activities are deemed unlawful and are thus prohibited:

- Discharges of sediment
- Discharges of pet waste
- Discharges of vegetative clippings
- Discharges of other landscaping or construction-related wastes.

Priority projects shall design the timing and application methods of irrigation water to minimize the runoff of excess irrigation water into the storm water conveyance system. The following techniques may be incorporated into the site design to reduce excessive irrigation.

- Employ rain shutoff devices to prevent irrigation after precipitation
- Design irrigation systems to each landscaped area's specific water requirements
- Design flow reducers or shutoff valves triggered by a pressure drop to control water loss in the event of broken sprinkler heads or lines

6.3 – Urban Housekeeping

Fertilizer applied by homeowners, in addition to organic matter such as leaves and lawn clippings; all result in nutrients in storm water runoff. Consumer use of excessive herbicide or pesticide contributes toxic chemicals to runoff. Homeowners should be educated as to the proper application of fertilizers and herbicides to lawns and gardens.

The average household contains a wide variety of toxins such as oil/grease, antifreeze, paint, household cleaners and solvents. Homeowners should be educated as to the proper use, storage, and disposal of these potential storm water runoff contaminants.

Per the RWQCB Order, the following housekeeping activities are deemed unlawful and are thus prohibited:

- Discharges of wash water from the cleaning or hosing of impervious surfaces including parking lots, streets, sidewalks, driveways, patios, plazas, and outdoor eating and drinking areas (landscape irrigation and lawn watering, as well as non-commercial washing of vehicles in residential zones, is exempt from this restriction)

- Discharges of pool or fountain water containing chloride, biocides, or other chemicals.

- Discharges or runoff from material storage areas containing chemicals, fuels, grease, oil, or other hazardous materials.

- Discharges of food-related wastes (grease, food processing, trash bin wash water, etc.).

6.4 – Automobile Use

Urban pollutants resulting from automobile use include oil, grease, antifreeze, hydraulic fluids, copper from brakes, and various fuels. Homeowners should be educated as to the proper use, storage, and disposal of these potential storm water contaminants.

Per the RWQCB Order, the following automobile use activities are deemed unlawful and are thus prohibited:

- Discharges of wash water from the hosing or cleaning of gas stations, auto repair garages, or other types of automotive service facilities.

- Discharges resulting from the cleaning, repair, or maintenance of any type of equipment, machinery, or facility including motor vehicles, cement-related equipment, port-a-potty servicing, etc.

- Discharges of wash water from mobile operations such as mobile automobile washing, steam cleaning, power washing, and carpet cleaning.

The Homeowners Association should make all homeowners aware of the aforementioned RWQCB regulations through a homeowners' education program. A monitoring program should also be implemented to insure compliance.

CHAPTER 7 – SITE BMP DESIGN (VORTECHS TREATMENT UNITS)

CHAPTER 7 – SITE BMP DESIGN VORTECHS TREATMENT UNITS

7.1 – BMP Location

The Otay Ranch Village 7 and La Media development includes two flow-based BMP treatment units and the Village 4 project development has incorporated one flow-based BMP treatment unit (shown on BMP Location Map located on the following page) into the site design. These treatment devices will be sized to treat the 85th percentile flow, as determined by the Rational method.

7.2 – Determination of Design Treatment Flows

The following table summarizes the parameters used for determination of design flows to the flow-based treatment unit.

DESIGN RUNOFF DETERMINATION SUMMARY TABLE

Treatment Unit	Runoff Coefficient (C)	Rainfall Intensity (in/hr)	Drainage Area (acres)	85th Pct. Design Flow (cfs)
Unit #1 – Node 133, La Media	0.70	0.2	76.4	10.7
Unit #2 – Node 128, SW Corner of site	0.65	0.2	46.9	6.10
@ SW Corner of Park Site	0.49	0.2	44	4.3

7.3 – Treatment Unit Selection

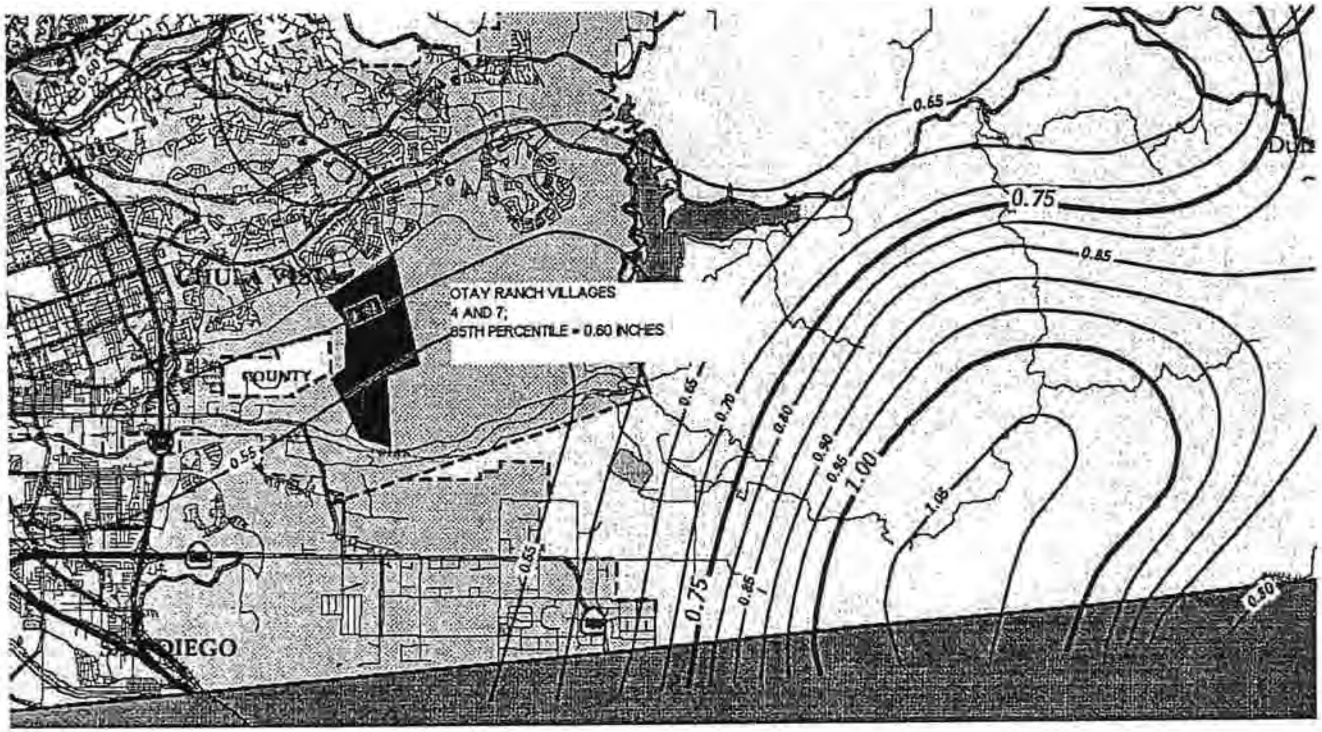
The proposed Vortechs units (or approved equivalent systems) are offline precast treatment units. The 85th percentile design flow rates will be treated while flows in excess of this flow rate bypass the swirl chamber (treatment) and proceed downstream. The decision on the selection of storm water treatment units was made based on the storm water treatment system's ability to adequately treat the design runoff flow.

Based on the results of the rational method 85th percentile flow calculations, a Vortechs Model 7000 has been selected for Unit #1, Model 5000 has been selected for Unit #2 and Model 4000 has been selected to treat runoff from the park site. The treatment capacity for the Vortechs systems are provided in a table at the end of this chapter.

The treatment units will be sized to treat flows greater than the 85th percentile storm, since additional storm water will be diverted by the diversion structure for storm events in excess of the 85th percentile storm. Calculations used to determine the

100-year diverted flow into the treatment unit will be included in subsequent reports, when the storm drain design becomes more defined and further hydrologic and hydraulic analysis of each storm drain system is completed.

Chapter 7 – 85th Percentile Rainfall Isopluvial Map


OTAY RANCH VILLAGES
4 AND 7;
85TH PERCENTILE = 0.60 INCHES
COUNTY
0.65
0.75
0.85
0.70
0.80
0.90
0.95
1.00
1.05
0.55
0.50
DIEGO

Chapter 7 – 85th Percentile Rational Method Calculations

85TH PERCENTILE PEAK FLOW AND VOLUME DETERMINATION
Modified Rational Method - Effective for Watersheds < 1.0 mi^2
Hunsaker & Associates - San Diego

Note: Only Enter Values in Boxes - Spreadsheet Will Calculate Remaining Values

Project Name	Otay Ranch Village 7, Neighborhood R-2
Work Order	0025-346
Jurisdiction	City of Chula Vista

BMP Location Unit #1: La Media Road, Node # 133

85th Percentile Rainfall = **0.60** inches
(from County Isopluvial Map)

Developed Drainage Area =	76.4	acres
Natural Drainage Area =	0.0	acres
Total Drainage Area to BMP =	**76.4**	acres

Dev. Area Percent Impervious =	60	%
Overall Percent Impervious =	**60**	%

Dev. Area Runoff Coefficient =	0.70
Nat. Area Runoff Coefficient =	0.35
Runoff Coefficient =	**0.70**

Time of Concentration = **13.4** minutes
(from Drainage Study)

RATIONAL METHOD RESULTS

Q = CIA where
- Q = 85th Percentile Peak Flow (cfs)
- C = Runoff Coefficient
- I = Rainfall Intensity (0.2 inch/hour per RWQCB mandate)
- A = Drainage Area (acres)

V = CPA where
- Q = 85th Percentile Runoff Volume (acre-feet)
- C = Runoff Coefficient
- P = 85th Percentile Rainfall (inches)
- A = Drainage Area (acres

Using the Total Drainage Area:

C =	0.70	
I =	0.2	inch/hour
P =	0.60	inches
A =	76.4	acres
Q =	**10.69**	**cfs**
V =	**2.67**	**acre-feet**

Using Developed Area Only:

C =	0.70	
I =	0.2	inch/hour
P =	0.60	inches
A =	76.4	acres
Q =	**10.69**	**cfs**
V =	**2.67**	**acre-feet**

85TH PERCENTILE PEAK FLOW AND VOLUME DETERMINATION
Modified Rational Method - Effective for Watersheds < 1.0 mi^2
Hunsaker & Associates - San Diego

Note: Only Enter Values in Boxes - Spreadsheet Will Calculate Remaining Values

Project Name	Otay Ranch Village 7, Neighborhood R-2
Work Order	0025-346
Jurisdiction	City of Chula Vista

BMP Location	Unit #2: Southwest corner of development, Node # 128

85th Percentile Rainfall =	**0.60**	inches
(from County Isopluvial Map)		
Developed Drainage Area =	46.9	acres
Natural Drainage Area =	0.0	acres
Total Drainage Area to BMP =	**46.9**	acres
Dev. Area Percent Impervious =	50	%
Overall Percent Impervious =	**50**	%
Dev. Area Runoff Coefficient =	0.65	
Nat. Area Runoff Coefficient =	0.35	
Runoff Coefficient =	**0.65**	
Time of Concentration =	**12.0**	minutes
(from Drainage Study)		

RATIONAL METHOD RESULTS

Q = CIA	where	Q =	85th Percentile Peak Flow (cfs)
		C =	Runoff Coefficient
		I =	Rainfall Intensity (0.2 inch/hour per RWQCB mandate)
		A =	Drainage Area (acres)
V = CPA	where	Q =	85th Percentile Runoff Volume (acre-feet)
		C =	Runoff Coefficient
		P =	85th Percentile Rainfall (inches)
		A =	Drainage Area (acres

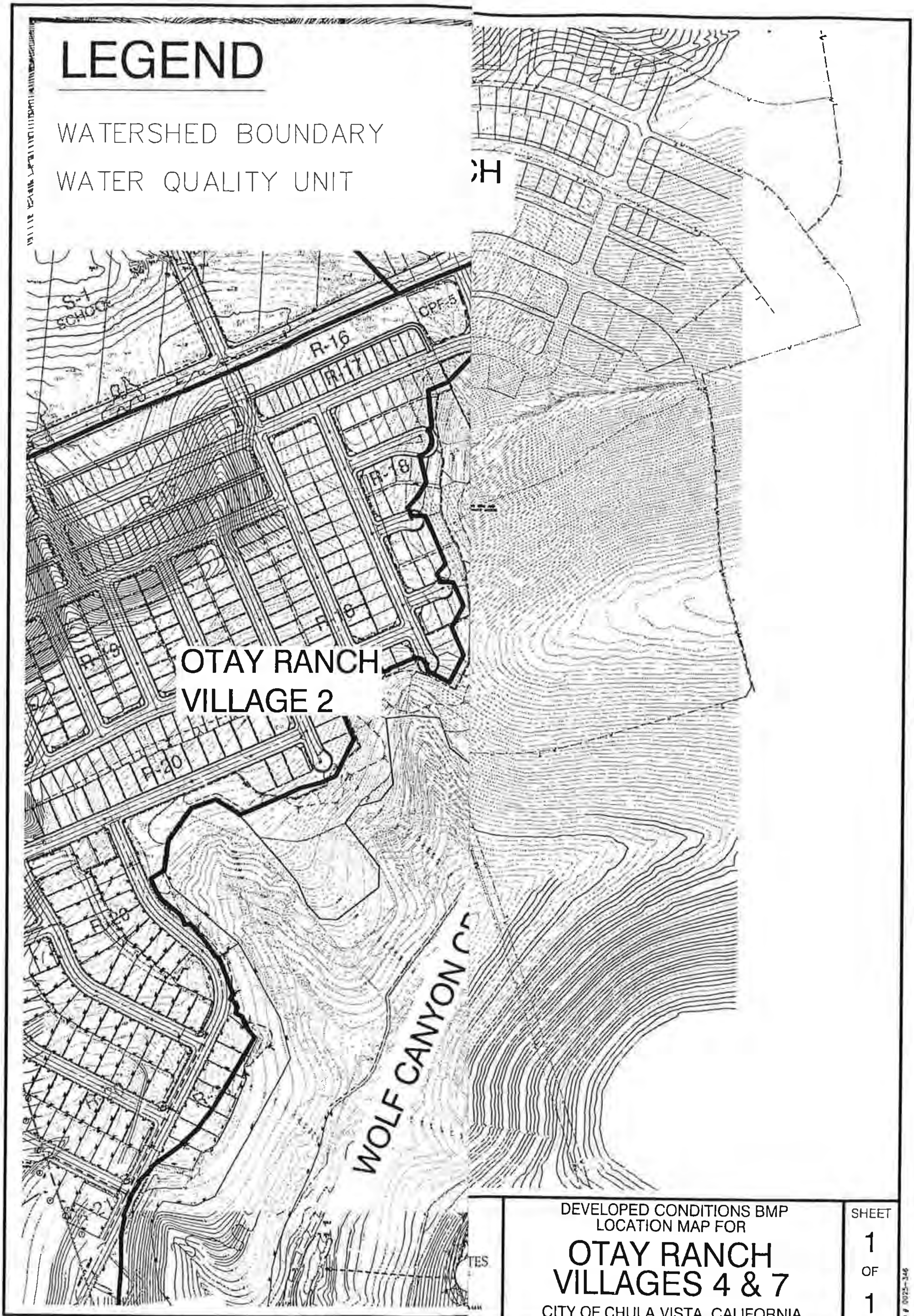
Using the Total Drainage Area:

C =	0.65	
I =	0.2	inch/hour
P =	0.60	inches
A =	46.9	acres
Q =	**6.10**	**cfs**
V =	**1.52**	**acre-feet**

Using Developed Area Only:

C =	0.65	
I =	0.2	inch/hour
P =	0.60	inches
A =	46.9	acres
Q =	**6.10**	**cfs**
V =	**1.52**	**acre-feet**

Chapter 7 – BMP Location Map



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Chapter 7 – Vortechs Capacity Table

APPROXIMATE VORTECHNICS CAPACITIES

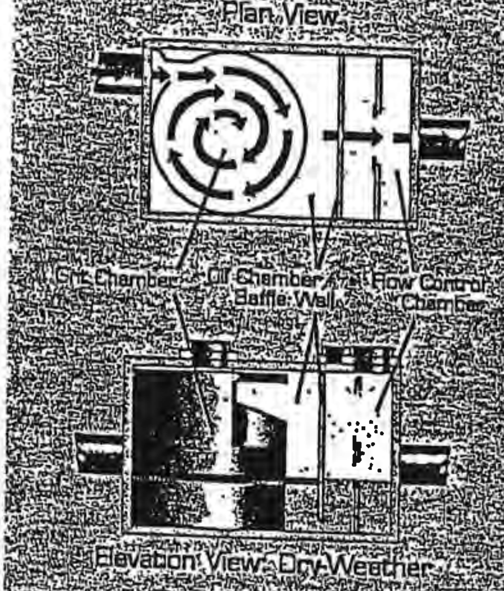
MODEL	Q treatment (CFS)	AREA (ACRES)				
		COMML.	CONDO.	8-10 DU	5-8 DU	3-5 DU
Model 1000	1.6	10	14	15	18	23
Model 2000	2.8	17	24	26	31	39
Model 3000	4.5	27	38	41	49	62
Model 4000	6.0	37	51	55	66	83
Model 5000	8.5	52	72	78	94	117
Model 7000	11.0	73	102	110	132	165
Model 9000	14.0	93	129	140	168	210
Model 11,000	17.5	117	162	175	210	263
Model 16,000	25.0	167	231	250	300	375

* Equipment costs include delivery to jobsite (FOB) and assembly (not installation).
THIS DOES **NOT** INCLUDE A WEIR DIVERSION BOX.

** Equipment and installation. Installation costs (25% - 30%) will vary depending on site conditions.

Chapter 7 – Vortechs System Data

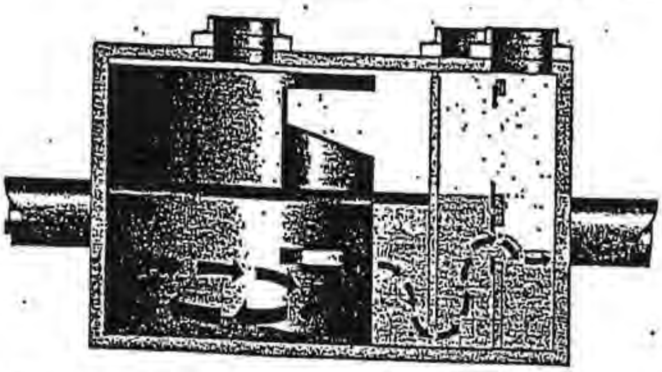
Features & Operation

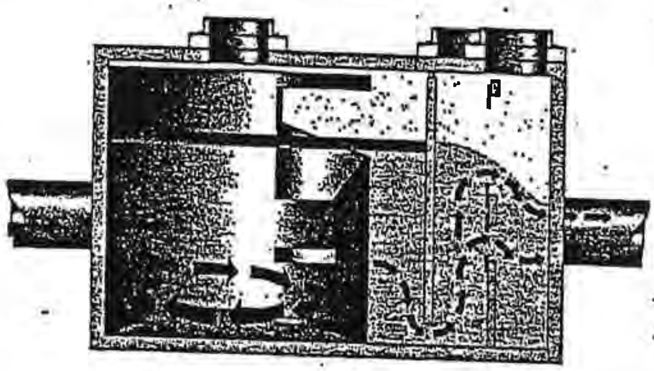


Grit Chamber
The swirling motion created by the tangential inlet directs settleable solids toward the center of this chamber. Sediment is caught in the swirling flow path and settles back onto the pile after the storm event is over.

Oil Chamber & Baffle Wall
The center baffle traps floatables in the oil chamber, even during clean-out. Highly resistant to flow surges.

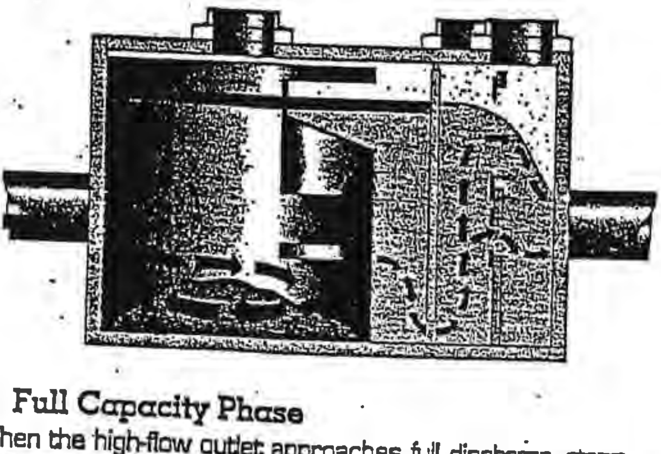
Flow Control Chamber
The weir and orifice flow controls: 1) Raise level and volume in the system as flow rate increases, and 2) gradually drain the system as flow rate subsides.

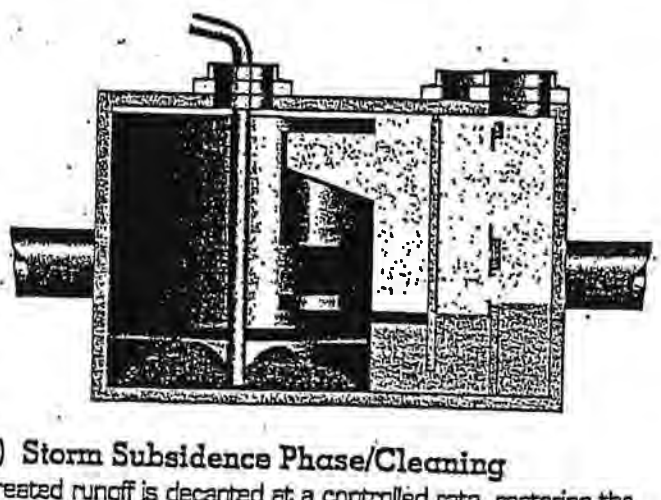




1) Initial Wet Weather Phase
During a two-month storm event the water level begins to rise above the top of the inlet pipe. This influent control feature reduces turbulence and avoids resuspension of pollutants.

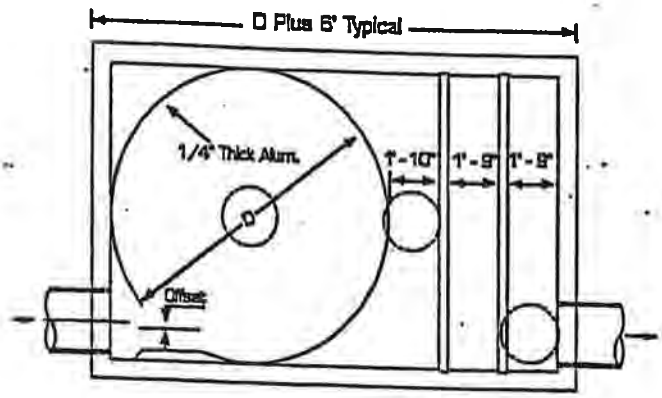
2) Transition Phase
As the inflow rate increases above the controlled outflow rate, the tank fills and the floating contaminant layer accumulated from past storms rises. Swirling action increases at this stage, while sediment pile remains stable.



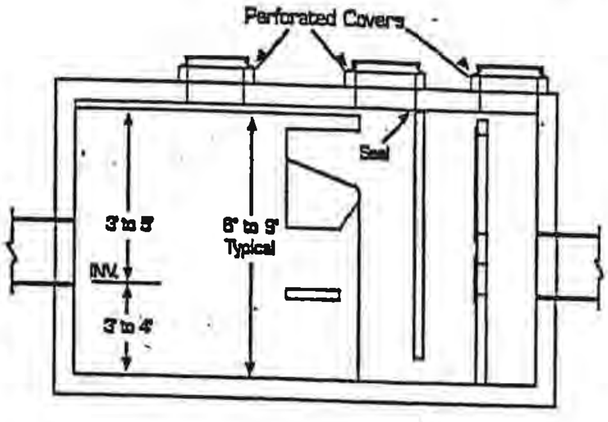


3) Full Capacity Phase
When the high-flow outlet approaches full discharge, storm drains are flowing at peak capacity. The Vortechs System is designed to match your design storm flow and provide treatment throughout the range of storm events without bypass-

4) Storm Subsidence Phase/Cleaning
Treated runoff is decanted at a controlled rate, restoring the water level to a low dry-weather volume and revealing a conical pile of sediment. The low water level facilitates inspection and cleaning, and significantly reduces



Plan View



Elevation View

To begin the design of your Vortechs System, refer to the sizing chart below and complete a Specifier's Worksheet to provide details about your site and design flows. Then simply fax or mail the worksheet to Vortechnics with your site plan, and we'll produce detailed Vortechs System scale drawings free of charge.

Vortechs™ Model	Grit Chamber Diameter/Area ft/ft²	Peak Design Flow[A] cfs	Sediment Storage[B] yds³	Approx. Size[C] L x W ft
1000	3/7	1.6	.75	9 x 3
2000	4/13	2.8	1.25	10 x 4
3000	5/20	4.5	1.75	11 x 5
4000	6/28	6.0	2.5	12 x 6
5000	7/38	8.5	3.25	13 x 7
7000	8/50	11.0	4.0	14 x 8
9000	9/64	14.0	4.75	15 x 9
11000	10/79	17.5	5.5	16 x 10
16000	12/113	25.0	7.0	18 x 12

Engineering Notes

A) For In-line Vortechs Systems without a bypass, sizing criteria is based on providing one square foot of grit chamber surface area for each 100 gpm of peak design storm flow rate (e.g., 10-year storm). For more details about Vortechnics sizing criteria refer to Vortechnics Technical Bulletin 3.

B) Sediment storage volume assumes a 3 foot sump.

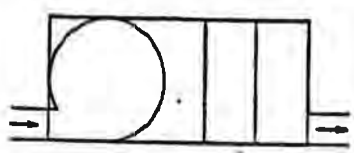
C) Construction details may vary depending on the specific application. Any alterations to the sizing chart specifications will appear on Vortechnics dimensional and shop drawings. Please call Vortechnics for the weight of specific Vortechs systems if needed.

Special Note: Oil storage capacity, when it is needed to meet a specific requirement for spill containment, can be sized to meet the storage requirement with the selected model. Vortechnics technical staff will optimize system geometry to meet containment requirements within a correctly sized Vortechs System.

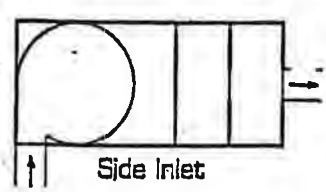
Metric Specification Chart available by calling Vortechnics at (207) 878-3662.

Vortechs System Inlet/Outlet Configurations

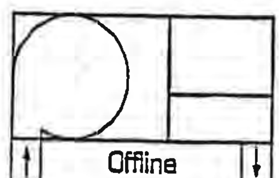
Vortechs Systems can be configured to accommodate various inlet and outlet pipe orientations. The inlet pipe can enter the end or side of the tank at right angles - outlet pipes can exit the end or the side of system at most angles.



End Inlet



Side Inlet



Offline

Pretreatment

SECTION 02721

STORMWATER TREATMENT SYSTEM

PART 1.00 GENERAL

1.01 DESCRIPTION

A. Work Included:

The Contractor, and/or a manufacturer selected by the Contractor and approved by the Engineer, shall furnish all labor, materials, equipment and incidentals required and install all precast concrete stormwater treatment systems and appurtenances in accordance with the Drawings and these specifications.

B. Related work described elsewhere:

1. Unit Masonry
2. Miscellaneous Metals
3. Waterproofing

1.02 QUALITY CONTROL INSPECTION

A. The quality of materials, the process of manufacture, and the finished sections shall be subject to inspection by the Engineer. Such inspection may be made at the place of manufacture, or on the work site after delivery, or at both places, and the sections shall be subject to rejection at any time if material conditions fail to meet any of the specification requirements, even though sample sections may have been accepted as satisfactory at the place of manufacture. Sections rejected after delivery to the site shall be marked for identification and shall be removed from the site at once. All sections which have been damaged beyond repair during delivery will be rejected and, if already installed, shall be repaired to the Engineer's acceptance level, if permitted, or removed and replaced, entirely at the Contractor's expense.

B. All sections shall be inspected for general appearance, dimensions, soundness, etc. The surface shall be dense, close textured and free of blisters, cracks, roughness and exposure of reinforcement.

C. Imperfections may be repaired, subject to the acceptance of the Engineer, after demonstration by the manufacturer that strong and permanent repairs result. Repairs shall be carefully inspected before final acceptance. Cement mortar used for repairs shall have a minimum compressive strength of 4,000 psi at the end of 7 days and 5,000 psi at the end of 28 days when tested in 3 inch diameter by 6 inch long cylinders stored in the standard manner. Epoxy mortar may be utilized for repairs.

1.03 SUBMITTALS

A. Shop Drawings

The Contractor shall be provided with dimensional drawings and, when specified, utilize these drawings as the basis for preparation of shop drawings showing details for construction, reinforcing, joints and any cast-in-place appurtenances. Shop drawings shall be annotated to indicate all materials to be used and all applicable standards for materials, required tests of materials and design assumptions for structural analysis. Design calculations and shop drawings shall be certified by a Professional Engineer retained by the system manufacturer or contractor and licensed in the state where the system is to be installed. Shop drawings shall be prepared at a scale of not less than 1/4" per foot. Six (6) hard copies of said shop drawings shall be submitted to the Engineer for review and approval.

B. Affidavit on patent infringement

The Contractor shall submit to the Engineer, prior to installation of the stormwater treatment system, an affidavit regarding patent infringement rights stating that any suit or claim against the Owner due to alleged infringement rights shall be defended by the Contractor who will bear all the costs, expenses and attorney's fees incurred thereof.

PART 2.00 PRODUCTS

2.01 MATERIALS AND DESIGN

A. Concrete for precast stormwater treatment systems shall conform to ASTM C 857 and C 858 and meet the following additional requirements:

1. The wall thickness shall not be less than 6 inches or as shown on the dimensional drawings. In all cases the wall thickness shall be no less than the minimum thickness necessary to sustain HS20-44 loading requirements as determined by a Licensed Professional Engineer.

2. Sections shall have tongue and groove or ship-lap joints with a butyl mastic sealant conforming to ASTM C 990.

3. Cement shall be Type III Portland cement conforming to ASTM C 150.

4. Pipe openings shall be sized to accept pipes of the specified size(s) and material(s), and shall be sealed by the Contractor with a hydraulic cement conforming to ASTM C 595M

5. Internal metal components shall be aluminum alloy 5052-H32 in accordance with ASTM B 209.

6. Brick or masonry used to build the manhole frame to grade shall conform to ASTM C 32 or ASTM C 139 and the Masonry Section of these Specifications.

7. Casting for manhole frames and covers shall be in accordance with The Miscellaneous Metals Section of these Specifications.

8. All sections shall be cured by an approved method. Sections shall not be shipped until the concrete has attained a compressive strength of 4,000 psi or util 5 days after fabrication and/or repair, whichever is the longer.

9. A butimen sealant in conformance with ASTM C 990 shall be utilized in affixing the aluminum swirl chamber to the concrete vault.

2.02 PERFORMANCE

Each stormwater treatment system shall adhere to the following performance specifications at the specified design flows, as listed below:

Table 2.02

Vortechs Model	Swirl Chamber Diameter (ft)	Design Treatment Capacity (cfs)	Sediment Storage (yd^3)
1000	3.67	2.3	1.00
2000	4	2.8	1.25
3000	5	4.5	1.75
4000	6	6.0	2.50
5000	7	8.5	3.25
7000	8	11.0	4.00
9000	9	14.0	4.75
11000	10	17.5	5.50
16000	12	25.0	7.00

Each stormwater treatment system shall include a circular aluminum "swirl chamber" (or "grit chamber") with a tangential inlet to induce a swirling flow pattern that will accumulate and store settleable solids in a manner and a location that will prevent re-suspension of previously captured particulates. Each swirl chamber diameter shall not be less than the diameter listed in Table 2.02 (neglecting chamber wall thickness).

Each stormwater treatment system shall be of a hydraulic design that includes flow controls designed and certified by a professional engineer using accepted principles of fluid mechanics that raise the water surface inside the tank to a pre-determined level in order to prevent the re-entrainment of trapped floating contaminants.

Each stormwater treatment system shall be capable of removing 80% of the net annual Total Suspended Solids (TSS). Individual stormwater treatment systems shall have the Design Treatment Capacity listed in Table 2.02, and shall not resuspend trapped sediments or re-entrain floating contaminants at flow rates up to and including the specified Design Treatment Capacity.

Individual stormwater treatment systems shall have usable sediment storage capacity of not less than the corresponding volume listed in Table 2.02. The systems shall be designed such that the pump-out volume is less than ½ of the total system volume. The systems shall be designed to not allow surcharge of the upstream piping network during dry weather conditions.

A water-lock feature shall be incorporated into the design of the stormwater treatment system to prevent the introduction of trapped oil and floatable contaminants to the downstream piping during routine maintenance and to ensure that no oil escapes the system during the ensuing rain event. Direct access shall be provided to the sediment and floatable contaminant storage chambers to facilitate maintenance. There shall be no appurtenances or restrictions within these chambers.

The stormwater treatment system manufacturer shall furnish documentation which supports all product performance claims and features, storage capacities and maintenance requirements.

Stormwater treatment systems shall be completely housed within one rectangular structure.

2.03 <u>MANUFACTURER</u>

Each stormwater treatment system shall be of a type that has been installed and used successfully for a minimum of 5 years. The manufacturer of said system shall have been regularly engaged in the engineering design and production of systems for the physical treatment of stormwater runoff.

Each stormwater treatment system shall be a Vortechs™ System as manufactured by Vortechnics, Inc., 41 Evergreen Drive, Portland, Maine 04103, phone: 207-878-3662, fax: 207-878-8507; and as protected under U.S. Patent # 5,759,415.

<u>PART 3.00 EXECUTION</u>

3.01 <u>INSTALLATION</u>

A. Each Stormwater Treatment System shall be constructed according to the sizes shown on the Drawings and as specified herein. Install at elevations and locations shown on the Drawings or as otherwise directed by the Engineer.

B. Place the precast base unit on a granular subbase of minimum thickness of six inches after compaction or of greater thickness and compaction if specified elsewhere. The granular subbase shall be checked for level prior to setting and the precast base section of the trap shall be checked for level at all four corners after it is set. If the slope from any corner to any other corner exceeds 0.5% the base section shall be removed and the granular subbase material re-leveled.

C. Prior to setting subsequent sections place butimen sealant in conformance with ASTM C990-91 along the construction joint in the section that is already in place.

D. After setting the base and wall or riser sections install the circular swirl chamber wall by bolting the swirl chamber to the side walls at the three (3) tangent points and at the 3-inch wide inlet tab using HILTI brand concrete anchors or equivalent 1/2-inch diameter by 2-3/4" minimum length at heights of approximately three inches (3") off the floor and at the mid-height of the completed trap (at locations of pre-drilled holes in aluminum components). Seal the bottom edge of the swirl chamber to the trap floor with the supplied aluminum angle flange. Adhere ¼" thick by 1" wide neoprene sponge material to the flange with half of it's width on the horizontal leg of the flange and half of it's width on the vertical leg. The aluminum angle flange shall be affixed to the floor with a minimum 3/8" diameter by 2-3/4" drop in wedge anchor at the location of the predrilled holes. Affix the swirl chamber to the flange with hex head ¼" x 1-1/2" zinc coated self- tapping screws at the location of the predrilled holes. Seal the vault sidewalls to the outside of the swirl chamber from the floor to the same height as the inlet pipe invert using butyl mastic or approved equal.

E. Prior to setting the precast roof section, butimen sealant equal to ASTM C990 shall be placed along the top of the baffle wall, using more than one layer of mastic if necessary, to a thickness at least one inch (1") greater than the nominal gap between the top of the baffle and the roof section.

The nominal gap shall be determined either by field measurement or the shop drawings. After placement of the roof section has compressed the butyl mastic sealant in the gap, finish sealing the gap with an approved non-shrink grout on both sides of the gap using the butyl mastic as a backing material to which to apply the grout. Also apply non-shrink grout to the joints at the side edges of the baffle wall.

F. After setting the precast roof section of the stormwater treatment system; set precast concrete manhole riser sections, to the height required to bring the cast iron manhole covers to grade, so that the sections are vertical and in true alignment with a 1/4 inch maximum tolerance allowed. Backfill in a careful manner, bringing the fill up in 6" lifts on all sides. If leaks appear, clean the inside joints and caulk with lead wool to the satisfaction of the Engineer. Precast sections shall be set in a manner that will result in a watertight joint. In all instances, installation of Stormwater Treatment Systems shall conform to ASTM specification C891 "Standard Practice For Installation of Underground Precast Utility Structures".

G. Plug holes in the concrete sections made for handling or other purposes with a nonshrink grout or by using grout in combination with concrete plugs.

H. Where holes must be cut in the precast sections to accommodate pipes, do all cutting before setting the sections in place to prevent any subsequent jarring which may loosen the mortar joints. The Contractor shall make all pipe connections.

DESIGN AND OPERATION

Basic Operation

The Vortechs System is sized on the basis of removing both sediment and floating pollutants from stormwater runoff. When the system is operating at its peak design capacity, the maximum service rate will be approximately 100 gallons-per-minute per square foot of grit chamber area (gpm/sf). The Vortechs System has been tested for flows up to and including this maximum rate and has been shown to produce positive removal efficiencies throughout this range.

The Vortechs System will provide a net annual removal efficiency in excess of 80% removal of Total Suspended Solids as they are typically encountered in runoff from urban environments. The Vortechs System will also effectively capture and contain floatables in stormwater runoff.

The tangential inlet creates a swirling motion that directs settleable solids into a pile towards the center of the grit chamber. Sediment is caught in the swirling flow path and settles back onto the pile after the storm event is over. Floatables entrapment is achieved by sizing the low flow control to create a rise in the water level in the tank that is sufficient to just submerge the inlet pipe in the 2-month storm.

The Vortechs System is designed to create a backwater condition within the system in order to maximize removal efficiencies. The amount of backwater varies and is determined by the Vortechnics staff. To prevent flooding, the final design of the system incorporates all site conditions.

Design Process

During the Vortechs System design process consideration is given to both the physical constraints of the site and the site-specific flows. Each system is designed differently based on these characteristics, and the internal flow controls are specifically designed to accommodate the expected flows.

The site engineer provides the Vortechs System rim and invert elevations, pipe sizes, design flow rate, and design storm recurrence interval. Another consideration is whether the system is in an on-line or off-line (i.e. bypassed) configuration. If regulatory authorities allow treatment of storm flows less than the conveyance capacity of the piping system, it may be possible to provide a Vortechs System in an off-line configuration which will result in a cost savings without a significant reduction in pollutant removal efficiency.

Sizing the System

Each system is custom designed based on the design conditions provided. The weir, orifice, sump depth, and height of tank will vary depending on the site conditions and performance requirements. The rim and invert elevations will impact the overall height of the unit, the sump depth, and the placement of the weir and orifice. Also affecting the placement of the weir and orifice is the pipe size, the orientation of the internal walls, and the potential for tailwater. The flow rates determine the size of the weir, orifice, and the baffle opening.

Size: The size of the system depends on whether or not the system is on-line or off-line. An on-line system will be chosen such that the design flow rate is equal to or less then the Vortechs rated design flow. For an off-line system, the 2-month flow rate is determined and the model number is chosen based on the grit chamber area such that 24 gpm/sf of flow is realized through the chamber.

Sump: Typically a three-foot sump depth is provided in Vortechs Systems. This depth is most common since it provides ample sediment storage and keeps the excavation depth to a minimum. However, because each Vortechs System is custom designed, the individual sump depths may vary to balance maintenance costs with capital costs.

Orifice: The function of the orifice is to raise the water level in the Vortechs System. This increases the area of the flow in the pipe, which decreases the velocity of the water flowing into the system. A reduction in turbulence is realized at the inlet; this aids in keeping the trapped sediment and floatables contained. In addition, the rise in water level causes the floatables to rise above the inlet and away from the baffle opening, thus preventing the floatables from becoming re-entrained and pulled under the baffle wall. The orifice is designed to pass a flow approximately equal to that of a 2-month storm event.

Weir: Any event greater than the 2-month event causes the water level in the Vortechs System to rise to the upper flow control, submerging the inlet. The upper flow control is normally a Cippoletti weir. A Cippoletti weir is a trapezoidal weir with 4 to 1 sloping sides. Like the orifice, the weir also causes the water level in the system to rise, which promotes sediment and floatable removal. As the water rises, the volume of water in the system increases, thus stabilizing the detention time and allowing sediment to settle out. The swirl is maintained by allowing continuous flow through the system via the weir and orifice. The weir is sized to pass the design flow rate minus the orifice flow at full head.

Baffle: The baffle opening is designed to maintain a velocity such that re-entrainment of floatables and re-suspension of sediment is minimized. The baffle opening is at least 6 inches to ensure against clogging. The largest opening of 15 inches is chosen to maximize the distance between the floatable layer and the baffle opening. This keeps the floatables trapped and maintains the oil storage volume. In most applications, the flow under the baffle wall is approximately 1.0 foot per second.

Bypass: For systems in an off-line configuration, a weir crest length and elevation is calculated for the diversion structure that will be installed upstream of the specified Vortechs System. The goal is to achieve a water surface elevation during the 100-year storm that is at the same elevation as the top of the Vortechs Cippoletti weir. The area of flow over the bypass weir is calculated based on the 100-year flow. From this area, the height of flow is solved for a given weir length. Since the area of flow remains constant, the height of flow over the weir varies with the bypass weir length. See *Technical Bulletin 3A* for more information.

Flow Control Calculations

Vortechs Model 5000 System

The Vortechs System W.Q.S. 1 is a Model 5000 with a 7.0-foot diameter grit chamber. In this application, the runoff rate for a rainfall event with a return frequency of 10 years is 6.13 cubic feet per second (cfs). The system design flow is 2751 gpm (6.13 cfs). The surface area of the grit chamber is 38.5 square feet, therefore the peak operating rate is 2751 divided by 38.5 or 72 gpm/sf.

The low flow control is a trapezoidal orifice ($Q_{orifice}$). Since the inlet is a 24-inch diameter pipe, the orifice must raise the water level 24 inches, or 2.0 feet, in a 2-month storm to submerge the inlet pipe. According to Vortechnics Technical Bulletin #3, the 2-month storm flow rate is approximately equal to the 10-year flow rate divided by 7. The orifice calculation based on the full design flow is as follows:

$Q_{2\text{-}month} = Q_{10\,year} \div 7 = 6.13 \div 7 = 0.88$ cfs

$Q_{orifice} = C(A)(2gh)^{0.5} = 0.56(0.14)(2.0 \times 32.2 \times 2.0)^{0.5} = 0.89$ cfs 4

Where C = Orifice contraction coefficient = 0.56 (based on Vortechnics laboratory testing)
A = Orifice flow area, ft^2 (calculated by Vortechnics technical staff)
h = Design head, ft (equal to the inlet pipe diameter)

A Cippoletti weir configuration is utilized as the high flow control (Q_{weir}) which is conservatively designed for the system design flow (Q_{design}) of 6.13 cfs. The weir calculations are as follows:

$Q_{weir} = 6.13$ cfs

$Q_{weir} = C(L)(H)^{1.5} = 3.37(0.50)(2.42)^{1.5} = 6.34$ cfs 4

Where C = Cippoletti Weir coefficient = 3.37 (based on Vortechnics laboratory testing)
H = Available head, ft (height of weir)
L = Design weir crest length, ft (calculated by Vortechnics technical staff)

MAINTENANCE

The Vortechs System requires minimal routine maintenance. However, it is important that the system be inspected at regular intervals and cleaned when necessary to ensure optimum performance. The rate at which the system collects pollutants will depend more heavily on site activities than the size of the unit, e.g., heavy winter sanding will cause the grit chamber to fill more quickly but regular sweeping will slow accumulation.

Inspection

Inspection is the key to effective maintenance and it is easily performed. Vortechnics recommends ongoing quarterly inspections of the accumulated sediment. Note that is not unusual for sediment accumulation to be relatively light in the first year as initial sediment loads in new storm drainage systems may be diverted to catch basin sumps. Pollutant deposition and transport may vary from year to year and quarterly inspections will help insure that systems are cleaned out at the appropriate time. Inspections should be performed more often in the winter months in climates where sanding operations may lead to rapid accumulations, or in equipment washdown areas. It is very useful to keep a record of each inspection. A simple form for doing so is provided.

The Vortechs System only needs to be cleaned when inspection reveals that it is nearly full; specifically, when sediment depth has accumulated to within six inches of the dry-weather water level. This determination can be made by taking 2 measurements with a stadia rod or similar measuring device: one measurement is the distance from the manhole opening to the top of the sediment pile and the other is the distance from the manhole opening to the water surface. If the difference between the two measurements is less than six inches the system should be cleaned out. Note: to avoid underestimating the volume of sediment in the chamber, the measuring device must be lowered to the top of the sediment pile carefully. Finer, silty particles at the top of the pile typically offer less resistance to the end of the rod than larger particles toward the bottom of the pile.

In Vortechs installations where the risk of large petroleum spills is small, liquid contaminants may not accumulate as quickly as sediment. However, an oil or gasoline spill should be cleaned out immediately. Oil or gas that accumulates on a more routine basis should be removed when an appreciable layer has been captured.

Cleaning

Cleanout of the Vortechs System with a vacuum truck is generally the most effective and convenient method. Cleanout should not occur within 6 hours of a rain event to allow the entire collection system to drain down. Properly maintained Vortechs Systems will only require evacuation of the grit chamber portion of the system, in which case only the manhole cover nearest to the system inlet need be opened to remove water and contaminants. However, all chambers should be checked to ensure the integrity of the system. In installations where a "clamshell" is being utilized for solids removal, prior to removing the grit, absorbent pads or pillows can be placed in the oil chamber to remove floating contaminants. Once this is done, sediment may then be easily removed with the clamshell.

In some cases, it may be necessary to pump out all chambers. An important maintenance feature built into Vortechs Systems is that floatables remain trapped after a cleaning. A pock t of water between the grit chamber and the outlet panel keeps the bottom of the baffle submerged, so that all floatables remain trapped when the system begins to fill up agai . Therefore, in the event of cleaning other chambers it is imperative that the grit chamber t drained first. Manhole covers should be securely seated following cleaning activities, to ensure that surface runoff does not leak into the unit from above.

CHAPTER 8 - References

ATTACHMENTS

OVERSIZED EXHIBIT
"DEVELOPED CONDITIONS HYDROLOGY MAP"

This exhibit is on file at the City of Chula Vista, Planning Department located at 276 Fourth Avenue, Chula Vista, CA 91910

APPENDIX E-5

WATER AND RECYCLED WATER STUDY

VILLAGE 7 CONCEPTUAL

WATER AND RECYCLED WATER STUDY

March 10, 2004

PBS&J Project No.: 491067

Prepared For:

McMillin Land Development

Prepared By:

PBSJ

9275 Sky Park Court, Suite 200
San Diego, CA 92123

REGISTERED PROFESSIONAL ENGINEER
MARK B. ELLIOTT
No. 42064
Exp. 3/31/04
CIVIL
STATE OF CALIFORNIA

By: Mark B. Elliott, P.E.
Project Director

Acknowledgements

PROJECT TEAM

McMILLIN COMPANIES		619-477-4117
Frank Zaidle	Engineering Manager	
Todd Galarneau	Project Manager	
PBS&J		858-874-1810
Mark B. Elliott	Principal-In-Charge	
Dan Brogadir	Project Manager	
Jennifer Bileck	Project Engineer	

Table of Contents

Chapter 1
Introduction

1.1 Purpose

This report provides an overview of the existing and planned potable water and recycled water services for the proposed Otay Ranch Village 7 master-planned development (Project). This study summarizes existing and planned regional water facilities that will serve the Project, estimated water demands, and conceptual on-site potable and recycled water distribution systems.

This document is prepared to support the project's preliminary development plan and SPA application. A sub-area master plan (SAMP) will be prepared for review and approval by the Otay Water District (District) concurrently with the processing of the tentative map. The SAMP will provide more detailed information on the project such as project phasing, pump station and reservoir capacity requirements, and hydraulic computer modeling and analysis to determine flow requirements and finalize recommended pipe sizes.

1.2 Project Overview

The Project is located in the City of Chula Vista within the Otay Ranch General Planning Area. Birch Road to the north, the future extension of State Route 125 to the east, the future extension of La Media Road to the west, and future Rock Mountain Road to the south bound the site.

Planned land uses in Village 7 include single- and multi-family residential, commercial, community purpose, schools, and parks. Figure 1-1 shows the planned land uses based on a revised Project Site Utilization Plan (Cinti Land Planning, 03/08/2004).

1.3 Site Topography

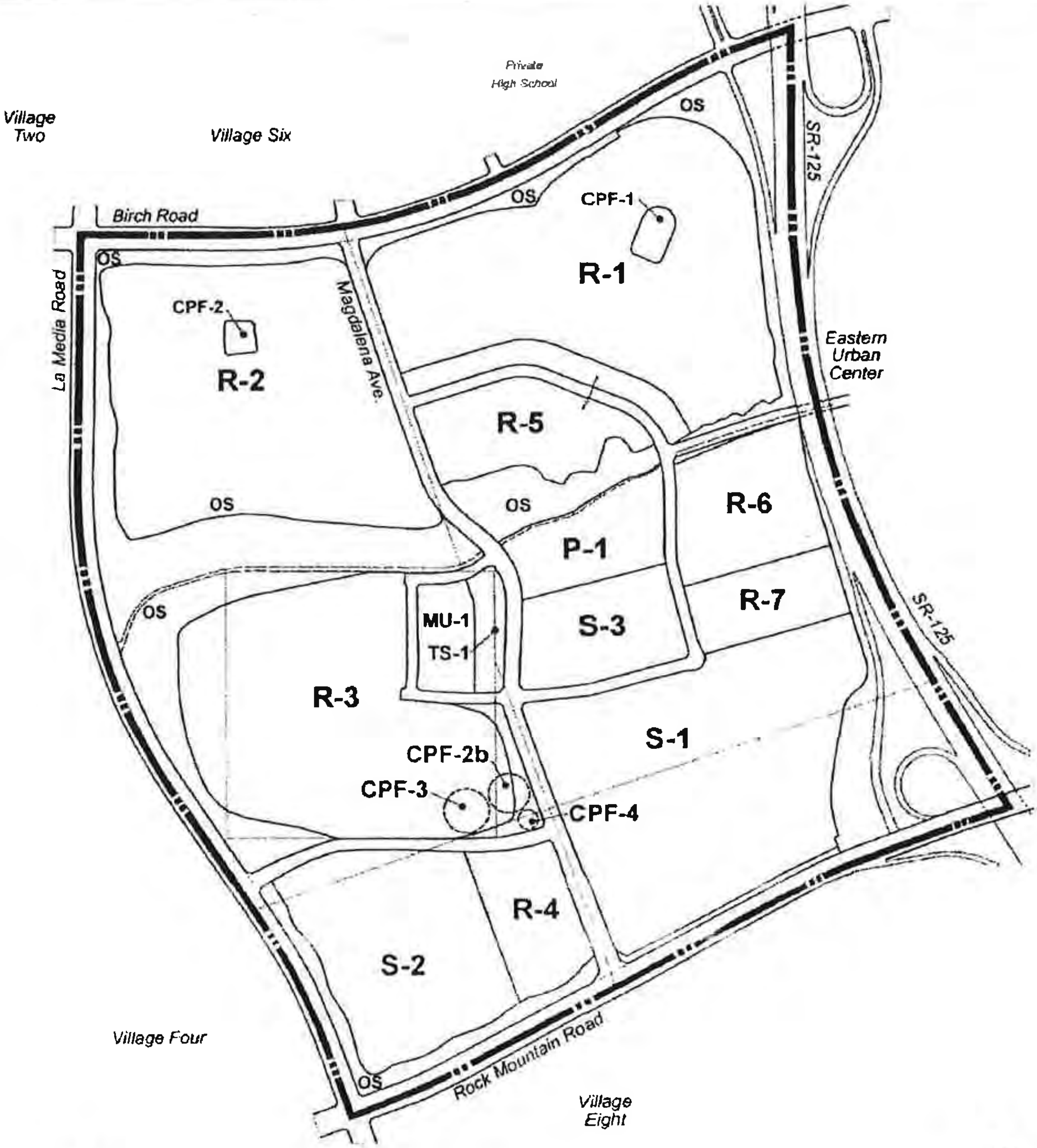
Proposed graded elevations will range from a low of approximately 410 feet in Wolf Canyon along the western boundary of the site to a high of approximately 570 to 580 feet in the northeast and southeast corners of the site. Natural drainage on the property is generally from east to west. Based on proposed graded elevations, the development is situated within the District's 711 and 980 Potable Water Pressure Zones and the 680 and 944 Recycled Water Pressure Zones.

1.4 Water Supply and Service

The District will provide potable and recycled water service to the Project. Prior to provision of service, the development will be required to annex into District Improvement Districts 22 and 27. The District has existing and planned facilities in the vicinity of the project allowing water service to be provided by expanding the existing system. This report will provide recommendations for improvements in each of the zones needed to provide water service to the Project.

1.5 Long Term Water Availability

The proposed Village 7 development is consistent with land use and water demand assumptions in the District's Water Resource Master Plan (July 2002). As required by the City the District will conduct a water audit to verify that the District has planned and sized water supply facilities to meet Village 7 needs. Furthermore, the District receives its treated water from the San Diego County Water Authority (SDCWA) which has developed water supply plans based on the District's future needs. The SDCWA has prepared a Water Facilities Master Plan (2003) which recommends projects to meet long term water supply needs. These projects include securing water supply from Colorado River and local water supply development, including sea water desalination and water reclamation. Accordingly these planned water supply projects will secure a long term water supply and satisfy SB 610.



LEGEND

MU	MIXED USE/COMMERCIAL
TS-1	TOWN SQUARE
R-1 - R-4	SF RESIDENTIAL
R-5 - R-7	MF RESIDENTIAL
CPF-1 - CPF-4	COMMUNITY PURPOSE FACILITY
S-1	HIGH SCHOOL
S-2	MIDDLE SCHOOL
S-3	ELEMENTARY SCHOOL
C-1	COMMERCIAL
OS	OPEN SPACE
P-1	PARK

PROPOSED LAND USE
FIGURE 1-1

I:\design\49-KearnyMesa\491067\491067-LU-sewer.dwg March 10, 2004

Chapter 2
Water Demands

2.1 Potable Water Demand Projections

The design criteria implemented to evaluate the potable water system for the Project are established in accordance with the Otay Water District Water Resources Master Plan (2002). The design criteria are utilized for analysis of the existing water system as well as for design and sizing of proposed improvements and expansions to the existing system to accommodate demands in the study area.

2.1.1 Pressure Zones

The Otay Water District has established criteria to determine pressure zone boundaries within new and existing developments. The criteria, as defined in the District Master Plan, establishes the minimum and maximum allowable pressures within the water distribution piping system under specified system operating parameters. Minimum pressure criteria are based on maximum day and fire flow requirements while maximum pressure limitations are imposed to protect internal residential and commercial building water piping from failure under static and transient operating conditions. Maintaining water pressures within the limitations summarized in Table 2-1 will also protect the water distribution system piping, valves, pumps, and other appurtenances from premature failure or increased maintenance requirements.

Generally, the potable water distribution system is designed to maintain static pressures between 65 pounds per square inch (psi) and 200 psi. The potable water distribution system has been designed to yield a minimum of 40 psi residual pressure at any location under peak hour demand flows, and a minimum residual pressure of 20 psi during maximum day demand plus fire flow conditions. Potable water mains are sized to maintain a maximum velocity of 10 feet per second (fps) under a maximum day plus fire flow scenario and a maximum velocity of 6 fps under peak hour flow conditions.

Table 2-1
Distribution System Pressure Limitations

Operating Condition	Criteria	Pressure
Static	Minimum Pressure	65 psi
Static	Maximum Pressure	200 psi
Peak Hour	Minimum Pressure	40 psi
Max Day plus Fire Flow	Minimum Pressure	20 psi

2.1.2 Water Demand

Based on unit demand factors specified in the District Master Plan, average potable water demands for the Project were estimated and are presented in Table 2-2.

Table 2-2
Village 7 Potable Water Demands

Land Use	Net Area (Ac)	Dwelling Units	Unit Demand [1]	Average Daily Demand (gpd)
SF Residential	152.3	1,053	500 gpd/du	526,500
MF Residential	37.9	448	300 gpd/du	134,400
Elementary School	11.1		1,785 gpd/ac	19,814
Middle School	26.3		1,785 gpd/ac	46,946
High School	55.8		1,785 gpd/ac	99,603
Commercial	3.7		1,785 gpd/ac	6,605
Town Square	1.9		2,155 gpd/ac	4,095
Park	7.6		2,155 gpd/ac	16,378
CPF	4.1		893 gpd/ac	3,661
TOTAL	300.7	1,501		858,000

(1) Unit demand factors from draft *Otay Water District Water Resources Master Plan* (July 2002)

2.2 Recycled Water Demand Projections

Landscape systems generally require a minimum of 65 psi at the meter to obtain adequate coverage of the irrigated area. It is expected that this minimum pressure can be achieved at all locations within the Project. The primary criteria for sizing recycled water lines is the ability to meet peak hour recycled water demands while maintaining the maximum pipeline velocity between 5 to 8 fps.

Table 2-3 provides the projected recycled water demand for the Project.

Table 2-3
Village 7 Recycled Water Demand

Land Use	Net Area (Ac)	Percent Irrigated	Irrigated Area (Ac)	Unit Demand [1]	Average Daily Demand (gpd)
MF Residential	37.9	15%	5.7	2,152 gpd/du	12,234
Elementary School	11.1	20%	2.2	2,152 gpd/ac	4,777
Middle School	26.3	20%	5.3	2,152 gpd/ac	11,320
High School	55.8	20%	11.2	2,152 gpd/ac	24,016
Commercial	3.7	10%	0.4	2,152 gpd/ac	796
Town Square	1.9	25%	0.5	2,152 gpd/ac	1,022
CPF	4.1	100%	4.1	2,152 gpd/ac	8,823
Park	7.6	100%	7.6	2,152 gpd/ac	16,355
TOTAL			37		79,344

(1) Unit demand factors from draft *Otay Water District Water Resources Master Plan* (July 2002)

Chapter 3
Regional Water Facilities

3.1 Supply

The Project will receive supply from the District's Central Service Area. Potable water is supplied to the Central Service Area by the San Diego County Water Authority via the Second San Diego Aqueduct. Water is delivered at Aqueduct connections No. 10 and No. 12 and is conveyed by gravity to District reservoirs with a high water level of 624 feet. Water is then pumped from the 624 Zone to the 711 and 980 Zones via the Central Area Pump Station (711-1 Pump station) and the EastLake Pump Station (980-1 Pump Station), respectively.

Recycled water supply is currently available from the Ralph W. Chapman Water Recycling Facility located near the intersection of Singer Lane and Highway 94. The plant has a practical capacity of 1.0 million gallons per day (MGD). The recycled water is delivered from the plant to storage ponds with a high water level of 944 feet situated within the District Use Area located north of Proctor Valley Road. Potable water is currently used to supplement the recycled water supply during summer and dry weather month conditions.

Recycled water supply is also anticipated to be available from the City of San Diego's 15.0 MGD South Bay Water Reclamation Plant. This supply will be delivered to the District at a grade of 450 feet and then pumped to the 680 and 944 Zones via the planned 450-1 and 680-1 pump stations.

3.2 Transmission Facilities

Potable and recycled water service to the Project will be provided through extension of existing transmission mains in La Media and Birch Roads. Existing potable water mains include a 16-inch 711 Zone main in Olympic Parkway that will be extended in La Media Road and Hunte Parkway/Birch Road to the Project boundary, and a 16-inch 980 Zone main in Olympic Parkway that will be extended through Village 6 and Hunte Parkway/Birch Road toward SR125.

Existing recycled mains that will be extended to the project include a 20-inch 680 Zone main and 12-inch 944 Zone main in Olympic Parkway will be extended in La Media and Hunte Parkway/Birch Road.

3.3 Storage Facilities

Operational and emergency potable water storage for the Project will be provided in the existing 711 reservoirs located in the EastLake Greens subdivision (711-1 and 711-2 Reservoirs) and the District Use Area (711-3 Reservoir) and 980 Zone Reservoirs located in the District Use Area (980-1 and 980-2 Reservoirs).

Chapter 4
Proposed Water System

4.1 Recommended Water Projects

The Project can receive water service by expanding the existing potable and recycled water systems. These extensions will be constructed either as part of adjacent subdivision development or concurrent with development of the project. Per District policy, construction of regional water mains in La Media Road, Hunte Parkway, and Birch Road will be funded by the District as Capital Improvement Program projects. Based on District planning criteria, redundant sources of potable water supply to the Project will be required prior to occupancy of the development.

The project is proposed to be phased from North to South. The potable water system has been developed as shown to provide a redundant supply for each new unit of development.

4.2 Recommended On-Site Improvements

Figure 4-1 illustrates the conceptual on-site potable and recycled water distribution systems for the Project. Sizes of the on-site facilities will be determined based on final site layout and design criteria specified in the District Master Plan. As previously discussed, a Sub-Area Master Plan will be prepared concurrent with development of the Project tentative map that will address the sizing and phasing of onsite and off-site water facilities for the project based on hydraulic analysis of the proposed water system.

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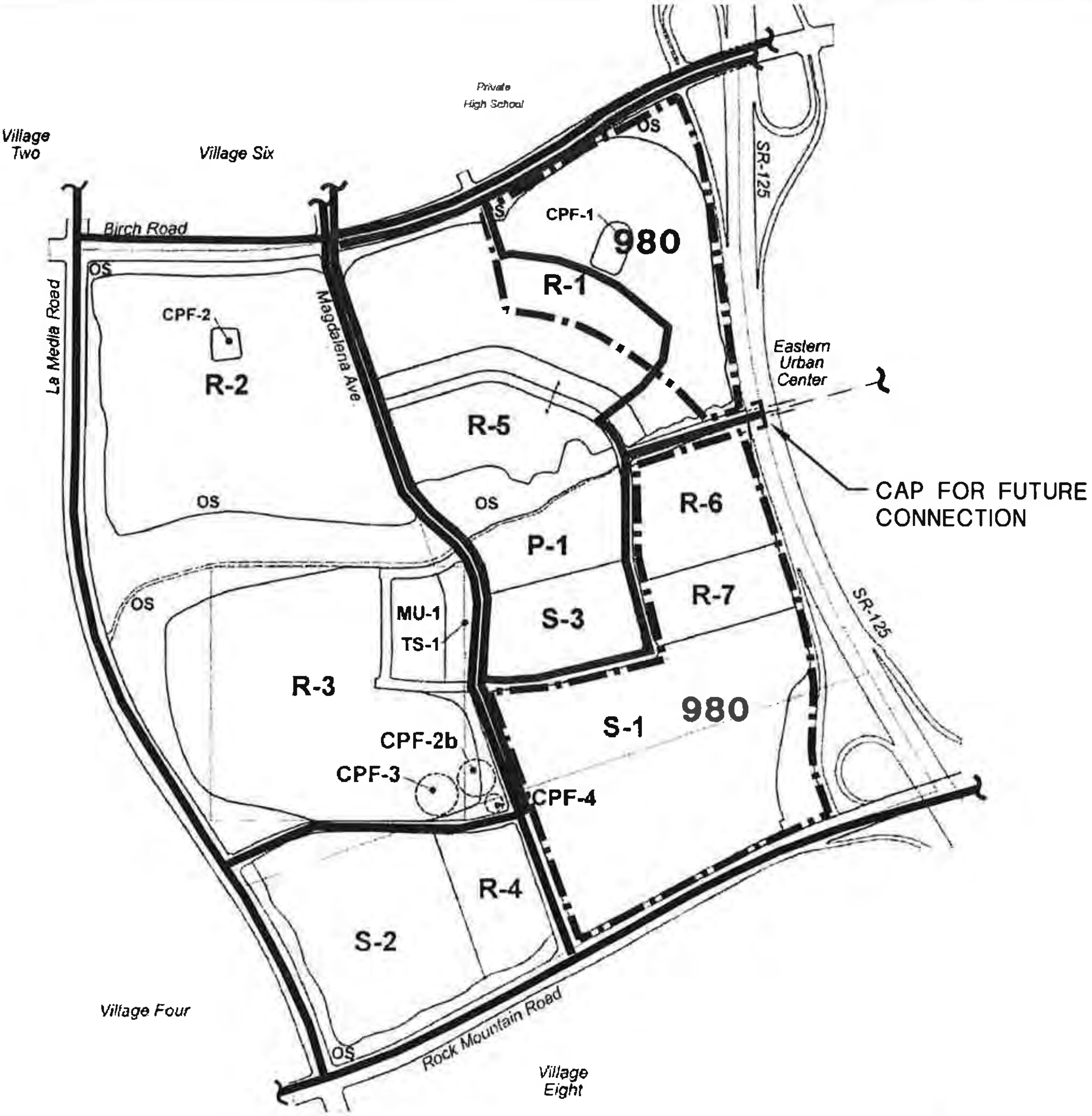

LEGEND

- 711 ZONE WATER MAIN
- 980 ZONE WATER MAIN
- 980 ZONE BOUNDARY

NO SCALE

NOTE:
PROJECT IS WITHIN 711 ZONE SERVICE AREA UNLESS OTHERWISE NOTED.

POTABLE WATER SYSTEM
FIGURE 4-1

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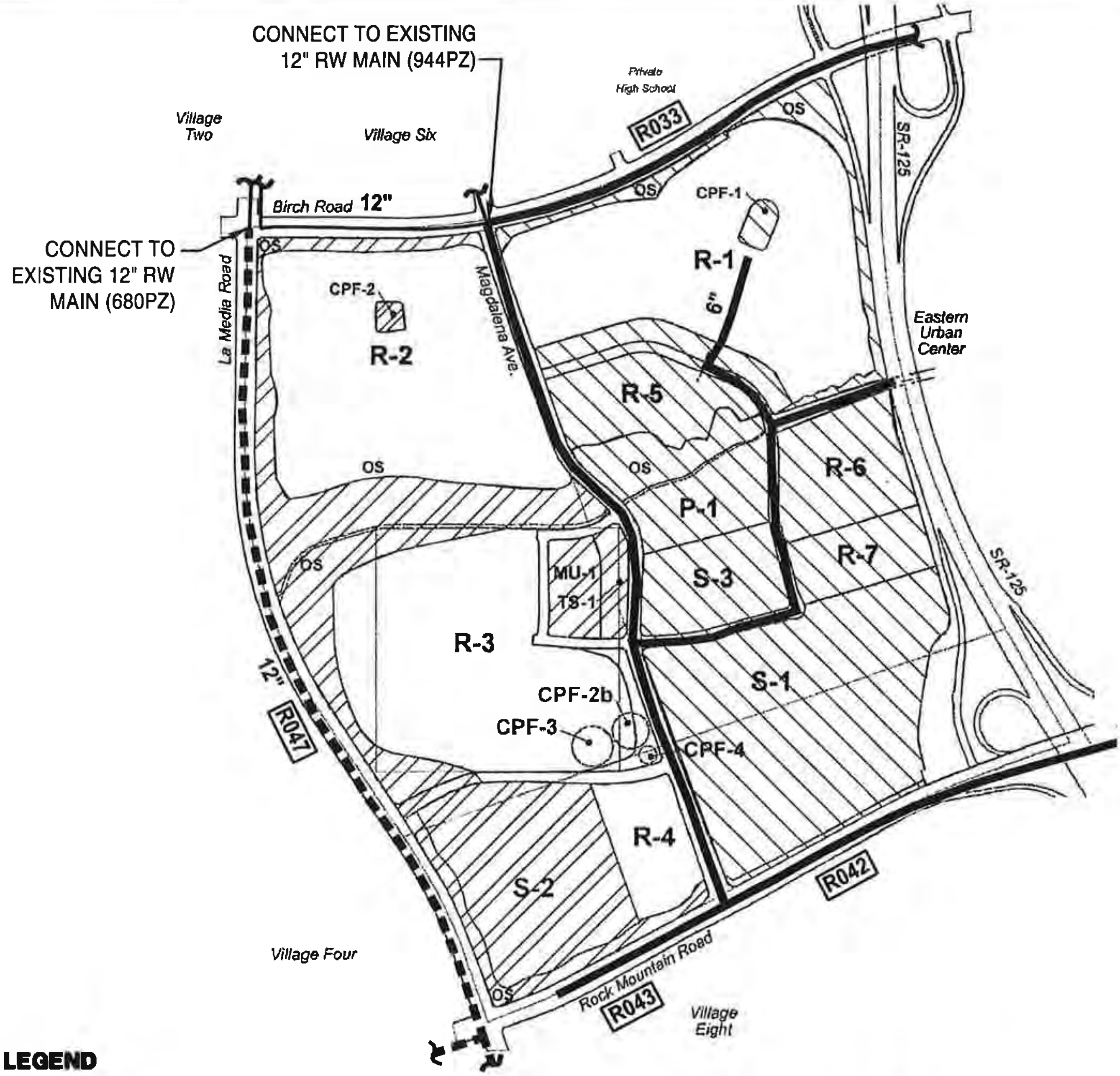

LEGEND

(hatch)	680 ZONE SERVICE AREA
(hatch)	944 ZONE SERVICE AREA
(dashed line)	680 ZONE MAIN
(heavy line)	944 ZONE MAIN
(thin line)	EXISTING RW MAIN
R047	CIP No.

NOTE: ALL PIPELINES 8" UNLESS OTHERWISE NOTED.

N

NO SCALE

RECYCLED WATER SYSTEM
FIGURE 4-2

APPENDIX E-6

RECYCLED WATER LETTER
AND
WATER SUPPLY AND ASSESSMENT REPORT

...Dedicated to Community Service

2554 SWEETWATER SPRINGS BOULEVARD, SPRING VALLEY, CALIFORNIA 91978-2096
TELEPHONE: 670-2222, AREA CODE 619

May 13, 2004 W.O. 9605

Ms. Marni Borg
Environmental Projects Manager
Planning Department
City of Chula Vista
276 Fourth Avenue
Chula Vista, California 91910

SUBJECT: Water Supply Assessment and Verification Report for Otay Ranch Village Seven

Dear Ms. Borg:

This letter is in response to a request by McMillin Otay Ranch, LLC (McMillin), that Otay Water District (WD) provide the City of Chula Vista with correspondence to clarify the recycled water demand projections within the Water Supply Assessment and Verification (WSA&V) Report for Otay Ranch Village Seven. The Otay WD's Board of Directors approved the subject report on March 3, 2004.

The Otay WD has reviewed the updated recycled water demand information for Village Seven provided by McMillin. The revised total projected recycled water demand for Village Seven is lower than that identified for Village Seven within the Otay WD's Water Resources Master Plan. Because the Water Resources Master Plan formed the primary basis for the WSA&V Report for Village Seven, the updated demand figures do not alter the water supply analysis or the conclusions regarding recycled water supply or availability. Thus, the overall findings of the WSA&V Report for Village Seven remain valid.

If you have any questions, please feel free to contact me at (619) 670-2242.

Sincerely,

James Peasley

James Peasley, P.E.
Engineering Planning Manager

JFP:lrp

cc: Todd Galarneau, McMillin Otay Ranch

P:\WORKING\Developer\WO 9605\CCV WSAV Report Village 7 Letter, 5-13-04.doc

OTAY WATER DISTRICT

WATER SUPPLY ASSESSMENT AND VERIFICATION REPORT

Otay Ranch Village Seven
Sectional Planning Area Plan

Prepared by:

Otay Water District
Psomas
in consultation with
San Diego County Water Authority

January 2004

Otay Water District
Water Supply Assessment and Verification Report
January 2004

Otay Ranch Village Seven
Sectional Planning Area Plan

Table of Contents

Otay Water District
Water Supply Assessment and Verification Report
January 2004

Otay Ranch Village Seven
Sectional Planning Area Plan

Section 1 - Purpose

This Water Supply Assessment and Verification Report (WSA&V Report) has been prepared by the Otay Water District (Otay WD) in consultation with the San Diego County Water (Water Authority) and the City of Chula Vista pursuant to Public Resources Code Section 21151.9 and California Water Code Sections 10631, 10657, 10910, 10911, 10912, and 10915 referred to as SB 610 and Business and Professions Code Section 11010 and Government Code Sections 65867.5, 66455.3, and 66473.7 referred to as SB 221. SB 610 and SB 221 amended state law, effective January 1, 2002, to improve the link between information on water supply availability and certain land use decisions made by cities and counties. SB 610 requires that the water purveyor of the public water system prepare a water supply assessment to be included in the environmental documentation of certain proposed projects. SB 221 requires affirmative written verification from the water purveyor of the public water system that sufficient water supplies are available for certain residential subdivisions of property prior to approval of a tentative map.

The City of Chula Vista requested the WSA&V Report as part of the environmental review of the Otay Ranch Village Seven Sectional Planning Area (SPA) Plan (Project). The Project description is provided in Section 3 of this WSA&V Report. The City of Chula Vista also requested, since the SB 610 and SB 221 requirements are substantially similar, that Otay WD prepare both the Water Supply Assessment and Water Verification concurrently. This WSA&V Report is intended for use by the City of Chula Vista in its evaluation of the Project under the California Environmental Quality Act process. This WSA&V Report evaluates water supplies that are or will be available during normal, single-dry year, and multiple dry water years during a 20-year projection to meet existing demands, expected demands of the Project, and reasonably foreseeable planned future water demands served by Otay WD.

This WSA&V Report documents the following: 1) an identification of existing water supply entitlements, water rights, water service contracts, or agreements relevant to the identified water supply for the proposed Project, 2) water received in prior years pursuant to those entitlements, rights, contracts, and agreements, and 3) a description of the quantities of water received in prior years by the Otay WD.

Section 2 - Findings

The WSA&V Report identifies that the water demand projections for the proposed Project are included in the water demand forecasts within the Urban Water Management Plans and other water resources planning documents of the Otay WD, the Water Authority, and the Metropolitan Water District of Southern California (Metropolitan). Water supplies necessary to serve the demands of the proposed Project, along with existing and other projected future users, as well as the actions necessary to develop these supplies, have been identified in the water supply planning documents of the Otay WD, the Water Authority, and Metropolitan. This WSA&V Report demonstrates and verifies that there are sufficient water supplies over a 20-year planning horizon to meet the projected demand of the proposed Project and the existing and other planned development projects within the Otay WD.

This WSA&V Report includes, among other information, an identification of existing water supply entitlements, water rights, water service contracts, or agreements relevant to the identified water supply for the proposed Project.

As requested by the City of Chula Vista, the Otay WD has prepared this WSA&V to verify that sufficient water supplies meets projected water demands of the Project and the Otay WD for a 20-year planning horizon, and in single- and multiple-dry years.

Based on a normal water supply year, the five-year increments for a 20-year projection indicate projected water supply will meet the estimated water demand (36,658 acre-feet (ac-ft) in 2005 to 64,690 ac-ft in 2025). Based on dry year forecasts, the estimated water supply will also meet the projected water demand, during single- and multiple-dry years scenarios. For a single dry year, a supply of 68,799 ac-ft within the Otay WD service area is necessary. For multiple-dry years, a supply of 39,001 ac-ft, 40,719 ac-ft, and 42,505 ac-ft, respectively, is necessary to meet demand.

Together, these findings verify that there is a sufficient water supply to serve the proposed Project and the existing and other planned projects of Otay WD in both normal and dry year forecasts. An adequate supply is further confirmed by the March 2003, Metropolitan produced document entitled, "Report on Metropolitan's Water Supplies, A Blueprint for Water Reliability" (March 2003 Report), which states that Metropolitan will have adequate supplies to meet dry-year demands within its service area over the next 20 years.

Section 3 - Project Description

The McMillin Otay Ranch LLC has submitted an application to the City of Chula Vista for development of Otay Ranch Village Seven. The Project encompasses approximately 420.8 acres and contains various land uses as proposed by the McMillin Otay Ranch LLC. The City of Chula Vista has publicly announced its intent to initiate the preparation of an

Environmental Impact Report (EIR) for the Project in conformance with the California Environmental Quality Act and as set forth in Public Resources Code 21065. The Project is located in the City of Chula Vista within San Diego County. The Project is included within the City of Chula Vista Otay Ranch General Development Plan (GDP), Otay Subregional Plan, Volume 2 document. The Project land areas are located within what is defined as the Otay Valley Parcel of the Otay Ranch GDP.

The approximately 23,000 acre Otay Ranch is a master-planned community that includes a broad range of residential, commercial, retail, and industrial development interwoven with civic and community uses, such as libraries, parks, and schools, together with an open space preserve system consisting of approximately 11,375 acres. The Otay Ranch GDP was adopted by the Chula Vista City Council and the San Diego County Board of Supervisors on October 28, 1993, which was accompanied by a Program Environmental Impact Report EIR-90-01 (SCH #89010154). Village Seven is a part of the designated 14 villages and five planning areas within the Otay Ranch GDP area.

Under the implementation program for Otay Ranch, SPA plans are required to be approved by the City of Chula Vista before final development entitlements can be considered. The current proposed Project SPA Plan is intended to further refine the development standards, land plans, goals, objectives, and policies of the Otay Ranch GDP.

The proposed Project area is located within Otay Ranch, in the eastern portion of the City of Chula Vista. Village Seven is generally bounded to the west by the future La Media Road, to the north by Birch Road, to the east by the future State Route 125, and to the south by the future Rock Mountain Road.

The McMillin Otay Ranch LLC proposed development concept for the Project is generally planned as a mixed density residential community. The various land uses include residential development, three schools, a park, commercial, community purpose facilities, a transit stop, a town square, open space, and circulation elements. The planned residential products total approximately 1,501 dwelling units and incorporate 1,053 single-family units and 448 multi-family units. The Project plan also includes 3.2 acres of commercial land uses, 6.3 acres of community purposes facilities, a 11.1 acre elementary school site, a 25.2 acre middle school, a 52.0 acre high school site, 7.9 acres for a park, 62.4 acres of open space, and 69.0 acres of land use for circulation elements. The entire Project is comprised of several phases.

The City of Chula Vista has identified that the following discretionary actions are contemplated as part of the proposed Project:

A. Adoption of Sectional Planning Area Plan

The SPA Plan would implement the land use plans, goals, objectives, and polices of the City of Chula Vista General Plan and the Otay Ranch GDP for Village Seven. Development parameters, urban design criteria, circulation pattern, open space and recreational concept, and infrastructure requirements would also be addressed by the SPA Plan.

B. Approval of Tentative Tract Map(s)

The Tract Map(s) establish the layout of land uses, developable and open space lots, and infrastructure requirements for Village Seven. The Tract Map(s) corresponding to two land ownerships within Village Seven would be addressed in the EIR for the Project.

The projected potable and recycled water demands associated with the Project have considered all of the above discretionary actions and are incorporated into and used in this WSA&V Report. The water demands for the proposed Project are included in the projected water demand estimates provided in Section 5 – Historical and Projected Water Demands.

Section 4 – Otay Water District

The Otay WD is a municipal water district formed in 1956 pursuant to the Municipal Water District Act of 1911 (Water Code §§ 71000 et seq.). In addition to water service, the Otay WD also provides sewer service to a limited portion of its jurisdiction. The Otay WD joined the Water Authority as a member agency in 1956 to acquire the right to purchase and distribute imported water throughout its service area. The Water Authority is an agency responsible for the wholesale supply of water to its 23 public agencies members in San Diego County.

The Water Authority currently obtains all of its imported supply from Metropolitan, but is in the process of diversifying its available supplies. On October 10, 2003, the Water Authority along with the Imperial Irrigation District (IID), Coachella Valley Water District (CVWD), Metropolitan, and various other parties, executed a series of agreements commonly known as the Quantification Settlement Agreements (QSA). Those agreements, when implemented, will provide the Water Authority with up to 277.7 thousand acre-feet per year (ac-ft/yr) of water conserved by the IID and by the lining of the All American Canal (AAC) and the Coachella Canal (CC). See Sections 6.1.1 and 6.2.1 of this WSA&V Report for additional information.

The Otay WD service area is generally located within the south central portion of San Diego County and includes approximately 125 square miles. The Otay WD serves portions of the unincorporated communities of southern El Cajon, La Mesa, Rancho San Diego, Jamul, Spring Valley, Bonita, and Otay Mesa, the eastern portion of the City of Chula Vista and a portion of the City of San Diego on Otay Mesa. The proposed Project is located within the Otay WD service area.

Data obtained from the Department of Water Resources' California Irrigation Management Information System (Weather Station No. 147) indicates that the average annual reference evapotranspiration over the past five years is 52 inches with an annual precipitation rate of 10.2 inches.

Population growth within the Otay WD service area is expected to increase from the current figure of approximately 143,000 to an estimated 243,000 by 2020, and is estimated to be 277,000 at ultimate build out of the service area. Data on projected population and growth rate projections within the Otay WD was obtained from the San Diego Association of Governments (SANDAG) regional growth forecasts. SANDAG serves as the regional, intergovernmental planning agency that provides forecasted population and housing figures. Land use information used to develop water demand projections are based upon Specific or Sectional Planning Areas, the Otay Ranch GDP, San Diego County Community Plans, and City of San Diego, City of Chula Vista, and County of San Diego General Plans. The City of Chula Vista, the City of San Diego, and the County of San Diego are the three land use planning agencies within the Otay WD jurisdiction.

4.1 Urban Water Management Plan

In accordance with the California Urban Water Management Planning Act, the Otay WD Board of Directors adopted an Urban Water Management Plan (UWMP) in December 2000 and it was subsequently submitted to the California Department of Water Resources (DWR). As required by law, Otay WD's UWMP includes projected water supplies required to meet future demands through 2020. In accordance with Water Code Section 10910 (c)(2) and Government Code Section 66473.7 (c)(3), information from Otay WD's UWMP along with updated supplemental information from Otay WD's current Water Resources Master Plan have been utilized to prepare this WSA&V Report.

Section 5 – Historical and Projected Water Demands

The projected demands for Otay WD service area are based on Specific or Sectional Planning Areas, the Otay Ranch GDP, San Diego County Community Plans, and City of San Diego, City of Chula Vista, and County of San Diego General Plans, which are incorporated in SANDAG's most recent growth forecast data, which includes figures on future population, housing, and employment. This land use information is utilized in the preparation of the Otay WD's Water Resources Master Plan and Urban Water Management Plan to develop the forecasted demands. The Water Authority and Metropolitan also use SANDAG's most recent regional growth forecast to calculate future demands within their respective service areas. This provides for consistency between the retail and wholesale agencies water demand projections, thereby ensuring that adequate supplies are being planned for Otay WD's existing and future water users. In addition, SANDAG's growth forecasts are based on the land use policies of the cities and county within the San Diego County region, so planned growth is included in the water demand forecasts of Otay WD.

The historical and projected potable water demands for Otay WD service area are shown in Table 1.

Table 1
Historical and Projected Potable Water Demands (acre-feet)
Incorporating Water Conservation BMP Efforts[1]

Water Use Sectors	1990	1995	2000	2005	2010	2015	2020	2025[2]	Ultimate[3]
Single Family Residential	*	10,604	15,331	17,773	20,604	23,886	27,690	31,000	33,300
Multi-Family Residential	*	1,880	1,986	2,302	2,669	3,094	3,578	4,000	4,300
Commercial & Industrial	*	1,650	3,043	3,528	4,090	4,741	5,496	6,200	6,660
Institutional & Governmental	*	1,680	2,088	2,421	2,807	3,254	3,772	4,100	4,400
Landscape	*	3,983	6,259	7,256	8,412	9,752	11,305	13,200	14,200
Agricultural	*	487	171	198	230	267	310	200	140
Total	20,077	20,284	28,878	33,478	38,812	44,994	52,151	58,700	63,000

[1] Otay WD's 2000 UWMP and Water Resources Master Plan (WRMP).
[2] The year 2025 demand projection based on theWRMPdata.
[3] Ultimate demand data based on land use coverage applied to entire Otay WD service area.
* Detail by sector is unavailable for 1990.

The historical and projected recycled water demands for Otay WD service area are shown in Table 2.

Table 2
Historical and Projected Recycled Water Demands (acre-feet)
Incorporating Water Conservation BMP Efforts[1]

Water Use Sectors	1990	1995	2000	2005	2010	2015	2020	2025[2]	Ultimate[3]
Single Family Residential	0	*	*	0	0	0	0	0	0
Multi-Family Residential	0	*	*	198	242	286	330	374	443
Commercial & Industrial	0	*	*	131	160	189	218	248	294
Institutional & Governmental	0	*	*	250	306	361	417	472	559
Landscape	0	*	*	2,601	3,172	3,744	4,325	4,896	5,804
Agricultural	0	0	0	0	0	0	0	0	0
Total	0	915	1,274	3,180	3,880	4,580	5,290	5,990	7,100

[1]Otay WD's 2000 UWMP and WRMP.
[2]The year 2025 demand projection based on the WRMP data.
[3]Ultimate demand data based on land use coverage applied to the Otay WD recycled water service area, exclusive of the Otay Mesa service area.
*Detail by sector is unavailable for 1995 and 2000.

The Otay WD water demand projection methodology utilized a component approach. This was done by applying representative values of water usage to each land use type and then summed, culminating in a total ultimate water demand for the entire Otay WD service area. This is called the water duty method, and the water duty is the amount of water used in acre-feet per acre per year. This approach was used for all the land use types except residential development where a demand per dwelling unit was applied. In addition, water users such as golf courses, schools, jails, prisons, and hospitals were identified and specific water demands allocated.

To determine water duties for the various types of land use, the entire water meter database of the Otay WD was utilized and sorted by the appropriate land use types. The metered consumption records were then examined for each of the land uses, and water duties were determined for the various types of commercial, industrial, and institutional land uses. For example the water duty factors for commercial and industrial land uses were estimated using 1,785 and 893 gallons per day (gpd) per acre respectively. Residential water demand was established based on the same data but computed on a per-dwelling unit basis. The focus was to ensure that for each of the residential land use categories (very low, low, medium, and high densities), the demand criteria used were adequately represented based upon actual data. This method was used because residential land uses constitute a substantial percentage of the total planning area of the Otay WD.

By applying the established water duties to the proposed land uses, the projected water demand for the Otay WD planning area at ultimate development was determined. Projected water demands for the intervening years were determined using growth rate projections consistent with data obtained from SANDAG and the experience of the Otay WD.

Using the land use demand projection criteria as established in the Otay WD Water Resources Master Plan, the projected potable water demand for the proposed Project is shown in Table 3, which totals 0.84 million gallons per day (mgd) or 936 ac-ft/yr. The projected recycled water demand for the proposed Project is provided in Table 4, which totals 0.07 mgd or 77 ac-ft/yr, representing about 8% of total Project demand. These demand projections are consistent with the projected water demand included in the Otay WD Urban Water Management Plan and Water Resources Master Plan.

Table 3
Village Seven Projected Potable Water Annual Average Demands[1]

Location	Land Use Description	Dwelling Units	Area	Demand (gpd)
Village Seven	Single-Family Residential	1,053 units	151.4 acres	526,500
Village Seven	Multi-Family Residential	448 units	32.3 acres	134,400
Village Seven	Commercial		3.2 acres	5,712
Village Seven	Community Purpose Facilities		6.3 acres	11,246
Village Seven	Elementary School		11.1 acres	19,814
Village Seven	Middle School		25.2 acres	44,892
Village Seven	High School		52.0 acres	92,820
Village Seven	Park		7.9 acres	0
Village Seven	Open Space		62.4 acres	0
Village Seven	Circulation		69.0 acres	0
Totals		**1,501 units**	**420.8 acres**	**835,384**

[1]Land use information based upon Village Seven draft site utilization plan prepared by Cinti Land Planning.

Table 4
Village Seven Projected Recycled Water Annual Average Demands[1]

Location	Land Use Description	Irrigated Area	Demand (gpd)
Village Seven	Multi-Family Residential	32.3 acres	10,441
Village Seven	Commercial	3.2 acres	690
Village Seven	Community Purpose Facilities	6.3 acres	2,715
Village Seven	Elementary School	11.1 acres	4,784
Village Seven	Middle School	25.2 acres	10,861
Village Seven	High School	52.0 acres	22,412
Village Seven	Park	7.9 acres	17,025
Totals		**130.8 acres**	**68,928**

[1]Land use information based upon Village Seven draft site utilization plan prepared by Cinti Land Planning.

5.1 Demand Management (Water Conservation)

Demand management, or water conservation, is frequently the lowest-cost resource available to any water agency. Water conservation is addressed in the Otay WD Urban Water Management Plan, as an element of the long-term strategy for meeting the water needs. The goals of the Otay WD water conservation program are to do the following: 1) reduce the demand for imported water, 2) demonstrate continued commitment to the Best Management Practices (BMP), and 3) ensure a reliable water supply.

The Otay WD is signatory to the Memorandum of Understanding (MOU) Regarding Urban Water Conservation in California, which created the California Urban Water Conservation Council (CUWCC) in 1991 in an effort to reduce California's long-term water demands. Water conservation programs are developed and implemented on the premise that water conservation increases the water supply by reducing the demand on available supply, which is vital to the optimal utilization of a region's water supply resources. The Otay WD participates in many water conservation programs designed and typically operated on a shared-cost participation program basis among the Water Authority, Metropolitan, and their member agencies. The demands shown in Tables 1, 2, 3, and 4 take into account implementation of water conservation measures within Otay WD service area.

As a requirement for development projects within the City of Chula Vista and Otay WD, water conservation measures will be incorporated into the Project including the State-mandated 14 Best Management Practices for water conservation such as installation of ultra low flow toilets (ULFT), development of a water conversation plan for all landscape improvements, and the use of recycled water, all of which are typical requirements of development projects.

As one of the first signatories to the MOU Regarding Urban Water Conservation in California, the Otay WD has made BMP implementation for water conservation the cornerstone of its conservation programs and a key element in its water resource management strategy. As a member of the Water Authority, Otay WD also benefits from regional programs performed on behalf of its member agencies.

Current Otay WD conservation programs are saving approximately 950 ac-ft/yr of water within its service area. The vast majority of water savings, approximately 94%, currently are obtained through conservation efforts from the residential ULFT and large landscape programs. The Otay WD has planned to gradually shift emphasis towards residential landscaping and clothes washers as these programs continue to evolve. This is because opportunities for ULFTs will decline and large landscape water efficiency is increasingly emphasized and practiced. The resulting savings directly relate to additional available water in the San Diego region for beneficial use within the Water Authority service area, including the Otay WD.

In partnership with the Water Authority, the City of Chula Vista, and developers, the Otay WD's water conservation efforts are expected to grow and expand. Based upon an analysis of water savings as a percentage of overall demand during the last six years, the Otay WD expects to reduce water demand within a range of 1,400 to 2,200 acre-feet, which represents about three to five percent of the Otay WD's expected 2020 water demand.

The BMP programs implemented by Otay WD and regional BMP programs implemented by the Water Authority that benefit all member agencies, include the following:

- **BMP 1 - Water Survey Programs for Single-Family and Multi-Family Residential Customers**

  The Residential Survey Program is free to residents, both single and multi-family and has been available since 1995. The survey includes an indoor water use review, assistance with identifying indoor leaks, and a complete educational packet, which includes information about other water conservation programs. The survey also includes a meter leak detection test, irrigation system maintenance check, individualized seasonal suggestions of watering schedules, and soil check, information about proper lawn maintenance measures, and tips about low-water use landscaping where appropriate.

- **BMP 2 - Residential Plumbing Retrofit**

  The Otay WD continues to distribute showerheads at outreach events as well as at the main office upon request. Since the Otay WD service area is relatively new and most of the dwellings were built after 1992 and have newer plumbing fixtures, participation in the residential plumbing retrofit BMP is approaching saturation.

- **BMP 3 - System Water Audits, Leak Detection, and Repair**

Each local agency, including the Otay WD, maintains an active distribution system-auditing program. The industry standard, based on the American Water Works Association, for unaccounted-for water loss is no more than nine to ten percent. The Otay WD typically experiences about a six percent water loss, which is well below the industry standard threshold. The Otay WD regularly conducts ongoing internal distribution system leak detection surveys. The comparison of water sold to water purchased also helps in the detection of water loss. The Otay WD has incorporated this BMP into its operations and maintenance procedures, and established a six-year rotation schedule. Otay WD crews survey at least three to four miles of water main and laterals per year on an ongoing basis.

- **BMP 4 - Metering with Commodity Rates for All New Connections and Retrofit of Existing Connections**

The Otay WD is 100 percent metered for all customer sectors, including separate meters for single-family residential, commercial, large landscapes, institutional, and governmental facilities. Any unmetered use generally occurs at fire hydrants or from distribution system breaks. Estimates are made and accounted for each occurrence of a known unmetered water use event.

The Otay WD has an inclining block rate structure, with a lifeline allotment of 5 billing units per customer per month for residential customers. A billing unit is one hundred cubic feet (748 gallons), commonly abbreviated as HCF. Since 1990, commercial, industrial, and institutional customers are also required to have separate irrigation meters for both potable and recycled water.

The Otay WD water meter replacement, calibration, and maintenance program has been practiced for decades. The purpose is to maintain low levels of accounted for water loss. This is accomplished through scheduled water meter replacement and calibration efforts. The calibration of meters larger than two inches is generally performed on an annual basis. Water meters two inches in diameter and smaller are generally replaced once every ten years on the average. Meter calibration and periodic replacement insures that customers are paying for all of the water they consume, and therefore encourages conservation.

- **BMP 5 – Large Landscape Conservation Programs and Incentives**

The City of Chula Vista in cooperation with the Otay WD enforces water efficient landscaping requirements for all commercial, industrial, and institutional developments. A registered landscape architect is required to design the landscape plans to include automatic irrigation systems, rain shutoff devices, in-line check valves to prevent low head drainage, and separate landscaping meters. The use of xeriscape landscaping techniques, maximizing the use of drought tolerant (low water consuming) plants, and appropriate maintenance are reviewed and if needed, recommendations are provided.

As part of the Otay WD program, a California Irrigation Management Information System (CIMIS) weather station, located at the U.S. Olympic Training Center facility (Weather Station No. 147), is available to download precipitation and evapotranspiration levels to coincide with the customers' irrigation scheduling. This will assist in reducing overall annual irrigation water use and water quality benefits through management of runoff. In addition, Otay WD will be partnering with the Water Authority to begin offering an incentive to commercial and residential customers for the installation of weather-based irrigation controllers. These controllers will have the ability to adjust the irrigation schedule with historical seasonal or real-time weather data.

Since 1990, irrigation surveys are conducted for the large landscape customers (currently defined as one acre or greater), at no charge to the customer through the Professional Assistance for Landscape Management (PALM) program, sponsored by the Water Authority. During the survey, the survey team examines the irrigation system for distribution uniformity, matched irrigation components, and controller scheduling. The team calculates and recommends a water budget for the site, based on the size of the landscape, the plant material, and the climate. The Otay WD continues to be one of the few and first water agencies in the state that maintains a landscape water budget program for its landscape customers. The Otay WD will continue to implement this BMP by review of customers' water use and water budget and by offering ongoing follow-up evaluations to customers whose total water use exceeds their total annual water budget. In addition, in cooperation with the Water Authority, the Otay WD will soon participate in a new program providing incentives to improve sprinkler efficiency, known as the Commercial Landscape Incentive Program (CLIP).

- **BMP 6 – High-Efficiency Washing Machine (HEWM) Voucher Program**

Since 1995, the High Efficiency Washing Machine (HEWM) Voucher Program has been available to Otay WD customers. New technology in washing machine design provides for more efficient use and water savings. Over the past few years, an increasing number of residential customers have taken advantage of the $125 voucher offer. The HEWM's installed in multi-family laundry rooms and laundromats are eligible to receive a $300 voucher through the commercial HEWM program. Vouchers are offered for residential, commercial, institutional, and industrial customers.

- **BMP 7 – Public Information Programs**

Water conservation public information programs consist of newsletters, annual water quality Consumer Confidence Reports, brochures, bill inserts, bill messages, event staffing, web page maintenance, an annual Water Wise Landscape Contest, and active participation in the Water Conservation Garden (Garden). The Garden is a 4.25-acre site located at Cuyamaca College and has been opened to the public since late 1999. The Garden functions as a learning facility to further the education of visitors on how to effectively achieve water savings through xeriscape landscape techniques. The Garden is utilized for public and mid-week school tours, teacher in-service training, special events, seminars, classes, workshops, and community events.

The Otay WD regularly encourages its customers to visit the Garden for landscape ideas and attend classes. The Otay WD staff regularly develops bill inserts and messages in the water bill promoting landscape water efficiency and the Garden, and frequently writes articles on the Garden in its quarterly newsletter called the "Pipeline". In addition, a "Welcome to Otay" brochure is distributed to all residential customers new to the Otay WD, promoting the Garden, water-wise landscape practices, and other applicable water conservation programs.

- **BMP 8 – School Education Programs**

The Otay WD works with all school districts in its service area to educate students about water issues through curriculum-based educational programs. The Otay WD offers a full-service school education program, including classroom presentations, Garden tours, awards materials, and science fair participation. Grants for school site demonstration gardens and bus transportation are available as well. The Otay WD participates in and coordinates educational programs sponsored by the Water Authority, including teacher in-service training and Water Authority mini-grants.

- **BMP 9 – Conservation Programs for Commercial, Industrial, and Institutional Accounts**

Since 1995, the Otay WD has provided vouchers for water efficient devices to its commercial, industrial, and institutional accounts through shared-funding programs with the Water Authority and Metropolitan. Vouchers for $95 are available for low-flow and water-less urinals, $300 for commercial clothes washers installed in laundromats and multi-family common areas, $95 for commercial ULFTs, and $500 for cooling tower conductivity controllers. Incentives are now also available for multi-load commercial clothes washers, pre-rinse sprayers, and x-ray photo processing machines.

In addition, the Otay WD works closely with developers and the City of Chula Vista staff to provide materials and information on the latest water efficient technologies. The Otay WD staff developed a list of water-wise publications and provides them to developers that are creating new homeowner packets as required by the City of Chula Vista. The Otay WD works closely with the City of Chula Vista staff to evaluate the water conservation plans of new developments and to encourage the installation of new technologies such as weather-based irrigation controllers and dual-flush toilets in new construction.

- **BMP 10 – Wholesale Agency Assistance Program**

This BMP applies only to wholesale agencies. The Water Authority provides conservation-related technical support and information to its member agencies, including ULFT and High Efficiency Clothes Washer Program vouchers, residential surveys, partial funding for water efficient devices in commercial, institutional, and industrial properties, large turf irrigation, and conservation-related rates and pricing. The Water Authority typically manages the programs on behalf of its member agencies and contributes 25% of the cost for the incentive or survey. The Otay WD contributes another 25% of the cost, while Metropolitan typically provides 50% of the incentive.

- **BMP 11- Conservation Pricing**

Water rates vary among classifications of usage. The rates for residential customers are based on an accelerated block structure, where as more units are consumed, a higher unit rate is charged. Non-residential irrigation customers are charged a flat rate per unit. The recycled water rate is set at 85% of the potable water rate to provide an economic incentive for the use of the recycled water supply.

- **BMP 12 – Conservation Coordinator**

In accordance with the MOU, the Otay WD established a full-time Water Conservation Coordinator position in 1991. In addition, the Otay WD has a full-time Water Conservation Specialist position.

- **BMP 13 – Water Waste Prohibition**

The Otay WD has a "No Waste" ordinance, which is actively enforced. Enforcement of the ordinance includes a water use investigator to educate customers and if necessary, issue warnings and citations for violations.

- **BMP 14 – Residential ULFT Replacement Program**

The Otay WD established an Ultra-Low Flush Toilet (ULFT) Replacement Program in 1991. Residential customers are eligible to receive $75 off the cost of a ULFT toilet. In addition, a $95 voucher is available toward the purchase of a dual-flush toilet, which has been found to use 30% less water than a standard ULFT. The Otay WD worked closely with the Water Authority to develop a pilot incentive program to encourage builders to install dual-flush toilets in new construction. Currently, a $50 voucher is available to builders for every dual flush toilet installed.

Additional conservation or water use efficiency measures or programs practiced by the Otay WD include the following:

- **Supervisory Control and Data Acquisition System**

The Otay WD implemented and has operated for many years a Supervisor Control and Data Acquisition (SCADA) system to control, monitor, and collect data regarding the operation of the water system. The major facilities that have SCADA capabilities are the water supply source, transmission network, pumping stations, and water storage reservoirs. The SCADA system allows for many and varied useful functions. Some of these functions provide for operating personnel to monitor the water supply source flow rates, reservoir levels, turn on or off pumping units, etc. The SCADA system aids in the prevention of water reservoir overflow events and increases energy efficiency.

- **Water Conservation Ordinance**

California Water Code Sections 375 et seq. permit public entities which supply water at retail to adopt and enforce a water conservation program to reduce the quantity of water used by the people therein for the purpose of conserving water supplies of such public entity. The Otay WD Board of Directors established a comprehensive water conservation program pursuant to California Water Code Sections 375 et seq., based upon the need to conserve water supplies and to avoid or minimize the effects of any future shortage. A water shortage could exist based upon the occurrence of one or more of the following conditions:

1. A general water supply shortage due to increased demand or limited supplies.
2. Distribution or storage facilities of the Water Authority or other agencies become inadequate.
3. A major failure of the supply storage and distribution facilities of the Metropolitan, the Water Authority, or of the Otay WD occurs.

The Otay WD water conservation ordinance finds and determines that the conditions prevailing in the San Diego County area require that the available water resources be put to maximum beneficial use to the extent to which they are capable, and that the waste or unreasonable use, or unreasonable method of use, of water be prevented and that the conservation of such water be encouraged with a view to the maximum reasonable and beneficial use thereof in the interests of the people of the Otay WD and for the public welfare.

Section 6 - Existing and Projected Supplies

The Otay WD currently does not have an independent potable water supply source. The Otay WD is a member public agency of the Water Authority. The Water Authority is a member public agency of Metropolitan. The statutory relationships between the Water Authority and its member agencies, and Metropolitan and its member agencies, respectively, establish the scope of Otay WD's entitlements to water from these two agencies.

The Water Authority through two aqueducts, referred to as Pipeline No. 4 and the La Mesa Sweetwater Extension Pipeline, currently supply the Otay WD with 100 percent of its potable water. The Water Authority in turn, currently purchases all of its water from Metropolitan. Due to the Otay WD reliance on these two agencies, this WSA&V Report includes information on the existing and projected supplies, supply programs, and related projects of the Water Authority and Metropolitan. The Water Authority and Metropolitan are actively pursuing programs and projects to diversify their water supply resources. This information, along with a description of local recycled water supplies available to the Otay WD, is discussed below.

6.1 Metropolitan Water District of Southern California

In March 2003, Metropolitan produced a document entitled, "Report on Metropolitan's Water Supplies, A Blueprint for Water Reliability". The objective of the March 2003 Report is to provide the member agencies, retail water utilities, cities, and counties within their service area with information that may assist in their compliance with Water Code Sections 10910 through 10914 and Government Code Sections 65867.5, 66455.3, and 66473.7. The March 2003 Report states that the approach to evaluating water supplies and demands is consistent with Metropolitan's 2000 Regional UWMP. Metropolitan utilizes SANDAG's regional growth forecast in calculating regional water demands for the Water Authority's service area.

6.1.1 Availability of Sufficient Supplies and Plans for Acquiring Additional Supplies

Metropolitan is a wholesale supplier of water to its member public agencies and obtains its supplies from two primary sources: the Colorado River Aqueduct (CRA), which it owns and operates, and the State Water Project (SWP). The purpose of the March 2003 Report is to document the availability of these existing supplies and additional supplies necessary to meet future demands. To ensure a thorough analysis of the water supplies available to serve the proposed Project along with existing and future water demands, supplemental information to the March 2003 Report is included in the following paragraphs.

Colorado River Aqueduct Deliveries

The March 2003 Report includes a description of Metropolitan's 550,000 ac-ft/yr base apportionment water (Priority 4) along with the Colorado River supply projects that are necessary to maintain a full CRA. One of the actions that were finalized following distribution of the March 2003 Report is approval of the Quantification Settlement Agreements (QSA) and other related agreements. Signing of the QSA and related agreements will now allow implementation of Colorado River supply projects identified in Metropolitan's March 2003 Report. Information on these activities is discussed below.

The QSA is an integral part of California's Colorado River Water Use Plan to reduce dependency on Colorado River supplies. The QSA resolves long-standing disputes regarding priority and use of river water and creates a baseline for implementing water transfers. Implementation of the QSA also enables California to receive the benefit of special surplus criteria for Colorado River supplies to significantly increase the probability of surplus deliveries and provide a "soft-landing" for California while it reduces its take on the Colorado River.

Written Contracts or Other Proof

The following is a list of major QSA-related agreements and actions pertinent to water supply reliability in San Diego County along with the date that each were executed. Copies of agreements are on file in the corresponding agencies.

- Passage of SB 654 (Machado), SB 317 (Kuehl), and SB 277 (Ducheny) (September 2003). The California Governor signed these bills into law, which was necessary to carry out the actions contained in the QSA and related agreements.

- Quantification Settlement Agreement by and among Imperial Irrigation District, Metropolitan, and Coachella Valley Water District (October 10, 2003). This Agreement and related agreements are intended to consensually settle longstanding disputes regarding the priority, use, and transfer of Colorado River water and to establish by agreement the terms for the further distribution of Colorado River water among agencies for up to seventy-five (75) years. The agreement will also assist the agencies in meeting their water demands within California's apportionment of Colorado River water by identifying the terms, conditions, and incentives for the conservation and distribution of Colorado River water within California.

- Colorado River Delivery Agreement among the Department of the Interior, Coachella Valley Water District, Imperial Irrigation District, Metropolitan, and Water Authority (October 10, 2003). This Agreement provides federal authorization for water deliveries pursuant to the QSA. With approval by the Secretary of Interior, the Interim Surplus Guidelines have been reinstated.

- Allocation Agreement among the United States, Metropolitan, Coachella Valley Water District, Imperial Irrigation District, the Water Authority, and the San Luis Rey Indian Water Rights Settlement Parties (October 10, 2003.) This Agreement allocates water from the lining of the All American and Coachella Canals and assigns the right to 77.7 thousand acre-feet of conserved water per year from Metropolitan to the Water Authority in accordance with the Agreement.

Federal, State, and Local Permits/Approvals

- Final Program Environmental Impact Report (June 2002) for Implementation of the Colorado River Quantification Settlement Agreement. In June 2002, the three California Colorado River agencies (Metropolitan, IID and CVWD) certified the Program Environmental Impact Report (PEIR) for the QSA.

- Addendum to Final PEIR for Implementation of the Colorado River Quantification Settlement Agreement (October 2003). The Addendum to the Final PEIR was approved by the agencies during the months of September and October 2003. The modifications to the QSA require only minor changes to the evaluation in the certified Final PEIR to make it adequate under California Environmental Quality Act (CEQA) and do not require preparation of a subsequent EIR pursuant to CEQA.

- Conservation Agreement among the Bureau of Reclamation, Imperial Irrigation District, Coachella Valley Water District, and San Diego County Water Authority (October 10, 2003). This agreement is for the purpose of establishing the rights and obligations of the parties to implement the provisions of the Species Conservation Program. IID has commenced development of a habitat conservation plan (HCP) in accordance with the Federal and California Endangered Species Act, related to implementation of water conservation projects identified in the QSA. The HCP is not expected to be completed for up to three years after the execution of the QSA and the parties desire to participate with the Bureau of Reclamation in the implementation of the Species Conservation Program for the purpose of obtaining incidental take authorization pending completion of the HCP.

Integrated Resources Plan

Metropolitan is updating its Integrated Resources Plan (IRP), which will identify local supply production more fully and will include a buffer supply to mitigate against the risks associated with implementation of local and imported supply programs. Future supply reliability relies not only upon actions by Metropolitan to secure reliable imported supplies, but local agencies developing local projects identified in the future resource mix. Supply reliability associated with execution of the QSA is included in this update.

Metropolitan's Capital Investment Plan

As part of Metropolitan's annual budget approval process, a Capital Investment Plan is prepared. The cost, status, and progress of Metropolitan's infrastructure projects to deliver existing and future supplies are documented in this Plan. The financing of these projects is approved through the annual budget approval process.

6.2 San Diego County Water Authority

In accordance with the Urban Water Management Planning Act, the Water Authority adopted an UWMP in December 2000. The plan demonstrates that with implementation of the projects identified in the plan, adequate supplies will be available to meet future demands.

To ensure adequate supplies to meet future growth in the San Diego region, the Water Authority uses SANDAG's most recent regional growth forecast in calculating regional water demands. The existing and future demands of the Otay WD are included in the Water Authority's projections.

6.2.1 Availability of Sufficient Supplies and Plans for Acquiring Additional Supplies

The Water Authority currently purchases all its supplies from Metropolitan, but is pursing projects to diversify its supply. There are 27 member agencies that purchase supplies from Metropolitan, with the Water Authority being the largest customer. The Water Authority has executed a 10-year Purchase Order for Imported Water Supply from Metropolitan with an initial base demand of 556,399 acre-feet. Section 135 of Metropolitan's Act defines the preferential right to water for each member agency. As calculated by Metropolitan, the Water Authority currently has a preferential right to about 15.54% of Metropolitan's supply and uses approximately 28%. At any time under preferential rights, Metropolitan could allocate water without regard to historic water use or dependence on Metropolitan. Metropolitan has stated, consistent with Section 4202 of their Administrative Code, that they are prepared to provide the Water Authority's service area with adequate supplies of water to meet expanding and increasing needs in the years ahead. When and as additional water resources are required to meet increasing needs, Metropolitan will be prepared to deliver such supplies. To seek clarification regarding the current application and legality of Section 135, the Water Authority filed a lawsuit in Superior Court in January 2001. The suit is currently pending in the Court of Appeal. The historical annual imported water deliveries from Metropolitan are contained in Section 2.3 of the Water Authority's 2000 UWMP.

The Water Authority has made large investments in Metropolitan's facilities over the last 50 years and therefore will continue to include imported supplies from Metropolitan in the future resource mix. As discussed in the Water Authority's 2000 UWMP, the Water Authority is planning to diversify its supply portfolio and reduce purchases from Metropolitan. Implementation of water conservation measures within the Water Authority's service area is one of the most cost-effective means of reducing demands. The Water Authority's plan for achieving conservation savings and the estimated amount of future savings is discussed in the Water Authority's 2000 UWMP.

To meet future demands and diversify supplies, the Water Authority is implementing a water transfer with IID, implementing the All American Canal and Coachella Canal lining projects, and planning for the desalination of seawater. Table 5 summarizes the planned yields from these supply projects.

Table 5
SDCWA Projected Regional Water Supplies
Normal Year (acre-feet/year)

Water Supply Sources	2005	2010	2015	2020	2025
Water Authority/IID Transfer	30,000	70,000	100,000	190,000	200,000
AAC and CC Lining Projects	0	77,700	77,700	77,700	77,700
Seawater Desalination[1]	0	56,000	56,000	56,000	56,000
Total Projected Supplies	30,000	203,700	233,700	323,700	333,700

[1] The Water Authority is currently pursuing a 50 mgd seawater desalination facility at the Encina Power Plant in the City of Carlsbad that will yield approximately 56,000 acre-feet per year. According to the Water Authority's draft Water Facilities Master Plan, the facility could be expanded to 80 – 100 mgd in the future and/or other facilities constructed to increase this supply source.

These supplies are considered "drought-proof" supplies and should be available at the yields shown in Table 5 in both dry and multi-dry year scenarios. The status of the projected regional water supplies is detailed further within the following section.

6.2.1a The Water Authority-IID Water Conservation and Transfer Agreement

On April 29, 1998, the Water Authority signed a historic agreement with IID for the long-term transfer of conserved Colorado River water to San Diego County. Under the Water Authority-IID Agreement, Colorado River water will be conserved by Imperial Valley farmers, who voluntarily participate in the program, and then transferred to the Water Authority for use in San Diego County. The water to be conserved is part of IID's Colorado River rights, which are among the most senior in the Lower Colorado River Basin. Imperial Valley farmers will conserve the water by employing extra-ordinary conservation measures.

<u>Implementation Status</u>

On October 10, 2003, the Water Authority and IID executed an amendment to the original 1998 Water Authority-IID Water Transfer Agreement. The purpose of the amendment is to modify certain aspects of the 1998 Agreement to be consistent with the terms and conditions of the QSA and related agreements and to modify other aspects to temporarily lessen the environmental impacts of the transfer of conserved water. The Amendment was expressly conditioned upon approval and implementation of the QSA.

A restructuring of the IID transfer for the first 15 years of the agreement was needed to avoid potential impacts to the Salton Sea from reduced agricultural flows to the Salton Sea that are caused by the agricultural conservation measures in the Imperial Valley. State and federal regulatory agencies have stated that IID should maintain baseline salinity levels at the Sea for 15 years while they develop and begin to implement a plan to restore the Sea. The amendments contemplate that IID will conduct a combined temporary fallowing and system improvement program during the first 15 years of the transfer. In the sixteenth year of the agreement, all temporary fallowing would end and all water for transfer would be produced through on-farm and system conservation measures.

On November 5, 2003, IID filed a complaint in Imperial County Superior Court seeking validation of 13 contracts associated with the IID/Water Authority water transfer and the QSA. A validation lawsuit allows a public agency to bring an action in Superior Court to validate actions. This suit will provide the agencies with certainty and will facilitate implementation of the water transfer and the QSA. In another related legal action, Imperial County and various private parties filed suits in Superior Court, alleging violations of CEQA, the California Water Code, and other laws in connection with approval of the QSA, the water transfer, and related agreements. The IID, Coachella Valley Water District, Metropolitan, and the Water Authority are defending these suits and coordinating to seek validation of the contracts. Implementation of the transfer provisions will continue during the litigation.

Expected Supply

With execution of the QSA and related agreements, delivery of 10,000 acre-feet into San Diego County from the transfer will occur. The quantities will increase annually to 200,000 acre-feet by the year 2021 and remain fixed for the duration of the transfer agreement. The initial term of the agreement is for 45 years, with a provision that either agency may extend the agreement for an additional 30-year term under certain circumstances.

Transportation

The Water Authority entered into a water exchange agreement with Metropolitan on October 10, 2003 to transport the Water Authority/IID transfer water from the Colorado River to San Diego County. Under the exchange agreement, Metropolitan will take delivery of the transfer water through its CRA. In exchange, Metropolitan will deliver to the Water Authority a like quantity and quality of water. The Water Authority will pay Metropolitan's applicable rate for each acre-foot of exchange water delivered. According to the water exchange agreement, Metropolitan will make delivery of the transfer water for 35 years, unless the Water Authority elects to extend the agreement another 10 years for a total of 45 years.

Cost/Financing

The costs associated with the transfer are proposed to be financed through the Water Authority's rates and charges. In the restructured agreement between the Water Authority and IID, the price for the transfer water will start at $258 per acre-foot and increase each year at a set price.

In accordance with the October 2003 amended exchange agreement between Metropolitan and the Water Authority, the initial cost to transport the conserved water is $253 per acre-foot. Thereafter, the rate shall be equal to the charge or charges set by Metropolitan's Board of Directors pursuant to applicable law and regulation and generally applicable to the conveyance of water by Metropolitan on behalf of its member agencies.

The Water Authority will pay IID up-front payments of $20 million, including $10 million to help offset socioeconomic impacts associated with temporary land fallowing. At the end of the fifth year of the agreement, the Water Authority will prepay IID $10 million for future deliveries of water. IID will credit the Water Authority for its up-front payment during years 16 through 30.

As part of implementation of the QSA and water transfer, the Water Authority entered into an environmental cost sharing agreement. The agreement specifies that the Water Authority will contribute a total of $64 million for the purpose of funding environmental mitigation costs and contribute towards the Salton Sea Restoration Fund.

Written Contracts or Other Proof

The expected supply and costs associated with the transfer are based primarily on the following documents. Copies of agreements are on file in the corresponding agencies.

- Agreement for Transfer of Conserved Water by and between IID and the Water Authority (April 29, 1998). This Agreement provides for a market-based transaction in which the Water Authority would pay IID a unit price for agricultural water conserved by IID and transferred to the Water Authority.

- Amendment to Agreement between IID and the Water Authority for Transfer of Conserved Water (October 10, 2003). Consistent with the executed QSA and related agreements, the amendments restructure the agreement and modify it to minimize the environmental impacts of the transfer of conserved water to the Water Authority.

- Amended and Restated Agreement between Metropolitan and Water Authority for the Exchange of Water (October 10, 2003). This agreement was executed pursuant to the QSA and provides for delivery of the transfer water to the Water Authority.

- Environmental Cost Sharing, Funding, and Habitat Conservation Plan Development Agreement among IID, CVWD, and Water Authority (October 10, 2003). This Agreement provides for the specified allocation of QSA-related environmental review, mitigation, and litigation costs for the term of the QSA, and for development of a Habitat Conservation Plan.

- Quantification Settlement Agreement Joint Powers Authority Creation and Funding Agreement (October 10, 2003). The purpose of this agreement is to create and fund the QSA Joint Powers Authority and to establish the limits of the funding obligation of CVWD, IID, and Water Authority for environmental mitigation and Salton Sea restoration pursuant to SB 654 (Machado).

Federal, State, and Local Permits/Approvals

- EIR for Conservation and Transfer Agreement. As lead agency, IID certified the Final EIR for the Conservation and Transfer Agreement on June 28, 2002.
- Addendum to EIR for Conservation and Transfer Agreement. IID as lead agency and Water Authority as responsible agency approved addendum to the EIR in October 2003.
- EIS for Conservation and Transfer Agreement. Bureau of Reclamation issued a Record of Decision on the EIS in October 2003.
- Federal Endangered Species Act Permit. The U.S. Fish and Wildlife Service issued a Biological Opinion on January 12, 2001, that provides incidental take authorization and certain measures required to offset specified impacts on the Colorado River regarding such actions.
- California Endangered Species Act Permit. Application for Section 2081 permit is pending with California Department of Fish and Game.
- State Water Resources Control Board (SWRCB) Petition. SWRCB adopted Water Rights Order 2002-0016 concerning IID and Water Authority's amended joint petition for approval of a long-term transfer of conserved water from IID to the Water Authority and to change the point of diversion, place of use, and purpose of use under Permit 7643.

6.2.1b All American Canal and Coachella Canal Lining Projects

On September 25, 2003, the Water Authority Board voted to accept assignment of the Metropolitan's water rights to 77,700 ac-ft/yr from projects that will line the All American Canal (AAC) and Coachella Canal (CC). The projects will stop the loss of water that currently occurs through seepage and that conserved water will go to the Water Authority. This will provide the San Diego region with an additional 8.5 million acre-feet of water over the 110-year life of the agreement.

Implementation Status

The AAC lining project is at the pre-design phase. The lining project consists of constructing a concrete-lined canal parallel to 23 miles of the existing AAC from Pilot Knob to Drop 3. NEPA and CEQA documentation is complete, environmental mitigation measures have been identified and Endangered Species Act consultations are pending. Completion of final design will take about two years and an additional four years for construction. Completion of the entire project could be achieved by summer of 2010. The first portions (Reaches 2 and 3) of the project could be completed about one year earlier, in mid-2009.

The CC lining project is at 90 percent design. NEPA and CEQA documentation is complete, but may require amending to account for a somewhat different alignment of the new lined parallel canal and the construction of 26 new siphons that were not identified in the current environmental documentation. Endangered Species Act consultations are underway. Completion of the final design will take about six months and an additional four years for construction. Completion of the entire project could be achieved by mid-2008.

Expected Supply

The U.S. Secretary of the Interior will determine the total amount of water available for allocation upon completion of construction of each reach of canal, based on amounts estimated in the Final Environmental Impact Statement/Environmental Impact Reports (FEIS/EIRs) of each project. The AAC lining project FEIS/EIR estimates that 67,700 acre-feet of Colorado River water will be available per year for allocation upon completion of construction of the AAC lining project. The CC lining project FEIS/EIR estimates that 26,000 acre-feet of Colorado River water will be available per year for allocation upon completion of construction of the CC lining project. The October 10, 2003 Allocation Agreement states that 16,000 ac-ft/yr of conserved canal lining water will be allocated to the San Luis Rey Indian Water Rights Settlement Parties. The remaining amount, an estimated 77,700 ac-ft/yr, will be available to the Water Authority in approximately 2010. According to the Allocation Agreement, IID does have call rights to a portion (5,000 ac-ft/yr) of the conserved water upon termination of the QSA for the remainder of the 110 years of the Allocation Agreement and upon satisfying certain conditions. The term of the QSA is for up to 75 years.

Transportation

The October 10, 2003 Exchange Agreement between the Water Authority and Metropolitan also provides for the delivery of the conserved water from the canal lining projects. The Water Authority will pay Metropolitan's applicable rate for each acre-foot of exchange water delivered. In the Agreement, Metropolitan will delivery the canal lining water for the term of the Allocation Agreement (110 years).

Cost/Financing

The total estimated budget requirements for AAC and CC lining projects is approximately $327 million. Under California Water Code Section 12560 et seq., the Water Authority would receive $200 million for construction of the projects. In addition, under California Water Code Section 79567, $20 million could also be available for the lining projects. The Water Authority would be responsible for additional expenses above the grants funds provided by the state.

In accordance with the amended exchange agreement between Metropolitan and the Water Authority, the initial cost to transport the canal lining water is $253 per acre-foot. Thereafter, the rate shall be equal to the charge or charges set by Metropolitan's Board of Directors pursuant to applicable law and regulation and generally applicable to the conveyance of water by Metropolitan on behalf of its member agencies.

In accordance with the Allocation Agreement, the Water Authority will also be responsible for a portion of the net additional Operation, Maintenance, and Repair (OM&R) costs for the lined canals. The Secretary of Interior, working with the Canal Lining Projects OM&R Coordinating Committees, will determine the additional costs of operation, maintenance, and repair of the AAC and CC.

Any costs associated with the lining projects as proposed, are to be financed through the Water Authority's rates and charges.

Written Contracts or Other Proof

The expected supply and costs associated with the transfer are based primarily on the following documents: documents. Copies of agreements are on file in the corresponding agencies.

- U.S. Public Law 100-675 (1988). Authorized the Department of the Interior to reduce seepage from the existing earthen AAC and CC. The law provides that conserved water will be made available to specified California contracting water agencies according to established priorities.

- Allocation Agreement among the United States of America, The Metropolitan Water District of Southern California, Coachella Valley Water District, Imperial Irrigation District, San Diego County Water Authority, the La Jolla, Pala, Pauma, Rincon, and San Pasqual Bands of Mission Indians, the San Luis Rey River Indian Water Authority, the City of Escondido, and Vista Irrigation District (October 10, 2003). This agreement includes assignment of Metropolitan's rights and interest in delivery of 77,700 acre-feet of Colorado River water previously intended to be delivered to Metropolitan to the Water Authority. Allocates water from the AAC and CC lining projects for at least 110 years to the Water Authority, the San Luis Rey Indian Water Rights Settlement Parties, and IID, if it exercises its call rights.

- Amended and Restated Agreement between Metropolitan and Water Authority for the Exchange of Water (October 10, 2003). This agreement was executed pursuant to the QSA and provides for delivery of the conserved canal lining water to the Water Authority.

- California Water Code Section 12560 et seq. This Water Code Section provides for two hundred million dollars to be appropriated to the Department of Water Resources to help fund the canal lining projects in furtherance of implementing California's Colorado River Water Use Plan.

- California Water Code Section 79567. This Water Code Section identifies twenty million ($20 million) as available for appropriation by the California Legislature from the Water Security, Clean Drinking Water, Coastal, and Beach Protection Fund of 2002 (Proposition 50) to DWR for grants for canal liming and related projects necessary to reduce Colorado River water use. According to the Allocation Agreement, it is the intention of the agencies that those funds will be available for use by the Water Authority, IID, or CVWD for the AAC and CC lining projects.

The following agreements are currently executed to facilitate funding and construction of the AAC and CC lining projects. In accordance with the Allocation Agreement, Metropolitan has agreed to assign their rights associated with the agreements to the Water Authority.

- California Department of Water Resources – Metropolitan Funding Agreement (2001). Reimburse Metropolitan for project work necessary to construct the lining of the CC in an amount not to exceed $74 million.

- California Department of Water Resources – IID Funding Agreement (2001). Reimburse IID for project work necessary to construct a lined AAC in an amount not to exceed $126 million.

- Metropolitan – CVWD Assignment and Delegation of Design Obligations Agreement (2002). Assigns design of the CC lining project to CVWD.

- Metropolitan – CVWD Financial Arrangements Agreement for Design Obligations (2002). Obligates Metropolitan to advance funds to CVWD to cover costs for CC lining project design and CVWD to invoice Metropolitan to permit the Department of Water Resources to be billed for work completed.

Federal, State, and Local Permits/Approvals

- AAC Lining Project Final EIS/EIR (March 1994). A final EIR/EIS analyzing the potential impacts of lining the AAC was completed by the Bureau of Reclamation (Reclamation) in March 1994. A Record of Decision was signed by Reclamation in July 1994, implementing the preferred alternative for lining the AAC. A re-examination and analysis of these environmental compliance documents by Reclamation in November 1999 determined that these documents continued to meet the requirements of the National Environmental Policy Act and the California Environmental Quality Act and would be valid in the future.

- CC Lining Project Final EIS/EIR (April 2001). The final EIR/EIS for the CC lining project was completed in 2001. Reclamation signed the Record of Decision in April 2002.

6.2.1c The Water Authority's Seawater Desalination Project at Encina

The Water Authority's proposed Seawater Desalination Project at Encina (Desal Project) consists of a 50 mgd reverse osmosis desalination plant sited adjacent to the Encina Power Station in the City of Carlsbad and the pipelines and ancillary facilities necessary to convey product water from the plant to local and regional water distribution systems.

Implementation Status

The seawater desalination plant component of the project may be developed, financed, constructed, and operated for the Water Authority, through a design-build-operate procurement or an extended turnkey arrangement. The final project agreements cannot be executed until after completion of CEQA compliance for the Desal Project. The Water Authority is currently proceeding with an EIR for the Desal Project.

The seawater desalination conveyance facilities component will be constructed and owned by the Water Authority, as determined since inception of the Desal Project. A number of alternatives have been initially evaluated for delivery of desalinated water to the local and regional water supply distribution facilities. Various distribution system improvements (pipelines, pump stations, and other appurtenances) would be required to meet the projected demands to be served from the seawater desalination plant facility. The Water Authority is currently proceeding with a feasibility and alignment study for the conveyance facility component of the Desal Project.

The Water Authority is also working with the City of Carlsbad to develop a Memorandum of Understanding (MOU) that would address the delivery of desalinated seawater to the City of Carlsbad's water distribution system along with land use compatibility and project impact mitigation.

Expected Supply

The Desal Project is anticipated to produce 56,000 acre-feet annually of new water supply generated from seawater drawn in by the Encina Power Station cooling water circulation system from the Pacific Ocean via the Agua Hedionda Lagoon. The Desal Project would provide a new source of high quality water that would meet or exceed state and federal standards.

Written Contracts or Other Proof

The expected supply and cost associated with the proposed desalination project is currently based on the following document. Copies of agreements are on file in the corresponding agencies.

- Summary of Key Terms and Conditions (term sheet) Between the San Diego County Water Authority and Poseidon Resources Corporation for the Development of the Desalination Plant Component of the Carlsbad Seawater Desalination Project. The Water Authority Board of Directors approved this term sheet with Poseidon in November 2002.

Cost/Financing

The total estimated capital cost of the Desal Project is about $272 million in 2001 dollars, which includes the treatment facility and the cost to deliver the desalinated seawater into the local and regional distribution systems. According to the term sheet, the Water Authority will purchase water from the desalination plant for the first five years of operation. At the end of this five-year period, the Water Authority would purchase the plant using tax-exempt municipal financing and assume ownership.

The Water Authority is pursuing external funding to offset the capital and operating cost of the Desal Project including funding through the Metropolitan Water District of Southern California Seawater Desalination Program (SDP), state funding through the recently passed Proposition 50, as well as federal funding opportunities. The Water Authority hopes to secure SDP funding in 2004.

Federal, State, and Local Permits/Approvals

Table 6 provides a list of the major permits and discretionary actions required for the Desal Project and the anticipated schedule for completion of the permitting process.

Based on the estimated completion dates also shown in Table 6, the Water Authority anticipates Desal Project construction complete by 2007. Following a six-month start-up period, the Desal Project is scheduled to be on-line in 2008.

Table 6
List of Major Permits and Discretionary

Permit or Discretionary Action	Purpose	Scope	Scheduled Completion
Certification of Environmental Impact Report	Satisfy the requirements of the California Environmental Quality Act.	Those aspects of the proposed Desal Project that may affect environmental quality.	2004
Endangered Species Act Compliance (ESA)	Satisfy ESA requirements.	Proposed distribution facilities.	2004
Local Land Use Regulations* (*For any private portions of the project subject to GC 53091)	Satisfy the plans, policies, and ordinances of the City of Carlsbad as applied to the Desal Project site.	Land use, aesthetics.	2005
Domestic Water Supply Permit	Satisfy the requirements of the state and federal Safe Drinking Water Acts.	Source water and product water quality, treatment plant reliability and monitoring program.	2005 (Conceptual approval)
National Pollutant Discharge Elimination System Permit	Satisfy the requirements of the federal Clean Water Act, California Water Code, Ocean Plan, and Comprehensive Water Quality Control Plan for the San Diego Region.	Proposed discharge of concentrated seawater to the Pacific ocean via existing cooling water discharge system.	2005
Coastal Development Permit	Satisfy the requirements of the California Coastal Act and the federal Coastal Zone Management Act.	Those aspects of the proposed Desal Project that may affect coastal resources.	2005
Right-of-Way Acquisition for conveyance facilities	Acquire land necessary for construction of conveyance facilities.	Proposed distribution facilities.	2006

Water Authority's Capital Improvement Program and Financial Information

The Water Authority's annual Capital Improvement Program (CIP) budget document includes a description of each of the projects and programs being implemented to ensure existing and future facilities are adequate to deliver water supplies to the region. The project costs along with information on the activities that need to be completed are included in the CIP document. In addition, the Water Authority Board of Directors is provided a quarterly report on the status of development of the projects. As described in the Water Authority's annual budget, a combination of long- and short-term debt and cash (pay-as-you-go) will provide funding for capital improvements. Additional information is contained in the five-year forecast included in the Water Authority annual budget. The Water Authority's annual report also contains selected financial information and summarizes the Water Authority's investment policy.

6.3 Otay Water District

The Otay WD's Water Resources Master Plan and UWMP contains a comparison of projected supply and demands through the year 2020. Projected potable water resources to meet demands as planned are to be supplied entirely with imported water received from the Water Authority. Recycled water resources to meet projected demands as planned are to be supplied from local wastewater treatment plants. The Otay WD currently has no local supply of potable water or groundwater resources. The development of potential groundwater supplies is a possibility for consideration in the future to allow for less reliance upon imported water. The supply and demand forecasts contained within this WSA&V Report do not consider local groundwater development as a supply resource.

6.3.1 Demonstrating the Availability of Sufficient Supplies and Plans for Acquiring Additional Supplies

Section 5 subdivision 11 of the County Water Authority Act states that the Water Authority "as far as practicable, shall provide each of its member agencies with adequate supplies of water to meet their expanding and increasing needs." The Water Authority provides between 75 to 95 percent of the total supplies used by its 23 member agencies, depending on local weather and supply conditions. Historic imported water deliveries from the Water Authority to Otay WD and recycled water deliveries from Otay WD's Ralph W. Chapman Water Recycling Facility (RWCWRF) are shown in Table 7. Since the year 2000, recycled water demand has exceeded supply capabilities of the RWCWRF, which is typically limited to about 1,100 ac-ft/yr. The current and near term supply shortfall will be met by supplementing, that is adding potable water supplied by the Water Authority into the recycled water storage system. The recycled water system will continue to be supplemented with potable water until the additional source of recycled water supply from the City of San Diego's South Bay Water Reclamation Plant (SBWRP) is available. The supply of recycled water from the SBWRP is expected to begin in the fall of 2006 with construction completion and operation of the transmission, storage, and pump station systems necessary to receive the SBWRP recycled water.

Table 7
Historic Imported and Local Water Deliveries
Otay Water District

Calendar Year	Imported Water (acre-feet)	Recycled Water (acre-feet)	Total (acre-feet)
1980	12,558	0	12,558
1985	14,529	0	14,529
1990	20,077	0	20,077
1995	20,284	614	20,898
2000	28,878	1,274[1]	30,152

[1] The recycled water storage system was supplemented with potable water.

Imported and Regional Supplies

The availability of sufficient imported and regional water supplies to serve existing and planned uses within Otay WD is demonstrated in the above discussion on Metropolitan and the Water Authority's water supply reliability. The Otay WD currently takes delivery of about 32,000 ac-ft/yr of supplies from the Water Authority. This is expected to increase to about 52,151 ac-ft/yr by 2020. These figures take into account the amount of local supply (i.e. conservation and recycling) that is expected to meet demands within Otay WD service area.

Recycled Water Supplies

Wastewater collection, treatment, and disposal services provided by the Otay WD is limited to a relatively small area known as the Jamacha Basin, located within the Sweetwater River watershed, which is upstream of the Sweetwater Reservoir. Water recycling is defined as the treatment and disinfection of municipal wastewater to provide a water supply suitable for non-potable reuse. The Otay WD owns and operates the Ralph W. Chapman Water Recycling Facility (RWCWRF), which produces recycled water treated to a tertiary level for landscape irrigation purposes. The recycled water market area of the Otay WD is located primarily within the eastern area of the City of Chula Vista. The Otay WD distributes recycled water to a substantial market area that includes but is not limited to the U.S. Olympic Training Center, the EastLake Golf Course, and eventually to the proposed Project as well. The proposed Project will be incorporated within the Otay WD recycled water distribution system.

The Otay WD projects that annual average demands for recycled water will increase to about 5,290 ac-ft/yr by 2020 and are estimated to be 7,100 ac-ft/yr at ultimate build out. About 1,100 ac-ft/yr would be generated by the RWCWRF, with the remainder supplied to Otay WD by the City of San Diego's SBWRP.

Written Agreements, Contracts, or Other Proof

The supply and cost associated with deliveries of recycled water from the SBWRP is based on the following document, which is included in Appendix A.

- Agreement Between the Otay Water District and the City of San Diego for Purchase of Reclaimed Water from the South Bay Water Reclamation Plant. The agreement provides for the purchase of 6,721 ac-ft per year of recycled water from the SBWRP at a price of $350 per acre-foot. The Otay WD Board of Directors approved the final agreement on June 4, 2003 and the San Diego City Council approved the final agreement on October 20, 2003.

Recycled Water System Facilities

The Otay WD has and continues to construct recycled water storage, pumping, transmission, and distribution facilities to meet projected recycled water market demands. For over 12 years, millions of dollars of capital investment have been invested. The transmission, storage, and pumping capital improvements to receive and transport the recycled water from the City of San Diego's SBWRP are currently budgeted and have proceeded into the environmental documentation and preliminary design phases. These facilities are scheduled for completion and to be placed into operation at the end of 2006.

Financing

The capital improvement costs associated with the SBWRP supply are financed through Otay WD's water meter capacity fee rate structure. The Otay WD recycled water sales revenue, along with Metropolitan and the Water Authority's recycled water sales incentive programs are used to pay for the wholesale cost of the recycled water supply and the operating and maintenance expenses of the recycled water system facilities.

Federal, State, and Local Permits/Approvals

The Otay WD has in place an agreement with Metropolitan for their recycled water sales incentive program for supplies from the SBWRP. A copy of this agreement is included in Appendix B. The Otay WD will be preparing the application documentation for the recycled water sales incentive program with the Water Authority for supplies from the SBWRP. All permits for the construction of the recycled water facilities to receive, store, and pump the SBWRP supply will be acquired through the typical planning, environmental approval, and design processes.

The California Regional Water Quality Control Board San Diego Region (RWQCB) "Waste Discharge Requirements For Otay Water District, Jamacha Basin Facility, San Diego County" was adopted on June 29, 1992 (Order 92-25) to establish discharge requirements for the RWCWRF and it includes recycled water use provisions for the potential market areas. The Order prescribes waste discharge requirements and reclamation requirements governing the